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Technical Guidelines for Aseismic Design of Nuclear Power Plants

Translation of JEAG 4601-1987

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Brookhaven National Laboratory

Prepared for
U.S. Nuclear Regulatory Commission

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ABSTRACT

This document is a translation, in its entirety, of the Japan Electric Association (JEA) publication entitled "Technical Guidelines for Aseismic Design of Nuclear Power Plants - JEAG 4601-1987." This guideline describes in detail the aseismic design techniques used in Japan for nuclear power plants. It contains chapters dealing with: (a) the selection of earthquake ground motions for a site, (b) the investigation of foundation and bedrock conditions, (c) the evaluation of ground stability and the effects of ground movement on buried piping and structures, (d) the analysis and design of structures, and (e) the analysis and design of equipment and distribution systems (piping, electrical raceways, instrumentation, tubing and HVAC duct). The guideline also includes appendices which summarize data, information and references related to aseismic design technology.

FOREWORD

We are very glad to complete the English Translation of "Genshiryoku Hatsudensho Taishin Sekkei Shishin," JEAG 4601-1987 by the great effort of the Brookhaven National Laboratory (BNL) staff and others.

Dr. Walter Kato of BNL, whom I first met in the mid-1960's in Tokyo, mentioned to us the possibility of this type of effort approximately ten years ago. Dr. John Stevenson more recently contacted me about undertaking the translation of this document. Dr. James Costello of the U.S. Nuclear Regulatory Commission and others have encouraged and supported the project. Dr. Charles Hofmayer and Dr. Young Park have been developing the idea for this publication.

The members of our committee, Dr. Muneaki Kato, Mr. Rokuro Endo and others, worked on the review of the draft of the English Translation and prepared comments on it.

The Business Office of the Japan Electric Association, with the permission of the Ministry of International Trade and Industry, developed the copyright clearance for this publication with the U.S. Nuclear Regulatory Commission staff.

I greatly appreciate the continuous and extra effort by all of the above to make this publication possible.

We have also published JEAG 4601-1991 which is a supplement to the 1987 version. In addition, we are planning to revise JEAG 4601-1984 (Classification of Importance Level/Allowable Stress Edition) at the end of 1995 and publish it with some other supplements on new techniques.

April 1994

Heki Shibata, Chairman
Seismic Design Division
Nuclear Power Engineering Committee
Japan Electric Association

ACKNOWLEDGEMENTS

The permission granted by the Japan Electric Association (JEA) to publish the translation of JEAG 4601-1987 is greatly appreciated. In the event of any doubt regarding the translation, the original Guideline in Japane published by JEA should be consulted and remains the final authority.

The original translation from Japanese to English was prepared by the Ralph McElroy Company, Austin, Texas. The translation was further edited by Brookhaven National Laboratory (BNL) based on its own staff's review of the Japanese text and comments received from a JEA Code Division Committee chaired by Professor Heki Shibata. The translated text was also reviewed by an expert panel consisting of the following individuals:

- Dr. Robert Cloud, Robert L. Cloud & Associates, Inc.
- Dr. Carl Costantino, City College of New York
- Dr. I.M. Idriss, University of California, Davis
- Dr. Robert Kennedy, Structural Mechanics Consulting, Inc.
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- Dr. John Stevenson, Stevenson & Associates

The final translated text, including tables and figures, were prepared by the Graphic Arts Group at BNL.

This work was sponsored by the Division of Engineering, Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission. The NRC Project Manager was Dr. James F. Costello.

Electrotechnical Guide (JEAG)

Nuclear Power Edition

TECHNICAL GUIDELINES FOR ASEISMIC DESIGN OF NUCLEAR POWER PLANTS

JEAG 4601-1987

Survey Committee for Electrotechnical Standard

Japan Electric Association (JEA)

Recommendation

With the fast progress in the technology of nuclear power generation, the Ministry of International Trade and Industry is improving the technical standards and related codes, guides and so forth for nuclear power generating facilities in consideration of the progress achieved in technology and evolution in the social situation of society. In this process, the Ministry is making use of the initiative of the companies in the industry to ensure the highest level of safety.

In this process, the Electrical Technical Standard Survey Committee of the Japan Electric Association (JEA) has amended the present guideline by listening to opinions on the aseismic design techniques of nuclear power plants from various related parties and updating the content of the guideline. This is truly a useful task.

This guideline describes in detail the aseismic design techniques for nuclear power plants. It is believed that it is very useful for all persons engaged in design, operation/maintenance and other practical jobs in nuclear power plants.

This guideline was drafted carefully by the foremost specialists in various fields and was summarized by the Electrical Technical Standard Survey Committee. In order to perform design, construction and operation of the aseismic design technology for nuclear power plants, it is expected that the national technical standards, the related codes, guides and so forth and the various items defined in the guideline will be followed.

August 1987

Kunikazu Aisaka, Deputy Director-General
Agency of Natural Resources and Energy
Ministry of International Trade and Industry

Foreword

In order to develop the guidelines for aseismic design of nuclear power plants, the Special Committee on Nuclear Power of the Electrical Technical Standard Survey Committee set up an "Aseismic Design Sub-Committee" in January, 1968 to perform the evaluation. The draft of "Technical Guidelines for Aseismic Design of Nuclear Power Plants: JEAG 4601-1970" was formed by discussion at the Aseismic Design Sub-Committee, and it was acknowledged by the Special Committee on Nuclear Power in May, 1970, and by the Electrical Technical Standard Survey Committee in July, 1970. There were certain points, such as definition of the design earthquake and allowable stress during an earthquake, for which further discussion was needed. For these points, basic research was performed as projects of peaceful applications of nuclear power under contract with the Science and Technology Agency.

In this state, the Japan Electric Association (JEA) was requested by the Ministry of International Trade and Industry in 1974 to perform an investigation named "Guideline for Handling Earthquake Force for Aseismic Design," a topic closely related to the said definition of the design earthquake and allowable stress during an earthquake. The progress in this evaluation was inspected by the Special Committee on Nuclear Power, with attention paid to the results of the research work performed as projects of peaceful applications of nuclear power. Then, the "Special Committee on Aseismic Safety Evaluation of Nuclear Power" was set up under the Electrical Technical Standard Survey Committee, which presented an interim report in April, 1975. According to the conclusion drawn by the "Equipment/Piping Allowable Stress Subcommittee" set up in the Aseismic Design Sub-Committee in 1968, as well as the aforementioned interim report, in order to evaluate the "Guideline of Load Combination and Stress Evaluation" and the "Guideline of Classification of Importance Levels in Aseismic Design of Nuclear Power Generating Equipment and Its Application Range," an "Allowable Stress Division" and an "Aseismic Safety Importance Level Classification Division" were set up under the aforementioned Special Committee in November, 1975. They performed amendments for the classification of importance level and allowable stress of "Technical Guidelines for Aseismic Design of Nuclear Power Plants: JEAG 4601-1970" and furnished a new version of "Technical Guidelines for Aseismic Design of Nuclear Power Plants: Classification of Importance Level/Allowable Stress Edition, JEAG 4601-Supplement-1984" which was published in 1984.

JEAG 4601-1970 has its major points of description concentrated on the basic knowledge accumulated in the prior aseismic designs performed for actual nuclear power plants since 1960. In order to streamline the content and description scheme of the supplemented edition and to add new findings, an amendment was needed. As a result, the "Aseismic Design Sub-Committee" was set up again in January, 1984 under the Nuclear Specialty Committee of the Electrical Technical Standard Survey Committee.

On the basis of the preparatory session on compiling the guidelines of the present paper and the discussion with various related institutions, the first formal session of this division reached the following conclusions:

That is, for this round of amendment, the principle is to emphasize the technical guideline character by describing the main points of aseismic design for the so-called licensable features on the basis of the acknowledged experience accumulated up to now. As far as the new methods are concerned, for those which have passed the various tests and the examination of the "Special Study Committee on Aseismic Design of Nuclear Power Station" of this society, they are reported to the Ministry of International Trade and Industry, and they are described clearly for the further applications. As far as the other results of research work are concerned, they are listed in tables together with their publication times, and further development and applications are to be made for them.

The present guidelines are mainly applicable for light water reactors. However, they are believed to be also applicable for the so-called "ATR" and other pressure pipe type reactors, except for their special portions. In addition, the basic items are believed to be applicable for the future fast breeder reactors.

The original draft of this content compiled under the aforementioned guidelines had a huge volume, which, however, has been condensed to the present volume. For many examples and background of the content, the explanation may be insufficient. In such cases, the readers are referred to the cited references.

During this amendment process, all of the technical results achieved on the said background are taken into consideration with an effort to form the newest "Aseismic Design Technical Guidelines." Instead of formulating it as a detailed encyclopedia, a "guideline" style is adopted to formulate the final draft, which is not a mere collection of formulas. The Aseismic Design division sponsored a Steering and Editorial meeting to discuss the various chapters in an overall way. Also, for discussion of each of the various special fields, a Seminar on Seismic Motion, a Seminar on Civil Structures, a Seminar on Buildings and Structures, and a Seminar on Equipment/Piping System were convened, and many authors have devoted great effort to compiling this book. The draft has been amended several times based on the various seminars. The final version was reached after 40 sessions of the Steering and Editorial Meeting, 16 sessions of the Seminar on Seismic Motion, 16 sessions of the Seminar on Civil Structures, 27 sessions of the Seminar on Equipment/Piping System, and 7 sessions of the Division's Conference for Evaluation.

We here express our heartfelt thanks to the related government officers, to all the authors many of whom took part in the job in their companies and failed to have their names listed here explicitly, to the various companies in the nuclear power generation field and other fields, who made many comments on this book, to the persons in the Business Office of this Society, and, in particular, to the persons in charge of the various seminars for their great effort in making this book possible.

August 1987

Heki Shibata, Chairman
Seismic Design Division
Nuclear Power Specialty Committee
Electrical Technical Standard Survey Committee

About the Electrotechnical Guide

The Technical Standards based on the Electricity Utilities Industry Law define the legal codes for the minimum level needed for guaranteeing the safety for electrical apparatus.

The "Electrotechnical Code (JEAC)" formulated by the Electrical Technical Standard Survey Committee provides a specific explanation of the Technical Standards. It supplements items that are not described explicitly in the standards, and describes in an easily understandable way. It can be used as references for a provisional approval of exceptional cases/items. As a civil code, it classifies and defines the mandatory, advised and recommended items according to their contents and characteristics for persons in charge of design, construction and maintenance management of electrical apparatus.

On the other hand, there are many topics still in the research and development phase, which are believed to be necessary for the new techniques to be improved and necessary for forming safety code. It is difficult or inappropriate to define them in general. For example, there are the following cases:

(1) The case when there are few results and examples available including in foreign countries, regarding a new technique that is to be formulated in the code.

(2) The case when some items are necessary for safety, yet the theories and methodology related to the methods, countermeasures, etc., may not be establishable and it is difficult to formulate them for general applications.

(3) The case when it is difficult to clearly classify between the mandatory, advised and recommended items in which the research and development are necessary.

(4) The case when it may be inappropriate to make formulation in the consideration of the social situation.

In these cases, it is difficult to standardize. However, it is still desirable to formulate them in a general way to ensure safety. In this case, they are summarized in the "Electrotechnical Guide." Hence, in principal, it is desired that the "Electrotechnical Guide" be followed as an Electrotechnical Code (JEAC). However, it is necessary to pay attention to the following items:

(1) They ought to be interpreted appropriately so that they will not hamper progress in technology when they are actually adopted.

(2) The content must be fully understood to avoid errors in design and construction and so forth.

(3) Items and methods which are not described in the guideline but are appropriate for ensuring safety may also be adopted.

The Electrotechnical Guidelines were formulated by the Electrical Technical Standard Survey Committee organized by the related governmental agencies and with many authoritative specialists in all of the related fields taking part, with much effort and time used on this job. It is believed that it will be used by the many persons working in this field.

In order to facilitate future improvement in the guidelines, please send your opinions and requirements to the Japan Electric Association (JEA).

Points of attention in using this guideline

This guideline covers the knowledge accumulated up to 1986 It has many pages and is an overall collection of aseismic design technology for nuclear power plants. In order to use it effectively, please pay attention to the following points.

(1) Configuration

This guideline consists of the following chapters, attached data and appendices:

Chapter 1. Basic items

Chapter 2. Earthquake and basic earthquake ground motion

Chapter 3. Geological and ground survey

Chapter 4. Safety evaluation of ground and aseismic design of underground structures

Chapter 5. Aseismic design of building structures

Chapter 6. Aseismic design of equipment/piping systems

Chapter 7. Prospects of future technical topics

Attached data -
1. Licensing and related laws
2. Testing/inspection
3. Earthquake detecting equipment
4. Inspection/service after earthquake

Appendix -
1. List of various tests and research
2. Improvement of standardization programs
3. Aseismic specifications of various power plants
4. Recent survey report of intra-plate earthquakes
5. Basic references/reference books
6. List of summaries of seismic-related codes at the Institute of Nuclear Safety of Nuclear Power Engineering Corporation

The Attached Data include materials closely related to aseismic design technology, while the Appendices summarize the data, information, and references related to aseismic design technology.

(2) Nomenclature

The nomenclature is in principle the same for the various chapters. However, when a certain object is referred to by different terms in different fields (chapters), if it is determined that the customary different terms can be used better than the unified term, they are adopted for their respective fields.

(3) Citation

(a) Reference cited in each chapter are denoted by superscripts [brackets in this translation] in the text and are listed at the end of each chapter.

(b) For citations of test/research results listed in Appendix 1 "List of various tests and research" and citations of the results of the survey on the national improved standardization lis'e. Appendix 2 "Improvement of standardization programs," they are denoted as follows as superscript [brackets] in the text.

Citations from Appendix 1 "List of various tests and research"

K-C-1, 2, 3 (related to Chapter 2)
K-D-1, 2, 3 (related to Chapters 3 and 4)
K-K-1, 2, 3 (related to Chapter 5)
K-KI-1, 2, 3 (related to Chapter 6)

Citations from Appendix 2 "Improvement of standardization programs"

H-K-1, 2, 3, (related to Chapter 5)
H-KI-1, 2, 3 (related to Chapter 6)

For example, the symbol (K-C-1) indicates citation of the content listed in column K-C-1 in Appendix 1.

When the Electrical Technical Standard Survey Committee, Special Committee, Research Sub-Committee, etc., were working to formulate the guidelines, instructions were obtained from the Deputy Director-General of the Agency of Natural Resources and Energy, Ministry of International Trade and Industry, Manager of Nuclear Power Safety Administration Division of Public Utilities Department, Director of Nuclear Power Safety Examination Division, as well as the following governmental branches:

Industrial Location and Environmental Protection Bureau, Ministry of International Trade and Industry

Machinery and Information Industries Bureau, Ministry of International Trade and Industry

Bureau of Public Business, Regional Bureau of International Trade and Industry.

Standards Department, Agency of Industrial Science and Technology.

Nuclear Safety Bureau, Science and Technology Agency

Research Institute of Industrial Safety, Labor Standards Bureau, Ministry of Labor

Communications Policy Bureau, Ministry of Posts and Telecommunications

Land Transport Engineering (and Safety) Department, Regional Transport Bureau, Ministry of Transport

Fire Research Institute, Fire Defense Agency, Ministry of Home Affairs

Fire Prevention Division, Tokyo Fire Department

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TECHNICAL GUIDELINES FOR ASEISMIC DESIGN OF NUCLEAR POWER PLANTS

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Appendix

Chapter 1. Basic items

1.1 Basic ideas

1.1.1 Purpose of aseismic design

The purpose of the aseismic design of nuclear power plants is to design the facilities appropriately so that no excessive exposure to radiation takes place to the public and employees, in case of a major earthquake at the nuclear power plant. For this purpose, a more strict aseismic design should be performed for facilities, the damage of which would cause exposure to radiation, and for facilities which are designed to prevent discharge of the radioactive substances, than the other facilities of the power plant.

In addition to the aforementioned purpose of preventing radioactive exposure, rest of the facilities of the power plant with little relation to radioactive exposure, should also be designed to avoid any damage caused by earthquake. In this case, however, a trade-off may be considered between the interruption of power generation and destruction of facilities caused by damage and the cost increase due to the aseismic design.

1.1.2 Aseismic design and safety design

For safety design of a nuclear power plant [1.1.2-1], it is required that the facilities be designed to avoid excessive radioactive exposure to the public and employees, even in the case when various design conditions, including natural phenomena, are taken into consideration. One of these natural phenomena is the earthquake. It is required that the aforementioned safety requirement be satisfied even in the case of a major earthquake at the power plant. In other words, aseismic design is performed as a link in the whole chain of safety design.

1.2 Summary of aseismic design

1.2.1 Procedure of aseismic design

Items of the aseismic design of the various facilities of a nuclear power plant include determination of the design seismic motion for the site, confirmation of stability and survey of the ground during earthquake, stability of the support ground for the facilities, aseismic designs of the underground structures, buildings/structures, equipment, etc. They involve many fields, such as seismology, civil engineering, architecture, mechanical engineering, etc. As a result, with the aseismic capability taken into consideration for the overall layout of a plant and its construction plan, design of each facility is performed in its respective field. We will present detailed explanation of the various fields in the following chapter. At present, we only discuss the overall procedure of aseismic design.

As pointed out in section 1.1.1 "Purpose of aseismic design," it is necessary to ensure that the various facilities of the power plant do not cause a major accident due to failure in their safety mechanism during a major earthquake. For this purpose, design should be performed with the following procedure: (1) determination of the earthquake which may affect the site and should be taken into consideration in design; (2) determination of the earthquake ground motion at the site due to the aforementioned earthquake; (3) calculation of the ground motion input to the peripheral ground and the facilities; (4) calculation of the seismic force, stress, strain, deformation, etc., at the peripheral ground of the various facilities and at the various facilities caused by the seismic motion; (5) cross-sectional design for structures, and confirmation of aseismic capability by comparing calculated stresses with allowable stresses.

Determination of basic earthquake ground motion

The earthquake motions assumed for the site of the nuclear power plant include basic earthquake ground motions S_1 and S_2 with different intensities. Basic earthquake ground motion S_1 is assumed at the rock outcrop of the site based on past earthquakes, or earthquakes due to an active fault with a frequent activity, and referred to as the maximum design earthquake. On the other hand, basic earthquake ground motion S_2, which is beyond basic earthquake ground motion S_1, is assumed at the rock outcrop of the site due to an earthquake caused by active faults with a less frequent activity, an earthquake due to a seismo-tectonic structure at the site, or a shallow-focus earthquake (magnitude 6.5), and referred to as the extreme design earthquake. The basic earthquake ground motions are determined from the response spectra and/or simulated seismic waves.

Safety evaluation of ground and seismic design of underground structures

For the foundation soil of the nuclear reactor building, peripheral slope of the nuclear reactor building, and important outdoor underground structures, the seismic performance is evaluated on the basis of the basic earthquake ground motion from the viewpoint that the support function for class A and As facilities should not be degraded and there should be no secondary effect on maintaining functions of these facilities.

Safety evaluation of the foundation soil and peripheral slope is performed using the sliding-plane method or other conventional method with an appropriate soil model set up on the basis of geological survey, soil investigation, and test results. If needed, evaluation may also be performed by making static analysis and dynamic analysis using the finite element method or other methods which is suitable for treatment of more complicated conditions.

For the important outdoor underground structures, design of the structures is performed after an investigation of the safety of the support ground.

Seismic design of buildings/structures

For buildings and structures, depending on the seismic importance, design is performed for the dynamic seismic force or the dynamic seismic force calculated from the story shear coefficient, both for the standard seismic motion. Usually for buildings and structures, there are few cases when the safety of themselves are required directly. Instead, it is required that there should be no degradation in the function of the equipment which is supported on or contained in the buildings and structures designed according to their aseismic class. The design is performed for static seismic force calculated from the story shear coefficient and/or dynamic seismic force calculated from the standard seismic motion. The input seismic motion used for design of the nuclear reactor facilities is the seismic motion at the lower boundary of the analytical model, which is made considering the conditions of the site. For the building response analysis, either the spring model or an FEM model may be used to evaluate the dynamic soil-structure interaction effect. For basic earthquake ground motion S_1, since it is within the elastic range, a linear analysis is performed for the design. On the other hand, for basic earthquake ground motion S_2, a nonlinear elastoplastic analysis is performance considering the foundation uplifting and the material nonlinearity. If needed, the load due to the dynamic seismic force and the load due to the static seismic force may be combined with the other loads for stress analysis, structural design and evaluation.

Aseismic design of equipment/piping systems

For the equipment and piping systems, depending on the seismic importance, design is performed for the dynamic seismic force or the static seismic force calculated from story shear coefficient. Since these seismic forces are transferred from the support structure, the dynamic seismic force is often calculated using the floor response spectra. In addition, if needed, the load due to these seismic forces is combined with the other loads to calculate the member forces and the stress using the strength of material methods. The calculated stress is then compared with a pre-determined allowable stress to confirm the seismic safety. Also, for active machines such as pumps, etc., which are needed to perform the safety function, tests should be conducted to confirm the ability to maintain their functions during an earthquake.

Table 1.2.2-1. Definitions of classification of aseismic importance.

Class As	Facilities, damage of which may cause loss of coolant; facilities, which are required for emergency shutdown of the nuclear reactor and are needed to maintain the shutdown state of the reactor in a safe state; facility for storage of spent fuel; and nuclear reactor containment.
Class A	Facilities, which are needed to protect the public from the radioactive hazard in the case of a nuclear reactor accident, and facilities, malfunction of which may cause radioactive hazard to the public, but are not classified as Class As.
Class B	Facilities, which are related to the highly radioactive substance, but are not classified as Class As and Class A.
Class C	Facilities, which are related to the radioactive substance but are not classified in the above aseismic classes, and facilities not related to radioactive safety.

1.2.2 Classification of aseismic importance

In order to realize the purpose of aseismic design described in section 1.1.1 in a rational way, various facilities are classified based on their importance from the safety viewpoint, and design is then performed accordingly. Table 1.2.2-1 lists the definitions of the classification of aseismic importance. Table 1.2.2-2 lists the classification of functions. The basic ideas are as follows.

According to "Regulatory Guide for Aseismic Design of Nuclear Power Reactor Facilities: Japan Nuclear Safety Commission, July 20, 1981" [1.1.1-1] (referred to as "Evaluation Guideline" hereinafter), there are basically three classes: A, B, and C, with a portion of Class A called Class As. In the present guideline, we take Class As as a separate class in the 4-class classification (As, A, B, C).

For a nuclear power plant, technical measures should be taken to prevent loss of coolant and to maintain the nuclear reactor in its fully shutdown state without degradation in its functions, even in the case when the extreme design earthquake or the maximum design earthquake[(1)] takes place. Also, since the maximum design earthquake may take place more frequently than the extreme design earthquake, it is necessary to take technical measures to ensure maintenance of the functions of the facilities needed to prevent discharge of a large amount of radioactive substances, even in the case when the maximum design earthquake occurs while a coolant-loss accident is taking place at the power plant.

Depending on the frequency of occurrence of the earthquake, the safety state of the power plant is related to the intensity of earthquake and is defined as follows:

[(1)]Extreme design earthquake: This earthquake is stronger than the following "maximum design earthquake." It is supposed to be an earthquake which is selected from the earthquakes caused by an active fault, an earthquake caused by a seismic geological structure, and a shallow-focus earthquake, and which has the largest effect on the site.

Maximum design earthquake: This earthquake is selected from the past earthquakes and an earthquake caused by an active fault with a frequent activity, and it has the largest effect on the site.

For details, please see Chapter 2 "Section 2.1 Earthquake and standard seismic motion."

Table 1.2.2-2. Classification by function.

Class As	(i) Piping and equipment that form the "nuclear reactor coolant pressure boundary" (defined in the same way as in "Evaluation Guidelines for Safety Design of Light Water Reactor"). (ii) Equipment for storage of spent fuel. (iii) Equipment to rapidly induce negative reactivity for emergency shutdown of nuclear reactor, and equipment for maintaining the shut-down state of the nuclear reactor. (iv) Equipment used to remove decay heat from the reactor core after shutdown of the nuclear reactor. (v) Equipment which becomes the pressure barrier in case of accidental damage to the nuclear reactor coolant pressure boundary and directly prevents discharge of radioactive substances.
Class A	(i) Equipment needed to remove the decay heat from the reactor core after accidental damage to the nuclear reactor coolant pressure boundary. (ii) Equipment which is needed to suppress outward dissipation of radioactive substance released in an accident, but is not classified as (v) in the above Class As. (iii) Others.
Class B	(i) Equipment which contains or can contain the primary coolant which is in direct contact with the nuclear reactor coolant pressure boundary. (ii) Equipment which contains radioactive waste, except those which have a small content, or rupture of which due to the storage scheme leads to a radioactive effect to the public much smaller than the allowable annual doses outside the peripheral monitoring region. (iii) Equipment related to radioactive substances other than radioactive waste, and damage of which may bring excessive radioactive exposure to public and employees. (iv) Equipment for cooling the spent fuel. (v) Equipment which is used to suppress outward dissipation of radioactive substance in the case when it is released, but which is not classified as Class As or Class A.
Class C	(i) Equipment which is used to control the reactivity of the nuclear reactor, but which is not classified as Class As, A, or B. (ii) Equipment which contains or is related to radioactive substances, but which is not classified as Class As, A, or B. (iii) Equipment not related to radioactive safety, etc.

(i) In the case of extreme design earthquake

{1} Loss of coolant accident should not be induced.
{2} It should be possible to shutdown the nuclear reactor and to maintain the safe shutdown.
{3} Even in the unlikely case when loss of coolant accident takes place and the extreme design earthquake also takes place within a rather long period after the accident, the nuclear reactor containment should be able to maintain its function.

(ii) In the case of maximum design earthquake

{1} Items (i)-{1}, {2} in the above should be satisfied.
{2} Even in the unlikely case when the maximum design earthquake takes place right after a loss of coolant accident, the function for preventing discharge of a large amount of radioactive substances should still be maintained.
{3} Facilities, the damage of which causes discharge of a large amount of radioactive substances, should be able to maintain their function.

From the aforementioned basic consideration, the aseismic importance is classified and defined. In addition, classification of equipment is performed as related to the functions indicated in the classification of functions. From the classification of equipment and classification of functions, the aseismic importance is defined.

The equipment is mainly classified as follows:

{1} *Primary equipment:* System equipment directly related to function.

{2} *Auxiliary equipment:* Equipment indirectly related to function and playing an auxiliary role.

{3} *Direct support structures:* Support structures which directly support the primary equipment and auxiliary equipment, and support structures which directly receive the loads of the aforementioned equipment.

{4} *Indirect support structures:* Reinforced concrete or steel-frame support structures (buildings/structures) which receive loads transferred from the direct support structures.

{5} *Equipment for which inter-equipment influence should be considered:* Equipment for which damage of equipment in the lower category affects equipment in the upper category.

For the primary equipment, auxiliary equipment and direct support structures, the aseismic importance is defined as required by the safety function of the primary equipment. On the other hand, for the indirect support structures and equipment for which inter-equipment influence should be considered, since the safety requirements are determined as related to the other equipment, it is necessary to confirm that there is no problem under the standard seismic motion corresponding to the aseismic importance of the related equipment. This standard seismic motion is called seismic motion for evaluation. In addition, for the ground on which said equipment is installed, it is necessary to handle it in a similar way as the indirect support structures. As far as the peripheral ground, such as the back slope, is concerned, when its failure would affect equipment which is important for safety, it is necessary to handle it in a similar way as the equipment for which inter-equipment influence is considered.

For further details of the aseismic importance classification, please see "Technical Guidelines for Aseismic Design of Nuclear Power Plants: Volume for Importance Classification and Allowable Stress, JEAG 4601-Supplement-1984" [1.1.1-2] (referred to as "JEAG 4601 Supplement-1984" hereinafter).

1.2.3 Design seismic force

For a nuclear power plant, the facilities important for safety must be able to withstand the dynamic seismic force in the case of the extreme design earthquake or the maximum design earthquake; and, they should also be able to withstand the static seismic force depending on the aseismic importance. According to "Evaluation Guidelines," Table 1.2.3-1 lists the correspondence between the aseismic importance of the facility and the basic earthquake ground motion, which is the base for calculating the seismic force, story shear coefficient and seismic intensity.

For the building/structure as indirect support structure, it is necessary to confirm that it can maintain the support function against the basic earthquake ground motion corresponding to the aseismic class of the equipment supported. As pointed out in section 1.2.2 "Classification of aseismic importance," the basic earthquake ground motion is also called seismic motion for evaluation. Also, for the ground that supports facilities important to safety, it is necessary to confirm the ability to maintain the function of supporting the building and structure. For the back slope and other peripheral ground, damage of which may have a secondary effect on facilities important to safety, it is necessary to confirm that they would not fail due to the standard seismic motion.

1.2.4 Summary of earthquake and basic earthquake ground motion

(1) Summary of "Evaluation Guidelines"

Determination of the seismic motion used in the aseismic design of nuclear reactor facility (known as basic earthquake ground motion) is performed to satisfy the "Evaluation Guidelines." According to the "Evaluation Guidelines," seismological and geological knowledge are judged from the engineering point of view on the base of the past experience of safety evaluation; the earthquake ground motion used for the nuclear reactor facility is determined from the engineering judgement based on the updated knowledge in the seismology and seismic engineering fields from the viewpoint of ensuring the seismic safety of the nuclear reactor facilities against any conceivable earthquakes.

The basic earthquake ground motions can be divided into two types S_1 and S_2 according to their intensities. They are defined at the rock outcrop of the sites. The earthquakes that cause basic earthquake ground motions S_1 and S_2 are called the maximum design earthquake and the extreme design earthquake, respectively.

In addition, in the explanation of the "Evaluation Guidelines," definition of terminology, points for attention in evaluation of basic earthquake ground motion and evaluation standards of active faults, etc., are presented and are used as the standards in making judgment regarding the basic earthquake ground motion.

(2) Summary of earthquakes (maximum design earthquake, extreme design earthquake)

When the seismic design of the nuclear reactor facilities is performed, two types of earthquakes, namely, the maximum design earthquake and the extreme design earthquake are taken into consideration. Also, since the spectral characteristics of the seismic motion of the rock outcrop depend on the hypocentral distance, both short-distance and long-distance earthquakes should be taken into consideration for the above two design earthquakes.

a. The earthquake that causes basic earthquake ground motion S_1 is basically determined from the earthquake history. However, the earthquakes caused by active faults which have a high level of activity and may affect the site in the near future are also taken into consideration. Among these earthquakes, the earthquake which gives the largest influence on the site is called the maximum design earthquake.

The review of historical earthquakes is performed on the basis of the earthquake catalog. In particular, for the historical earthquakes that caused V-grade or higher-grade (of the seismic intensity scale by the Japan Meteorological Agency) effects on the site or its nearly region, detailed investigation is performed on the damage state, focus, and earthquake scale on the basis of the various earthquake catalogs and the many earthquake data used

Table 1.2.3-1. Correspondence between aseismic importance of facility and basic earthquake ground motion, static seismic coefficient, etc.[1,2]

	Aseismic importance	Basic earthquake ground motion, story shear coefficient, static seismic coefficient	Horizontal[5,6,7]	Vertical[8,9,10]
Building/ structure[3]	As	Basic earthquake ground motion	A_{S2}	½ A_{S2}
	As, A	Basic earthquake ground motion	A_{S1}	½ A_{S1}
		Story shear coefficient, static seismic coefficient	3.0 C_I	C_V
	B	Basic earthquake ground motion	—	—
		Story shear coefficient	1.5 C_I	—
	C	Basic earthquake ground motion	—	—
		Story shear coefficient	C_I	—
Equipment/ piping system[4]	As	Basic earthquake ground motion	A_{S2}	½ A_{S2}
	As, A	Basic earthquake ground motion	A_{S1}	½ A_{S1}
		Static seismic coefficient	3.6 C_I	1.2 C_V
	B	Basic earthquake ground motion	—	—
		Static seismic coefficient	1.8 C_I	—
	C	Basic earthquake ground motion	—	—
		Static seismic coefficient	1.2 C_I	—

(1)For Class As and Class A facilities, the horizontal seismic force and the vertical seismic force due to the basic earthquake ground motion are combined both in the unfavorable direction; and the horizontal seismic force and vertical seismic force caused by the story shear coefficient or the static seismic coefficient are combined in the unfavorable direction.

(2)The static horizontal seismic force of the underground portion of the building/structure is calculated by the horizontal seismic coefficient K specified for the underground portion. The static horizontal seismic force of the underground portion of the equipment/piping system is calculated from the value 20% larger than the horizonal seismic coefficient of the building/structure at the location where said equipment is set. (For details, please see Chapters 5 and 6.)

(3)For building/structure, the horizontal seismic force is calculated from the story shear coefficient; the vertical seismic force is calculated from the vertical seismic coefficient.

(4)The static horizontal seismic force of the equipment/piping system is calculated by regarding the story shear coefficient of the structure at the location of mounting as the seismic coefficient.

(5)A_{S2}: Acceleration acting on the facility due to basic earthquake ground motion S_2.

(6)A_{S1}: Acceleration acting on the facility due to basic earthquake ground motion S_1.

(7)C_I: Story shear coefficient (for details see Chapters 5 and 6).

(8)C_V: Vertical seismic coefficient for calculating static seismic force (for details, see Chapters 5 and 6).

(9)1/2 A_{S2}: 1/2 the value of the maximum acceleration amplitude of basic earthquake ground motion S_2 is taken as the vertical seismic coefficient.

(10)1/2 A_{S1}: 1/2 the value of the maximum acceleration amplitude of basic earthquake ground motion S_1 is taken as the vertical seismic coefficient.

as the basis for forming these catalogs, and they are considered as the maximum design earthquake. When there exists a "blank period" in the history of local earthquakes or a "blank region" in the seismic activities near the site, a special attention and investigation are necessary.

As far as the active faults which are taken into consideration in determining the maximum design earthquake are concerned, just as in the case of historical earthquakes, the active faults which give V- or higher-grade seismic intensity on the site are considered for investigation. The activity of the active fault is evaluated by adding engineering judgment to the reliable geological proof. Also, for evaluation of the active faults, it is necessary to investigate and study the relation between the active faults and the past earthquakes or microtremors. From the viewpoint of the influence on the site, evaluation of the earthquake scale due to the active fault and assumption of the location of the center of the seismic energy release are important items for investigation. According to the explanation in the "Evaluation Guidelines," the specific rules for determining the active faults as the cause of the maximum design earthquakes are as follows: those which are related to the historical earthquakes; those which belong to Class A active faults, had activity in the recent 10,000 years, and are predicted to be able to cause an earthquake in the near future; and those which are found to be significantly active at present by observation of micro-earthquake.

b. The earthquake that causes basic earthquake ground motion S_2 is an earthquake beyond the maximum design earthquake from the seismological point of view; it is the earthquake that gives the largest influences on the site and is called the extreme design earthquake. The objects that are taken into consideration for the extreme design earthquake include earthquake caused by active fault, earthquake caused by seismic earth structure, and the shallow-focus earthquake.

The active faults which are taken into consideration for the extreme design earthquake include Class A active faults except those taken as objects for the maximum design earthquake, active faults which belong to Class B or Class C and with possible activity within past 50,000 years, etc. The earthquakes based on the seismo-tectonic structure are considered with relationship to the occurrence of historical earthquakes and the active faults near the peripheral region of the site. They are used to determine the upper limits of the seismic scale on each of the earthquake-generating regions in the islands of Japan and their peripheral seas.

For the shallow-focus earthquake, in order to guarantee the safety margin for the facility, an earthquake of magnitude 6.5 is considered in design for the entire country.

(3) Summary and evaluation of basic earthquake ground motion

The basic earthquake ground motion is represented by the response spectrum at the rock outcrop of the site and the simulated seismic motions are obtained by curve-fitting with the spectrum. In order to determine the basic earthquake ground motion, sufficient investigation and evaluation should be made of the maximum amplitude, frequency characteristics, duration, and amplitude envelope time function, etc.

Since the maximum amplitude and frequency characteristics of the earthquake ground motion are closely related to the magnitude, focal distance, and earthquake ground characteristics of the soil at the site, detailed survey and investigation should be performed on the historical earthquakes, earthquakes due to active faults, and location of the center of earthquake energy release.

In principle, the maximum amplitude of the earthquake ground motion is represented in terms of velocity. It may be determined by using Kanai's formula, etc., and/or by using the theoretical analysis values based on the fault model. Also, it is useful to estimate the intensity of historical earthquake ground motion from the data on damage, such as falling tombstones, etc., for the references, and the statistical expected value can be used to estimate the peak amplitude of the earthquake ground motion.

1.2.5 Summary of geological/ground survey

(1) Summary

In order to guarantee the safety of the nuclear power plant, it is necessary to perform careful survey and tests of its geology and soil and to perform reliable analysis and design. In addition, the geology and soil have various different patterns, depending on the specific site. Hence, it is preferred that appropriate survey, test, analysis, evaluation and design be performed for the specific geology and soil on the basis of a good understanding on the specific site.

As indicated by its name, the geological/ground survey consists of geological survey and ground survey. The purpose of the geological survey is to make a geological survey on the site and the wide region surrounding the site to have a knowledge of the properties and activities of the faults that should be taken into consideration for the seismic design, and to provide detailed geological data for the peripheral region of the foundation of the structures. On the other hand, the purpose of the ground survey is to make a detailed survey on the ground of the building and its periphery, so that the ground is classified from an engineering point of view to provide the properties of the ground needed for the design.

(2) Geological survey

The geological survey includes survey of a wide region and survey of the site.

a. Survey of wide region

The survey of wide region is performed for a region within about 30 km of the site. For this region, the geological structure is clarified and the fault activity is evaluated for faults which may have activity in the Quaternary period. The survey methods include reviewing references, interpretation of aerial photographs, surface geological survey, wide-range elastic wave survey, sonic wave survey (sea), etc. Several methods may be used for measuring the ages of the faults, such as ^{14}C method, fission track method, quartz particle surface structural analysis, ESR age measurement method, etc. The survey results are summarized to form a geological diagram for the wide region. As far as the faults are concerned, evaluation is performed on S_1-class faults, S_2-class faults, and faults not to be considered for seismic design, separately.

b. Survey of the site

Survey of the site is implemented for the site and its vicinity. The geological structure in this region is clarified. In addition to the survey methods for the wide region, the survey methods also include boring, pit, trench, etc. The age of the fault is measured using the same methods as those used for the wide range survey. Based on the survey results, geological map, geological soil column profile, geological cross-sections, etc., are prepared, and the geological distribution and structure, activity history of fault, bedrock classification, etc., are evaluated. As far as the bedrock classification is concerned, the hardness of the bedrock, the properties of the geological discontinuity planes, and other geological factors are taken into consideration, so that the strength of the ground and its deformation characteristics can be correctly apprehended in an easy way.

(3) Ground survey/test

The ground survey/test is implemented to find the properties of the soil needed for evaluating the soil stability. Depending on the implementation stages, it can be divided into the following three types: (1) Survey in the preliminary design stage, (2) survey in the design stage, and (3) survey in the detailed design stage. On the other hand, depending on the type of structures, it can be divided to the following three types: (1) survey of the nuclear reactor building foundation soil, (2) survey of peripheral slopes of the nuclear reactor building, and (3) survey of ground for important outdoor underground structures.

a. Survey stages

(a) Survey in the preliminary design stage

In the preliminary design stage, the geologically classified ground structure and the general features of its three-dimensional distribution are apprehended in consideration of the mechanical properties needed to draw the preliminary site plan of the nuclear reaction facilities.

(b) Survey in the design stage

In the design stage, the properties of the ground for the individual structures and slopes are studied using the conventional method. Also, the ground and the soil characteristics of the soil layers needed for evaluating the seismic input for design are clarified.

(c) Survey in the detailed design stage

In the detailed design stage, properties are measured to perform more detailed safety evaluation in the case when the safety cannot be fully evaluated using the conventional methods.

b. Types of survey ground

(a) Nuclear reactor building foundation ground

Survey of the nuclear reactor building foundation ground is performed by boring survey, pit survey, rock test, bed rock test, elastic wave velocity test, etc. In the survey, the ground is classified to several classes. For each class, the properties (elastic wave velocity, shear strength, deformation coefficient, dynamic shear stiffness, damping constant, liquefaction strength, etc.) are derived.

(b) Survey of peripheral slopes of nuclear reactor building

Survey of the peripheral slopes of nuclear reactor building is performed by boring survey, elastic wave velocity test, pit survey, etc. In addition, if needed, on-site tests, etc., are also performed to obtain the mechanical properties needed for evaluating the stability of the soil.

(c) Survey of important outdoor underground structures

Survey of the important outdoor underground structures is performed by boring survey, elastic wave velocity test, etc. The various properties needed are derived while the survey results of the nuclear reactor building foundation ground are taken into consideration.

(d) Representation method of properties and applications in design

As far as the representation methods of properties and their applications in design are concerned, care should be taken in the following aspects: (1) static strength characteristics, (2) static deformation characteristics, (3) dynamic strength characteristics, (4) dynamic deformation characteristics and damping characteristics, as well as (5) evaluation of scatter.

1.2.6 Summary of safety evaluation of ground and seismic design of underground structures

Evaluation of seismic stability of nuclear reactor building foundation ground, nuclear reactor peripheral slope, and important outdoor civil structures is performed observing the following guidelines based on the standard seismic motion.

{1} The supporting function for buildings and equipment including those of Class A and Class As should not be degraded or

{2} There should be no secondary effect on the retention of function of these buildings and equipment.

(1) Nuclear reactor building foundation ground

a. Safety evaluation

Depending on necessity, the following analyses are implemented for the safety evaluation of the foundation ground.

{1} Analysis using sliding-plane method and other conventional methods
{2} Static analysis
{3} Dynamic analysis

If the analysis result can satisfy the safety evaluation standard value, usually further detailed analysis and examination can be omitted.

b. Design seismic force

(a) Seismic force for static evaluation

{1} Ground seismic coefficient

In principle, the design horizontal seismic coefficient (K_H) of the ground is determined using the following formula at the ground surface. Or, the equivalent seismic coefficient may be calculated by considering the vibration characteristics of the ground for the basic earthquake ground motion.

$$K_H = n_1 \cdot K_0$$

where K_0: standard design horizontal seismic coefficient (taken as 0.2)
n_1: correction coefficient for the site (taken as 1.0)

The design vertical seismic coefficient (K_V) is set as $K_V = K_H/2$, and is assumed to act together with the horizontal seismic coefficient in the unfavorable direction at the same time. $K_H = 0.2$ can be applied for bedrock with an S-wave velocity higher than about 500 m/s and the maximum acceleration from the basic earthquake ground motion, S_2, lower than 500 Gal. However, since there are various different types of soils, care should be exercised when it is applied.

{2} Seismic force acting on soil by building

The horizontal seismic force acting on the soil by the building is taken as the static seismic force based on the Evaluation Guideline," or the seismic force due to basic earthquake ground motion S_2, whichever is larger.

The vertical force acting on the soil by the building is calculated by assuming a constant seismic coefficient in the vertical direction (taken as 1/2 the maximum horizontal acceleration amplitude in the case of seismic force caused by basic earthquake ground motion S_2) in consideration of the vibration characteristics of building/structure with a vertical seismic coefficient of 0.3. In the case when dynamic analysis is also performed, it is possible to omit the static evaluation using the seismic force due to basic earthquake ground motion S_2.

(b) Seismic motion used in dynamic evaluation

The horizontal seismic motion used in dynamic analysis is set by transforming the basic earthquake ground motion S_2 defined at the rock outcrop of the site to the lower boundary of the analysis model. It is assumed that

For each class of facilities, the calculation method of the seismic force is determined according to its importance. For the equipment/piping support structures and other facilities, the seismic motion for evaluating the function retention is determined according to the importance of the facilities supported. In the design of buildings/structures, the seismic load calculated by the aforementioned method, the load constantly acting on the nuclear reactor facilities, the load acting on the facility when the nuclear reactor facility is in operational state (including abnormal transient stage during operation), and the load acting on the facility in case of an accident of the nuclear reactor facility are considered. The seismic force should be combined with the constant load and operational load (conventional operation load and abnormal transient operation load). The load with relatively long duration should be combined with the seismic force caused by the basic earthquake ground motion S_1.

The allowable limit of the building/structure with respect to the combined state of the seismic force and other load is determined according to the seismic importance class. In the building/structure that support facilities having different seismic classes, the allowable limit is determined to ensure a safety margin for the ultimate strength and to ensure that the functions of the supported facilities are not degraded due to the deformation of buildings/structures. The ultimate strength capacity is determined to ensure that the strength of each building or structure has an appropriate safety margin corresponding to the importance with respect to the required horizontal strength capacity.

(2) Seismic response analysis

The nuclear reactor building is a typical rigid structure. In addition, it is a complicated structure made of various structural materials in various structural forms. In performing seismic response analysis, it is important to model a structure as detailed as possible depending on the purpose of analysis and to calculate the response quantities as required for design using appropriate analysis methods.

a. Soil-structure interaction

Since the nuclear reactor building is a rigid structure, the interaction with the ground is larger than that of a conventional building, therefore it is necessary to incorporate the influence of the ground, such as the embedment effect, etc., into the vibration model in an appropriate way. In many cases, the influence of the ground is analyzed using a sway/rocking model, with the ground beneath the foundation mat replaced by equivalent horizontal and rotational springs. For the case when the effects of embedment depth, backfill soil, and the peripheral ground are separately considered, or when the adjacent buildings are considered, the ground may also be represented by the finite-element model (FEM) or the discrete-mass system model. In the recent investigation, the boundary treatment in the semi-infinite ground analysis is performed using the rational boundary element method (BEM), together with the FEM model for the peripheral ground of the building by using the substructure method.

For the vibration model, the in situ test data and laboratory test data are used to evaluate the dynamic ground stiffness, damping, and other ground properties for analysis. As the theoretical approach, based on the assumption of a homogeneous elastic body, the ground compliance, vibration admittance theory, etc., are used to derive the horizontal, vertical and rotational springs. Each spring, however, must take into consideration the dissipation effect of the vibration energy into ground. As a result, they are represented by frequency-dependent complex stiffness. Also, when the ground is handled using the FEM model or discrete-mass model, it is possible to assign different elastic constants for different layers.

In the seismic response analysis, the way to input the standard seismic motion at the rock outcrop is very important, since it has a great influence on the seismic design. Usually, in the case when it is possible to neglect the embedment since the nuclear reactor building is set on a rock outcrop, the basic earthquake ground motion itself can be taken as the input seismic motion. However, in the other cases, response analysis of the ground should be performed using a one-dimensional wave theory for the basic earthquake ground motion at the rock outcrop to determine the input seismic motion to the soil-structure interaction model by considering the site topography, soil layering and embedment depth.

b. Seismic response analysis

To model the superstructure of the nuclear reaction building, a so-called bending/shear-type discrete-mass system model is used. In this model, the various parts are taken as multicantilevers standing on the foundation mat; or, the various parts are combined as a single cantilever, with masses concentrated at the floor position. To determine the stiffness of the various parts of the building, the bending/shear stiffness are evaluated considering the web/flange effects. In order to take into account the effect of wall openings and the 3-D effect of orthogonal walls, it is also possible to use FEM with the foundation mat and various building parts modeled as a continuous body. In modeling the superstructure, it is also important to evaluate the properties of the building/structure related to stiffness and damping. For stiffness, the evaluation method using the various elastic constants is available in the various standards of Architectural Institute of Japan. For damping, the conventional damping constants for the different types of structures related are used, and the damping problem is treated in the vibration equations as internal viscous damping, modal damping, strain-energy-proportional damping, complex damping, etc. The major structural elements of the nuclear reactor buildings include box-shaped or cylinder-shaped shear walls, for which the restoring force characteristic curves have been determined on the basis of many structural experiments.

In nonlinear seismic response analysis, a model of the shear wall is formed by the aforementioned bending/shear cantilevers, with their skeleton curves approximated by trilinear lines. The hysteresis curve may be assumed in the so-called peak-oriented type, origin-oriented type, degrading trilinear type, etc. In the case when a large overturning moment acts on the base portion of the nuclear reactor building, the geometric nonlinearity is considered using a rocking spring for uplifting of base mat.

The conventional solution methods for the linear vibration equations include modal spectral method, time history modal method, direct method, frequency response analysis method, etc. For the frequency response analysis method, first, the response in the frequency domain is calculated to consider the frequency dependency of stiffness and damping; then, the results are transformed to the time domain. In order to evaluate the building stability, it is necessary to determine the contact pressure and contact rate of the foundation mat using the linear/nonlinear response analysis results. In addition, it is necessary to evaluate slide, etc. In the design of equipment/piping systems, the time history responses of the floor and other necessary parts on which they are installed are necessary. The floor response spectra are calculated with the damping constants of the equipment/piping systems used as parameters.

(3) Stress analysis and structural design

a. Stress analysis

In order to select the stress analysis method and modeling method, much attention should be paid to the configuration and load conditions of the structure. For the buildings in a nuclear power plant, since the structural forms are complicated, and the thicknesses of walls and plates of the structural components are much larger than those of the conventional buildings, the stress analysis is mainly performed with the aid of FEM analysis.

Important items for the stress analysis are as follows:

{1} Input method and model of composite structure
{2} Formation of analytical model for the thick concrete structures, such as the foundation mat of a containing facility, etc.
{3} Evaluation of spring in stress analysis of foundation mat
{4} Treatment of soil pressure in stress analysis
{5} Handling of thermal stress in combination with S_1 seismic stress.

b. Cross-section design

Just as for conventional buildings, cross-section evaluation for the various parts of nuclear reactor buildings is performed using various standards in principle. However, since a nuclear reactor building has thicker walls than a conventional building and there exist parts with complicated shapes, special attention should be paid to the following points when design is performed.

{1} Evaluation method for combining stresses
{2} Cross-section design method of thick concrete parts, such as foundation mat, etc.
{3} Anchor bolt design method
{4} Flat slab structure design method
{5} Design method of seismic walls with openings
{6} Composite structure design method
{7} Splicing method of large-diameter reinforcing bars.

c. Evaluation of function integrity

It is necessary to evaluate the ability of the various parts to maintain function with respect to stresses due to the S_1 and S_2 earthquakes and under necessary load combinations. The objects of the evaluation include leakproofing function, function in preventing secondary accident, support function, etc.

As far as the limit values of the various parts of building in maintaining the aforementioned functions are concerned, there are as yet no quantitative specifications/standards. As a result, for designers at present, the commonly used criteria for function integrity are S_1 for the allowable stress design, and S_2 as well as S_1 + LOCA for the ultimate strength design. For the allowable limit values with respect to the various functional requirements, the basic guidelines are provided in this document.

d. Safety margin

For the safety margin with respect to the static strength capacity and the dynamic strength, the reference value is not defined in the "Evaluation Guideline." Up to now in the practical design, it is covered by introducing a sufficient margin. For the quantitative evaluation of the safety margin, it is desired that a clearer definition be provided in the future.

(4) Concrete vessel

The concrete vessel contains the nuclear reactor as well as other equipments/piping systems. In an accident, the vessel can prevent dissipation of the radioactive substances which are leaked out. It is a structure made of reinforced concrete or prestressed concrete. The design is performed on the basis of the various technical standards related. In the structural analysis, the stresses are calculated with respect to the load conditions listed in these standards.

1.2.5 Summary of seismic design of equipment/piping system

(1) Basic guideline of seismic design

a. Structural plan and seismic support plan

In principle, the equipment/piping system of the nuclear reactor facility is designed in the rigid frequency region just as the support structure and building. The seismic performance of the equipment/piping system usually depends significantly on its support structure and its configuration. As a result, it is important to have an appropriate seismic support plan to ensure necessary and sufficient seismic performance. For parts in the equipment/piping system with a certain freedom in configuration, from the seismic viewpoint, the center of gravity should be made as low as possible and the installation state should have high stability in the configuration plan. In the case when a part with low seismic importance is near a part with high seismic importance, it is necessary to recheck the configuration plan to assure that the part with high seismic importance is not affected by earthquake damage to the part with low seismic importance. In addition, since the seismic support plan is also related to equipment maintenance and service, in addition to arranging a necessary and sufficient optimum plan to ensure the seismic performance of the system, it is important to arrange a large enough safety margin in consideration of the uncertain characteristics of the seismic force.

For the structural bodies of the equipment/piping system related to the pressure portion in a light water reactor or converter reactor, usually, the seismic load is not a determining factor for the plate thickness, and the portion of stress from seismic force is smaller as compared with the operating stress of the equipment; however, since the seismic support structure is mainly designed according to the seismic force, it is necessary to perform appropriate strength design with the uncertain factors of the seismic force taken into consideration and to pay attention to ensure the stiffness at the seismic support point. In particular, great care should be exercised in the design of the anchorage, which is the most important portion when the seismic damage pattern is taken into consideration, as it is in the boundary region with the building design.

b. Seismic analysis and safety evaluation

For the equipment/piping system, it is necessary to make appropriate classification according to seismic importance. Then, it is necessary to ensure that the system is safe with respect to the design seismic force corresponding to the applicable seismic class (As, A, B, or C).

The design seismic forces that should be calculated include the seismic force due to the horizontal static seismic coefficient corresponding to the seismic class, the dynamic seismic force which is based on the appropriate seismic response analysis with respect to standard seismic motion S_1 for Classes As and A, and with respect to standard seismic motion S_2 for class As, as well as the seismic force calculated from the vertical seismic coefficient.

The basic idea for the seismic safety evaluation of the equipment/piping system is that it is necessary to ensure that the combined stress including the seismic stress from the aforementioned design seismic force and the stresses due to other loads that must be taken into consideration is within the allowable limit (design by analysis). However, in the case when there are problems related to the complexity of the system's analysis and reliability, or when it is necessary to evaluate the functional integrity of equipment which cannot be determined by the allowable stress limit, it is possible to make confirmation by performing appropriate vibration test (evaluation by test).

(2) Seismic response analysis and design seismic load

a. Response analysis method

In principle, the seismic response analysis of the equipment/piping system of seismic classes As and A is performed by adopting the spectral modal analysis method based on the design floor spectrum of the floor used for installing the aforementioned system. The design floor response spectrum adopted is usually that of the appropriate floor which is near the center of gravity of the system or has the most seismic support points. However, depending on the requirement of the seismic safety evaluation, it is also possible to perform multi-input analysis by using the related floor response spectra, or approximate analysis similar to the aforementioned analysis. For the combination of the response due to the vertical seismic force and the aforementioned horizontal dynamic response, the absolute sum method is adopted.

For the nuclear reactor vessel, nuclear reactor pressure containment and core structures, in principle, a time history response analysis method is adopted using an analysis model with the aforementioned structures integrated with the reactor building or using an analysis model similar to the substructure method with the aforementioned structures separated from the reactor.

For seismic Class As, in the analysis using basic earthquake ground motion S_2, it may be sufficient to perform elastic design, using the linear spectral modal analysis method based on the floor response spectrum for the S_2 earthquake. It is, however, also permissible to adopt the nonlinear time history response analysis method with inputs from the mounting points and seismic support points based on the appropriate restoring force characteristics of the system.

For seismic class B, if it is determined that there is a danger of resonance due to its natural frequency, dynamic evaluation is performed on the basis of the spectrum which is 1/2 the floor response spectrum for S_1 design, to confirm the seismic safety. Also, an approximation or simplified method may be used for seismic response analysis, so long as there is no problem in safety (examples of these methods include the constant pitch span method, response evaluation method using only fundamental frequency, etc.).

b. Analysis model

Usually, containers are modeled by a one-dimensional discrete-mass/flexural shear beam system; pipes are modeled by a three-dimensional discrete-mass/flexural torsional shear beam system; other equipment is modeled similarly. Also, for containers, it is necessary to analyze the ovalization. For a large-sized water tank, it is necessary to analyze the sloshing motion of the water. In order to perform analysis for these characteristics, a sufficiently detailed model is needed. In addition to the discrete-mass system (concentrated constant system), a continuous model (distributed constant system) or a combination system may also be considered. In addition, it is also possible to use finite element models.

For a seismic support system, if it is based on a rigid structure design, it is possible to assume rigid support points. On the other hand, in the case when the stiffness of the support structures, e.g., steel frame supports, is not very high, as compared with the stiffness of the equipment/piping system, the support stiffness should be taken into consideration. For the anchorage, based on the judgment on the mechanical characteristics, the stiffness of the anchorage should be considered. Properties of the various elements of the analytical model include average moment of inertia, effective shear cross-sectional area, and other geometric characteristics of the system, as well as modulus of elasticity and other material mechanical characteristics depending on the operation temperature. They should be evaluated appropriately, respectively.

As far as the damping constant is concerned, in principle, the conventional design damping constant is used. However, in the case when the system is an interaction system with different parts having different damping constants (such as an interaction system with container 1.0%-frame 2%-pipe 2.5%), it is possible to use the modal damping constants.

c. Design seismic load

For Type 1 equipment, Type 2 container, and Type 3 equipment, as well as other seismic Class As and Class A equipment, the seismic load for design is determined on the basis of the seismic loads (moment, shear force, axial force, etc.) obtained from the S_2, S_1 seismic response analyses and the static seismic force. The principle is that the seismic load due to basic earthquake ground motion S_1 or the seismic load due to the static seismic force, whichever is larger, is adopted.

As far as the static seismic force of the equipment system is concerned, in the case when the story shear force coefficient of the building in which it is installed is known (the seismic class of the building is taken as the same as the class of the equipment system), in principle, 1.2 times the coefficient is used as the design horizontal seismic coefficient in the calculation.

(3) Stress/strength analysis

a. Stress analysis of Class As, A equipment

In the case of the Type 1 and Type 2 containers, the stress analysis method adopted is usually based on the shell theory or finite element method, with the loads of the various operating states which should be combined with the seismic load acting at the same time. However, it is also possible to calculate the stress during earthquake and the stress in operating state separately, then adding them considering the stress type and the stress component direction.

For Type 1 and Type 3 piping system, the stresses are calculated separately for the various operating loads and are combined with the stress due to the seismic load using an absolute sum in consideration of the direction of the seismic force direction, stress component direction, etc. For the other equipment/piping systems, too, stress analysis should be performed with the simultaneous application of the operating loads and the seismic force with the stress direction taken into consideration. However, the method of calculating the stresses separately followed by addition can also be adopted as the simplified method on the safety side.

The primary stresses due to the seismic force refer to all of the internal forces needed to satisfy "the force equilibrium conditions" with the external seismic forces. It is necessary to perform a detailed analysis of these stresses to evaluate the maximum stress. The secondary stresses due to the seismic force meet the self-balance conditions. They should be evaluated when the location where they take place cannot be ignored from the viewpoint of the functional integrity of the equipment system.

In the case when evaluation is to be made of the fatigue caused by the seismic force, the cyclic numbers of an earthquake load is needed. In this case, it should be determined appropriately from the characteristics of the seismic response waveform characteristics of the floor on which the system is installed and the seismic response characteristics of the system.

b. Stress analysis of Class B and C equipment

For Class B and C equipment, since the static seismic force is determined independent of the seismicity of the site, the design analysis/evaluation methods of the equipment can be standardized. They are mainly classified into the following types: container/tank type, pump/blower type, and pipe/duct type. For these types of equipment, stress analysis/strength evaluation is performed on the basis of the calculation of the primary stress during an earthquake. Design of the Class B and C equipment can be performed based on the prescribed stress check points, stress calculation equations, and calculation formats. For Class B equipment with the danger of resonance, since dynamic evaluation is needed, the natural period should be calculated. The format includes evaluation using the dynamic seismic force if the system is not a rigid structure.

c. Stress analysis of support structures

The reaction force during earthquake for the support structure is calculated from the dynamic and static seismic force for Class As and A equipment and mainly from the static seismic force for Class B and C equipment. The support structure must be designed to withstand this seismic reaction force. For Class As and A support structures, it is necessary to guarantee not only the strength but also the necessary stiffness. In addition, design of the support structure is also closely related to the Steel Structure Design Standard of the Architectural Institute of Japan. Attention should be paid to this fact.

(4) Seismic safety evaluation

As far as the seismic safety evaluation of the equipment/piping system of a nuclear reactor facility is concerned, in the case when "design by analysis" is performed, it is necessary to ensure that the various stresses caused by the loads to be combined with the design seismic force corresponding to the seismic importance should be within the corresponding allowable stress limits. However, attention should be paid to the fact that depending on the equipment, its function may not be fully evaluated by using only the strength evaluation. In the case of "evaluation by test," in addition to the strength evaluation, evaluation should also be performed from the viewpoint of functional integrity. However, attention should be paid to assure the appropriateness of the similarity of test specimen, seismic input characteristics, etc.

References

[1.1.1-1] Nuclear Safety Survey Division, Bureau of Nuclear Safety, Science and Technology Agency (ed.): Guidelines of Nuclear Safety Committee, 1984, Taisei Publishing Co., pp. 376-391.

[1.1.1-2] Electrical Technical Standard Survey Committee: Technical Guideline of Seismic Design of Nuclear Power Plant: Volume for Importance Clarification and Allowable Stress, JEAG 4601-Supplement-1984, Electrical Society of Japan.

[1.1.2-1] Nuclear Safety Survey Division, Bureau of Nuclear Safety, Science and Technology Agency (ed.): Guidelines of Nuclear Safety Committee, 1984, Taisei Publishing Co., pp. 28-54.

Chapter 2. Earthquake and basic earthquake ground motion

2.1 Summary of earthquake and basic earthquake ground motion

Evaluation of the seismic motion used for seismic design of nuclear reactor facilities (known as basic earthquake ground motion) is performed to satisfy "Regulatory guide for Aseismic Design of Nuclear Reactor Facilities in Power Plants" (referred to as "Evaluation Guidelines" hereinafter) drafted in September, 1978 by the Atomic Energy Commission (partially amended in July, 1981 by the Nuclear Safety Commission).

According to the "Evaluation Guidelines," seismological and geological knowledge are judged from the engineering point of view on the basis of past experiences of safety evaluation; the basic earthquake ground motion is determined on the basis of the updated knowledge in the seismology and seismic engineering fields from the viewpoint of ensuring the seismic safety of the facilities used in the nuclear reactor against any possible earthquakes.

The basic earthquake ground motions can be divided into two types: S_1 and S_2, according to their intensities. They are defined at the rock outcrop of the sites.

The earthquakes that cause basic earthquake ground motions S_1 and S_2 are called the maximum design earthquake and the extreme design earthquake, respectively.

In addition, in the explanation of the "Evaluation Guidelines," definition of terminology, points for attention in evaluation of standard earthquake and evaluation standards of active faults, etc., are presented and are used as the standards for making judgment for the basic earthquake ground motion.

Figure 2.1-1 shows the items for investigation and the points for attention needed for evaluation of earthquakes. Figure 2.1-2 shows the items for investigation and the points for attention needed for determination of the basic earthquake ground motion. Figure 2.1-3 shows the flow for determining the specific basic earthquake ground motion.

2.2 Earthquakes

As pointed out above, when the basic earthquake ground motions S_1 and S_2 are to be determined, it is necessary to select the maximum design earthquake and the extreme design earthquake.

The maximum design earthquake is assumed to be the earthquake with the largest influence among the following earthquakes: earthquakes which once had an influence of Scale V or higher intensity, on the earthquake intensity scale of the Meteorological Agency, on the site or in its vicinity according to the historical data and are expected to take place again with the same influence on the site and its vicinity, and earthquakes due to active faults with a high activity which may have influence on the site in the near future.

The extreme design earthquake is supposed to be the earthquake with the largest influence among the earthquakes greater than the maximum design earthquake from the seismological point of view, with investigation made from the engineering point of view on the basis of the past earthquake state, properties of active faults in the vicinity of the site, and seismic geostructure.

In this section, we will discuss past earthquakes that should be taken into consideration, earthquakes due to active faults, and earthquakes caused by seismic geostructures.

2.2.1 Past earthquakes

First, in order to select the earthquakes which had an influence of Scale V or higher intensity on the site or its vicinity, a survey is made of the various earthquake catalogs which list the historical earthquakes. The

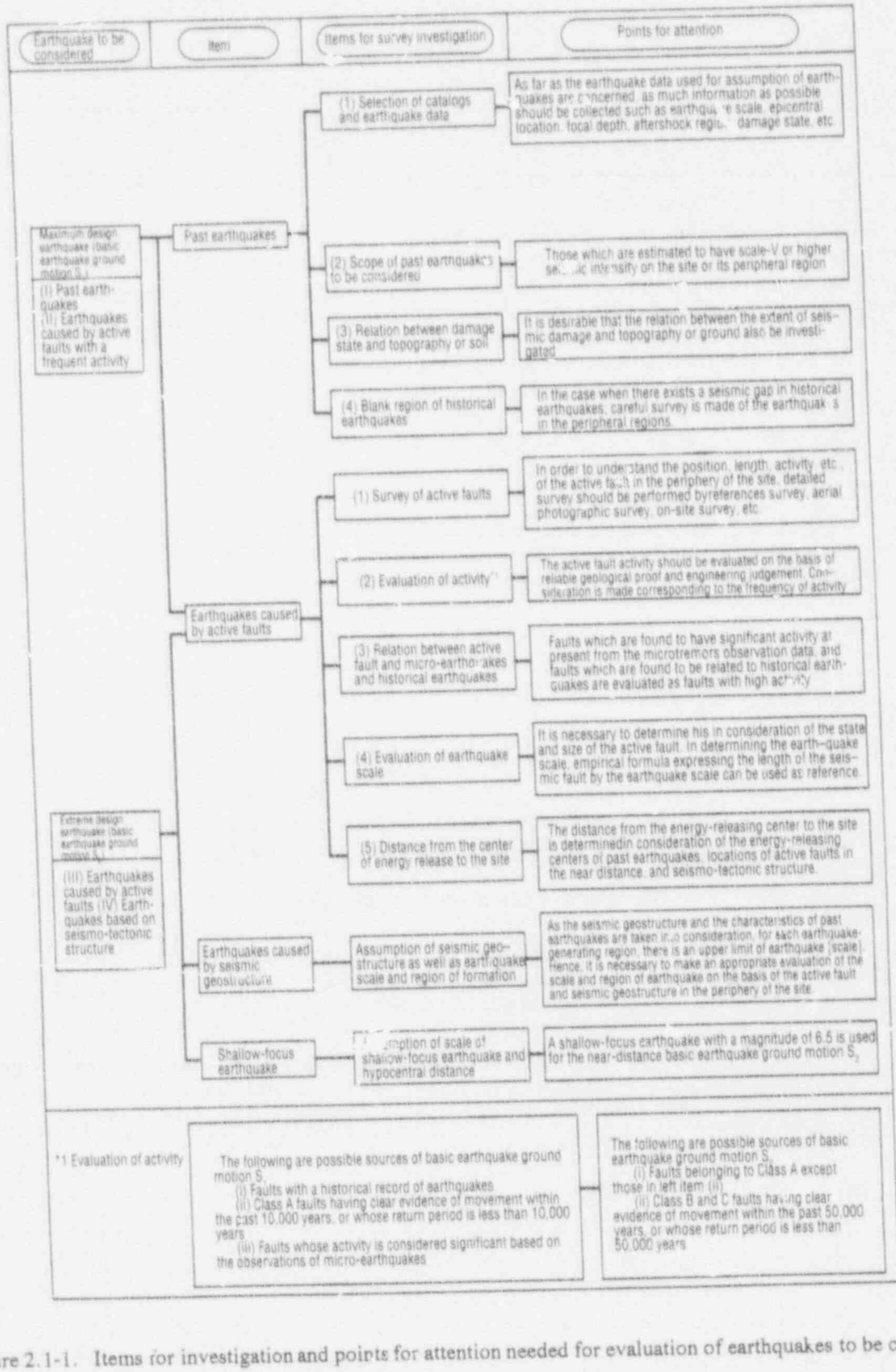


Figure 2.1-1. Items for investigation and points for attention needed for evaluation of earthquakes to be considered.

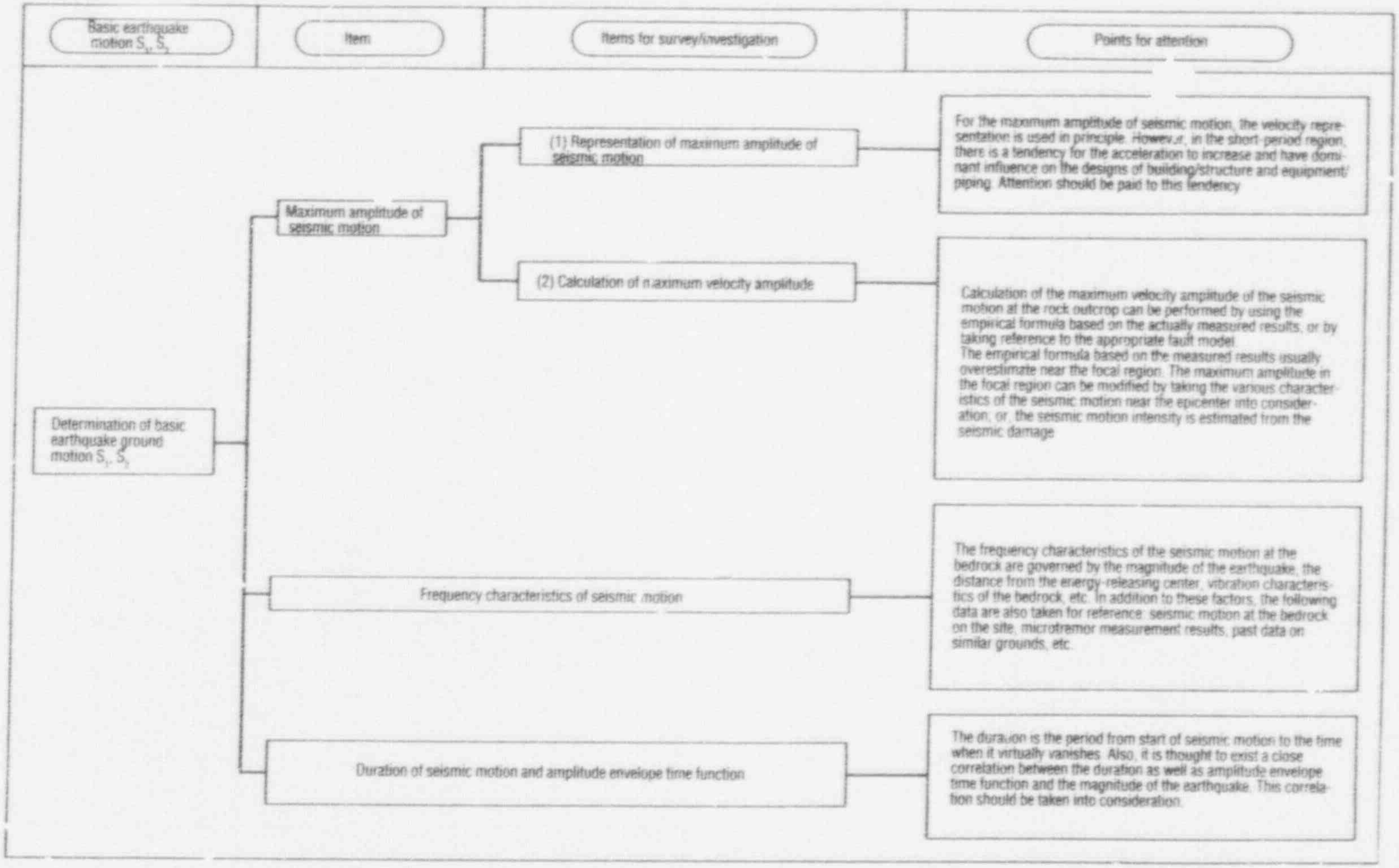


Figure 2.1-2. Items for investigation and points for attention needed for determination of basic earthquake ground motions.

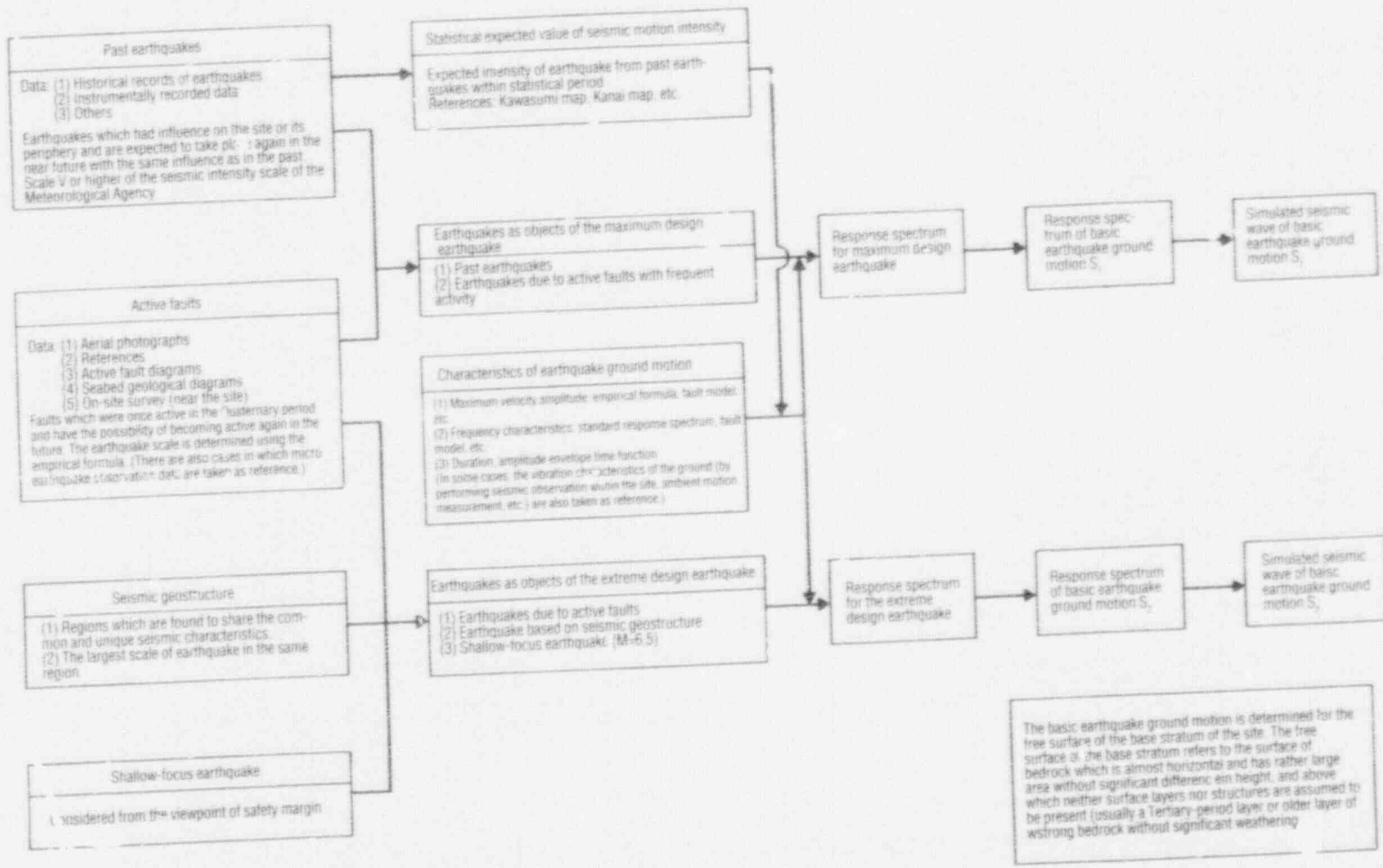


Figure 2.1-3. Flow sheet for determination of basic earthquake ground motion.

earthquake catalog with high reliability is used for making a survey of the earthquakes which had an influence of Scale V or higher intensity on the site and its vicinity with respect to magnitude, epicentral location, focal depth, aftershock region, resultant damage, etc. The survey region is usually within a radius of 200 km from the site, with emphasis on the seismic activity in the periphery of the site.

(1) Earthquake activity

As can be seen from the reference [2.2.1-1] which summarizes the numbers of strong earthquakes, severe earthquakes and catastrophic earthquakes in the history of Japan (Figure 2.2.1-1 to Figure 2.2.1-3), there exist significant differences in the earthquake activity between different regions. As a result, survey should be made with consideration of the seismic activity on the periphery of the site. In the following, we will present a brief description of the earthquake catalog used for survey.

a. Types of earthquake catalogs

(a) Historical records of earthquakes

There are many historical records on the resultant damage of earthquakes. The historical data before the Meiji era were assembled and sorted by Mr. Minoru Tayama as one of the research programs performed by the Earthquake Hazard Preventing Survey Institute set up in 1892 (Meiji 25). The data were first published in 1904 (Meiji 37) as "Historical Data of Earthquakes in Japan" [Dai Nippon Jishin Shiryo] [2.2.1-2]. Afterwards, in the Showa era [translator's note: after 1926], Kaneyoshi Musha made a significant supplement to these data and published "Supplemented Edition of Historical Data of Earthquakes in Japan" [2.2.1-3] (three volumes, 1941-1943), and "Earthquake Historical Data in Japan" [2.2.1-4] (1949). The two books listed the ancient earthquake data in the period from the start of written history to 1848 (Koka 4) and the period from 1848 (Kaei 1) to 1867 (Keio 3) for different types of earthquakes.

In addition, recently, the Earthquake Research Institute at University of Tokyo published "New Edition of Historical Data of Earthquakes in Japan" [2.2.1-5] based on new materials acquired in the recent survey of the historical records.

(b) Data of observation using instruments

The first earthquake observation using instruments in Japan was performed in 1872 (Meiji 5). Then, seismometers were installed in many locations in Japan. In 1884 (Meiji 17), earthquake survey started all over Japan. In the next year, for the first time, the observation results were published as "Earthquake Report of Central Meteorological Station." Later, the observation results were published in "Jishin Geppo" [Seismology Monthly], etc.

(c) Earthquake catalogs

Based on the above "earthquake historical data" and "data of observation using instruments," the data of the earthquake scales (magnitude) and the source characteristics (epicentral location, focal depth) are assembled to form "Earthquake Catalogs." At present, the major available earthquake catalogs are as follows:

(i) Rika Nenhyo [Annual of Natural Sciences] [2.2.1-6]
(ii) Shiryo Nippon Higaijishin Soran [Data Encyclopedia of earthquakes with damage in Japan] [2.2.1-7]
(iii) List of earthquakes with damage in Japan before 1975 [2.2.1-8] (Referred to as "Usami Catalog (1979)" hereinafter)
(iv) Nippon Fukin no M6.0 Ijono Jishin oyobi Higai jishin no Hyo [Table of earthquakes of M6.0 or higher and disastrous earthquakes in Japan and vicinity] [2.2.1-9] (Referred to as "Utsu Catalog (1982 b)" hereinafter)
(v) Jinshin Geppo [Earthquake Monthly]

Figure 2.2.1-1. Isolines of number of severe or higher earthquakes in history and average recurrence years at different places in Japan [2.2.1-1].

Figure 2.2.1-2. Isolines of number of violent or higher earthquakes in the history and average recurrence years at different places in Japan [2.2.1-1].

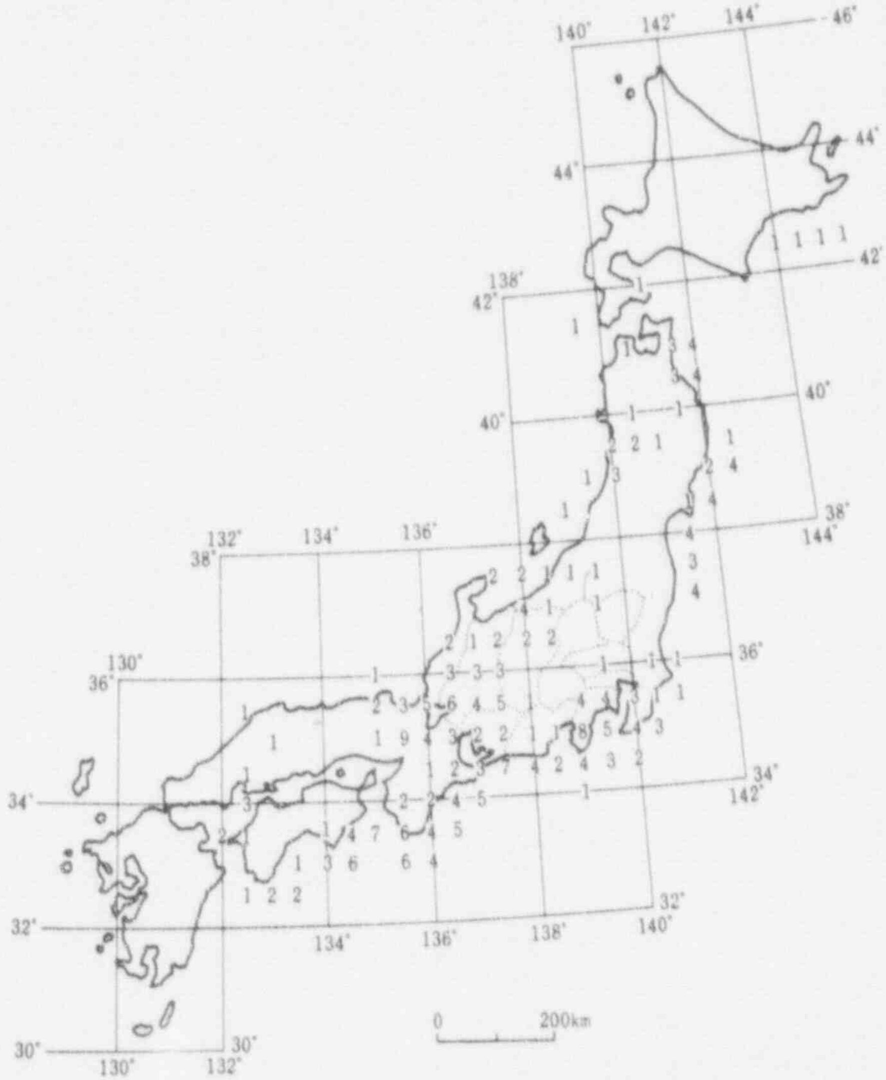


Figure 2.2.1-3. Numbers of catastrophic or higher earthquakes in the history at different places in Japan.

Among them, "Usami Catalog (1979)," "Utsu Catalog (1982 b)," and "Jishin Geppo" are highly reliable catalogs which include updated research results and are frequently used.

The relationship among the catalogs is shown in Figure 2.2.1-4. In the following, summaries of these catalogs will be presented.

(i) "Rika Nenpyo"

"Rika Nenpyo" is published annually. Its "Chronicle of earthquakes with damage in Japan and vicinity" lists the 433 earthquakes with damage which took place in the period from the start of written history to September, 1984. The minor earthquakes after 1884 (Meiji 17), however, are omitted.

For the magnitudes and focal locations of the earthquakes that took place before 1925, the data provided by Kawasumi are adopted. However, some data have been amended. For the earthquakes that took place in 1926 or later, the values of the Meteorological Agency are adopted. In the 1986 edition, the data were used which were made before the Meteorological Agency re-evaluated in the publication "Jishin Geppo, Appendix 6, Amended, Table of major earthquakes in Japan and vicinity (1926-1960)": published in 1982 [2.2.1-10].

(ii) "Shiryo Nippon Higai Jishin Soran"

Usami summarized earthquake damage on the basis of the various earthquake data for earthquakes from the start of written history to 1975.

For the magnitude and epicentral locations of earthquakes before 1926, the data listed in "Rika Nenpyo" are adopted. For earthquakes after 1926, the data provided by the Meteorological Agency are used. When the author prefers an amendment, the value is denoted.

With said amendment opinions taken into consideration and with data in "Shinsyu Nippon Jishin Shiryo," etc., added, a "Shinpen Nippon Higai Jishin Soran" [2.2.1-11] was published.

(iii) Usami Catalog (1979)

This catalog collects 61[illegible] earthquakes with damage in the period from the start of written history to August, 1975.

As far as the magnitudes and focal locations are concerned, for the earthquakes before 1884, the data (with amended opinions) in "Data--Encyclopedia of Earthquakes with Damage in Japan" are adopted; for the earthquakes in the period from 1885 to 1925, the data in "Utsu Catalog (1979)" [2.2.1-12] are adopted; and for the earthquakes after 1926, the data provided by the Meteorological Agency are adopted. In addition, the author made certain amendments for these data (see Figure 2.2.1-5).

Also, there are "Waga kuni ni okeru higai jishin no hyo (Amended edition)" [2.2.1-13] based on seismology, and "Kougakuteki jishindo settei no tameno Nippon Higai Jishin Ichiranpyo" [2.2.1-14] which was compiled to facilitate utilization of the data for engineering purposes.

(iv) "Utsu Catalog (1982 b)"

This catalog collects earthquakes with magnitudes over 6.0 and earthquakes that caused damage in Japan (including those with magnitude lower than 6.0), which took place in Japan and its vicinity in the 96 years from 1885 to 1980.

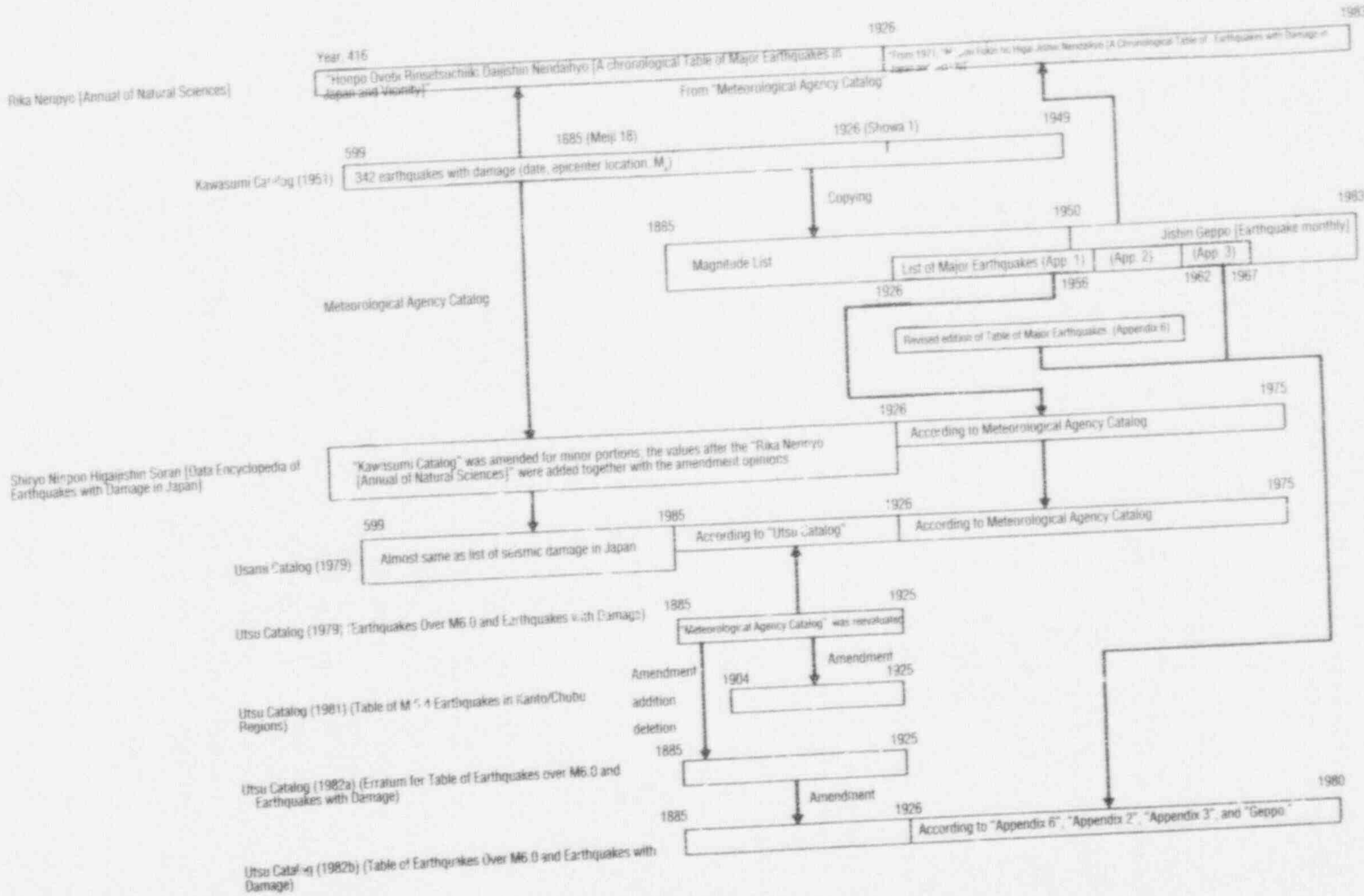


Figure 2.2.1-4. Diagram of relationship among earthquake catalogs.

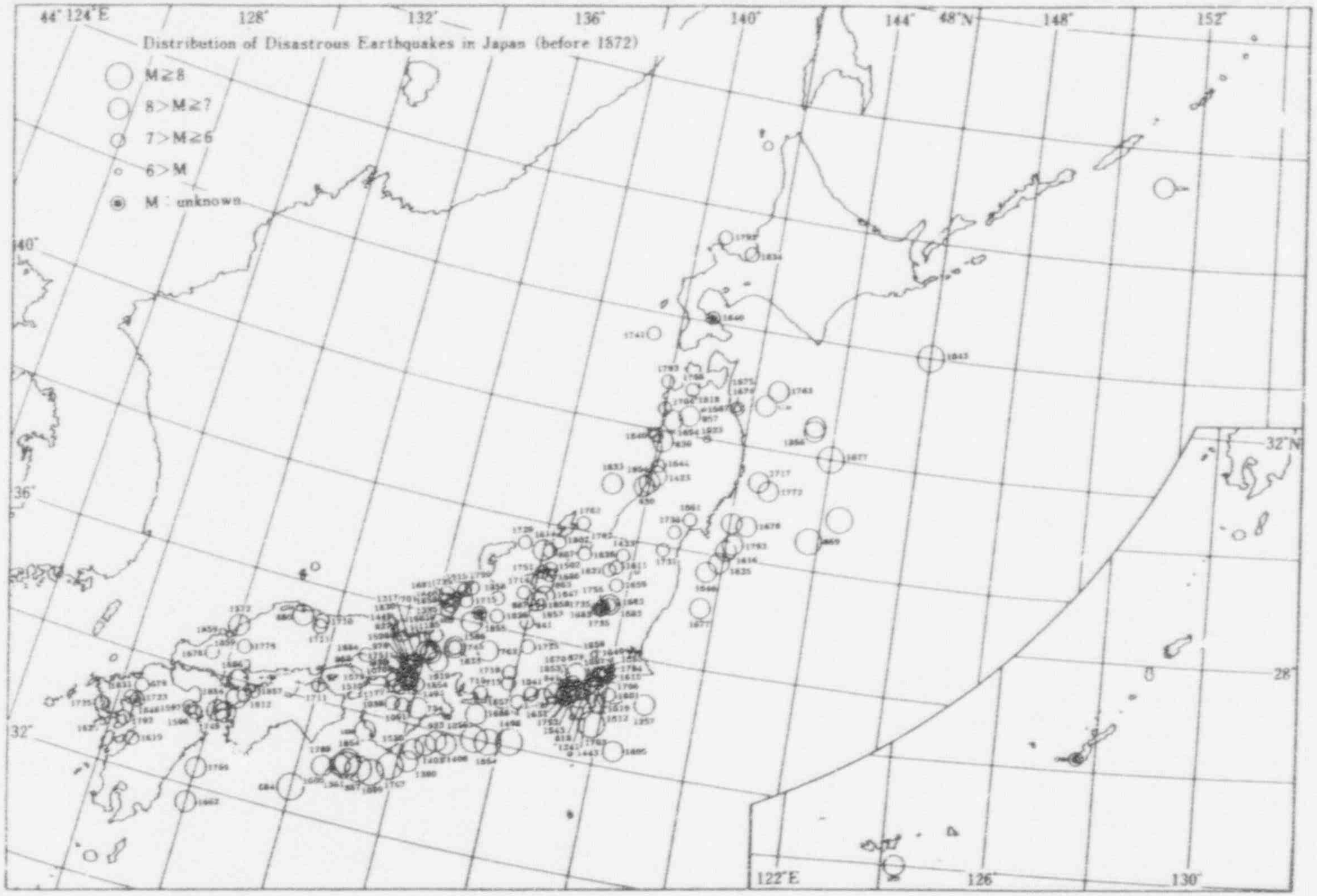


Figure 2.2.1-5. Distribution of disastrous earthquakes in Japan (before 1872).

As far as the magnitudes and epicentral location are concerned, for the earthquakes in the period from 1885 to 1925, the author made amendments of the catalog which was previously published by him ("Utsu Catalog (1979)"); for the earthquakes after 1926, the data of the Meteorological Agency are adopted in principle. Among them, for the earthquakes in the period of 1926-1960, the data published in "Jishin Geppo, Bessatsu 6, published in 1982," which include the source characteristics redetermined by the Meteorological Agency, are reflected.

(v) "Jishin Geppo" (Earthquake Monthly)

This is an earthquake catalog published by the Japan Meteorlogical Agency of the earthquakes taking place in each month. The determination of the magnitudes and epicentral locations for the earthquakes in the period of 1926-1960 was performed by manual operation; and for the earthquakes after 1961, computer processing was performed. But, for the earthquakes in the period of 1926-1960, computer processing was performed to make a redetermination of the epicentral element and in 1982, "Earthquake Monthly, Appendix 6" was published

b. Magnitudes

There are many definitions of magnitudes which represent the scales of the earthquakes. The magnitude scale widely used in Japan is the magnitude M of the Meteorological Agency [2.2.1-15]. For earthquakes not recorded on a seismograph, the equivalent magnitude evaluated from Kawasumi's magnitude M_k [2.2.1-16] is usually used. In some cases, however, the magnitude is estimated from the size of the enclosed area of isoseismals.

(a) Magnitude M of the Meteorological Agency

For earthquakes shallower than 60 km, the formula of Tsuboi (1954) is used; for earthquakes deeper than 60 km, the formula of Katsumata (1964) is used.

$$M = \log A + 1.73 \log \Delta - 0.83 \qquad \text{(Tsuboi)}$$

A: Maximum seismic motion amplitude (μ) (value combined from two horizontal components)
Δ: Epicentral distance (km)

$$M = \log A + K(\Delta, h) \qquad \text{(Katsumata)}$$

K(Δ, h): A function of epicentral distance and focal depth; it is derived by making M equal to what is derived from Tsuboi's formula for h = 25 km.

(b) Kawasumi's magnitude M_k

Kawasumi expressed the relation between the intensity and magnitude for the shallow earthquakes in Japan as follows, with Δ (km) indicating the hypocentral distance:

$$\left.\begin{aligned} I &= M_k + 2\ln(100/\Delta) - 0.00183(\Delta - 100) && \Delta \ge 100 \text{ km} \\ I &= M_k + 2\log(r_0/r) - 0.01668(r - r_0) && \Delta \le 100 \text{ km} \end{aligned}\right\}$$

where r is the focal distance (km), r_0 is r at Δ = 100 km, and $M = 0.5\ M_k + 4.85$. This formula is used to determine magnitude from the ancient earthquake data, and also is used to determine the level of seismic risk at various regions.

(2) Earthquake hazard history

According to the Seismic Intensity Scale of the Japan Meteorological Agency [2.2.1-6] (Table 2.2.1-1), seismic intensity V is defined as "earthquake with a degree of damage as follows: cracks are developed in walls; tombstones and stone lanterns fall; chimneys, stone walls, etc., are damaged." With a seismic intensity V, the earthquake starts to have damage to ordinary houses. On the basis of the highly reliable earthquake catalogs, magnitude-epicentral distance diagram (M-Δ diagram) can be drawn to show the relationship among magnitude (M), epicentral distance (Δ) and seismic intensity. From this diagram, the earthquakes with seismic intensities equal to or greater than V on the site and its vicinity can be selected. For these earthquakes on the site [and vicinity] with seismic intensities equal to or greater than V, survey is performed of the magnitude, epicenter location, focal depth, aftershock area, etc. The damage state is surveyed in detail on the basis of various historical earthquake data and articles/reports on earthquakes. In addition, it is desirable that a survey be made of the relation between the damage state of earthquake and the topography of ground.

Figure 2.2.1-6 shows an example of the M-Δ diagram determined based on the source characteristics, which were determined using the aforementioned catalogs.

The following formulas [2.2.1-7] are used to classify the seismic intensities:

$$\log r_{iv} = 0.41M - 0.75 \quad (2.2.1\text{-}4)$$

$$\log r_{v} = 0.5M - 1.85 \quad (2.2.1\text{-}5)$$

$$\log r_{vi} = 0.68M - 3.58 \quad (2.2.1\text{-}6)$$

where r_i is the radius (km) of the assumed circular region with seismic intensity over i. In Figure 2.2.1-6, r_i is denoted as Δ.

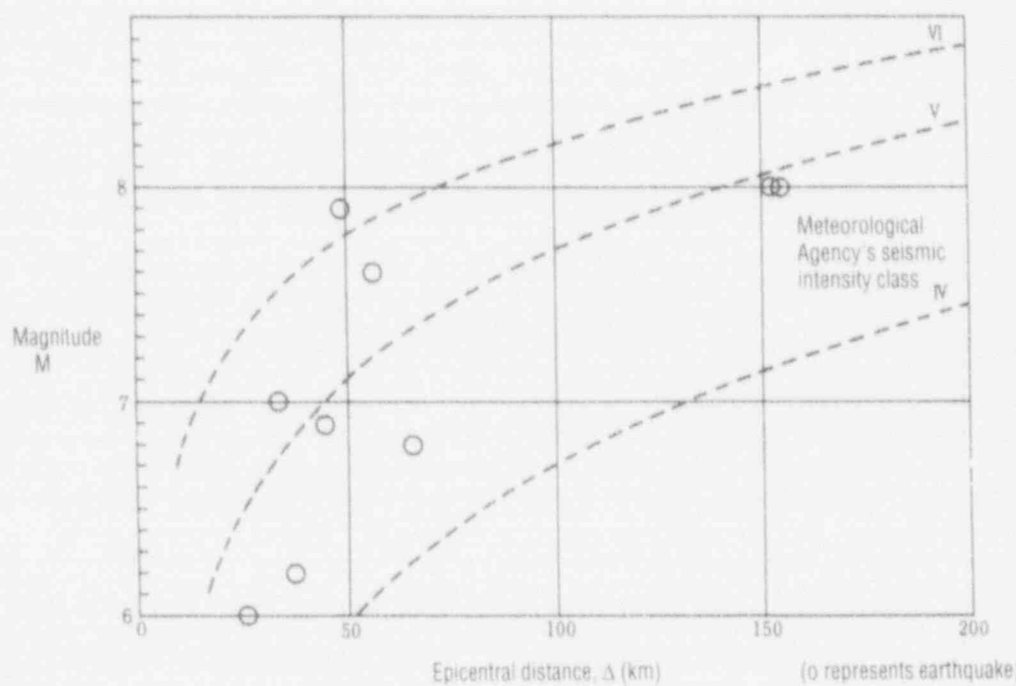


Figure 2.2.1-6. Magnitude vs. epicentral distance of earthquakes taking place in the vicinity of site (example).

As far as the seismic intensity is concerned, different countries have different methods. In Japan, the Meteorological Agency's method of classifying the seismic intensity is usually adopted. In this method, there are seismic classes ranging from 0 (insensible) to VII (catastrophic earthquake). The judgment is made on the basis of human sense, damage degree of wood houses, natural hazard phenomenon, etc. (see Table 2.2.1-1).

When the seismic intensity of the historical earthquake is to be estimated, comparison is made with the Meteorological Agency Seismic intensity Class Table. Also, reference is made with respect to "Table of Explanation of Seismic intensity Classes of Earthquakes" by Tokyo Metropolis [2.2.1-17], which has a more detailed description.

The items used for judgment in this table of explanation include the following 8 items: effects on humans, buildings, affiliated structures, indoor objects, fire utilities, transportation means, outdoor structures, and others. When the seismic intensity of the historical earthquake is estimated, the background of that era, the social status at that time, etc., should be taken into consideration appropriately.

For a specific region, there may exist seismic gaps in historical earthquakes. In this case, survey is made of earthquakes in the vicinity.

(3) Expected intensity of seismic motion

Several methods have been proposed to derive the statistically expected intensity of the seismic motion. Among them, the Kawasumi map and the Kanai map are frequently cited works for earthquakes that have taken place since the start of written history in Japan.

Figure 2.2.1-7 shows the expected acceleration values derived for standard ground by Kawasumi [2.2.1-1]. Figure 2.2.1-8 shows the expected velocity values at the bedrock derived by Kanai [2.2.1-18].

Since these values differ depending on the earthquake magnitude, re-evaluation of epicentral locations, range of the studied earthquake, and the survey period, the statistically expected values may be effectively calculated using the following methods.

- Velocity is calculated using Kanei's empirical formula. (See Equation (2.3.1-4) to be presented later.)
- The statistically expected velocity value is calculated using the following formula.

$$\frac{y}{Y} \sum_{v}^{\infty} N(v) = 1 \tag{2.2.1-7}$$

where, V: expected maximum velocity amplitude in y years
y: expected years
Y: statistical years
N(v): frequency spectrum of velocity amplitude v

From the obtained statistical expected values, the seismic activity on the site can be evaluated. (See Figure 2.2.1-9.)

(4) Past earthquakes that should be taken into consideration

On the basis of the aforementioned evaluations, the earthquakes which once took place and may occur again with influence on the site and vicinity should be selected according to the historical data, and are considered as the past earthquakes in determining the maximum design earthquake.

Table 2.2.1-1. Meteorological Agency's seismic intensity scale [2.2.1-6].

The Meteorological Agency Seismic Intensity Scale and Reference Items (1978) is adopted for determining seismic intensity in Japan. Other countries adopt other standards of seismic intensity classes. The numbers in the explanation column refer to the acceleration of the earth movement in units of Gal (cm/s^2). These acceleration values are included in the table for reference, although they are not formal seismic classes.

Meteorological Agency's seismic intensity class		Reference items
Scale	Explanation	
0	No feeling. Shocks too weak to cause human feeling, registered only be seismographs. (<0.8)	Suspended object is found to sway a little; crack sound can be heard; however, human body does not sense the sway, and it is insensible.
I	Slight. Extremely feeble shocks only felt by persons at rest or by those who are very sensitive to earthquakes. (0.8–2.5)	When [people are] at rest, a little sway can be felt, but the time is not long. Usually, standing people cannot feel it.
II	Light. Shocks felt by most persons, slight shaking of doors and Japanese latticed sliding doors (shoji). (2.5–8.0)	Suspended object can be seen moving. Although standing people can feel a small sway, walking people almost do not feel it. Sleeping people may be wakened.
III	Weak. Slight shaking of houses and buildings, rattling of doors and Japanese latticed sliding doors (shoji). (8.0–25)	People are alarmed, sleeping people are wakened, yet nobody escapes to outside as there is no horrible feel. Many people outdoors feel it, but some walking people may not feel it.
IV	Strong. Strong shaking of houses and buildings, overturning of unstable objects, spilling of liquids out of vessels, felt by walking people and many people rush outdoors. (25–80)	Sleeping people are wakened with a horrible feel. Electric light poles and other poles shake significantly. Roof tiles on conventional houses shift in position. However, there is as yet no damage to the houses. People have a slight dizzy feeling.
V	Severe. Cracks in the walls, overturning of gravestones, stone lanterns, etc., damage of chimneys and stone-fences. (80–250)	It is difficult to stand. Conventional houses begin to be damaged slightly. Weak ground cracks or sinks. Furniture not seated well falls.
VI	Violent. Demolition of houses by less than 30% in total number, land slips, fissures in the ground. Most people cannot stand. (250–400)	Walking is difficult. People can only crawl to move.
VII	Catastrophic. Demolition of houses by more than 30%, intense landslips, large fissures in the ground and faults. (>400)	

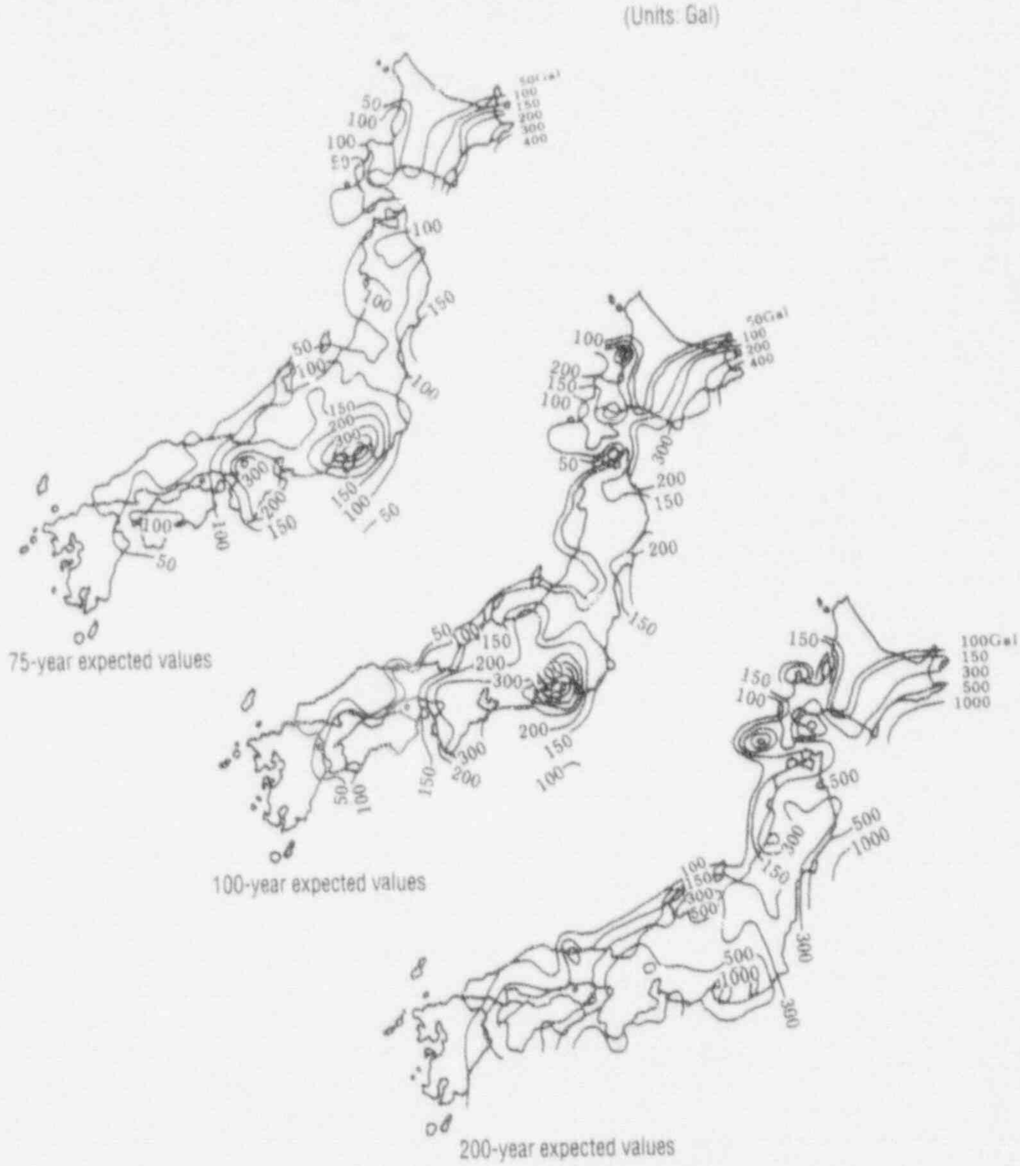


Figure 2.2.1-7. Kawasumi's expected acceleration diagram [2.2.1.1-1].

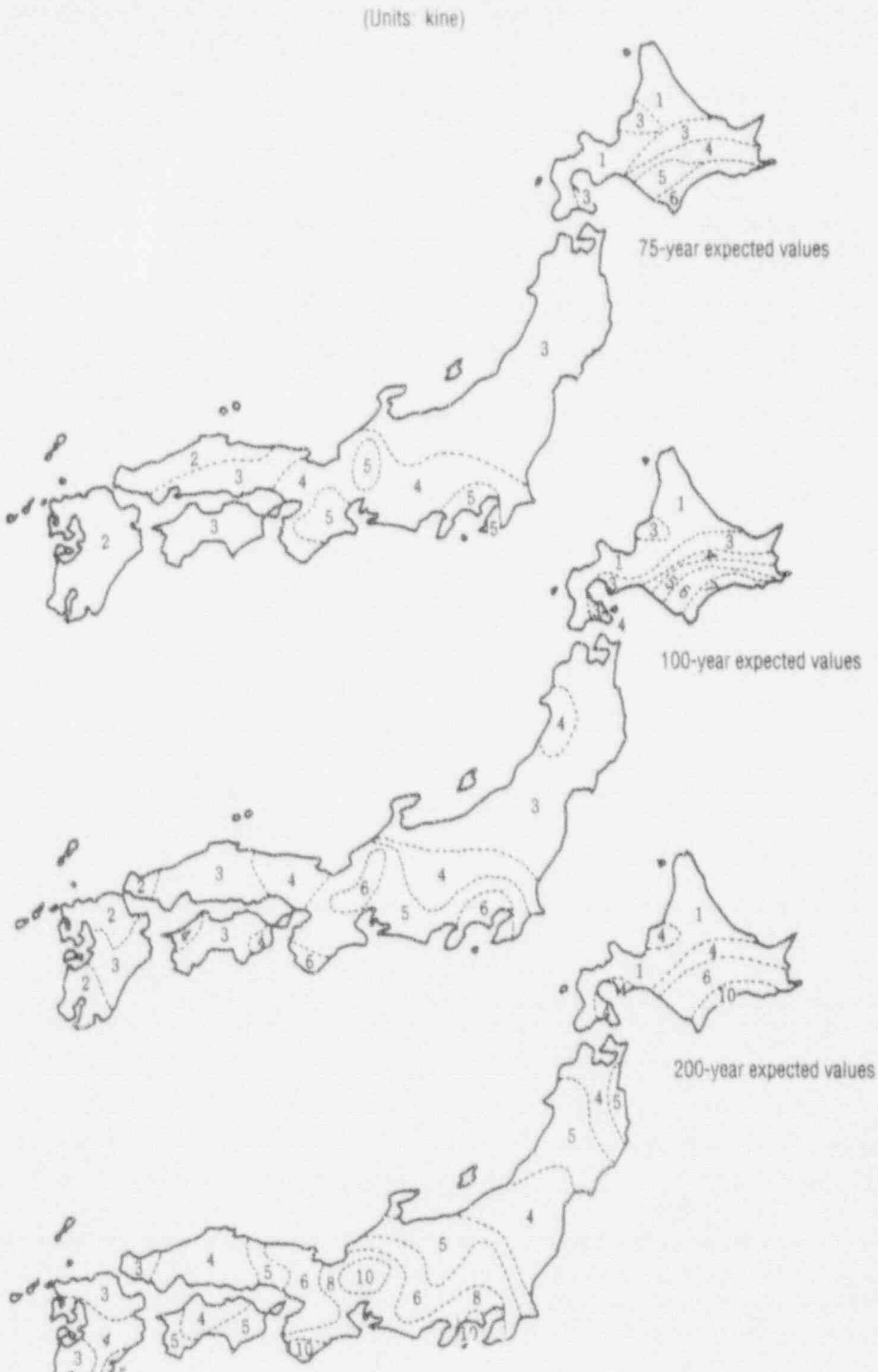


Figure 2.2.1-8. Kanai's expected velocity diagram.

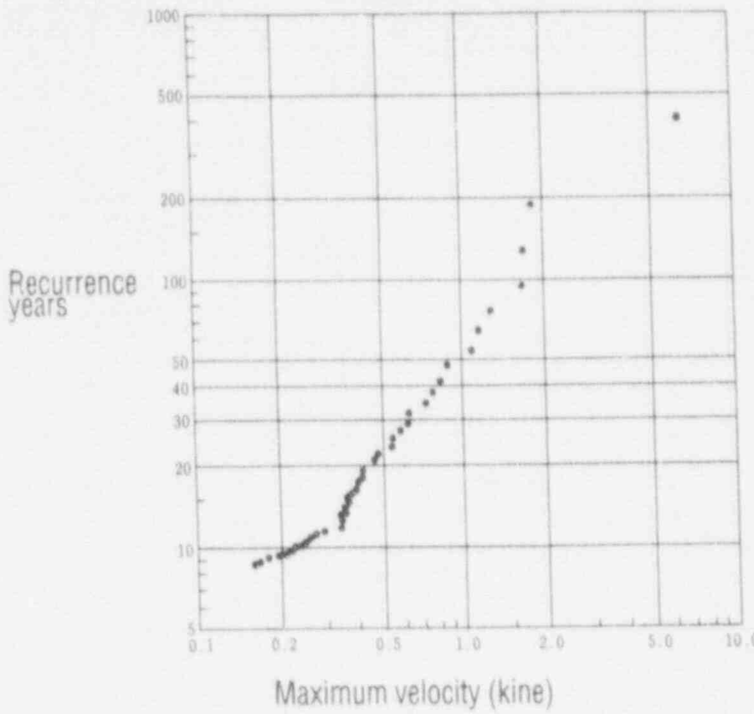


Figure 2.2.1-9. Expected velocity on the sit* (example).

2.2.2 Active faults

(1) Active faults

A plane in bedrock on its two sides slipping from each other is called a fault plane (or simply, fault).

As shown in Figure 2.2.2-1, the slip due to the fault movement is usually represented by a displacement vector on the fault plane. Depending on the relation between the direction of the vector and the ground surface (horizontal plane), faults can be divided into dip-slip faults and lateral strike-slip faults, which can be further divided as follows:

Dip-slip faults
- Normal fault
  - Upper bedrock above the fault plane (upper block) slips downward along the relative dip direction
- Reverse fault
  - Upper bedrock above the fault plane slips upward along the relative dip direction.

Lateral strike-slip faults
- Fault with right lateral strike-slip component
  - The bedrock on the far side of the fault plane slips to the right with respect to the bedrock on the closer side.
- Fault with left lateral strike-slip component
  - The bedrock on the far side of the fault plane slips to the left with respect to the bedrock on the closer side.

The actual fault movement is usually a mixture of dip-slip and the lateral strike slip.

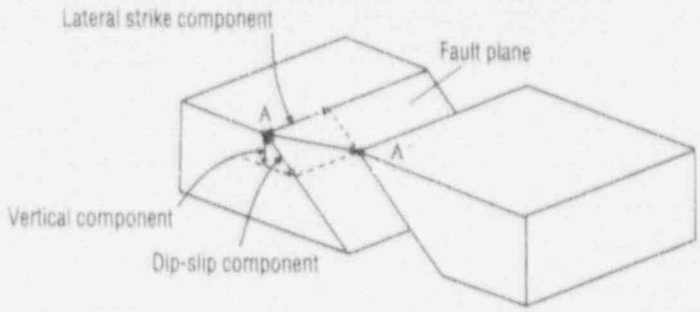


Figure 2.2.2-1. Schematic diagram of fault movement.

The fault activity may change in time. There are some faults which have been in repeated activity and will remain active in the future. There are also some faults which have stopped activity.

The so-called active faults refer to the faults which have been active in the Quaternary period (started about 1.70 million years ago [2.2.1-6]) and are expected to remain active in the future.

There are several publications which summarize the distributions of the active faults in Japan, such as "map and Catalog of active faults in Japan" [2.2.2-1], "Map of active faults in Japan" [2.2.2-2], and "Active faults in Japan--Map and data" [2.2.2-3]. In the following, their contents will be introduced briefly.

a. Map of active faults distribution in Japan (1976)

This map shows the distribution of active faults in Japan except Hokkaido. It provides the catalog of active faults together with the references.

b. Map of active faults in Japan (1978)

Geological Survey Institute summarizes the active faults in the entire land of Japan on the basis of the references published. With a prescribed standard, this map divides the active faults into seismic faults, active faults, and suspected active faults, and represents them respectively.

c. Active faults in Japan--map and data (1980)

This book was prepared by Active Fault Research Institute which was organized to perform an overall survey of the active faults in Japan according to a 3-year plan starting from 1976. This book defines the active faults on the land and sea bottom in Japan according to the same criteria. For each active fault, it summarizes the related properties for reference.

For the active faults on land, the 1/40,000 aerial photograph was taken as the basic data, with reference made to topographical maps, geological maps and references. If needed, on-site survey was performed, with results added for evaluation.

As far as the active faults on the seabed are concerned, the basic data are the records and seabed topographical maps prepared by the Hydrographic Division, Maritime Safety Agency, with the aid of continuous sonic wave survey in order to prepare "Basic map of the sea." Also, other seabed geological structural maps and references are taken as references.

Figure 2.2.2-2 summarizes the major active faults in Japan and its peripheral sea area.

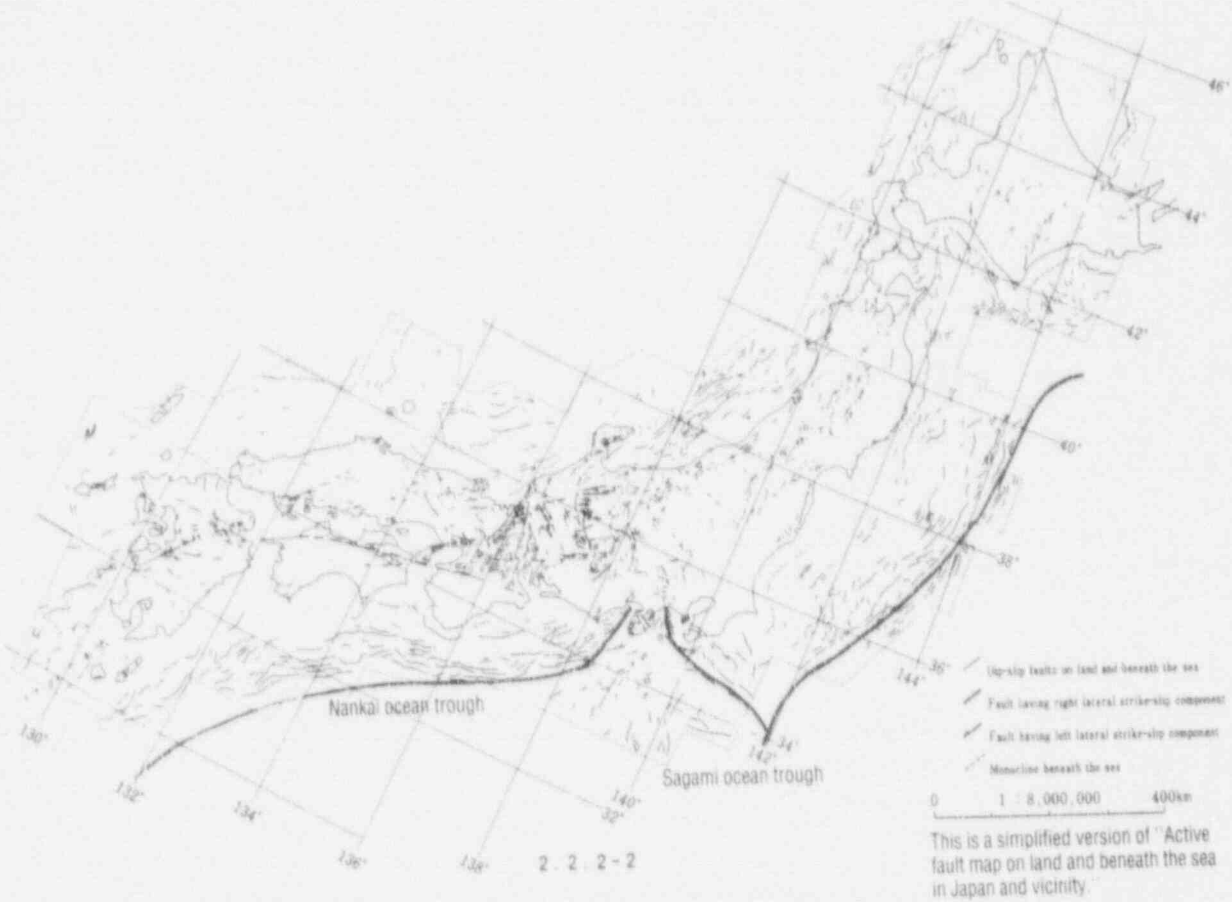


Figure 2.2.2-2. Major active faults on land and beneath the sea in Japan and vicinity.

In addition, new survey results are summarized in "Active Structural Map (1/500,000)" [2.2.2-4] (Geological Survey Institute), "Seabed Geological Structural Map (1/50,000)" (Maritime Safety Agency) and other maps. These maps and references which make detailed description of each active fault can be used effectively for surveying the specific region.

The active fault distribution maps cited here are all maps which summarize the active faults as well as structures that might be considered as active faults in Japan. Also, more detailed survey/investigation has been performed for each active fault, and the results have been published as references.

As can be seen from Figure 2.2.2-2 [2.2.2-3], for the active faults in Japan, the distribution density, distribution pattern, strike, length, fault type, etc., are different in different regions. The degree of activity also depends significantly on the region.

Due to the difference of active faults in different regions, Japan can be divided into several regions of active faults for research purposes. Figures 2.2.2-3 [2.2.2-5] and 2.2.2-4 [2.2.2-3] are examples of this classification.

The classification of active fault regions is closely related to the rock that forms the earth's crust, the stress state in the earth's crust and the seismic mechanism of the earthquake.

The activity of the active faults can be classified according to the value of the average dislocation speed as shown in Table 2.2.2-1. In this case, the average dislocation speed is derived by dividing the dislocation of an active fault by the years since formation of the dislocation.

(2) Active faults and past earthquakes

The relation between active faults and past earthquakes is most clearly displayed by the fault appeared on the ground surface during the earthquake. This fault is called a seismic fault. Even when no apparent faults are found on the ground surface, a fault which caused the earthquake can be identified under the ground. These faults, together with the above seismic faults, are sometimes called as source faults [2.2.2-7].

Table 2.2.2-2 [2.2.2-8] lists examples of earthquakes that took place in Japan accompanied with seismic faults.

The seismic faults are usually appear along the existing active faults, with their dislocation directions in agreement with those of the active faults as indicated by the topography.

However, in many cases, the relation between the historical earthquakes and the active faults is not clear. Hence, in order to relate the past earthquakes to the peripheral active faults, it is necessary to perform a detailed survey of the epicenter locations of past earthquakes, time of occurrence, scales and properties, as well as sizes and activities of the active faults, and to make a detailed evaluation of the relationship between them.

(3) Active faults and microtremors

Among the active faults, those for which the present activity is found significant by observation on microtremors are evaluated as active faults with high activity degrees in some cases. However, it is believed that direct correlation between occurrence of microtremors and the present activity of the active faults as a whole exists only in limited cases. Hence, when evaluation is to be made of the activity, it is necessary to perform a detailed survey/study in time and space of the geological data, such as occurrence status of micro-earthquakes, properties of active faults, etc.

Features of active fault regions

Active fault region		Fault density	Earthquake activity	Type of fault
I	Inner side of northeast Japan	Medium	Medium	Reverse fault
II	Inner belt of central Japan	High	High	Lateral strike-slip fault + reverse fault
III	Inner belt of southwest Japan (Chugoku, Kitakyushu)	Low	Low	Lateral strike-slip fault + normal fault
IV	Outer belt	Very low	Ultra low	
V	Minamikanto, Izu	High	High	Lateral strike-slip fault (+ reverse fault)

Figure 2.2.2-3. Example 1 of classification of active fault regions in Japan [2.2.2-5].

Active fault regions and characteristics of faults in them

Class		Fine class		Density	Length of major faults*	Activity of major faults	Typical fault type**	Note
I	Main portion of Hokkaido	a	Inner belt of Hokkaido main portion	Small	Small	C	Reverse	
		b	Outer belt of Hokkaido main portion	Small	Medium	B	Reverse	
II	Inner belt of northeast Japan	a	Continental slope of inner belt of northeast Japan	Large	Large	A?	Reverse	Beneath sea
		b	Land of inner belt of northeast Japan	Medium	Small	B	Reverse	Volcano region
III	Outer belt of Tohoku region			Very small	Medium	B	Reverse/lateral	
IV	Pacific Ocean slope of northeast Japan	a	South coastal region of Hokkaido	Large	Large	A?	Reverse/lateral?	Beneath sea
		b	Sanriku, Joban, Kashima offing	Large	Large	A?	Reverse	Beneath sea
		c	Sagami Ocean trough and vicintiy	Large	Large	A	Reverse/lateral	Mainly beneath sea
V	Izu-Ogasawara arc tip portion	a	Kanto Mountain district and vicinity	Medium	Small	B	Reverse/lateral	
		b	Izu Peninsular and vicinity	Large	Small	B	Lateral	Volcano region
BF	Fossamaguna west edge belt			Large	Small	A	Lateral/reverse	
VI	East part of southwest Japan inner belt	a	Noto Peninsula and vicinity	Small	Small	B·C	Reverse	Land and seabed
		b	Oki Ocean Trough and vicinity	Medium	Medium	B?	Reverse?	Beneath sea
		c	Chubu Mountain district	Large	Large	A	Lateral/reverse	
		BT	Tsuruga Bay–Ise Bay belt	Large	Medium	A·B	Lateral/reverse	
		d	Kinki triangular region	Large	Medium	B·A	Reverse/lateral	
		e	Kinki northwest region	Medium	Medium	B	Lateral/reverse	
VII	West part of southwest Japan inner belt	a	Chugoku, Setouchi, Kitakyushu	Small	Small	B·C	Lateral/reverse	
		b	Central Kyushu volcano region	Large	Small	B	Normal	Volcano region
BM	Median tectonic line belt			Large	Large	A	Lateral	
VIII	Southwest Japan outer belt			Very small	Small	B·C	Reverse/lateral	
IX	Pacific Ocean slope of southwest Japan			Large	Large	AA	Reverse/lateral	Beneath sea
X	North part of Okinawa Ocean Trough			Medium	Large	B?	Normal	Beneath sea
	Southwest islands			Large-very small (large diff. among regions)	Small	B·C	Normal	Land only
	Izu/Ogasawara islands			Small	Small	C	Reverse?	Land only

*Small: < 20 km; medium: 20-50 km; large: > 50 km. Original data are different for seabed and land. Lengths here are based on the data of a 1/2,000,000 map.

**Reverse: reverse fault; normal: normal fault; lateral: lateral strike-slip fault.

Figure 2.2.2-4. Example 2 of classification of active fault regions in Japan [2.2.2-3].

Table 2.2.2-1. Classification of activity degree of [illegible] faults using average dislocation speed [2.2.2-6].

Class	Average dislocation speed S (mm/year)
Class A	$1 \leq S$
Class B	$0.1 \leq S < 1$
Class C	$S < 0.1$

Table 2.2.2-2. Table of major earthquakes accompanied with seismic faults [2.2.2.-8].

Year/Month/Date	Earthquake	*M*	Seismic fault or location
1847. 5. 8	Zenkoji earthquake	7.4	Zenkoji fault, etc.
1891.10. 28	Nobi earthquake	8.0	Neodani fault, etc.
1894.10. 22	Shonai earthquake	7.0	Yadamezawa fault, etc.
1896. 8. 31	Rikuu earthquake	7.2	Senya fault, etc.
1918.11. 11	Omachi earthquake	6.1	Terakaito fault, etc.
1923. 9. 1	Great Kanto earthquake	7.9	Shimoura fault, etc.
1925. 5. 23	Tajima earthquake	6.8	Tai fault
1927. 3. 7	Kitatango earthquake	7.3	Gomura fault, etc.
1930.11. 26	Kitaizu earthquake	7.3	Tanna fault, etc.
1938. 5. 29	Kussharo earthquake	6.1	Kussharo fault, etc.
1943. 9. 10	Tottori earthquake	7.2	Sikano fault, etc.
1945. 1. 13	Mikawa earthquake	6.8	Fukouzu fault, etc.
1948. 6. 28	Fukui earthquake	7.1	Fukui seismic fault
1964. 6. 16	Niigata earthquake	7.5	Murakamioki seabed
1965 ~ 1968	Matsusiro earthquake swarm	~5.4	Matsushiro seismic fault
1974. 5. 9	Izu Peninsula offing earthquake	6.9	Irousaki fault, etc.
1978. 1. 14	Earthquake in sea near Izu Oshima Island	7.0	Inatori Omaezaki fault, etc.

(4) Earthquakes caused by active faults that should be taken into consideration.

The characteristics of active faults (e.g. the magnitude and frequency of earthquakes) differ considerably from fault to fault, and it is not practical to take all the active faults into consideration on an equal basis. For example, it is not necessarily proper from an engineering viewpoint to expect that the active fault with a very small probability of generating a strong earthquake, would generate another earthquake

Therefore, when the active faults are considered, their activities shall be evaluated first and they will be taken into account according to the degree of their activities.

Earthquakes which could be generated at active faults shall be classified as earthquakes producing the basic earthquake ground motions S1 or S2 depending on the activities of the faults. The following guidelines will be the bases for the evaluation of active faults.

a. The following shall be considered in the evaluation of sources generating the basic earthquake ground motion S1:

{1} Faults with a historical record of earthquakes.
{2} Class A faults having clear evidence of movement within the past 10,000 years, or whose return period is less than 10,000 years.
{3} Faults whose activity is considered significant based on the observation of microtremors.

b. The following items shall be considered in the evaluation of sources generating the basic earthquake ground motion S2:

{1} Faults belonging to Class A except those in above a.{2}
{2} Class B and C faults having clear evidence of movement within the past 50,000 years, or whose return period is less than 50,000 years.

For the active faults on the land of Japan, it is believed that the creep dislocation is small. Hence, the following relationship exists between the recurrence period R (years) of earthquake and the average dislocation speed of the fault S (mm/year) [2.2.2-6]:

$$R = D/(S \times 10^{-3}) \tag{2.2.2-1}$$

where D(m) represents the fault displacement amount in an earthquake; it is related to the earthquake magnitude by the following equation:

$$\log D = 0.6M - 4.0 \tag{2.2.2-2}$$

From Equations (2.2.2-1) and (2.2.2-2), the following relationship is derived which can be used to calculate the earthquake recurrence period R (years):

$$R = 10^{(0.6M-1)}/S \tag{2.2.2-3}$$

In addition, the following relationship [2.2.2-6] exists between length L (km) of seismic fault on the land of Japan and magnitude M of the earthquake:

$$\log L = 0.6M - 2.9 \quad (2.2.2\text{-}4)$$

Although it is possible to release all of the strain energy of the fault in a single round of earthquake, it is also possible for it to be released in several rounds of earthquakes, with the strain energy of a portion of the fault released in each earthquake. Hence, the earthquake magnitude M obtained from Equation (2.2.2-4) for an active fault with length L (km) is the maximum scale of the earthquake that can be induced by the active fault. When the seismic motion is to be evaluated at a site separated from the active fault by a certain distance, the earthquake scale is derived from the fault size, and the epicentral distance is determined by regarding the center of the fault as the epicenter.

Also, in the case when the source of past earthquakes, such as a seismic fault, is clear, on the basis of sufficient study on the scale and activity of the active fault as well as the scale of the past earthquakes, it is possible to use the corresponding past earthquake to represent the earthquake scale and hypocentral location for the active fault.

The specific survey methods of the active faults are described in detail in "Chapter 3. Geological and ground survey."

2.2.3 Seismo-tectonic structure

(1) Seismo-tectonic structure

The seismo-tectonic structure refers to the geological structure of a region which shares common properties related to generation of earthquakes, i.e., the earthquake scale, focal depth, earthquake generation mechanism and earthquake occurrence frequency.

Japan is made up of several regions different in geological structure and topography. Hence, the nature of the earthquakes in Japan has a strong region-dependency. This region-dependency of earthquakes is believed to be a reflection of the differences in the geological structure and mechanical characteristics of the geological structure of the region.

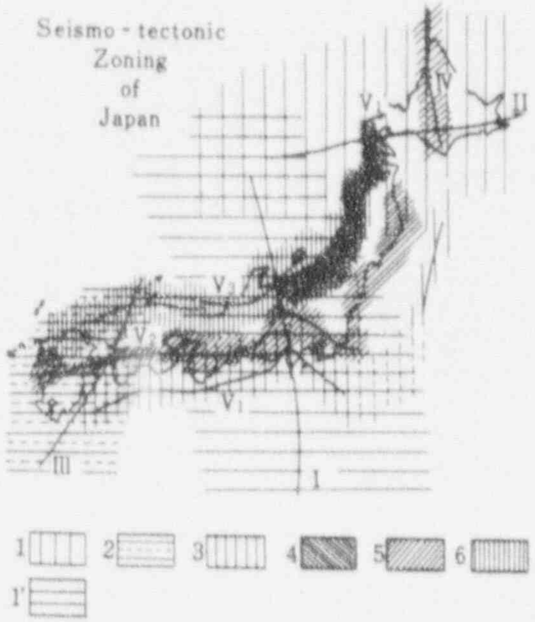
It is pointed out by many authors that the maximum scale of earthquake that can take place in a certain region depends on the geological structure of the region.

The first effort to divide Japan into several zones related to earthquake activity was made by Imamura [2.2.3-1] who discovered the earthquake belts from the earthquake occurrence maps. The concept is to make a qualitative judgment of the dependence of earthquake activity on the region from the geometrical distribution of the focuses.

It was Miyamura [2.2.3-2] who first made a formal investigation of the earthquake tectonic structure in Japan.

Miyamura formed a 3-D earthquake distribution map for the whole country of Japan and studied the zoning feature of the focal density. Based on analytical results and their relation to the history of development of the structure of crust in the Japan islands and their vicinity, he proposed that Japan be divided into 6 types of earthquake tectonic zones as shown in Figure 2.2.3-1.

Figure 2.2.3-2 illustrates the maximum scale of earthquakes that can take place in each zone, judging from the past earthquakes in Japan [2.2.3-3].



MIYAMURA 1967

For the earthquake zones in Japan, I–V represent the axes of island arcs, and 1–6 represent the earthquake zones. For zones 1–3, the earthquake zone slips downward from the crust bottom to the deep mantle portion along the parallel lines running from south/east to north/west and perpendicular to the respective axes. For zones 4–6, the earthquake zone is limited within the crust.

1: Earthquake zone (depth: 30–700 km) of young island arc most active at present (I, Izu/Ogasawara arc; II, Chishima arc).

2: Earthquake zone (depth: 30–300 km) due to island arc which is somehow aged but is still active at present (III, Ryukyu arc).

3: Earthquake zone (depth: 30–90 km) of Tertiary orogenic zone which is still active at present (V, outer zone of Honshu arc).

4: Earthquake zone (depth: <20 km) of Tertiary orogenic zone which is still slightly active at present (V_1, Uetu strike arc).

5: Earthquake zone (depth: <20 km) of orogenic zone of late Mesozoic era and late Paleozoic era which once saw late igneous activity and still makes slight plate movement at present (IV, Kabahuto/Hidaka arc; V_2, Central zone of Honshu arc).

6: Earthquake zone (depth: <40 km) of orogenic zone of Archaean era or Paleozoic era which was formed as plateau in the regenerated plate movement.

Figure 2.2.3-1. Earthquake zones proposed by Miyamura [2.2.3-3] (Courtesy MIYAMURA 1967).

Figure 2.2.3-2. Maximum scales of earthquakes that can take place in Japan [2.2.3-3].

(2) Earthquake induced by seismo-tectonic structure

For the earthquake that should be considered in relation to the seismo-tectonic structure, based on the zoning system shown in Figure 2.2.3-2, the maximum earthquake scale for the region is assumed, and the focal location is determined from the viewpoint of seismology and geology.

It should be pointed out that Figure 2.2.3-2 only indicates that the maximum scale of earthquake taking place in the region should not exceed the value shown in the figure; it does not indicate that the maximum-scale earthquake can take place anywhere in the region.

Hence, in the case when the extreme design earthquake is considered as related to the earthquake tectonic structure at a certain spot, it is a very important task to determine the epicentral position.

Because the geological structure and past earthquakes are surveyed in detail to clarify their characteristics in Japan, the epicentral position of the maximum possible earthquake may be determined considering active faults and past earthquakes.

Earthquakes taking place in Japan can be roughly classified into earthquakes taking place near the boundary of the plate on the Pacific Ocean side due to slip of the Pacific Ocean Plate and Philippine Sea Plate beneath the Eurasia Plate, and earthquakes taking place within the inland crust.

In the vicinity of the plate boundary, major earthquakes take place repeatedly in the same region with an interval of several tens of years to about 200 years. It is possible to define the maximum possible earthquake as related to the past earthquakes.

On the other hand, in the inland region, the interval of earthquakes taking place in the same region is long, and it is usually more difficult to correlate with the past earthquakes clearly as compared with the case of the plate boundary. However, for the inland region, it is possible to make a detailed survey of the active faults; hence, the maximum possible earthquake that may take place can be determined as related to the active faults.

(3) Shallow-focus earthquake

For the shallow-focus earthquake, although it is desirable that it be determined from the earthquake tectonic structure of the region and the occurrence characteristics of the earthquake, it is usually difficult to determine the earthquake scale and the focal position. Consequently, the shallow-focus earthquake is determined from the viewpoint of the aseismic design of nuclear power facilities that the design is performed to ensure safety even in the case when an earthquake takes place very near the site, instead of from the viewpoint as related to earthquakes that may actually take place.

As a result, as part of the design margin to ensure the seismic safety, for any site, a shallow-focus earthquake with a magnitude of 6.5 and with a hypocentral distance of 10 km is used as one of the extreme design earthquake.

2.3 Basic design earthquake ground motion

As pointed out in the above section, based on the past earthquakes, active faults and seismic geological structure, the maximum design earthquake and the extreme design earthquake are estimated, and then the basic earthquake ground motions S1, S2 can be determined at the rock outcrop surface. In this evaluation, both the near and distant earthquakes are taken into consideration according to the "Regulatory Guide for Aseismic Design." However, as the characteristics of seismic motion depends also on the epicentral distance in addition to the earthquake scale, care should be excercised in determining the ground motions.

2.3.1 Characteristics of earthquake ground motion

Among the various characteristics of seismic motions, the maximum amplitude of seismic motion, frequency characteristics, duration and amplitude envelope time function, etc., are explained in the following.

(1) Maximum amplitude of seismic motion

a. Maximum amplitude of seismic motion

As far as the maximum amplitude of seismic motion (maximum acceleration, maximum velocity, maximum displacement) is concerned, it is believed that the source characteristics and the characteristics of the wave propagation path are the major factors that affect the seismic motion. Various empirical formulas have been proposed as functions of the magnitude and the epicentral distance.

Among them, frequently cited formulas are described as follows:

(a) Kanai's formula [2.3.1-1]

Kanai once observed the seismic motion in the pit of Hitachi Mine at a position 300 m beneath the ground surface. Based on the records, the displacement spectrum at the point 100 km from the hypocenter was derived, and the following formula was found between maximum value $d_{ms}(\mu)$ of the displacement spectrum and its period $T_m(s)$:

$$d_{mu} = 537 T_m^{2.56} \qquad (2.3.1\text{-}1)$$

When the velocity amplitude spectrum was derived, it was found that for periods in the range from 0.05-0.2 s to T_m, the velocity amplitude is nearly constant, and the uniform energy distribution rule exists for the seismic motion at the bedrock. Based on these observations, the following formula was derived:

$$d_0 = T \times 10^{0.61M - 1.73\log X - 1.47} \qquad (2.3.1\text{-}1)$$

This formula is based on the observation records with magnitude of M4.1-5.1 and with focal distance of 40-200 km. The average P-wave velocity at the ground of the observation point is V_P = 5.4 km/s.

Afterwards, based on the records of the Matsushiro earthquake swarm, the following formulas were proposed for acceleration a_0 (Gal), velocity v_0 (kine), and displacement d_0 (cm) at the bedrock:

$$a_0 = \frac{1}{T} \times 10^{0.61M - (1.66 + 3.60/X)\log X + (0.167 - 1.83/X)} \qquad (2.3.1\text{-}3)$$

$$v_0 = 10^{0.61M - (1.66 + 3.60/X)\log X - (0.631 + 1.83/X)} \qquad (2.3.1\text{-}4)$$

$$d_0 = T \times 10^{0.61M - (1.66 + 3.60/X)\log X - (1.430 + 1.83/X)} \qquad (2.3.1\text{-}5)$$

where X is the focal distance (km), M is the magnitude, and T is the period of seismic motion (s).

(b) Okamoto's formula [2.3.1-2]

Based on the earthquake observation records obtained for Kinugawa Hydraulic Power Station, Okamoto proposed the following formula relating recorded maximum acceleration a (Gal), magnitude M, and epicentral distance Δ(km):

$$\log \frac{a}{640} = \left(-0.1036M^2 + 1.7244M - 7.604\right) \times \left(\frac{40 + \Delta}{100}\right) \qquad (2.3.1\text{-}6)$$

The data adopted correspond to an earthquake with magnitude M = 4.3-7.9 and epicentral distance Δ = 43-540 km. For earthquakes with larger M, the epicentral distances are as large as several hundred km. The P-wave velocity at the observation point is in the range of V_p = 3.4-3.6 km/s.

(c) Watabe's formula [2.3.1-3]

Based on the records obtained where ground can be regarded as rock sites (with the S-wave velocity V_S greater than 0.6 km/s), Watabe et al. proposed the following equations using peak acceleration records (74 records) and peak velocity values (numerically integrated):

$$A = 10^{0.440M - 1.38\log X + 1.04} \qquad (2.3.1\text{-}7)$$

$$V = 10^{0.607M - 1.19\log X - 1.40} \qquad (2.3.1\text{-}8)$$

where, A (Gal), V (kine).

The average value of the S-wave velocity weighted by the number of observation records is about 1.1 km/s. For horizontal components with two directions, the average value is used.

b. Estimation of seismic motion intensity from falling tombstones and damage to wood-structure buildings

At present, there are few observation records in the world near epicenters of major earthquakes. Hence, although the empirical formulas in the above section proposed on the base of actually measured results are suitable for the region outside the source region, they often overestimate ground motions in the focal region.

One way to estimate the intensity of seismic motion in the focal region is by using the damage caused by a past earthquake.

A useful method is to estimate the acceleration from falling tombstones and other monoliths (columns) [2.3.1-4] and to estimate the acceleration from the collapse rate of wooden houses.

(2) Frequency characteristics of seismic motion in bedrock

The frequency characteristics of seismic motion are determined by the combined effects of the source characteristics, characteristics of propagation path, characteristics of the local ground near the observation spot, characteristics of seismograph, etc.

In this section, we will discuss the evaluation methods of the frequency characteristics of the seismic motion at the bedrock where the effects of local soil can be excluded.

a. Osaki's method [2.3.1-5] [2.3.1-6]

Based on 84 sets of records of accelerations in Japan and abroad, as well as the data of falling tombstones, Osaki et al. prepared a pseudo-velocity response spectrum as a function of magnitude M and epicentral distance Δ as follows.

(a) The shapes of the pseudo-velocity response spectra (damping: 5%), normalized with respect to a maximum velocity value of 10 kine of seismic motion, is shown in Table 2.3.1-1 and Figure 2.3.1-1.

(b) The maximum velocity value of the seismic motion can be represented by the following formula:

$$V_{max} = 10^{0.61M - P\log X - Q} \qquad (2.3.1\text{-}9)$$

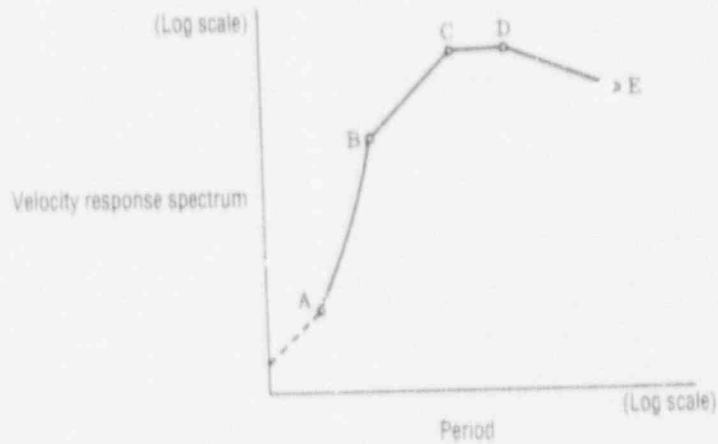
V_{max}: maximum velocity value at rock outcrop (kine)
$P = 1.66 + 3.60/X$
$Q = 0.631 + 1.83/X$
X: focal distance (km) = $(\Delta^2 + D^2)^{1/2}$
Δ: epicentral distance (km)
D: depth of energy releasing center (km) = $10^{0.353M-1.435}$

Table 2.3.1-1. Design response spectrum (damping fac'or = 5%).

Field	Magnitude M	Epicentral Distance Δ (km)	Control Points									
			A		B		C		D		E	
			T_A	S_V	T_B	S_V	T_C	S_V	T_D	S_V	T_E	S_V
Near	8	25	0.02	0.6	0.10	10	0.30	30	0.50	30	2.0	12
	7	10		0.7	0.10	11	0.23	24	0.45	24		7
	6	5		1.2	0.10	17	0.13	21	0.35	21		3
Intermediate	8	120		0.5	0.20	18	0.35	32	1.00	32		26
	7	45		0.5	0.13	11	0.33	28	0.80	28		19
	6	15		0.6	0.10	10	0.25	24	0.60	24		12
Far	8	350		0.5	0.22	26	0.37	44	1.20	44		42
	7	150		0.5	0.14	15	0.35	38	0.90	38		32
	6	60		0.5	0.10	10	0.33	33	0.70	33		20

T : Period (s)
S_V : 10-kine standardized response spectral value (kine)

T: Period (s)
S_V : 10-kine standardized response spectral value (kine)



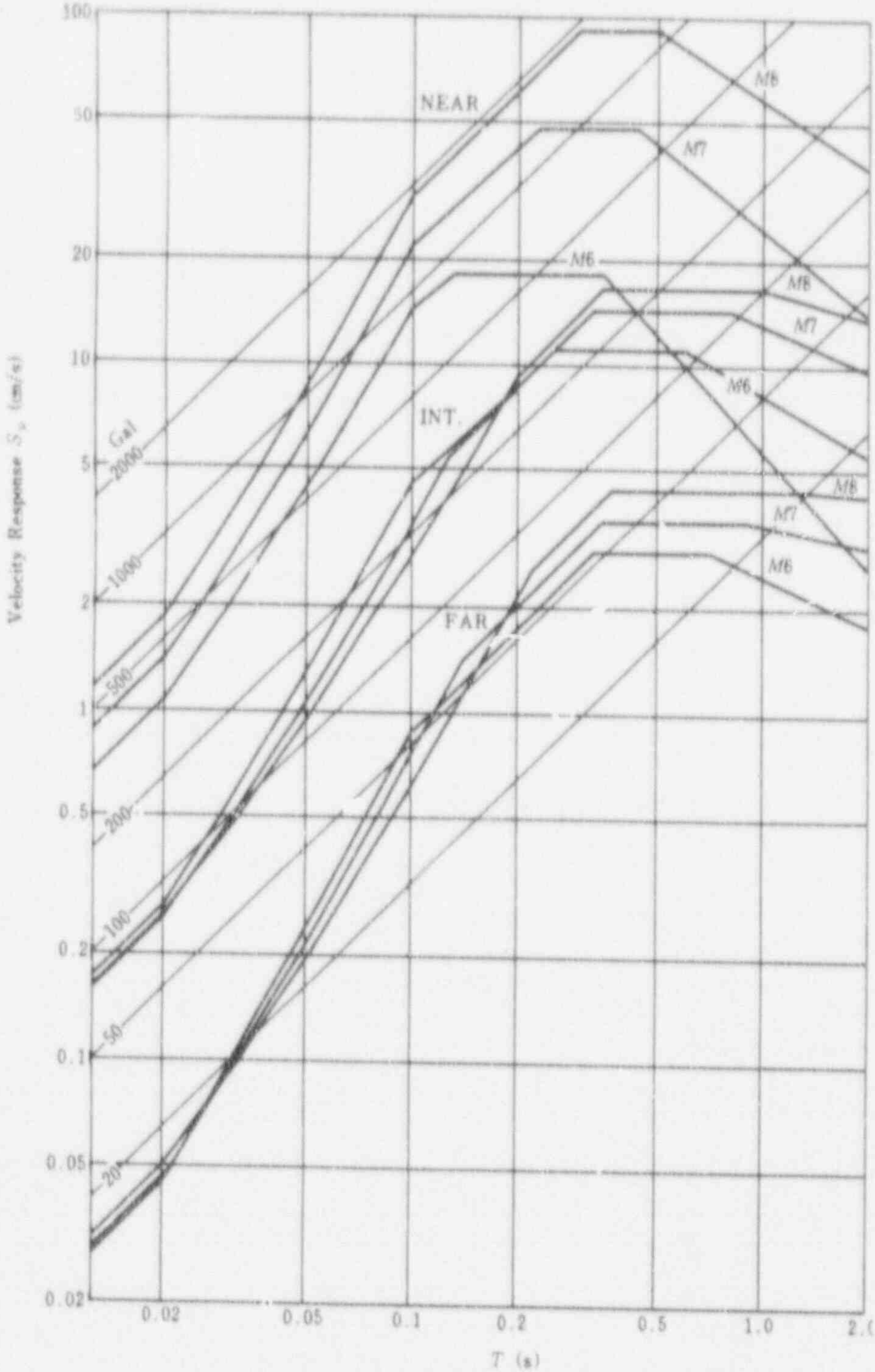


Figure 2.3.1-1. Design response spectra [2.3.1-5].

(c) When damping is other than 5%, the response spectral value should be amended by the following formulas:

$$\eta = 1/\sqrt{1 + 17(h - 0.05)\exp(-2.5T/T_0)} \qquad \text{when } T \geq T_B \tag{2.3.1-10}$$

$$\eta = 1.0 \qquad \text{when } T = T_A \tag{2.3.1-11}$$

h: damping coefficient
T: T_A, T_B, T_C, T_D, T_E in Table 2.3.1-1
T_0: effective duration of seismic motion (s) = $10^{0.31M-1.2}$

(d) When magnitude M and epicentral distance Δ are different from the values listed in Table 2.3.1-1, first, linear interpolation is performed for M; then, for Δ, interpolation is made with the logarithm of Δ.

b. Kobayashi's method [2.3.1-7]

Based on the idea that if the spectrum of seismic motion is averaged for a number of earthquakes, a spectrum similar to the ground amplification characteristics can be obtained, Kobayashi et al. derived the velocity response spectrum at the seismic bedrock (corresponding to an S-wave velocity of around 3.0 km/s) using magnitude M and focal distance X as follows:

(a) The velocity response spectrum of the seismic motion recorded at each observation point on the ground surface is divided by the amplification function of the soil, and the result is considered as the velocity response spectrum (damping: 5%) for the bedrock.
The following empirical formula is derived for the spectrum.

$$\log S_{v_0}(T) = a(T) \cdot M - b(T) \cdot \log X - c(T) \tag{2.3.1-12}$$

$S_{V_0}(T)$: velocity response spectrum (damping factor 5%) (kine)
M: magnitude
X: focal distance (km)
a(T), b(T), c(T): coefficients derived for each bedrock using the least squares method; they are functions of period T(s).

(b) For this empirical formula, coefficients a(T), b(T), and c(T) were later amended by Midorikawa and Kobayashi (1978) [2.3.1-8].

(c) In addition, Kobayashi and Midorikawa have (1981) [2.3.1-9] proposed the following empirical formula with seismic moment (M_0) instead of magnitude (M) used as the parameter:

$$\log S_{v_0}(T) = a(T) \cdot (\log M_0 - 26.6) - b(T) \cdot \log X + 2.36 \tag{2.3.1-13}$$

(3) Estimation of seismic motion with the aid of fault model

The area of the seismic fault plane increases as the scale of the earthquake increases. At magnitude 8, the area reaches a size similar to that of a prefecture (Translator's Note: size of a typical county in the U.S.). As a result, it is difficult to take the focus as a point.

Seismic waves are generated from the entire fault plane. They then overlap with each other and reach the observation point. Consequently, in the vicinity of the focal region, the intensity distribution of the seismic wave reaching the observation point no longer has a concentric circular form with the focus at the center, as predicted from the conventional empirical formula with the assumption of a point source; instead, it is closely related to the profile of the fault plane and the propagation direction of the rupture.

Since the focal region is not a point, but has a certain size, the amplitude of the seismic wave in the focal region or its vicinity does not increase significantly. Since the focus is distributed over the entire fault plane with a certain size, when the spot becomes relatively near the focal region, it becomes difficult to exhibit the distance attenuation phenomenon as predicted by assuming a point source. In this case, as the distance decreases, the amplitude does not increase so much.

Several efforts have been made to try to explain the short-period seismic motion characteristics, and several models have been proposed.

Generally speaking, the methods of the fault model can be divided into the following four types. In the following, these models will be explained briefly.

a. Deterministic model

In the "deterministic model," the source process is handled as a simple homogeneous process. It can be further classified to a "kinetic model" in which the focal process is defined only by the kinetic parameters, and a "stress relaxation model" in which the shear stress is analyzed as it relaxes while the shear destruction progresses on the fault plane. Typical research work for the "kinetic model" was performed by Haskell. The "deterministic model" is effective for the long-period components with periods longer than several seconds. However, it has a tendency to give a lower evaluation for the short-period component.

b. Probabilistic model

In "probabilistic model," the complicated focal process and rupture transfer process that actually take place are treated in a probabilistic manner, so that the short-period component can also be evaluated. It may be further divided into a model in which the dislocation function is considered as a random process with an assumed self-correlation function, a model in which the fault is represented by irregular subslips (e.g., by Sato).

- Sato's method [2.3.1-10]

A rectangular-shaped fault is divided into many small areas. For each small area, the rise time needed to reach the final dislocation amount and the rupture velocity are determined through stochastic perturbation in calculation of the seismic motion.

c. Semiempirical model

According to the "semiempirical model," the seismic records of medium and small earthquakes with identical or similar propagation path and source mechanism are superposed to simulate the seismic motion of a major earthquake. Irikura, Tanaka et al. have investigated this approach.

- Irikura's method [2.3.1-11]

In this method, the records of foreshocks or aftershocks are used. Based on the similarity rule between major earthquakes and microtremors, the small earthquakes records are superposed to synthesize the seismic motion of a major earthquake.

The superposition number is determined on the basis of the ratio of seismic moment with the following three factors taken into consideration: fault length, width, and rise time.

d. Engineering model

In the "engineering model," in order to evaluate the short-period component including in the focal region, modifications are made using empirical formulas from the engineering point of view. In this respect, Kobayashi and Midorikawa applied the empirical formula for point source on each small divided region on the fault plane; Suzuki, Tanaka, and Sato applied empirical formula to correct the short-period component based on th "kinetic model."

– Kobayashi and Midorikawa's method [2.3.1-12]

In this method, it is assumed that the seismic motion envelope function is made of the superposed pulses generated from the small elements on the fault plane; the shapes of the pulses are determined using the semiempirical formula related to the pulse obtained from the records of strong earthquakes; the pulses are then superposed to calculate the seismic motion envelope function. Since the envelope function calculated in this way is for the seismic wave at the bedrock, the spectrum for the surface of ground and the maximum acceleration are calculated by taking the amplification of the soil into consideration. This model is used relatively widely in earthquake damage prevention programs.

(4) Duration of seismic motion and time variation of amplitude envelope

Duration of seismic motion is also an important engineering parameter, just as the maximum value of seismic motion and its spectral characteristics. Various evaluation methods have been tried. However, the duration depends on various factors such as the rupture time (fault length, L/rupture velocity, V_R), the time needed for propagation of the seismic wave to the observation point, propagation path of the seismic wave, and, in particular, increase in the duration of seismic motion due to the repetitive reflection of the seismic wave at the local weak grounds in the vicinity of the observation point. It is thus difficult to derive the duration of the seismic motion with these factors taken into consideration.

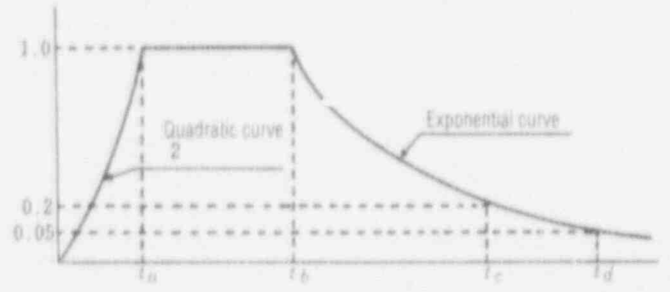
Among the several empirical formulas proposed on the base of observation records using only magnitude M as the variable, the following formula is often used:

$$\log T_d = 0.31M - 0.774 \qquad (2.3.1\text{-}14)$$

In this formula, the duration of the acceleration in each record is defined as the time from the rise start time to the time when the value falls to 10% of the peak value.

As shown schematically in Figure 2.3.1-2, for the envelope curve of the amplitude of the seismic wave, Jennings et al. [2.3.1-13], give the function forms that illustrate rise/fall of the various portions as well as their duration times of each portion.

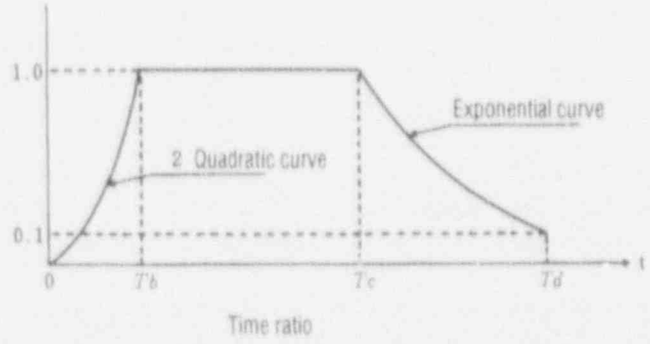
On the other hand, Osaki gives the duration time and time variation of the amplitude envelope as illustrated in Figure 2.3.1-3.



(Units: S)

Magnitude	t_a	t_b	t_c	t_d
4.5-5.5	2	2.5	3.5	10
5.5-6.5	2	4	10	12
7	4	15	30	50
8	4	35	80	120

Figure 2.3.1-2. Functions for defining the seismic waveform [2.3.1-13].



Time ratio

Magnitude	T_b / T_d	T_c / T_d
8	0.08	0.46
7	0.12	0.50
6	0.16	0.54

$T_d = 10^{0.31M-0.774}$

T_d = duration (s); M: magnitude

Figure 2.3.1-3. Duration and time variation of amplitude envelope [2.3.1-5].

2.3.2 Basic earthquake ground motion

(1) Earthquakes under consideration

When aseismic design is to be performed for a nuclear reactor facility, two types of earthquakes should be taken into consideration: the maximum design earthquake and the extreme design earthquake.

The maximum design earthquakes, which represent the basic design earthquake ground motion S1, are determined based on a seismological review of past earthquakes as well as highly active faults which may cause earthquakes in the near future.

The extreme design earthquakes, which represent the basic design earthquake ground motion S2, are determined on the basis of engineering judgement on active faults and the seismotectonic structure of the site and the surrounding region.

For earthquakes generating the basic design earthquake ground motions, both distant and close earthquakes shall be considered.

In addition, a shallow-focus earthquake shall be considered for the basic design earthquake ground motion S2.

Figure 2.1-3 shows the flow sheet for determining the basic earthquake ground motion.

(2) Input position of seismic motion

When aseismic design of a structure is to be performed, one of the important items to be considered is where to locate the position the seismic motion is defined. In the conventional method, the design input seismic motion is defined on the ground surface. In recent years, however, it has also been defined at foundation level, at supporting ground level, and at bedrock level as a result of the development and progress in the earthquake engineering and analysis of seismic motion.

The input position of seismic motion may also depend on the type of structure and its natural period. As a general rule for determining the seismic motion location, the input plane has a certain expanse in the space, the shear wave velocity on the plane is almost the same and the change in the shear wave velocity is less than that on the ground surface.

For aseismic design of a nuclear reactor facility, the basic earthquake ground motion is determined at free surface of the base stratum (rock outcrop) of the site as defined in the "Evaluation Guideline."

The rock outcrop (free surface of the base stratum) is a nearly flat surface of the base stratum expanding over a significant area, and above which neither surface layers nor structures are present. The base stratum is firm bedrock with a shear wave velocity, V_S, higher than 0.7 km/s (2300 fps), which was formed in the Tertiary or earlier era and which is not significantly weathered.

(3) Seismic motion characteristics on the site

The standard seismic motion used for the aseismic design of a nuclear reactor facility is determined on the basis of the seismic motion at the rock outcrop of the site.

When the basic earthquake ground motion is to be determined, the various properties of the seismic motion, such as maximum amplitude, frequency characteristics, duration, and time variation of amplitude envelope, need to be determined.

a. For the maximum amplitude of seismic motion, Kanai's empirical formula is usually used as the formula to evaluate the strength of the seismic motion at the bedrock, since it has relatively small difference between the calculated values and the observed values.

b. The frequency characteristics of seismic motion are determined on the basis of the design response spectrum (referred to as "standard response spectrum" hereinafter) which was proposed for nuclear reactor buildings or other rigid structures built on the bedrock according to aforementioned "2.3.1(2)a. Osaki's method" (see Table 2.3.1-1).
The maximum amplitude and frequency characteristics of the seismic motion are evaluated as a function of the magnitude of earthquake and the distance between the site and the focus where the energy is released.

c. The duration of seismic motion and the time variation of the amplitude envelope are shown in Figure 2.3.1-3.

Also, in order to determine the location of rock outcrop and the frequency characteristic of the seismic motion, results of the following survey items are also taken into consideration.

{1} Survey of elastic wave velocity: Survey of elastic wave velocity at the site. If needed, survey to a portion with a significant depth.
{2} Measurement of microtremor: Measurement of ambient micromotion at the site.
{3} Earthquake observation: Earthquake observation at the site.
{4} Existing data for similar grounds.

In the case when the epicentral distance is small compared to the size of the site, it is also possible to evaluate the seismic motion on the basis of the fault model, which takes the geometric dimensions of the fault and the rupture process into consideration. A typical method is shown in the above section "2.3.1(3)d. Engineering model."

Based on the results of recent research work [H-K-1], from the statistical analysis results of the earthquake observation data obtained for hard bedrock, it is found that for earthquakes with the same magnitude and focal distance, a clear difference in the response spectrum is developed due to difference in the shear wave velocity of the bedrock. As a result, when the frequency characteristics of the standard seismic motion are to be evaluated, on the basis of Figure 2.3.2-1, the standard response spectrum should be multiplied by a correcting coefficient corresponding to the shear wave velocity at the rock outcrop.

Correction coefficient R and shear wave velocity V_S of the ground are defined as follows.

{1} Correction coefficient R

R = 1.0 when V_S = 0.7 km/s, R = 0.8 when V_S = 1.5 km/s.

When 0.7 km/s < V_S < 1.5 km/s, linear interpolation is performed on the two log-scale axes. That is, the correcting coefficient is defined as follows with its shape shown in Figure 2.3.2-1.

$$\left.\begin{array}{ll} R = 1.0 & ;\ V_S = 0.7 \text{ km/s} \\ R = 0.8 & ;\ V_S \geq 1.5 \text{ km/s} \\ R = (V_S/0.7)^{-0.292} & ;\ 0.7 \text{ km/s} < V_S < 1.5 \text{ km/s} \end{array}\right\} \qquad (2.3.2\text{-}1)$$

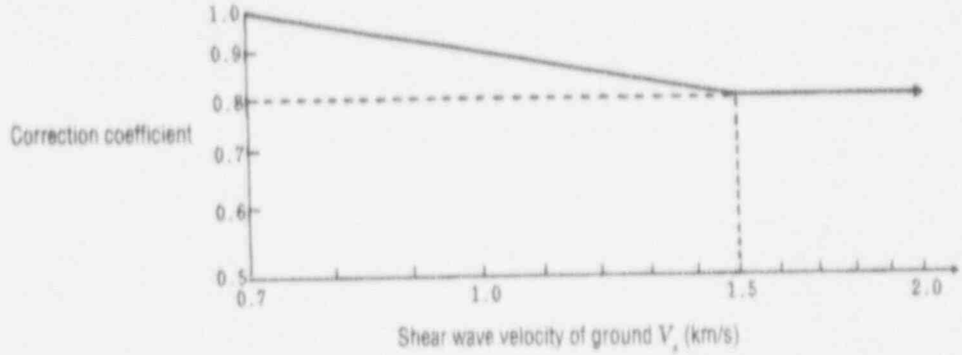


Figure 2.3.2-1. Shear wave velocity of ground vs. correction coefficient (V_S-R diagram).

{2} Shear wave velocity V_S of ground

The shear wave velocity at the rock outcrop is used to represent V_S. For periods longer than 1.0 sec, special consideration is needed when the aforementioned correction coefficient is used.

2.3.3 Generation of simulated seismic wave

For basic earthquake ground motions S_1 and S_2, the simulated seismic wave is generated to fit the corresponding response spectrum on the base of the duration and the time variation of the amplitude envelope as explained in the above section on seismic motion characteristics.

(1) Although there are many methods for generating the simulated seismic waves, the method commonly used at present is by superposing sinusoidal waves to curve-fit the desired response spectrum.

(2) The time history of acceleration wave X(t) as a function of time is represented by the following formula:

$$X(t) = E(t) \sum_{i=1}^{N} A_i \cdot \sin(\omega_i t + \phi_i) \qquad (2.3.3\text{-}1)$$

where X(t): time history of acceleration wave
E(t): amplitude envelope
N: number of superposed Ai
ω_i: angular frequency
A_i: amplitude of each frequency component
ϕ_i: phase angle

The aforementioned methods can be divided according to the phase charactericties and amplitude envelope curves.

{1} Method in which the simulated seismic wave is formed using the phase charac ristics of the actual earthquake.
{2} Method using the phase angle determined by a uniform random number and the amplitude envelope curve as shown in Figure 2.3.1-3.

{3} Method using the phase of a uniform random number and different amplitude envelopes for different period ranges.

{4} Method in which the phase characteristics are prepared as a mixture of pulse phase, exponential functional phase and random-number phase.

(3) The fitness to the target response spectrum can be evaluated as follows. Damping of the response spectrum is supposed to be 5%.

$$R(T) = \frac{S_{V_1}(T)}{S_{V_2}(T)} \geq 0.85 \quad (0.02 \leq T \leq 2.0) \qquad (2.3.3\text{-}2)$$

where, T: period (s)

$S_{V1}(T)$: Response spectral value of simulated seismic wave

$S_{V2}(T)$: Target response spectral value

If the above condition is not met, appropriate correction can be performed repeatedly until the condition is met.

Figure 2.3.3-1 shows the flow sheet of the above method (2)-{2}. Figure 2.3.3-2 shows an example of the generation of a simulated seismic wave.

2.4 Others

2.4.1 Earthquake prediction

(1) Earthquake prediction

Earthquake prediction refers to the prediction of the following parameters of the earthquake to take place: location, scale, and time, on the basis of crustal movement, seismicity, geomagnetism, underground water, etc.

For the major earthquakes which take place at the interplate with a repetition interval ranging from several tens of years to about 200 years, there exists a rather high possibility for prediction. On the other hand, for the earthquakes taking place in the intra-plate of Japan, since the recurrence period is estimated as about 1000 years, prediction is rather difficult.

In 1965, the Earthquake Prediction Research Project [2.1.1-1] was started under the suggestion of the Geodesy Council, the Ministry of Education. This was the first time that the prediction of earthquake was taken as a national project in Japan. Later, in 1968, a system for promoting the project was set up with The Coordinating Committee for Earthquake Prediction as the mainstay, using the Tokachioki earthquake as the turning point. In 1970, eight areas of specified and intensified observation were assigned. Amendment was made in 1978 as shown in Figure 2.4.1-1.

When a nuclear power plant is to be planned in one of these regions, sufficient survey should be carried out with respect to the reasoning for selecting the specific area.

(2) Large Scale Earthquake Countermeasures Act

As a background for drafting a law regarding special measures against large-scale earthquakes [2.1.1-2], efforts were made to provide prediction information on large-scale earthquakes in the Tokai region (Tokai

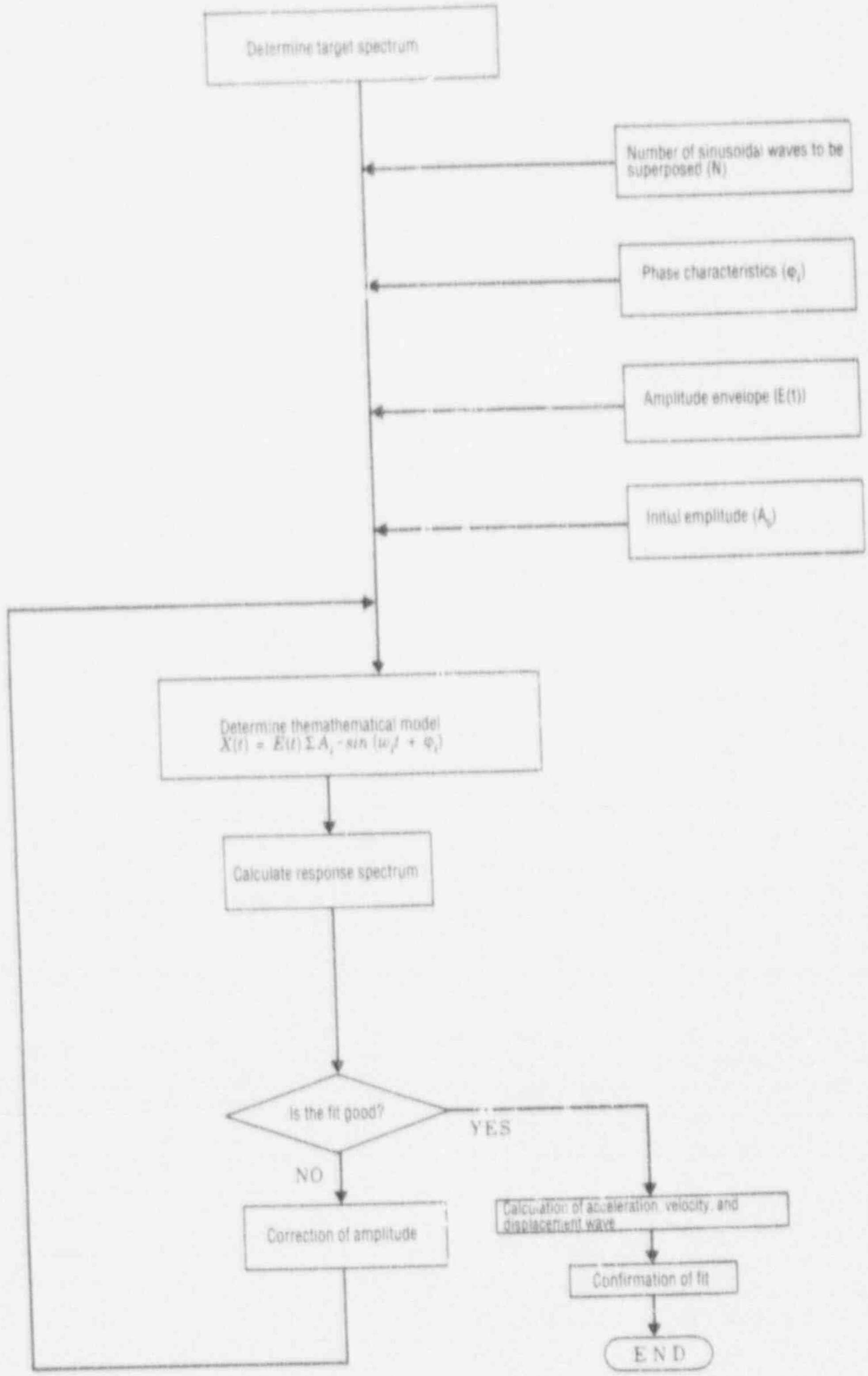


Figure 2.3.3-1. Flow sheet for generating simulated seismic wave.

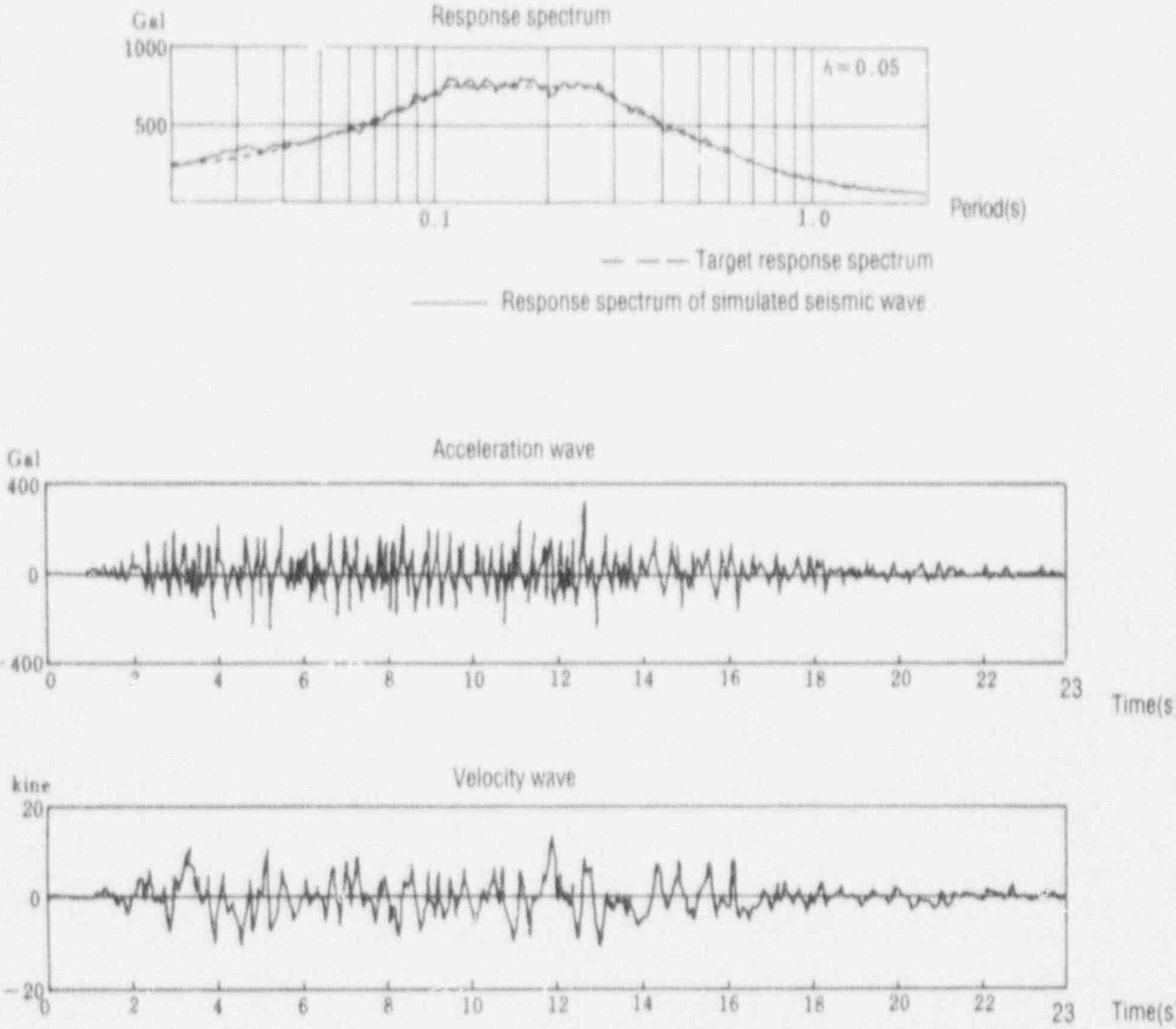


Figure 2.3.3-2. Example of formation of simulated seismic wave.

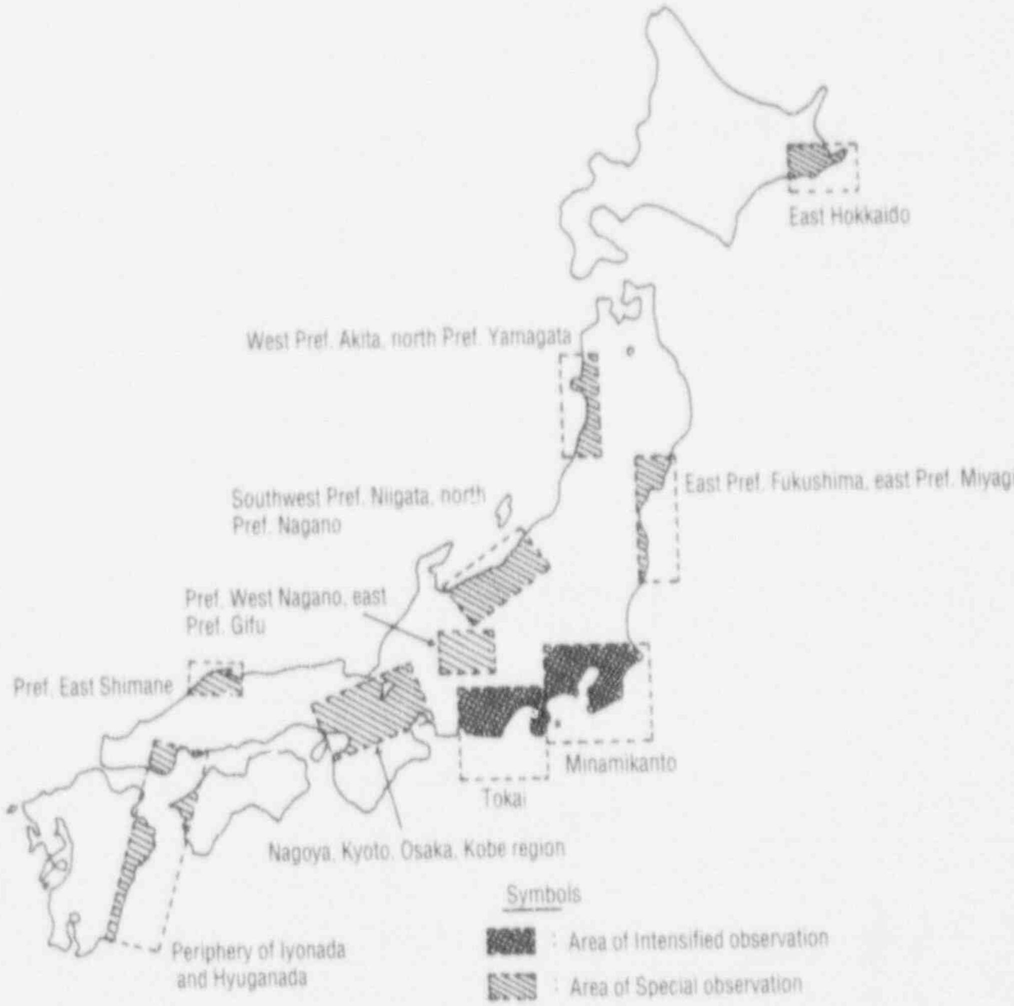


Figure 2.4.1-1. Special observation regions (amended on August 21, 1978) [2.1.1-1].

earthquake). For this purpose, measures for preventing damage have been proposed, and it became necessary to support these measures by legal means.

Based on the following facts with respect to the Tokai region, the Earthquake Prediction Liaison Council [2.1.1-3] believed it necessary to further strengthen the observation. Hence, in April 1977, Prediction Council for the Tokai Area was set up.

The facts include:

{1} About 120 years have passed since the Ansei Tokai earthquake in 1854. It is clear that there exists an seismic gap where no large-scale earthquake took place during this period.
{2} Significant subsidence has been found since the Meiji era in the area from Onaezaki to Suruga.
{3} Horizontal compression in the northwest-southeast direction with Suruga at its center is observed.

In addition, public opinion is strong on the counter measures against earthquake disaster with a demand for drafting a law regarding measures against earthquakes. As a result, Large Scale Earthquake Countermeasures Act was drafted and went into effect in December 1978.

This law is mainly characterized by the feature that it is a law of special measures against earthquakes before the damage takes place. The earthquake taken as the object is the earthquake with magnitude of about 8 for which the precursory phenomenon before the earthquake can be observed in relatively wide range.

At present, only the Tokai region is assigned as Area under Intensified Measures against Earthquake Disaster. It is believed, however, that in the future, with the development of earthquake prediction, other regions may also be assigned. When a nuclear power plant is to be planned in any region assigned, it is necessary to make a sufficient survey on the reasons for the siting and to take necessary measures.

2.4.2 Tsunami

Many nuclear power plants in Japan are located in coastal regions. Hence, when the site is selected and designed, the influence of a tsunami must be fully taken into consideration. Tsunami is mainly caused by the uplift and depression of ocean bottom in a wide range accompanying an earthquake.

In the ocean, the transfer velocity V of a tsunami is represented by $V = (gh)^{1/2}$ (g: gravitational acceleration, h: depth of the sea). It is about 200 m/s at a water depth of 5 km. Hence, it takes about 8 min for a tsunami generated in the ocean 100 km away from the coast to reach the coast. As it approaches the coast, the water becomes shallower, the transfer velocity becomes slower, and the wave becomes more concentrated. When a tsunami hits a V- or U-shaped bay, the wave may have a very large height in some cases.

In order to predict the height of a tsunami, the following measures are taken:

{1} Survey of past tsunami records
{2} Investigation using simplified formulas
{3} Numerical simulation
etc.

In the survey of past tsunami records, evaluation is performed by extracting the references which record the earthquakes that caused damage on the site and vicinity [2.1.2-1].

As far as the simplified formulas are concerned, there is the formula proposed by Iida [2.4.2-2]

$$m = 2.6M - 18.4 \tag{2.4.2-1}$$

where m is the tsunami's scale by Imai and Iida.

On the other hand, Abe [2.4.2-3] used the records of tide gauge to evaluate the wave height of tsunami generated on the Pacific Ocean side in the vicinity of Japan. He proposed the following formulas:

$$M_t = \log H + \log \Delta + 5.8 \tag{2.4.2-2}$$

$$M_t = \log H_2 + \log \Delta + 5.55 \tag{2.4.2-3}$$

where, M_t: tsunami magnitude
H: maximum single-side amplitude in tide detection record (m)
H_2: maximum double-side amplitude in tide detection record (m)
Δ: shortest distance between epicenter and observation point on the sea (km)

The above formulas can be used effectively in deriving the maximum height of a tsunami for Δ in the range of 100-3500 km. When they are used for historical earthquake for which M_t is unknown, attention should be paid to the correspondence with the magnitude.

For estimation using the simplified formulas, the influences of the ocean bottom topography, coast topography, and source mechanism are not taken into consideration. Hence, they only give a simple estimated value. In the case when a detailed study is needed, although it is difficult to determine the fault parameters, it is still effective to perform the numerical simulation of the tsunami on the basis of the fault model of the earthquake.

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Chapter 3. Geological and ground survey

3.1 Summary of geological and ground survey

In order to ensure the safety of the nuclear power plant, it is necessary to perform a careful survey and test of the geological conditions and ground, so that highly reliable construction works can be performed on the basis of reliable analysis and design. Since the actual geological conditions and grounds have various different types, for each type of geological condition and ground, appropriate survey, testing, analysis, evaluation, design, etc., should be performed with a good understanding of them.

This chapter refers to the report of "Survey/test methods of geological conditions and grounds of nuclear power plants and evaluation methods of aseismic stability of grounds" compiled by the Japan Society of Civil Engineers [3.1-1].

3.1.1 Summary of geological survey

The purpose of the geological survey in planning/construction of a nuclear power plant is to understand the activity of the faults which should be taken into consideration in the aseismic design and to clarify the detailed geology and geological structure in the periphery of the foundation of the structure.

For this purpose, geological survey is performed for the wide region in the periphery of the site (land and sea) and for the region within the site.

For the wide-range geological survey, appropriate reference survey and topographic survey are performed. On the base of the survey results, surface geological survey is performed along the faults and lineaments described in the references. In particular, in the region near the site, surface geological survey is implemented mainly by performing detailed survey on the outcrop of the fault.

For the geological survey within the site, on the basis of the reference survey, topographic survey, surface geological survey, etc., boring survey and pit survey are performed to obtain knowledge of the detailed geological structure, as well as rock distribution and rock type. If needed, geophysical prospecting, trench survey, etc., are also performed.

In these surveys, the activities of the faults to be considered are clarified, and the detailed geological conditions of the bedrock around the foundation of the nuclear reactor building are determined. On this basis, soil classification and formation of soil model are performed, with results used as the data for safety evaluation of the ground.

3.1.2 Summary of ground survey/test

On the basis of the results of the geological survey and the soil model, appropriate survey and testing of the soil are implemented. The soil as the survey items include ground of the foundation of the nuclear reactor building, peripheral slope of the nuclear reactor building, grounds of important outdoor underground structures, etc. The survey/test are performed correspondingly in the various design stages: basic planning stage, design stage, and detailed design stage.

In the basic planning stage, on the basis of the plans for arrangement of the nuclear reactor building and other structures, necessary surveys and tests are implemented to find the general properties of the geology and soil.

In the design stage, in order to investigate the stability of the soil of the object structures, necessary surveys and tests are performed, such as the detailed survey on the geological structure and rock type, and the physical tests of the rock and bedrock.

The survey/test in the detailed design stage is implemented in the case when the stability cannot be fully evaluated in the design stage. The items of survey/test depend on the specific ground.

3.2 Geological survey

3.2.1 Purpose and scope of survey

(1) Purpose of survey

The main purposes of the geological survey performed in planning and building of the nuclear power plant are as follows: clarification of the geological structure at the prescribed site and its peripheral region, preparation of the data used for selecting the site, clarification of the activity of faults, and investigation of the detailed geological state of the bedrock near the foundation of the structure.

In the survey of activity of faults, among the faults with various scales in the bedrock, the faults with high activity are identified; for these faults, the activity, distribution, position, and size of the faults that are needed to investigate the basic earthquake ground motion, are determined.

In the survey of the foundation bedrock for structures, the detailed geological structure, rock distribution and rock type are determined for the foundations of the nuclear reactor building and other major structures, and the results are used as the basic data for investigating the properties of the soil needed for design.

Figure 3.2.1-1 shows the flow chart of the geological survey.

(2) Scope of survey

The scope of geological survey is the range needed for drafting the construction plan and performing design of the nuclear power plant; the scope should meet the standards such as the evaluation guidelines. The specific surveys are divided to wide-area survey and survey on the site.

The range of the wide-area survey is within a radius of 30 km from the site.(1) For the land region, first reference survey is performed for the appropriate region that includes the aforementioned range to find the geological structure, etc., and faults and lineaments longer than 10 km are selected.(2)

Afterwards, topography survey is performed, and lineaments longer than 10 km are selected. For the vicinity of the site, lineaments shorter than 10 km but believed to have a large influence on the site are also extracted. In addition, major faults described in the references in the neighboring region within a radius of 30 km are also taken as objects for survey. In addition to the surface geological survey along these faults and lineaments,

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(1)According to the present handbook for safety evaluation of geology and bedrock of nuclear power plant [3.2.1-1], as an item for evaluation in safety examination, for the land within a radius of at least 30 km from the center of the site, geological diagram and its explanation should be furnished with appropriate evaluation.

(2)For faults with recorded activity in the Quaternary period in the references, faults with clear deformed terrain, and faults related to seismic activities, even if they are shorter than 10 km, they are still taken as the object for investigation.

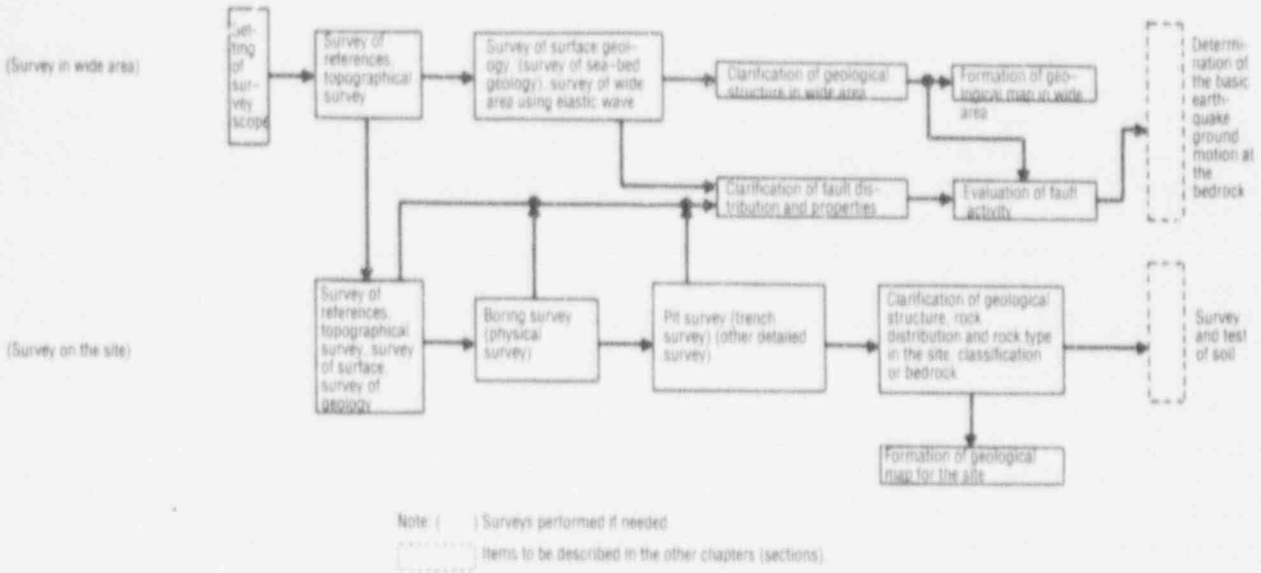

Figure 3.2.1-1. Flow chart of geological survey.

it is also necessary to perform a detailed surface geological survey in the region within a radius of 10 km from the site.

For the sea area, if needed, the survey is performed with the survey region, survey method and survey precision used for the land region adopted.

For the survey of the site, a more detailed geological survey is performed for the region within a radius of 200 m from the center of the site where the nuclear reactor building is to be built.

3.2.2 Wide-area survey

(1) Survey planning

a. Types of survey

There are the following methods for wide-area survey:

{1} Reference survey
{2} Topography survey
{3} Surface geological survey
{4} Seabed geological survey

b. Survey methods

(a) Reference survey

In the reference survey, the geological data within the region to be surveyed are surveyed and collected from the existing references. There are a variety of different references for topography/geology. One should collect as many of the published references as possible. Examples of the published literatures include the geological maps and their expositions published by the Geological Survey of Japan and prefectural authorities; and the literatures published in the journals related to geology, etc. Also, if needed, data not published may also be utilized.

Through reference survey, one should grasp the contents of description of the general items, such as topography, stratigraphy, geological structure, geological history, etc., the contents of description of the fault distribution, lineaments, standard topographical profiles, Quaternary-period crustal movement, etc., as well as the contents of description of active volcanoes, large-scale earth slips, etc.

Among them, for the faults and lineaments, those which are longer than 10 km (including those which are shorter than 10 km but are located near the site and have significant influence on the site) and those which have had activity in the Quaternary period (except those which have no recorded activity in the latter period of the Quaternary period) are extracted. For these faults and lineaments, the following data are sorted: location, direction, length, rupture width, displacement, sense, lineament features, presence/absence of records of activity, etc. Based on these results, each fault is classified as a fault estimated from references or a fault whose existence has been verified.

(b) Topography survey

In the topography survey, the lineaments are determined using the existing topographical map (1/50,000, 1/25,000, etc., published by the Geographical Survey Institute) and aerial photographs (1/40,000-1/10,000, etc., taken by the Forestry and Field Agency, Geographical Survey Institute).

As the lineaments are surveyed, lineaments longer than 10 km and lineaments with clear terrain displacement are extracted. [illegible] are sorted according to the features of the topography, and are used as data to study the existence and activi[illegible] faults. Table 3.2.2-1 lists examples of survey of the lineaments. Table 3.2.2-2, lists examples of the survey contents.

(c) Surface geological survey

Surface geological survey is implemented in the range described in Section 3.2.1 (2) "Scope of survey."

For the region within a radius of 10 km from the site, detailed surface geological survey is implemented to clarify the constituent rocks, stratigraphy, geological structures, etc. Also, in the survey along the faults and lineaments outside the radius of 10 km, together with the survey of the fault outcrop, the geological structure and rock distribution are also surveyed. It is desirable to confirm the presence of a fault by observing its outcrop. However, in the case when a clear outcrop cannot be observed, investigation is made from the peripheral geological structure.

For the faults and lineaments as the object of survey, appropriate sketches and descriptions of the fault outcrops are performed. Since the size of a fault, in particular, the length of a fault, is important in determining the scale of a possible earthquake, careful investigation is needed of the continuity. Figure 3.2.2-1 shows an example of a sketch of a fault outcrop.

To determine the activity of the fault, the characteristics of the upper layers on the fault, in particular, the Quaternary-period layers (layers which act as indices for the fault's activity years, such as terrace deposit, volcano ash layer, red earth, etc.), are surveyed. If needed, survey is also performed to determine the ages of these layers.

Table 3.2.2.-1. Examples of standards for judging the lineament [3.1-1].

Classification of lineament	Elements for juding		
	Terrace surface	Within mountain and hill	Lineaments continuation direction, continuation, altitude continuation, etc.
Lineaments with high possibility of dislocation	- Those for which on a clearly continuous terrace surface there are clear scarp, steep slope, and other linear continua which are free of breakage. - Those for which on a number of different terrace surfaces and other terrain surfaces, there exists a straight continuation of scarp and steep slope.	- Those for which on a clearly continuous terrace surface, there are clear scarp, saddle portion, and other linerar continua which form a uniform height discontinuity. - Those for which the river valleys and ridges are systematically bent in the same direction, with the bending amount being accumulative.	- The continuous direction is oblique or perpendicular to the directions of river-eroded scarps or sea-eroded scarps. - The direction of inclination is opposite to the general inclined direction of the topographical surface. - The continuous direction is identical to the direction of river-eroded scarp or sea-eroded scarp; the inclined direction is identical to the general inclined direction of the topographical surface. However, there exists a clear height discontinuity, and the difference in elevation is generally uniform with good continuity.
Lineaments with possibility of dislocation	- Those for which on a clearly continuous terrace surface, there are clear scarp, steep slope, and other linear continua which are almost free of breakage.	- Those for which on an estimated continuous terrace surface, there are scarp, saddle portions, and other linear continua which form a uniform height discontinuity. - Those for which although the river valleys and ridges are not clear, they are bent systematically in the same direction.	- The continuous direction is oblique or perpendicular to the directions of river-eroded scarps or sea-eroded scarps. - The direction of inclination is opposite to the general inclined direction of the topographical surface. - The continuous direction is identical to the direction of river-eroded scarp or sea-eroded scarp; the inclined direction is identical to the general inclined direction of the topographical surface. However, ther exists a clear height discontinuity, and the difference in elevation is generally uniform with good continuity.

Table 3.2.2.-1 *(Cont'd)*. Examples of standards for judging the lineament [3.1-1].

Classification of lineament	Elements for juding		
	Terrace surface	Within mountain and hill	Lineaments continuation direction, continuation, altitude continuation, etc.
Lineaments with low possibility of dislocation	- Those for which although there is a continuum of nearly linear scarps and steep slopes on the terrace surface, a portion of it is not clear.	- It is made of a nearly linear continuum of scarps, saddle portions, etc., with uniform height discontinuities observed on both sides. - A portion of the valleys and ridges are bent in the same direction.	- The continuous direction is inclined a little to the directions of river-eroded scarps or sea-eroded scarps. - The continuous direction is identical to the direction of river-eroded scarp or sea-eroded scarp; the inclined direction is identical to the general inclined direction of the terrain surface. However, there exists a height discontinuity, and the difference in elevation is generally uniform with good continuity. However, a portion is unclear.
Lineaments with little possibility of dislocation	- Those for which although there are scarps and steep slopes on the terrace surface, there are many discontinuous portions, and the pattern becomes unclear.	- It is made of nearly linear discontinuous portions of unclear scarps, saddle portions, etc., with unclear height discontinuities on its two sides.	- The height discontinuity is unclear; the continuous direction is identical or inclined in the directions of river-eroded scarps, sea-eroded scarps, and the general inclined direction of the topographical surface; a portion of it is broken to pieces.
Lineaments caused by factors other than dislocation	- Those for which no scarp or steep slope is seen on the topographical surface.	- It is made of discontinuous portions of unclear scarps, saddle portions, etc., with unclear height discontinuities on its two sides.	- Although a height discontinuity is observed, it is unclear. - The continuous direction is identical or inclined to the directions of the river-eroded scarps and sea-eroded scarps, and the general inclined direction of the topographical surface. It is often broken in a discontinuous pattern.

Table 3.2.2-2. Examples of the content of judging of lineaments related to identification of dislocation [3.1-1].

Items for judgment	Content of judgment
Presence/absence of a sense with a certain tendency	Whether there is a cumulative tendency of dislocation with the lineament at the boundary. In particular, for terrace surface with different heights, if the lineament has an accumulated difference in elevation, there is a high possibility of dislocation.
Degree of preservation of topography form	Is the topographical configuration that forms the lineament clear or not?
Continuity of lineament	Is the lineament continuous or not on the standard topography? Does it have a certain length?
Standard topography	Does the standard topography lineament include terrace, volcano foot, fan-shaped terrain or foothill mild slope? These terrains are believed to be formed in the late Quaternary period.
Topographical configuration	Does the lineament topographical configuration contain reverse scarp, reverse low scarp, wind gap, or bend? These terrain configurations are believed to be closely related to the dislocation.
Lineament direction	Is the lineament direction perpendicular or oblique to the direction of the conventional scarp? In the case of parallel direction, there is a high possibility of forming scarp by erosion.
Linearity of lineament	Is the lineament linear or not? If it is not linear, there is a high possibility of formation due to erosion and land slide.

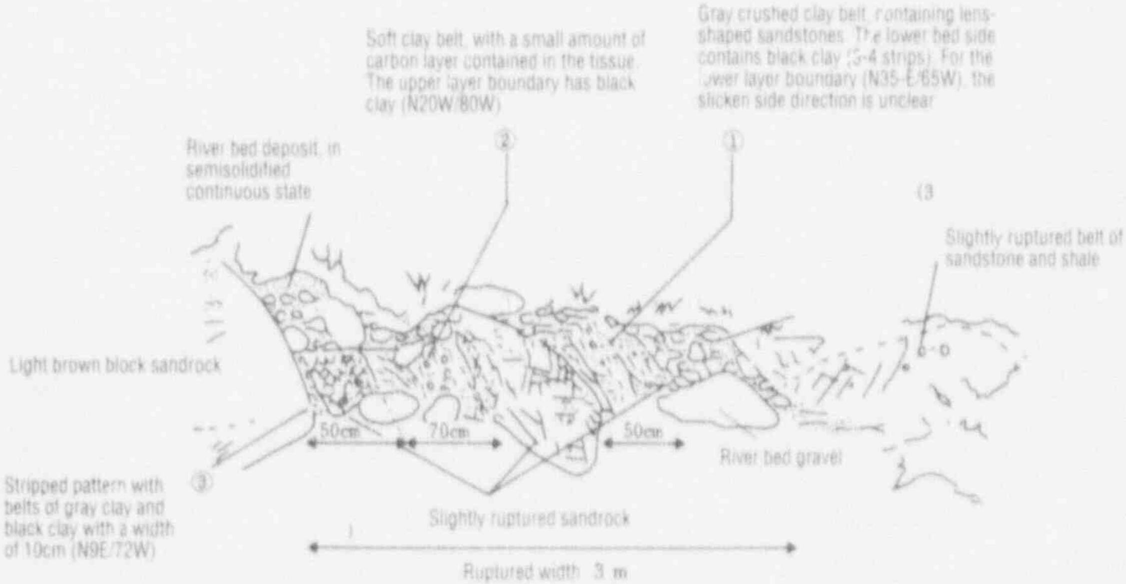

Figure 3.2.2-1. Example of sketch of fault outcrop.

In addition, survey is performed on the substances within the fault, depending on the requirement (see Section 3.2.3 (1) b "Survey methods").

(d) Seabed geological survey

In the case when there is no data on the seabed geological structure, in particular, data on faults, and in the case when it is necessary to evaluate the activity of a fault on the seabed as described in the references, seabed geological survey is performed. In this case, survey is performed on the stratigraphy, geological structure, presence/absence of fault, as well as the size, property and activity of the fault.

The references of the seabed geological structure are mainly provided by the official institutions, such as Waterway Division of Maritime Safety Agency, Geological Survey Institute, etc. The methods used in the surveys of the seabed by these institutions depend on the purpose of each specific case. In particular, for the sonic prospecting, there are unique features for each survey method including the survey depth, distance between measurement lines, and resolution. When investigation is to be made of the geological structures, such as faults, described in the references, it is necessary to have a sufficient understanding of the features of these methods.

Seabed geological survey is mainly performed by using sonic prospecting method. As the geological structure of the seabed is surveyed, information is obtained on the layer/rock distribution, fault distribution, their scales and properties, and activities in the seabed. Their relation with those in the land region is also clarified. Figure 3.2.2-2 shows an example of the flow chart for investigation of the activity of a fault in the seabed from the sonic prospecting and related items for investigation.

(e) Other surveys

In addition to the aforementioned types of surveys, in order to find the profile of layer wave velocity in the deep underground portion, wide-area elastic wave survey is implemented if needed. Usually, the depth where the layer wave velocity with a P-wave velocity is about 5-6 km/s, which is known as the seismic bedrock, to said layer are determined. The scale and precision of the survey are determined appropriately according to the geological structure in the periphery of the site.

(2) Evaluation of geology/geological structure

a. Items for evaluation and investigation

Based on the results of the surveys described above, evaluation is made as the following items are investigated.

{1} Layer name, distribution, properties, geological age, and geological structure
{2} Fault position, size, properties, and active age
{3} Others

For {1}, the various layers and rock types in the survey region are [illegible]d; their size and continuity in the horizontal and vertical directions are determined. In this way, the layer and rock distribution in said region can be identified. In addition, based on the sequence of layers, their overall relationship or absolute formation age, the formation sequence of the layer and rock can be determined. Figures 3.2.2-3 and 4 illustrate examples of the geological map and the lineament distribution map.

For {2}, based on the above {1} survey results of the object fault, its certainty of existence, size, and properties are clarified, and the results of survey and investigation of the fault length, activity history, and the final activity age are summarized. For the fault that cannot be confirmed on the surface, investigation is made with

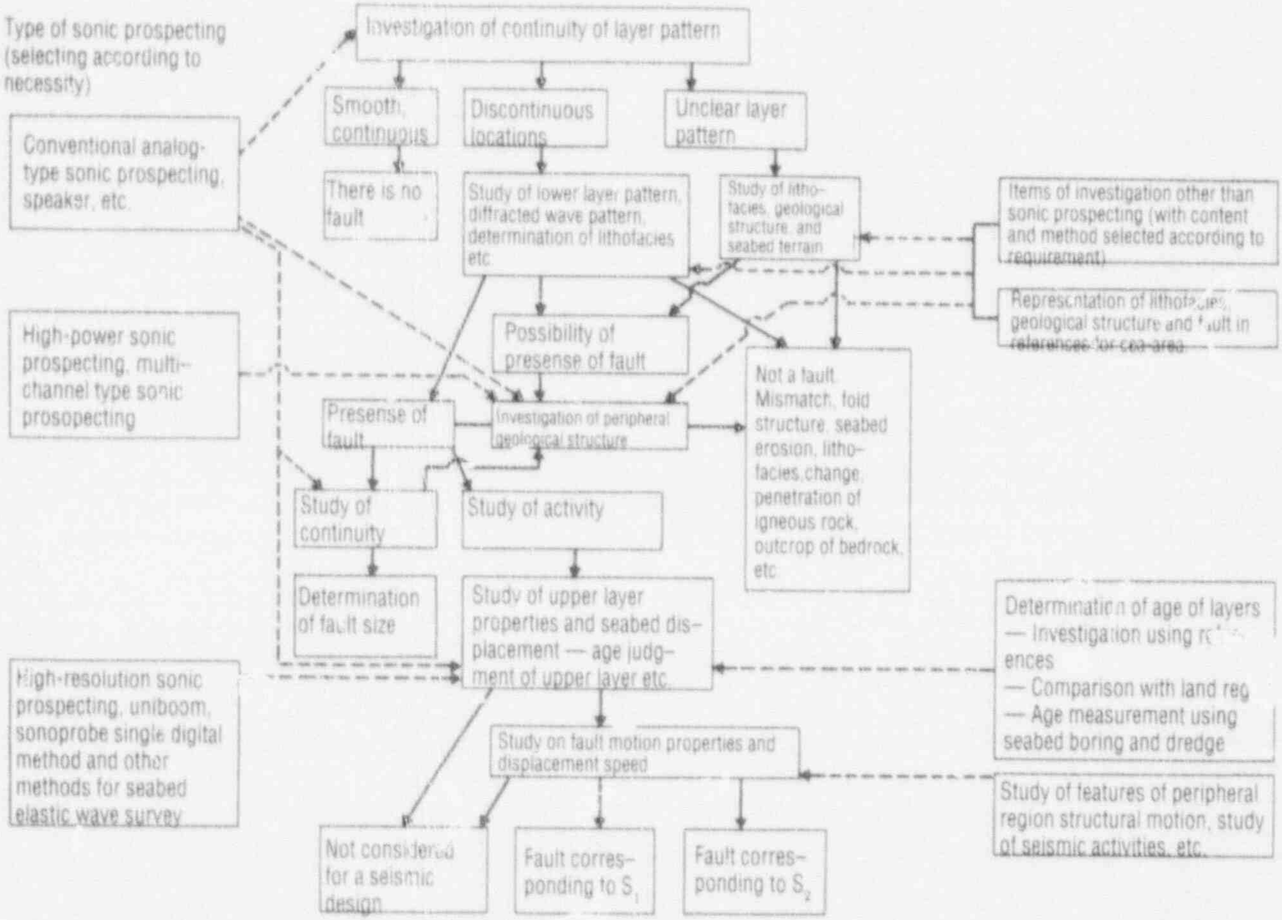

Figure 3.2.2-2. Example of flow chart of investigation of activity of seabed fault by sonic prospecting and related items of investigation [3.1-1].

Figure 3.2.2-3. Example of geological plan map of periphery of the site.

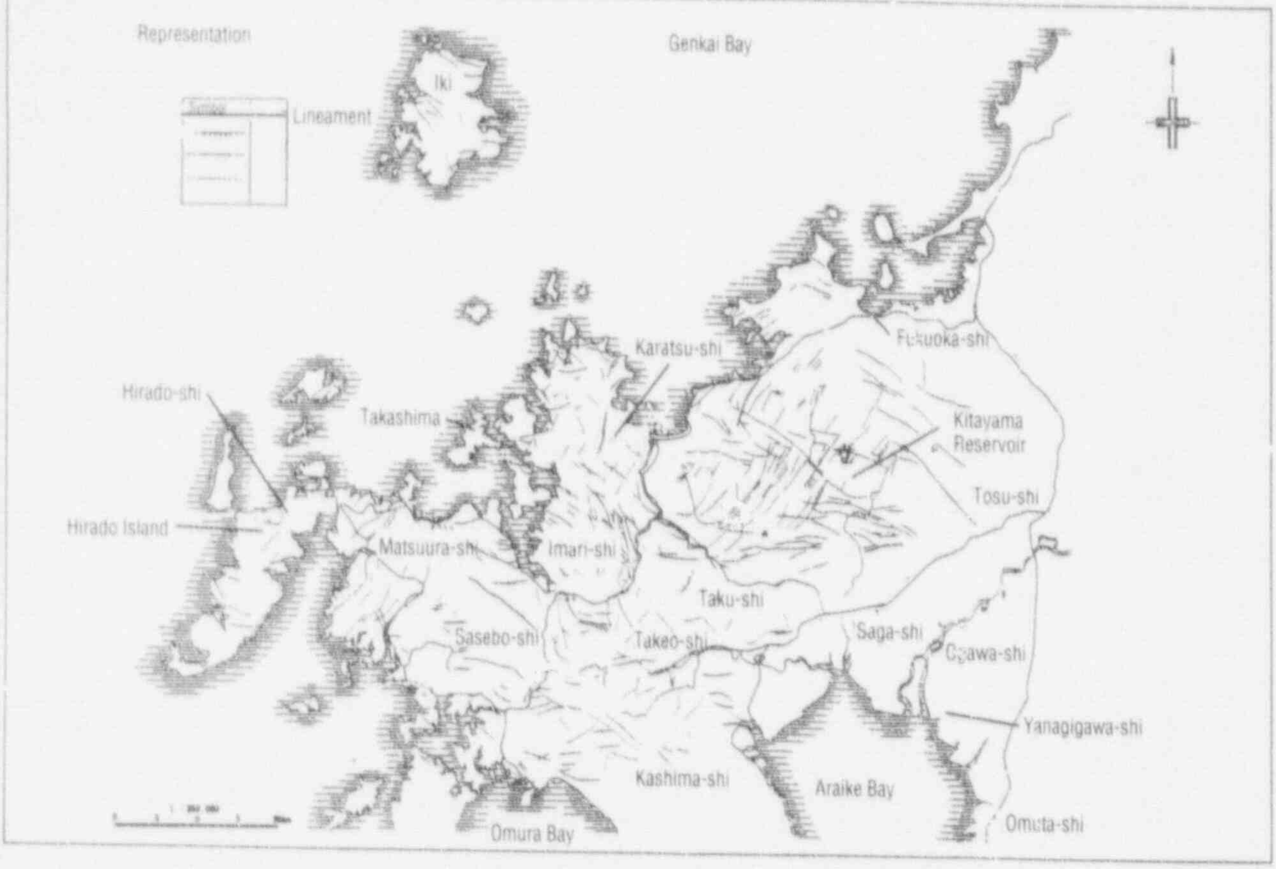

Figure 3.2.2-4. Example of lineament distribution map in the vicinity of the site.

reference to the available references, as well as the geological structure and rock distribution in the periphery of the fault. The results are classified, according to the final activity ages, to fault corresponding to S_1, fault corresponding to S_2,[1] and fault not considered for aseismic design, and are used as the data to determine the basic earthquake ground motion (see Figure 3.2.2-5).

b. Representation form

The results of the wide-area survey are represented as the following geological maps.

{1} Geological map with a scale of 1/200,000 for the range within radius 30 km from the site.
{2} Geological map with an appropriate scale for the range within radius of 10 km from the site.

3.2.3 Survey on site

(1) Survey planning

a. Summary of survey

The following methods are used to survey the site.

{1} Reference survey
{2} Topography survey
{3} Ground surface geological survey
{4} Boring survey
{5} Pit survey
{6} Physical survey
{7} Trench survey
{8} Fault active years survey

Among these, surveys {6}, {7} and {8} are performed depending on the necessity.

For the entire area of the site, reference survey, topography survey, and ground surface geological survey are mainly performed.

In the vicinity of the nuclear reactor building, boring survey is also performed. At the location where the nuclear reactor building is to be constructed, boring survey with a large boring depth and pit survey are performed.

As far as the peripheral slope of the nuclear reactor building is concerned, judging from the analysis results for the past cases of slope failure [3.2.3. 1-4], the slope for which the distance from the tail of the slope to the nuclear reactor building is {1} smaller than about 50 m, or {2} smaller than about 1.4 times the height of the slope[2] is taken as the peripheral slope of the nuclear reactor building. However, the actual slopes have various topography and geological structures, it is necessary to make a careful study when the actual survey is performed. For these peripheral slopes, the geological survey needed for safety evaluation of the slope is mainly performed by ground surface geological survey and boring survey.

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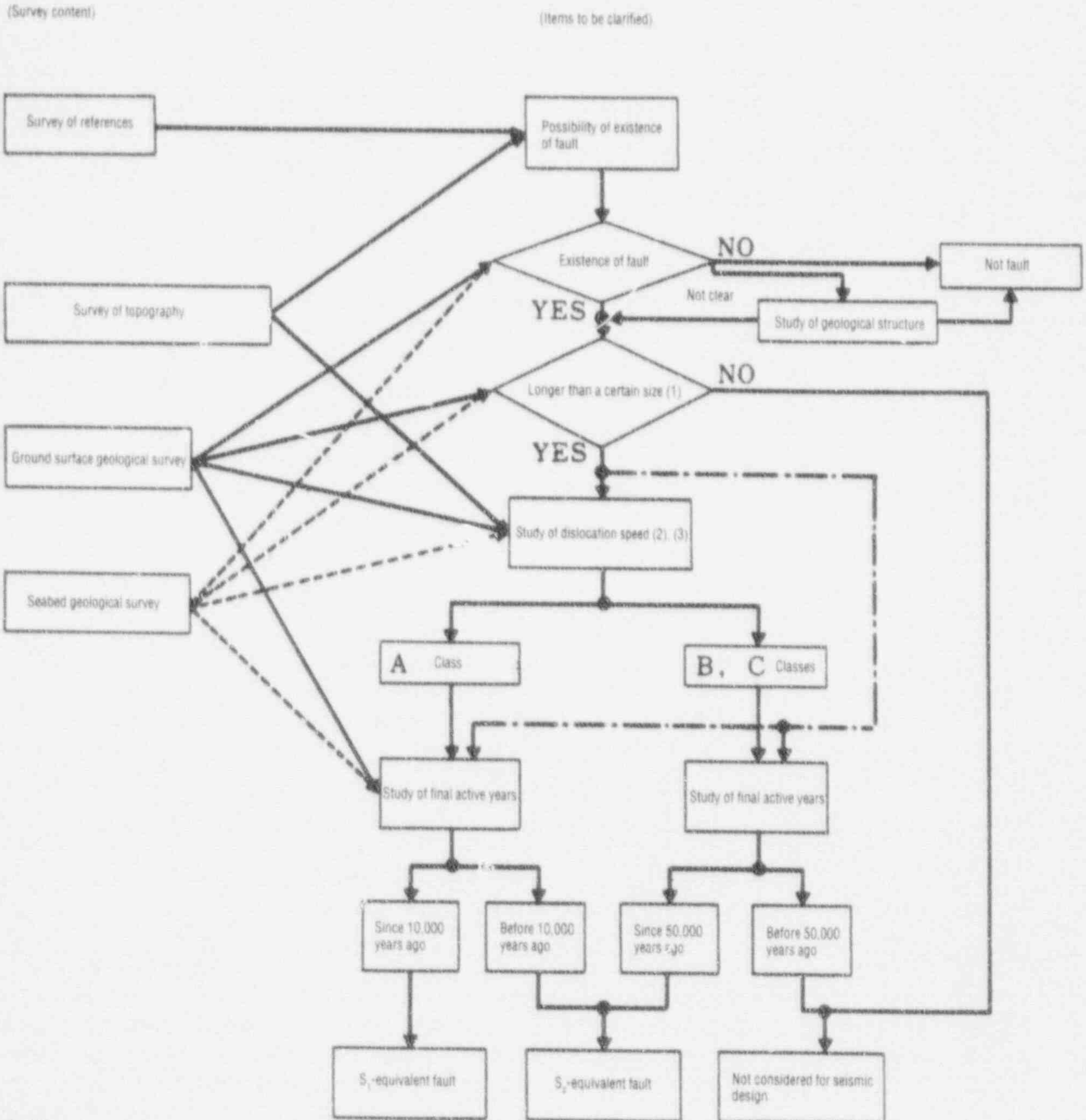
[1]Here, the faults that cause basic earthquake ground motions S_1 and S_2 are called fault corresponding to S1 and fault corresponding to S_2, respectively.

[2]The slope height refers to the difference in elevation between the tail of the slope and the highest point of the peripheral slope near the nuclear reactor building in the range which may affect the nuclear reactor building and other important facilities in case of slope failure.



Symbol
. . . . . Studied only when needed
Notes: (1) Longer than 10 kw for wide range: it becomes shorter in vicinity of site.
(2) Determination is usually difficult; in many cases, the route indicated by dot-dash line is adopted.
(3) Average displacement speed (S) (mm/year) is as follows:
$1 < S$ fir Class A
$0.1 < S < 1$ for Class B
$S < 0.1$ for Class C

Figure 3.2.2-5. Flow sheet of survey of fault activity.

To evaluate the geological condition of the planned site of an important outdoor underground structures, boring survey is mainly performed. Figure 3.2.3-1 illustrates an example of the geological survey of the site.

b. Survey methods

(a) Reference survey

The survey method is based on the reference survey described in "3.2.2 Wide-area survey." Depending on the requirement, the data in the vicinity of the site, including those unpublished, are collected and assorted.

(b) Topography/geological survey

Among the lineaments determined in the wide-area topography/lineament survey, those which are on the site and in its vicinity are extracted and assorted according to the items in Section "3.2.2. Wide-area survey." In the site and vicinity, depending on the requirement, a detailed topographical survey is carried out to study the topographical elements, and to perform topographical classification. For the lineament survey, lineaments including those considered to be too short in the wide area survey are used as the data for evaluation of the fault activity. Also, attention should be paid to the distribution of landslide topography, failure topography, and sand/stone avalanche deposit topography.

(c) Ground surface geological survey

The purpose of the ground surface geological survey is to collect the detailed ground surface geological data in the site and vicinity for determining the general directions and guidelines of survey in the site.

In the ground surface geological study, the following items are surveyed.

{1} Type, formation age, distribution, and conditions of layers of rock
{2} Weathering/deterioration of rock
{3} Distribution, size, and properties of fault ruptured zone
{4} Distribution, scale, and properties of joint
{5} Presence/absence and features of landslide, ground failure, etc.

In particular, for the discontinuous planes observed in the Quaternary or other new-era layers, detailed study should be performed to find out whether it is caused by fault or landslide. In the case when it is due to fault, observation should be made of the relation between the Quaternary layers (terrace deposit, volcanic ash layer, red soil, etc.) and the fault and used as data to understand the final active period of the fault.

(d) Boring survey

The range of boring survey and the boring interval are determined according to the purpose of the survey. For the site where the nuclear reactor building is to be constructed, considering that the range requiring engineering examination is about twice the width of the foundation of the building, the boring survey range is about 200 m from the center of the building's foundation. In principle, boring is conducted on the grid drawn within the range. The grid interval is usually set as 40-50 m in order to find the geological structure, rock distribution and rock type for the site where the building is to be constructed. However, it may be set wider in the case when the geology/geological structure is relatively simple.

For the boring survey on the site where the nuclear reactor building is to be constructed, at least 5 sets of all core boring should be implemented, with the depth determined in consideration of the survey range needed for analysis. For the portion just beneath the planned foundation, the boring depth should not be shorter than the

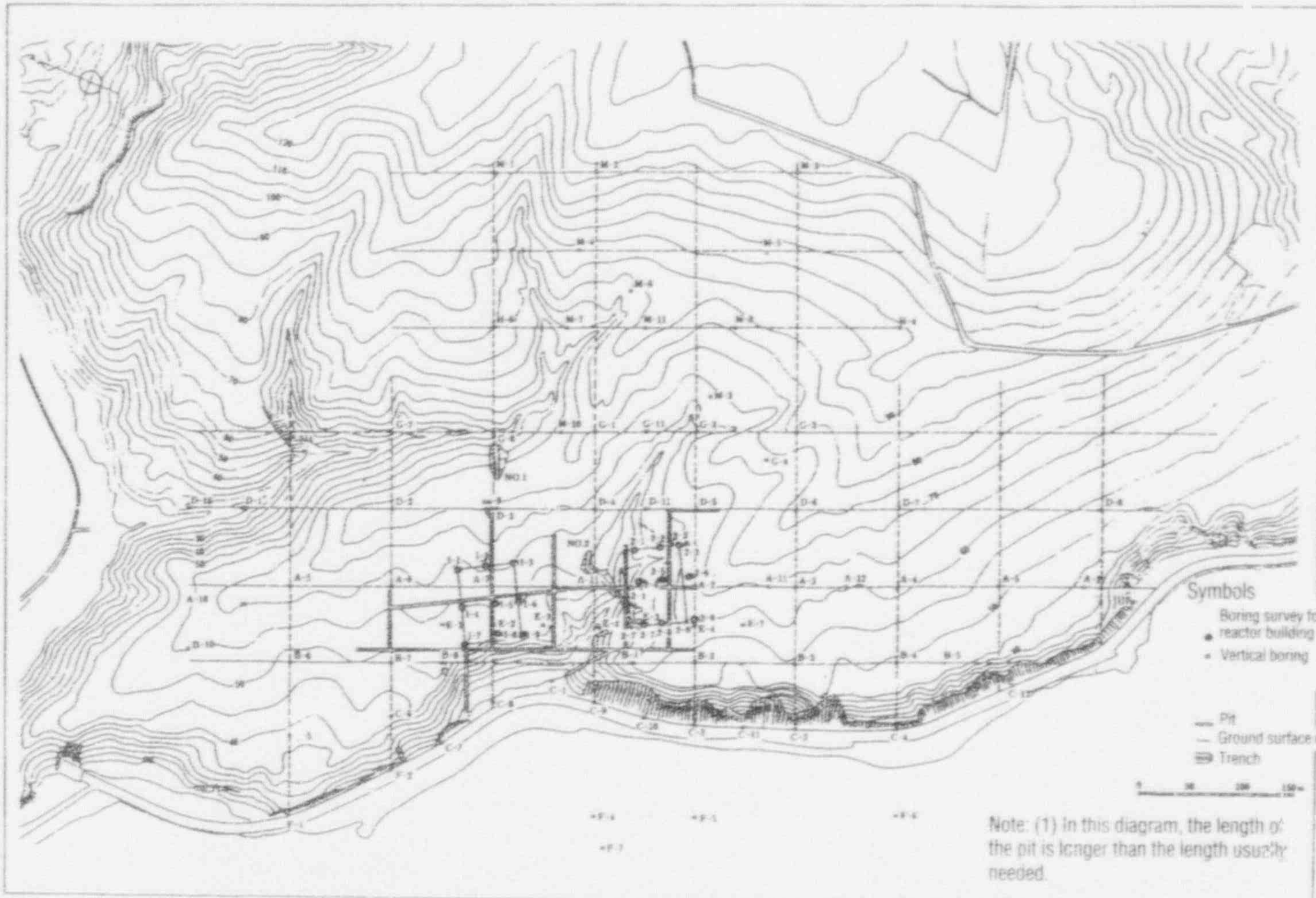


Figure 3.2.3-1. Example of map for geological survey positions in the site.

planned foundation width. In addition, in the case when the geological structure, distribution of rock and distribution of rock type are complex or when there are many types of rocks, more boring logs should be arranged.

Also, boring survey may also be performed for a general survey of the geological structure over the entire area of the site, and survey of the peripheral slope and the foundation of important outdoor underground structure, as well as for tracing the continuity of a specific fault. In this case, the range, number of logs and depth are determined according to the specific purpose.

In boring survey, survey is made of the following items: the type of rock, status of weathering and deterioration, rock type, cracks, rock classification; distribution, property and continuity of rupture zone; core sampling status; underground water status; boring hole status; etc. The results are summarized with an appropriate scale (Figure 3.2.3-2). In addition, pictures are taken of the cores.

(e) Pit survey

The main purpose of a pit survey is to obtain a detailed understanding of the geological structure, rock distribution and rock types at the site of the nuclear reactor building and in its vicinity, so that the position of the nuclear reactor building can be determined. For this purpose, in principle, the pits are excavated so that they cross at a right angle to each other above the elevation of the foundation of the reactor building, with a proper length considering the width of the planned foundation. In addition to the pit survey for the foundation of the reactor building, pit survey may also be performed to survey the peripheral slope, and to trace faults, depending on the requirement.

For the pit geological survey, the type of rock in the pit, weathering/deterioration state of the rock, rock classification, geological boundary, fault, rupture state of its periphery, etc., are surveyed, with the obtained results summarized in a pit unfolded diagram with a scale of 1/100 (see Figure 3.2.3-3).

(f) Physical survey

The physical survey methods include elastic wave survey (elastic wave propagation speed in bedrock), electrical survey (resistivity and other electrical properties), gravitational survey (density), magnetic survey (magnetic property), radioactive survey, etc.

(g) Trench survey

The purpose of the trench survey is mainly to clarify the following items:

{1} Confirmation of presence/absence of faults along the extended line from the existing fault.
{2} Confirmation of presence/absence of Quaternary activity for the fault to be evaluated.

The excavation position, size and number of trenches are determined according to the specific purpose. In the trench survey, survey is performed of the following items: presence/absence of fault on the side wall and, if present, the status of the fault, classification of the Quaternary layers, relation between the fault and Quaternary layers, dislocation displacement of the fault, etc. The results are summarized in geological unfolded map with an appropriate scale, sketch, etc.

(h) Survey of fault active age

In the survey of the fault active age, the age survey of the layers related to the fault is mainly performed. Also, depending on the requirement, evaluation of the intrafault material is also performed as reference.

NO. 116

EL. = 65.50m
L = 89.70m

Leveling rod (m)	Depth (m)	Elevation (m)	Columnar section	Geological name	Core sampling rate (%)	Maximum core length (m)	Note
40	40.00	EL. 29.50		Rhyolite		32	Silicon
						45	
						100	
						49	
						66	
45						26	
						21	
	47.90	EL. 21.60				28	
				Fine-grain diorite		33	The rock boundary is fine and tight.
	50.50	EL. 19.00				40	
50				Rhyolite		56	
						17	The rock boundary is not clear, with a gradual transition nature. Silicon.
						45	
						55	
55						32	
						29	
						55	
							Rupture belt, with width of 80 cm and slope angle of 75.° Made of clay and sand portions
60						12	Silicon with many cracks. The structure is weathered to a certain degree.
						15	
						29	
	64.10	EL. 5.40				16	
65				Mixture of rhyolite and fine-grain diorite		27	The structure is weathered to a certain degree.
						33	The structure is fresh. The slope angle is 56° at the boundary of the two types of rock.
						26	
	67.90	EL. 1.60				38	
				Fine-grain diorite		30	
70						43	
						26	
	72.10	EL. -2.60				27	
				Rhyolite		37	The rock boundary is fully tight. It is also fine.
75						36	
						28	
						27	
						66	
						31	In a gradual transitional way, it becomes silicon.
						30	
						31	The rock boundary is not clear, with a gradual transition nature.
80	80.00	EL. -10.50					

Figure 3.2.3-2. Example of boring columnar section.

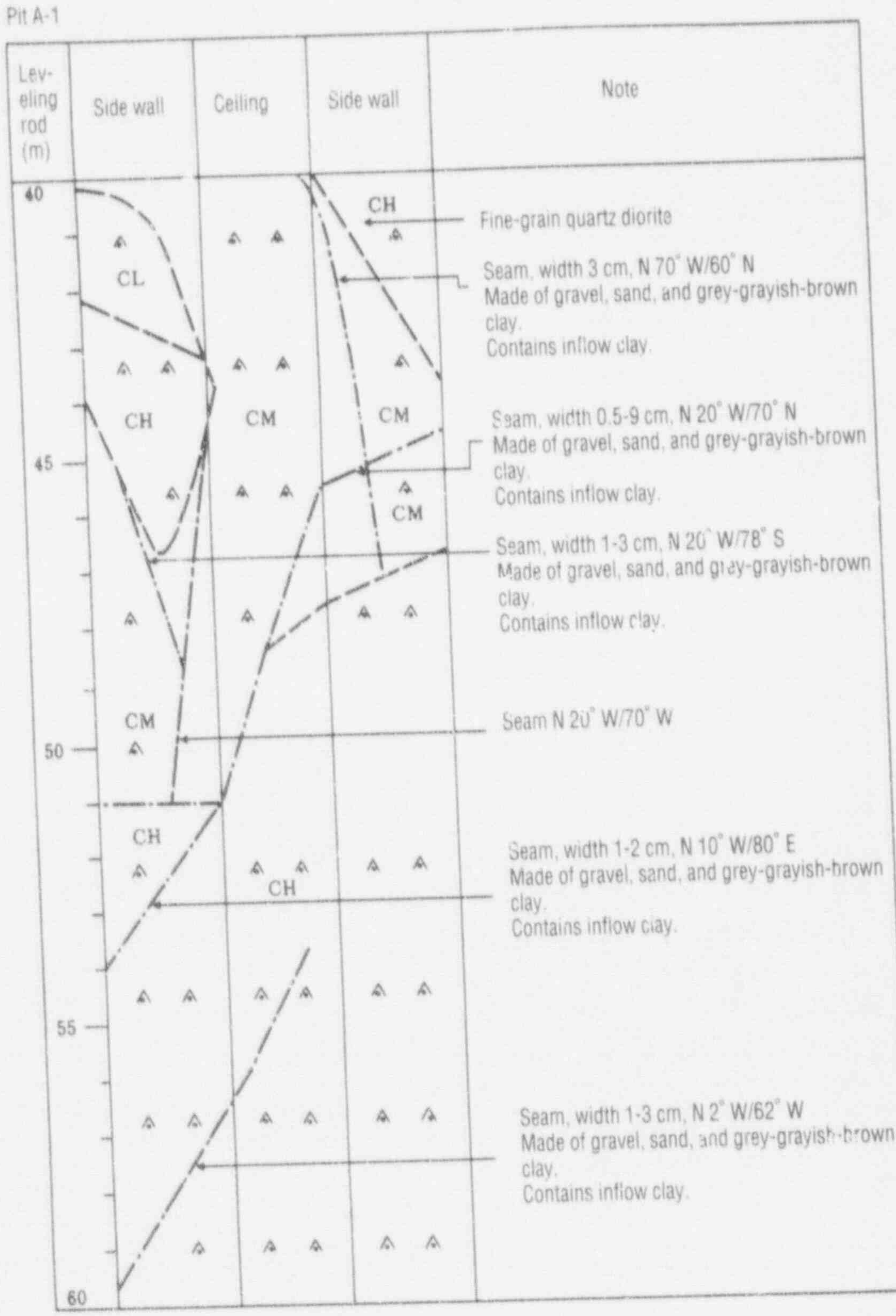


Figure 3.2.3-3. Example of pit unfolded map.

Methods used to determined the age of the layers dislocated by the fault or the age of the layers that covers the fault include 14_C method and other age measurement methods, as well as tephrochronology method, etc. Methods that may be used to study the intrafault material include surface textures analysis of quartz grains, fission track method, ESR dating method, etc.

(2) Evaluation of geology and geological structure

a. Evaluation items

Based on the survey results described in the above sections, evaluation is performed on the following items:

{1} Names, distributions, properties, and geological ages of rocks and unconsolidated deposits that form the ground, rock types, weathering/degradation status, geological structure
{2} Distribution and properties of fault rupture belt, history of fault activity, final active age
{3} Classification of bedrock
{4} Others (land slide, etc.)

b. Forms of representation

The results of survey of the site are represented by the following geological maps.

{1} Detailed geological maps with the planned reactor core position in the center

For a range with the planned reactor core position in the center and with a radius of about 200 m, the following maps are formed with a scale of 1/1,000 or smaller; at least one horizontal geological map at the level of the bottom surface of the foundation of the building; and at least 2 vertical geological maps which are perpendicular to each other and pass through the planned reactor core position.

{2} Wide-area geological maps

For the range within about 1 km from the center of the planned site, a horizontal geological map and vertical geological cross-sectional maps (at least in 2 orthogonal directions) with a scale of 1/5,000 or larger. In addition, depending on the requirement, representation is also made for distributions and directions of joints and seams, geological structure of peripheral slope, rock distribution and rock quality. Figures 3.2.3-4 and 3.2.3-5 illustrate examples of horizontal and vertical geological maps.

(3) Classification of bedrocks

a. Bedrock classification method

In principle, the bedrock classification of the ground for nuclear power plant is performed by classifying the rock type and degree of rock quality on the basis of the geological elements. The basic classification is useful for the detailed engineering tests to be performed later. Classification of the bedrock is basically a combination of classifications of rock type and the degree of rock quality. In some cases, for several classes with nearly the same engineering properties ~ the basis of the geological engineering judgment, it is also possible to handle them as the same bedrock class.

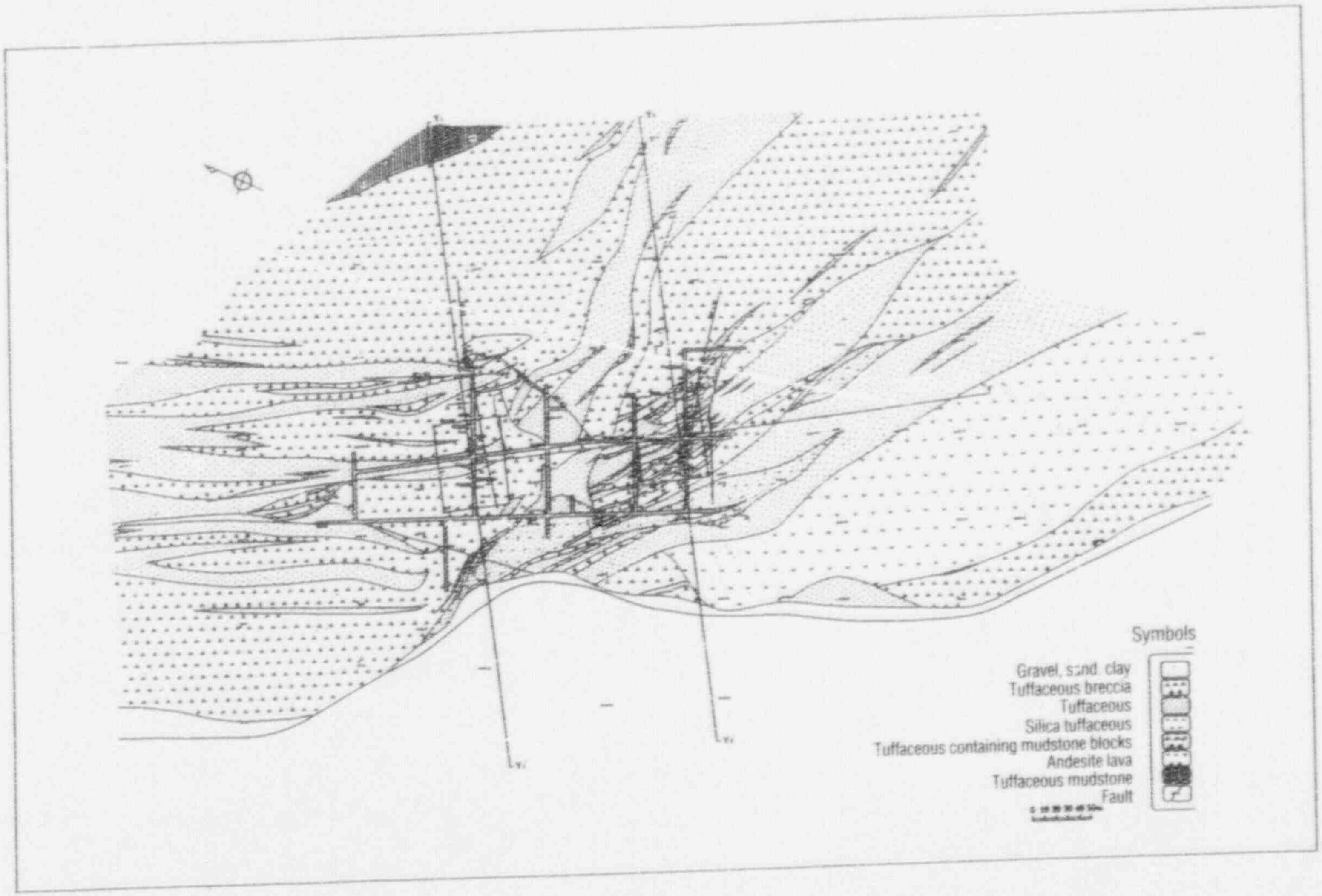


Figure 3.2.3-4. An example of a horizontal geological section map.

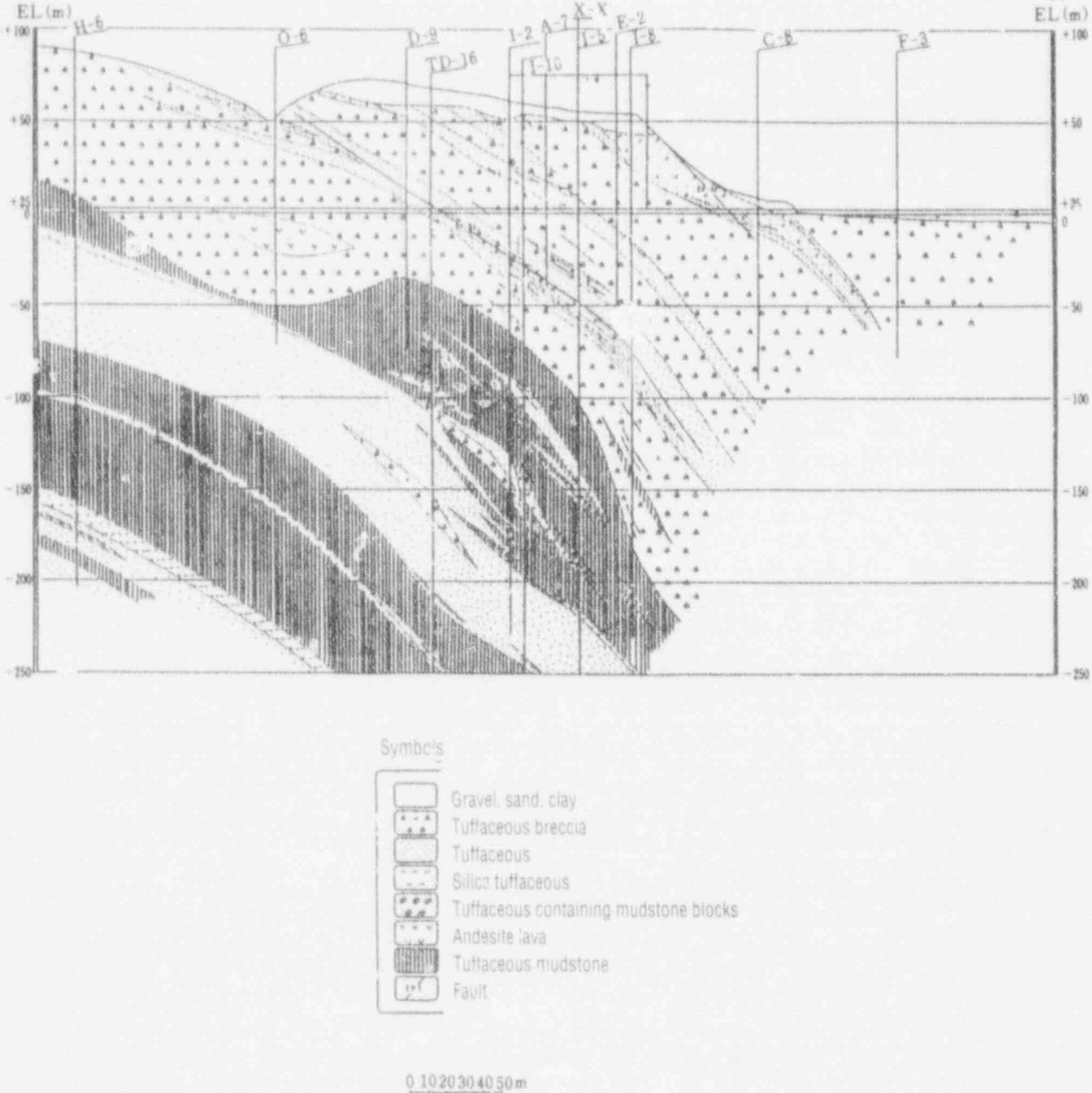


Figure 3.2.3-5. An example of a vertical geological section map.

Bedrock can be roughly classified as hard bedrock and soft bedrock. Table 3.2.3-1 lists the types and formation ages of the rocks that form the bedrock, various bedrock classification methods, and summary of the correspondence to the bedrock constituent materials described in section "3.3.4 Classification of soil and engineering characteristics and evaluation."

The engineering properties of the hard bedrock significantly depend on the degree of weathering/deterioration and the state of development of joint, schistosity and other separation planes. The hard bedrock classification method commonly used at present is a combination of the DENKEN-type 6-class bedrock classification method listed in Table 3.2.3-2 and the appropriate rock type classification. Of course, in addition to the element classification described in Table 3.2.3-2, depending on the actual geological elements at each specific location, it is necessary to set up an appropriate judgment standard suitable for the characteristics of the intrinsic geological elements to be classified for rock quality or the characteristics of the boring core.

On the other hand, for soft bedrock, the elements that define the properties of the bedrock depend more on the intrinsic composition, age, etc., of the forming rock, rather than weathering, joint, etc. The soft bedrock in broader sense includes bedrock which is made of rocks that originally belonged to the family of hard rocks but have been softened due to serious weathering. These bedrock can be handled as Class C or Class D in Table 3.2.3-2.

Among the soft bedrock, those which are made of rocks that are intrinsically soft are usually known as sedimentary soft rocks (including pyroclastic rocks). However, it is difficult to classify these bedrock using the same standard as for the hard bedrock. Hence, depending on the requirement, it is necessary to perform appropriate classification according to the geological element standard. For the soft bedrock (sedimentary soft bedrock) that should be evaluated as the foundation ground of the nuclear power plant, they can be classified roughly into three types according to the features of the geological factors and the engineering properties to be classified. These features of geometrical factors and engineering properties are summarized in Table 3.2.3-3.

For the soft bedrock classified as in Table 3.2.3-3, if further classification of the bedrock quality is needed, it is desirable to define an appropriate classification standard corresponding to the feature of the geological factors at the spot. Usually, for the semi-hard soft rocks (soft rock, Class-I), they may be classified into 2-3 classes of rock qualities with respect to weathering and joint status. For the new-period soft rocks (soft rock, Class-II) and heterogeneous soft rocks (soft rock, Class-III), they are usually classified into groups according to lithofacies or layering features. In particular, if needed, depending on the joint development status, etc., they may also be classified into two classes of rock quality.

b. Modeling of ground

The ground can be classified into the following types from the engineering point of view (see Figure 3.2.3-6).

{1} Isotropic ground
{2} Anisotropic ground
{3} Heterogeneous ground

The aforementioned classes are made mainly in consideration of model formation when the stability of the ground is to be investigated. The classes may be further divided as listed in Table 3.2.3-4. Depending on the actual situation, the model can be set up as a combination of these and other elements.

Classification of the models can be made in consideration of the correspondence between the features of geometrical factors and the ground property types. By selecting the appropriate model, it is possible to classify the various groups for performing tests on the properties of the ground in an easy and rational way (group dividing). It is also possible to obtain the distribution of ground properties for design purposes without considering the classification of the ground model.

Table 3.2.3-1. Application of bedrock classification method and engineering classification.

<table>
<tr><th colspan="4" rowspan="2">Type of structural rock</th><th colspan="3">Geological classification of bedrock (see 3.3.4(3))</th><th rowspan="2">Engineering classification of ground constituent material (see 3.3.4(4))</th><th rowspan="2">Engineering classification for property representation (see 3.3.5)</th></tr>
<tr><th colspan="2">Hardness of rock block</th><th>Bedrock classification method</th></tr>
<tr><td colspan="4">Igneous rock</td><td colspan="2" rowspan="5">Hard Rocks</td><td rowspan="5">DENKEN-type classification, etc.</td><td colspan="2" rowspan="5">- Hard rock: Rather fresh rock
- Weathered rock, deteriorated rock: Rocks degraded due to weathering and deterioration. Depending on the degree, the property may be represented as soft rock B or soft rock A (weathered soft rock).</td></tr>
<tr><td rowspan="7">Sedimentary rock</td><td colspan="3">Paleozoic stratum</td></tr>
<tr><td colspan="3">Mesozoic stratum</td></tr>
<tr><td colspan="3">Old tertiary period</td></tr>
<tr><td rowspan="4">New tertiary period</td><td rowspan="3">General sedimentary rock</td><td rowspan="2">Miocene stratum</td></tr>
<tr><td rowspan="3">Soft rocks (sedimentary soft rocks)</td><td>Class-I soft rock[(1)]</td><td>Classified into 2-3 classes of rock quality according to hard bedrock[(3)]</td><td rowspan="3">Soft rock: Rocks with uniaxial compressive strength (q_u) less than 100-200 kgf/cm^2 are handled as soft rock</td><td>Soft rock B[(2)]: (Same as left)</td></tr>
<tr><td>Pliocene stratum</td><td>Class-II soft rock</td><td>In principle, only rock type</td><td rowspan="2">Soft rock B[(2)]: With relatively large consolidation degree, or in unsaturated state

Soft rock A[(2)]: Soft and in saturated state</td></tr>
<tr><td colspan="2">Pyroclastic rock</td><td>Class-III soft rock (Those with hard substrate are treated as hard bedrock)</td><td>Groups are divided according to rock type and rock phase</td></tr>
</table>

[(1)]In some cases, depending on the consolidation degree of the forming rock, it may contain a portion of Old Tertiary period or Pliocene layer.

[(2)]When the rock test result is applied, rocks that correspond to the effective stress method are considered as A, those that correspond to the gross stress method are considered as B (see Table 3.3.5-1).

[(3)]Depending on the geological state at the spot, an appropriate classification system is set up.

Table 3.2.3-2. DENKEN-type classification of bedrock [3.2.3-5].

Name	Features
A	It is rather fresh, without weathering and deterioration of the rock-forming minerals and grains. The cracks and joints are well adhered, without traces of weathering along these planes. When it is rapped with a hammer for diagnosis, a clear sound can be heard.
B	The rock is hard without opening (such as 1-mm opening), crack or joint. The tissue is well adhered. However, the rock-forming minerals and particles are partially weathered and deteriorated. When it is rapped with a hammer for diagnosis, a clear sound can be heard.
C_H	Except quartz, the rock-forming minerals and grains are weathered. However, the rock is still rather hard. Usually, it is contaminated by limonite, etc.; adhesion among parts separated by joints and cracks is decreased a little. When it is hit hard with a hammer, rock lumps may be detached along the cracks. On the surface of the detachment plane, a thin layer of claylike substance is left. When it is rapped with a hammer for diagnosis, a slightly muffled sound can be heard.
C_M	Except quartz, the rock-forming minerals and grains are weathered and becomes softened to a certain degree. The rock also becomes softened to a certain degree. The adhesion between parts separated by joint or crack is decreased somehow. Rock lumps are detached along the cracks under the common level of rapping with a hammer. A claylike layer is left on the surface of the detachment plane. When it is rapped with a hammer for diagnosis, sound muffled to a certain degree can be heard.
C_L	The rock-forming minerals and [grains] are weathered and soft. The rock is also soft. The adhesion between parts separated by joint or crack is decreased. Rock lumps are detached along the cracks under light rapping with a hammer. Claylike substance is found left on the surface of the detachment plane. When it is rapped with a hammer for diagnosis, a muffled sound can be heard.
D	Rock-forming minerals and grains are weathered and seriously softened. The rock is rather soft. There is almost no adhesion between parts separated by joint or crack. Debris falls off under slight rap with a hammer. Claylike substance is found left on the detachment plane. When it is rapped with a hammer for diagnosis, a much muffled sound can be heard.

Table 3.2.3-3. Basic scheme of classification of soft bedrock [3.1-1].

Class	Features of rock blocks	Possibility of classification	Examples of ranges of physical properties of rock blocks corresponding to features of rock blocks[(1)]			Engineering properties requiring attention
			q_u kgf/cm^2	V_P km/s	V_S km/s	
Semi-hard soft rock (Class I soft rock)	Seedrock, shale, homogeneous tuff and their laminates, mainly of the New Tertiary Miocene epoch, with some of the Pliocene epoch and Old Tertiary period. For the portion with a high consolidation degree and freshness, the rock tissue is fine. However, when hit by a hammer, a muffled sound can be heard. Also, the structural grains on the surface may be deformed or separated easily. These are features different from hard rock.	It is possible to classify the rocks into 2-3 grades according to the degree of weathering and degree of development of joints. These grades correspond to the engineering properties.	400 ~ 50	3.5 ~ 2.0	1.9 ~ 0.9	Strength/deformation characteristics; anisotropicity in some cases.
New-period soft rock (Class II soft rock)	Mainly Pliocene-epoch mudstone, shale, sandstone and their laminates. The consolidation degree is small, and it may collapse easily when hit by a hammer. The rock tissue is homogeneous with a rather simple geological structure and few joints.	It is usually difficult or not needed. In some cases, however, classification may be performed corresponding to the level of development of joints.	100 ~ 10	2.3 ~ 1.6	1.0 ~ 0.5	Strength/deformation characteristics, creep characteristics, slaking characteristics, etc.
Heterogeneous soft rock (Class III soft rock)	Miocene or Pliocene-epoch volcano-ruptured rock, with a soft substrate so that the classification standard of hard rock cannot be applied. There is a significant portion with heterogeneous rock tissue which can be seen by the naked eye. The conglomerates are mainly made of volcanic rocks, or the same soft conglomerate as the substrate. It often forms a laminate with hard volcanic rocks, or is penetrated with the hard volcanic rocks.	Appropriate grouping can be made according to the lithofacies. Classification within the same lithofacies according to geological factors is difficult.	300 ~ 10	3.3 ~ 2.0	1.7 ~ 0.8	Depending on the properties of the substrate, standards of Class I or Class II are applied. In particular, emphasis is set on heterogeneous and dispersion.

[(1)]These are only rule-of-thumb values, not values for classification.

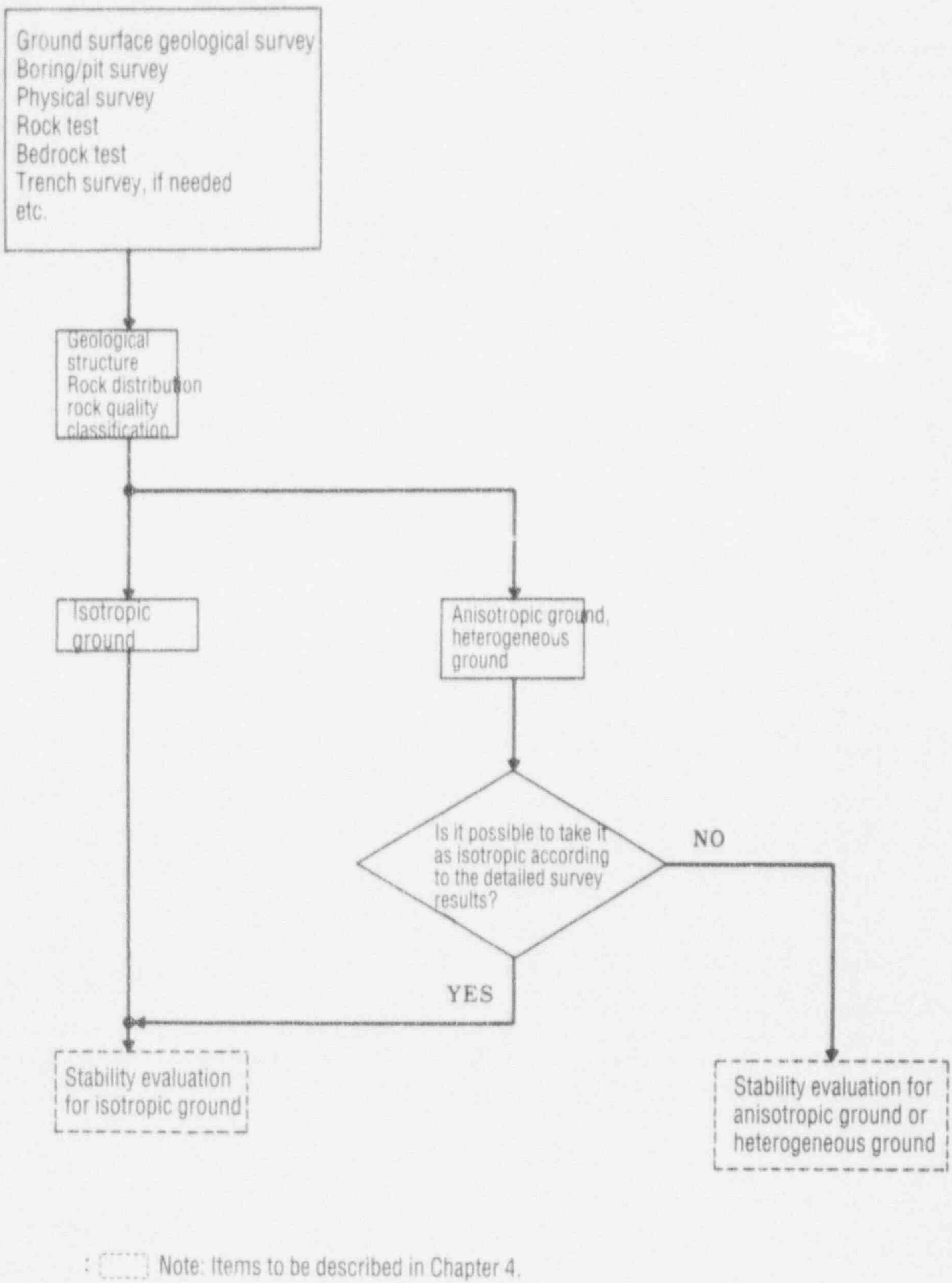
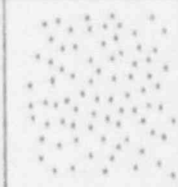
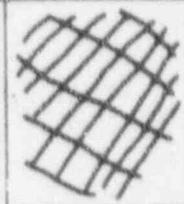
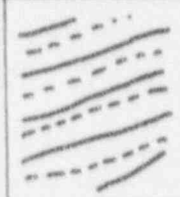
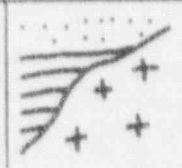
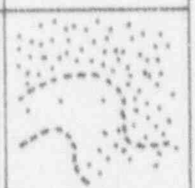
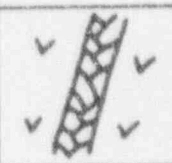


Figure 3.2.3-6. Classification of grounds.

Table 3.2.3-4. Ground models and their features [3.1-1].

Ground			Ground features	Symbols
Ground	Isotropic	Homogeneous ground	Generally speaking, homogeneous isotropic ground. For layers containing gravel, when the size of the gravel is much smaller than the width of the foundation of the structure, it is included in the family of homogeneous ground.	
	Isotropic / Anisotropic	Joint ground	Ground with significant joints. When the directions of the joints seem concentrated, it is taken as anisotropic ground, when the directions of the joints do not seem concentrated, it is taken as isotropic ground.	
	Anisotropic	Layered ground	Ground which is made of laminates of layers having different properties and displays anitropicity, of ground which displays anisotropicity as significant layering and schistosity exist in an otherwise homogeneous ground.	
	Heterogeneous	Hetero-homogeneous ground	Ground which has a multilayer structure or an irregular structure made of two or more types of layers having different properties.	
		Weathered/deteriorated ground	Ground with localized deterioration caused by weathering. Since the properties usually change in a gradual transition, it is necessary to arrange an apppropriate zoning scheme for the rock grade.	
		Ground containing fault rupture belt	Ground for which the existence of fault rupture belt or other weak layer is an important factor for evaluating the properties.	

3.3 Survey and soil test

3.3.1 Purpose of survey and test

The grounds that are taken as the objects of survey/test include ground of nuclear reactor building foundation, peripheral slope of nuclear reactor building, ground of important outdoor underground structure, etc. The purpose of the survey of the ground of nuclear reactor building foundation and the ground of important outdoor underground structure is to obtain properties of the soil needed for stability evaluation of the ground, evaluation of design earthquake ground motions, and evaluation of propagation characteristics of seismic wave motion. On the other hand, the purpose of the survey of the peripheral slope of the nuclear reactor building is to obtain the properties of the soil needed for the stability evaluation of the slope. The survey/test for each location is performed in several stages: basic planning stage, design stage, and detailed design stage.

In the basic planning stage, the foundation rocks are roughly identified, and the rock classification and their three-dimensional distribution are determined based on geological and mechanical characteristics required for performing basic layout of reactor facilities.

In the design stage, the soil properties needed for designing using the conventional method are evaluated for each structure and slope. The properties of the foundation bedrock and the soil properties of each layer, required for determining design input ground motions, are evaluated.

In the detailed design stage, additional properties are evaluated for the analytical methods needed to perform more detailed stability evaluation in the case when the safety cannot be evaluated sufficiently using the conventional method (see Chapter 4).

Figure 3.3.1-1 shows the flow sheet of the various survey stages.

3.3.2 Survey items and survey range

(1) Foundation ground of nuclear reactor building

Here, the foundation ground of the nuclear reactor building refers to the bedrock just beneath the foundation concrete mat of the nuclear reactor building and the peripheral ground.

a. Survey methods

The following methods can be used for the ground survey/test of the foundation ground of the nuclear reactor building.

{1} Survey of ground structure using boring and pit
{2} Laboratory rock test
{3} In situ bedrock shear test
{4} Bedrock deformation test
{5} Elastic wave velocity test

In addition, depending on the requirements, survey/test may also be performed using appropriate boring holes.

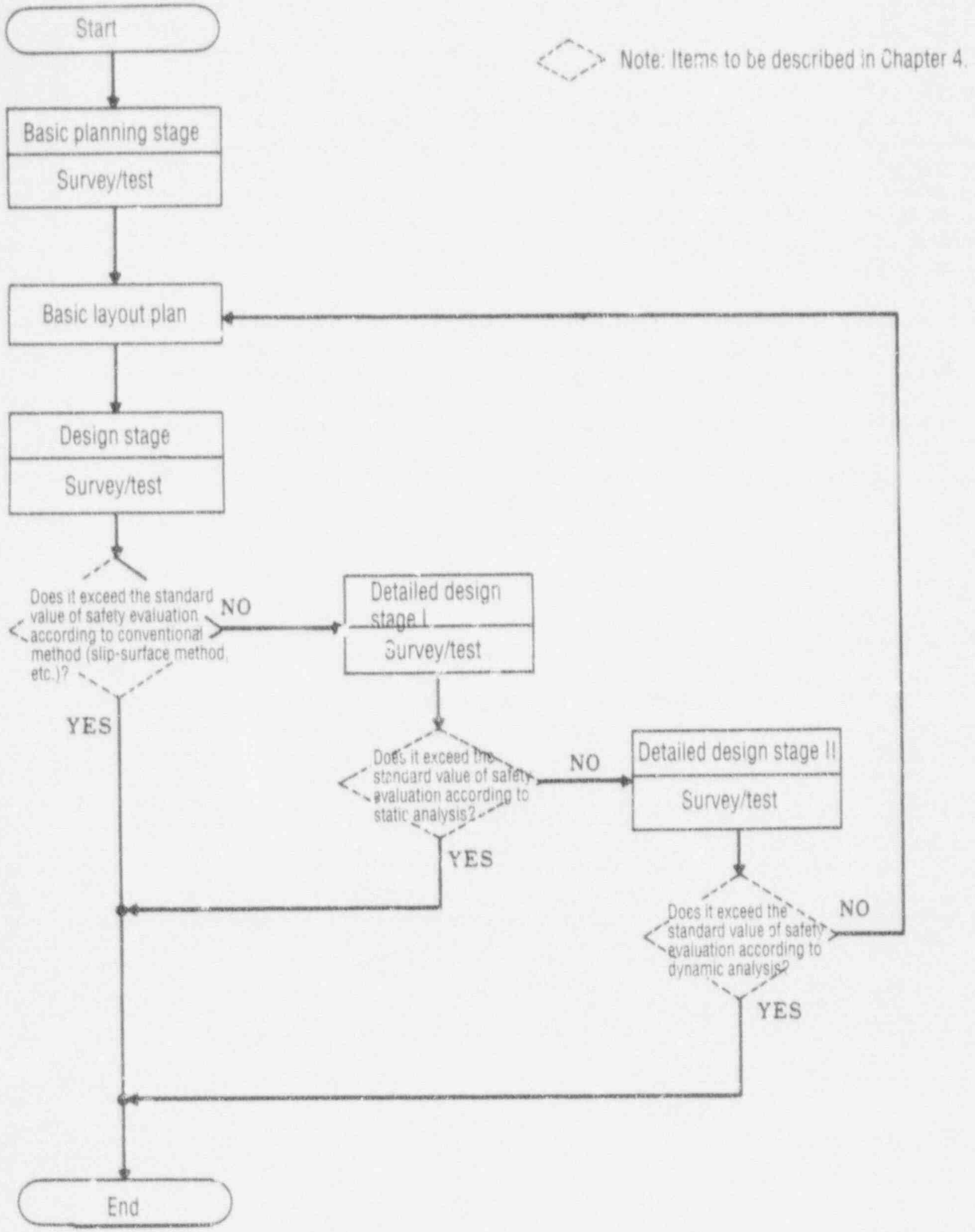


Figure 3.3.1-1. Survey stages and flow sheet of survey [3.1-1].

b. Area of investigations

The area of investigations is determined three-dimensionally considering the area where the stress state of the foundation rocks may be altered by complexity of the foundation rock structure, dead weight of facilities and seismic loads. And the size of the model used for foundation rock stability analyses should also be considered. The specific description can be found in section "3.2.3(1) Survey planning." Figure 3.3.2-1 shows the flow sheet of survey/test of the foundation ground of the nuclear reactor building. Table 3.3.2-1 shows the test example.

(2) Peripheral slope of nuclear reactor building

a. Survey methods

For the ground structure, all-core boring and elastic wave velocity test are performed. If needed, pit survey shall also be performed. For the soil properties, boring core or block sample is used to perform laboratory tests. If needed, in situ test may be performed. In the case of excavated slopes, in addition to the invesigation of variation of the underground water table during rainfall or snow melt, it is important to clarify mechanical properties of the surface materials and rocks required for assessing the stability of the slopes under the stress-released status after excavation. On the other hand, for the banking slope, it is important to evaluate the mechanical characteristics needed to evaluate the stability in the consolidated state. Figure 3.3.2-2 illustrates the flow sheet of ground survey/test for the peripheral slope of the nuclear reactor building.

b. Survey range

The survey range is determined in consideration of the size of the ground model for stability analysis according to the slope's shape, size, geological structure and the constituent materials of the slope.

(3) Ground of important outdoor underground structure

a. Survey methods

For the ground structure, boring survey and, if needed, elastic wave velocity test are performed. For the soil properties, in addition to the aforementioned survey, laboratory rock test using the boring core and in situ bedrock test shall be performed. In the case when the ground at the site is considered to be similar to the foundation ground of the nuclear reactor building, confirmation is performed by laboratory tests, etc. If the ground is found to be identical with the ground of the nuclear reactor building foundation, it is possible to use the properties derived for the foundation ground of the nuclear reactor building. In addition, for the backfill soil, it is necessary to derive the properties of the desired consolidated state. For sandy ground, it is necessary to derive the liquefication strength. Figure 3.3.2-3 shows the flow sheet of ground survey/test of the important outdoor underground structure.

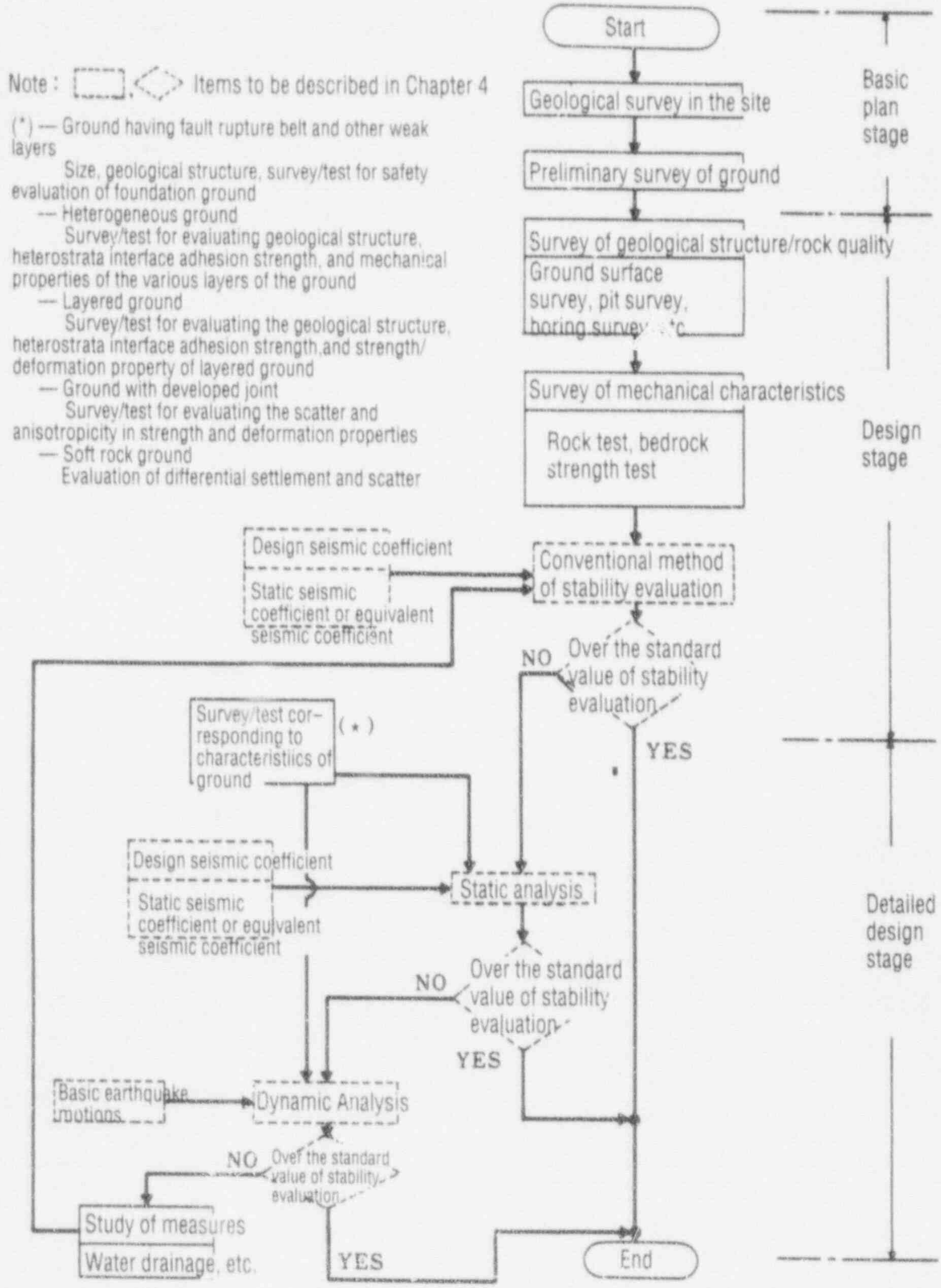


Figure 3.3.2-1. Example of flow sheet of survey/test and design of foundation ground of nuclear reactor building [3.1-1].

Table 3.3.2-1(a). Example (1) of survey/test of foundation ground of nuclear reactor building [3.1-1].

	Item	Point A (mudstone)	Point B (mudstone)	Point C (ryholite)	Point D (granite)	Point E (sandstone, gravel)	Point F (sandstone, mudstone layered together)	Results
Evaluation of rock distribution of foundation ground	Boring survey	9 holes	6 holes (exapand about 1000 m)	2B (B: foundation width) 1 hole; 1B: 4 or more holes (50-m mesh)	2B: 1 hole; 1B: 23 holes	30 holes	6 holes (expand: about 1,560 m), total 92 holes (expand: about 10,500 m)	Layer/rock distribution, underground water level, distribution/continuity of rupture belt
	Elastic wave velocity test in boring hole	5 holes	6 holes	—	1 hole (depth 300 m)	5 holes	16 holes (depth 100-300 m)	V_P, V_S, quality level, crack coefficient
	Pit survey	Perpendicular at the portion just over foundation bottom, about 200 m	Same as left, about 200 m	Same as left, about 670 m	Same as left, about 620 m	Same as left, about 320 m	Same as left, about 1,100 m	Layer/rock distribution, joint distribution, distribution/continuity of fault rupture belt
	Ground surface elastic wave velocity test Elastic wave speed test in pit	— Refraction method: interval of 5m; fan-like radiant method: interval of 2.5 m	— Refraction method: interval of 5m; fan-like radiant method: interval of 2.5 m	50-m mesh, refractive method, fan-like radiant method	— Refractive method, fan-like radiant method	— Refractive method: interval 2 m; fan-like radiant method: interval 2 m	— Refractive method: interval 2.5 m; fan-like radiant method: interval 2.5 m	V_P V_r, V_S, dynamic modulus of elasticity, dynamic Poisson ratio

Table 3.3.2-1(a) *(Cont'd)*. Example (1) of survey/test of foundation ground of nuclear reactor building [3.1-1].

	Item	Point A (mudstone)	Point B (mudstone)	Point C (ryholite)	Point D (granite)	Point E (sandstone, gravel)	Point F (sandstone, mudstone layered together)	Results
Evaluation of rock distribution of foundation ground	Physical test	About 270 pieces	About 260 pieces	About 300 pieces	About 180 pieces	About 560 pieces	In pit: mudstone about 120 pieces, sandstone about 120 pieces In hole: mudstone about 90 pieces, sandstone about 90 pieces	Specific gravity, water absorptivity, density (effective porosity)
	Uniaxial compression test	269 pieces $\phi=35\sim70$ mm, $h=80\sim135$ mm	258 pieces $\phi=50$ mm, $h=120$ mm	About 280 pieces $\phi=50$ mm, $h=120$ mm	95 pieces $\phi=50$ mm, $h=50$ mm	About 250 pieces $\phi=50$ mm, $h=50$ mm	In pit: mudstone 262 pieces, sandstone 265 pieces; In hole: mudstone 231 pieces, sandstone 154 pieces	Uniaxial compressive strength (q_u), static elastic modulus, Poisson ratio
	Ultrasonic velocity test of rock	—	170 pieces $\phi=50$ mm, $h=120$ mm	About 300 pieces	—	About 250 pieces	In pit: mudstone 202 pieces, sandstone 206 pieces; In hole: mudstone 231 pieces, sandstone 131 pieces	V_P, V_S, dynamic elastic modulus, dynamic Poisson ratio

Table 3.3.2-1(a) *(Cont'd)*. Example (1) of survey/test of foundation ground of nuclear reactor building [3.1-1].

	Item	Point A (mudstone)	Point B (mudstone)	Point C (ryholite)	Point D (granite)	Point E (sandstone, gravel)	Point F (sandstone, mudstone layered together)	Results
Evaluation of rock distribution of foundation ground	3-axial compression test	135 groups $\sigma=1,3,6,10$ kgf/cm^2, $\phi=35\sim50$ mm, h$=70\sim100$ mm	—	23 groups $\sigma=20\sim200$ kgf/cm^2 (5 stages) $\phi=30$ mm, h$=60$ mm	100 pieces $\phi=50$ mm, h$=100$ mm	About 120 pieces $\phi=35$ mm, h$=70$ mm	In pit: mudstone 157 pieces, sandstone 113 pieces; In hole: mudstone 223 pieces, sandstone 116 pieces $\phi=50$ mm, h$=100$ mm, $\sigma=1,3,5,8,13,20$ kgf/cm^2	$v_\theta(c)$, ϕ
	Tensile test	—	—	45 pieces $\phi=50$ mm, h$=50$ mm	60 pieces $\phi=50$ mm, h$=100$ mm	Press to crack, 190 pieces $\phi=50$ mm, h$=50$ mm	In pit: mudstone 40 pieces, sandstone 40 pieces; In hole: mudstone 90 pieces, sandstone 80 pieces $\phi=50$ mm, h$=40\sim50$ mm	Tensile (compressive) strength
	Schmidt rock hammer test	Measurement interval: 0.5 m; measurement range: 48 m; measurement point number: 5 points/location	Measurement interval: 0.5 m; measurement range: 40 m; measurement point number: 9 points/location		Measurement of pit wall with interval of 1 m	Measurement interval: 2.5 m; measurement range: 110 m; measurement point number: 46 location	Measurement interval: 5 m; measurement point number: 9 points/location	Distribution of scatter in resistance (evaluation of nonuniformity of ground)

Table 3.3.2-1(b). Example (2) of survey/test of foundation ground of nuclear reactor building.

	Item	Point A (mudstone)	Point B (mudstone)	Point C (ryholite)	Point D (granite)	Point E (sandstone, gravel)	Point F (sandstone, mudstone laminated with each other)	Results
Evaluation of bearing capacity	Bearing capacity test	10 Spots Load plate ϕ 30, 60,100 cm $\Delta\sigma_n$=5 kgf/cm^2	4 Groups Load plate ϕ 30 cm, $\Delta\sigma_n$=5 kgf/cm^2	2 Groups Load plate ϕ 30 cm	16 Spots Load plate ϕ 30, 60 cm	12 Spots Load plate ϕ 30 cm	Vertical direction: 4 spots; horizontal direction: 4 spots; load plate ϕ 30 cm	Yield value, limit bearing capacity
	Step load creep strength test C (3-axial accelerated creep test)	2 Groups ϕ=50 mm, h=125 mm σ_3=0,1,3,6,10 kgf/cm^2 $\sigma_1-\sigma_3$=2.5 kgf/cm^2	5 Pieces ϕ=50 mm, h=120 mm σ_3=0, 1, 3, 6, 10 kgf/cm^2 $\sigma_1-\sigma_3$=2.5 kgf/cm^2	—	—	—	Mudstone: 6 pieces; sandstone: 6 pieces ϕ=50 mm, h=100 mm σ_3=4, 6, 8 kgf/cm^2 $\Delta\sigma_1$: Mudstone 10 kgf/cm^2 : Mudstone 5 kgf/cm^2	Upper limit yield strength (long-term bedrock bearing capacity)
	Uniaxial creep failure test	3 Groups ϕ=50 mm, h=125 mm ϕ_n=22.5, 20.0, 17.5, 15.0 kgf/cm^2 Load	—	—	—	—	—	Uniaxial creep failure strength

Table 3.3.2-1(b) *(Cont'd)*. Example (2) of survey/test of foundation ground of nuclear reactor building.

	Item	Point A (mudstone)	Point B (mudstone)	Point C (ryholite)	Point D (granite)	Point E (sandstone, gravel)	Point F (sandstone, mudstone laminated with each other)	Results
Evaluation of slide	Bedrock shear test	6 Spots 600×600×300 mm $\Delta\sigma_n$=5 kgf/cm^2	4 Spots 600×600×200 mm σ_V=0~30 kgf/cm^2 $\Delta\sigma_a$=2.5 kgf/cm (30 min)	6 Spots 700×700 mm σ_V=0–30 kgf/cm^2 (6 stages)	Block: 4 spots; Lock: 3 spots 600×600 mm	Block: 5 spots; Lock: 3 spots 600×600 mm	Block flow meshes (4 pieces); attachment meshes (4 pieces)	Failure surface
	High-pressure 3-axial compression test (block sample)	7 Groups ϕ=50 mm, h=125 mm ϕ_3=0, 1, 3, 6, 10, 20, 40 kgf/cm^2	15 Spots (CU) ϕ=50 mm, h=120 mm σ_3=0, 1, 3, 6, 10, 30 kgf/cm^2	—	—	—	—	Failure surface
	Tensile test	6 Groups (pure tensile), 0.1 kgf/cm^2/min	5 Pieces (compressive failure) ϕ=50 mm, h=100 mm	—	—	—	—	Failure surface

Table 3.3.2-1(b) *(Cont'd)*. Example (2) of survey/test of foundation ground of nuclear reactor building.

	Item	Point A (mudstone)	Point B (mudstone)	Point C (ryholite)	Point D (granite)	Point E (sandstone, gravel)	Point F (sandstone, mudstone laminated with each other)	Results
Evaluation of settlement	Bedrock deformation test	10 Spots Load plate ϕ 60, 100 cm $\Delta\sigma_n=2$ kgf/cm^2 (15 min)	4 Spots Load plate ϕ 60 cm, $\Delta\sigma_n=2$ kgf/cm^2 (15 min)	7 Spots ϕ 30 cm, $\sigma_V=100$ kgf/cm^2, 50 cm, $\sigma_V=50$ kgf/cm^2, 100 cm, $\sigma_V=20$ kgf/cm^2	16 Spots Load plate ϕ 30, 60 cm	12 Spots Load plate ϕ 30 cm	Vertical, ϕ 30 cm, 4 pieces; horizontal, ϕ 30 cm, 4 pieces; vertical, ϕ 80 cm, 2 pieces	Secant elastic modulus, tangential elastic modulus, deformation coefficient
	Bedrock creep test	2 Spots Load plate ϕ 30 cm, $\sigma_n=6$, 12, 6, 0 kgf/cm^2, lasting for about 7 days	1 Spot Load plate ϕ 60 cm, $\sigma_n=6$ kgf/cm^2, 4-12 months	—	—	—	2 Spots Load plate ϕ 80 cm, $\Delta\sigma_n=16$ kgf/cm^2, 150 days	Creep coefficient

Table 3.3.2-1(c). Example (3) of survey/test of foundation ground of nuclear reactor building.

	Item	Point A (mudstone)	Point B (mudstone)	Results
Evaluation of settlement	High-pressure 3-axial compression test (block sample)	7 Groups (UU) $\phi=50$ mm, h=125 mm $\sigma_3=0,1,3,6,10,20,40$ kgf/cm^2	15 Pieces (W) $\phi=50$ mm, h=120 mm $\sigma_3=0,1,3,6,10,30$ kgf/cm^2	Deformation coefficient
	3-axial compression test (block sample), crude-grained tuff	Each 3 groups (CU) $\phi=35$ mm, h=80 mm $\sigma_3=0,1,3,6$ kgf/cm^2	—	Deformation coefficient
	3-axial creep test	2 Groups (UU) $\phi=50$ mm, h=125 mm $\sigma_1-\sigma_3=6$ kgf/cm^2 ($\sigma_3=0,0.5,1.5,3.0$ kgf/cm^2) $\sigma_1-\sigma_3=12$ kgf/cm^2 ($\sigma_3=0,1,3,6$ kgf/cm^2)	4 Pieces (UU) $\phi=50$ mm, h=120 mm $\sigma_1-\sigma_3=6$kgf/cm^2 $\sigma_3=0.1,3,6,10$ kgf/cm^2 30 ~ 40 Days	Creep coefficient
	Consolidation test	3 Groups P=2.5,5,10,20,40,160 kgf/cm^2	5 Pieces $\phi=50$ mm, h=100 mm P=Same as left	Consolidation yield stress
	Measurement of Poisson ratio (3-axial UU test)	1 Group $\phi=50$ mm, h=125 mm $\sigma_3=0,1,2,3$ kgf/cm^2	—	Poisson ratio
Others	Dynamic shear test	10 Pieces $\phi=108$ mm, h=30 mm Load: equivalent to load of soil cover $\gamma=10^{-5}\sim10^{-3}$ (1 Hz) $\gamma=10^{-4}$ (0.1,1,10 Hz)	9 Pieces $\phi=100$ mm, h=40 mm Same as left γ : Same as left γ : Same as left	Dynamic shear modulus (G) Damping (h) $G/G_0\sim\gamma$ $h\sim\gamma$
	Initial stress measurement	Horizontal two directions, vertical	—	Three-dimensional stress component. Check for presence of structural stress.
	3-axial compression test using layer boundary as parameter (UU)	3 Groups $\phi=35$ mm, h=80 mm $\sigma_3=1,3,6,10$ kgf/cm^2	—	Check of presence of strength anisotropicity with layer boundary as parameter.

Table 3.3.2-1(c) *(Cont'd)*. Example (3) of survey/test of foundation ground of nuclear reactor building.

	Item	Point A (mudstone)	Point B (mudstone)	Results
Others	3-axial compression test with water content ratio as parameter (UU)	1 Group $\phi=50$ mm, h=125 mm $\sigma_3=0,1,3,6,10$ kgf/cm^2	—	c, $\phi \sim$ w
	Uniaxial compression test with water content ratio as parameter	15 Pieces $\phi=50$ mm, h=125 mm	—	$q_u \sim$ w
	Ultrasonic velocity test with water content ratio used as parameter	3 Pieces $\phi=50$ mm, h=50 mm	—	V_P, $V_S \sim$ w
	Ultrasonic velocity test with propagation distance used as parameter	4 Pieces $\phi=50$ mm, h=20 ~ 60 mm	—	V_S, $V_P \sim$ Propagation distance

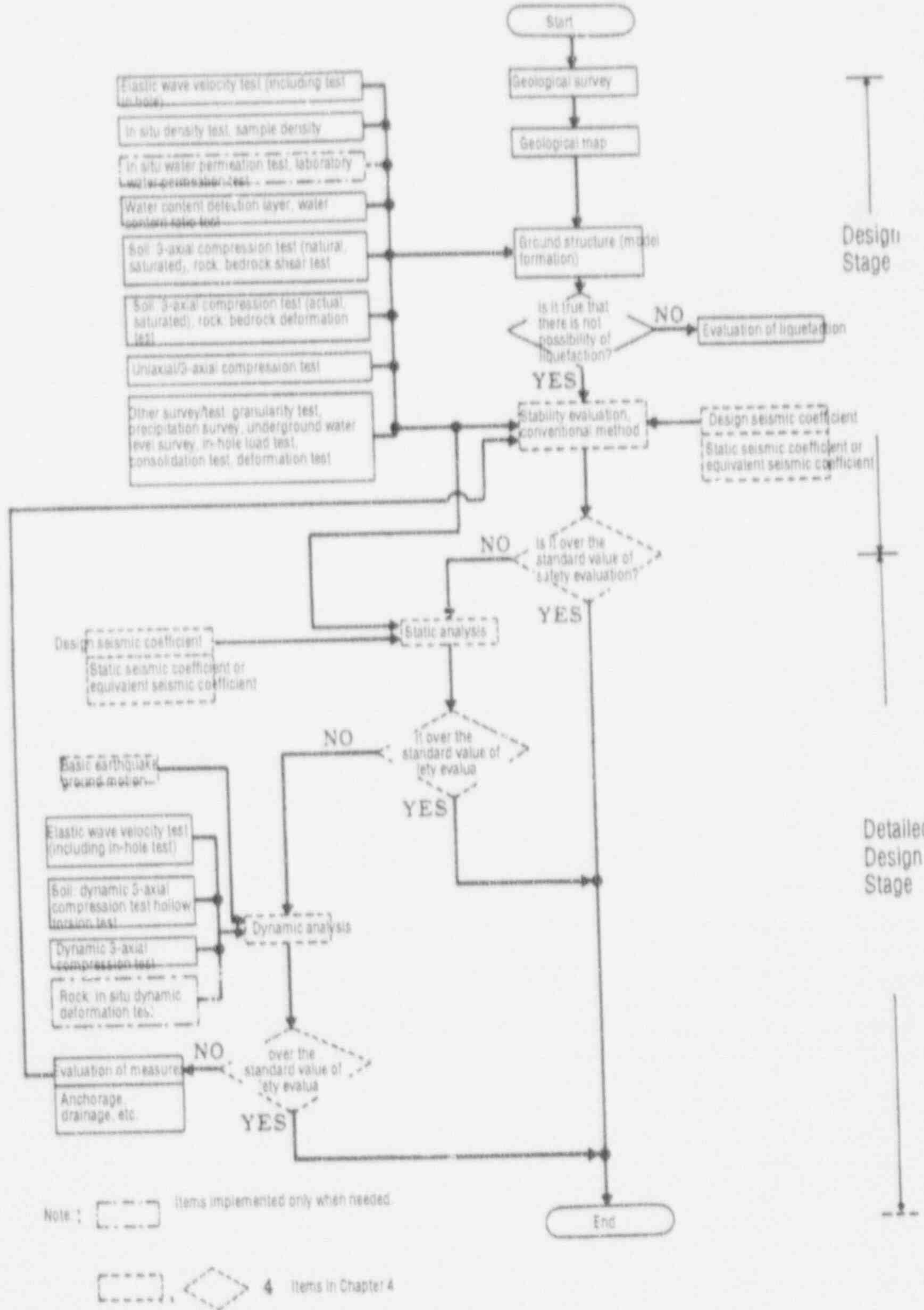


Figure 3.3.2-2. Example of flow chart of survey/test and design of the ground of the peripheral slope of the nuclear reactor building [3.1-1].

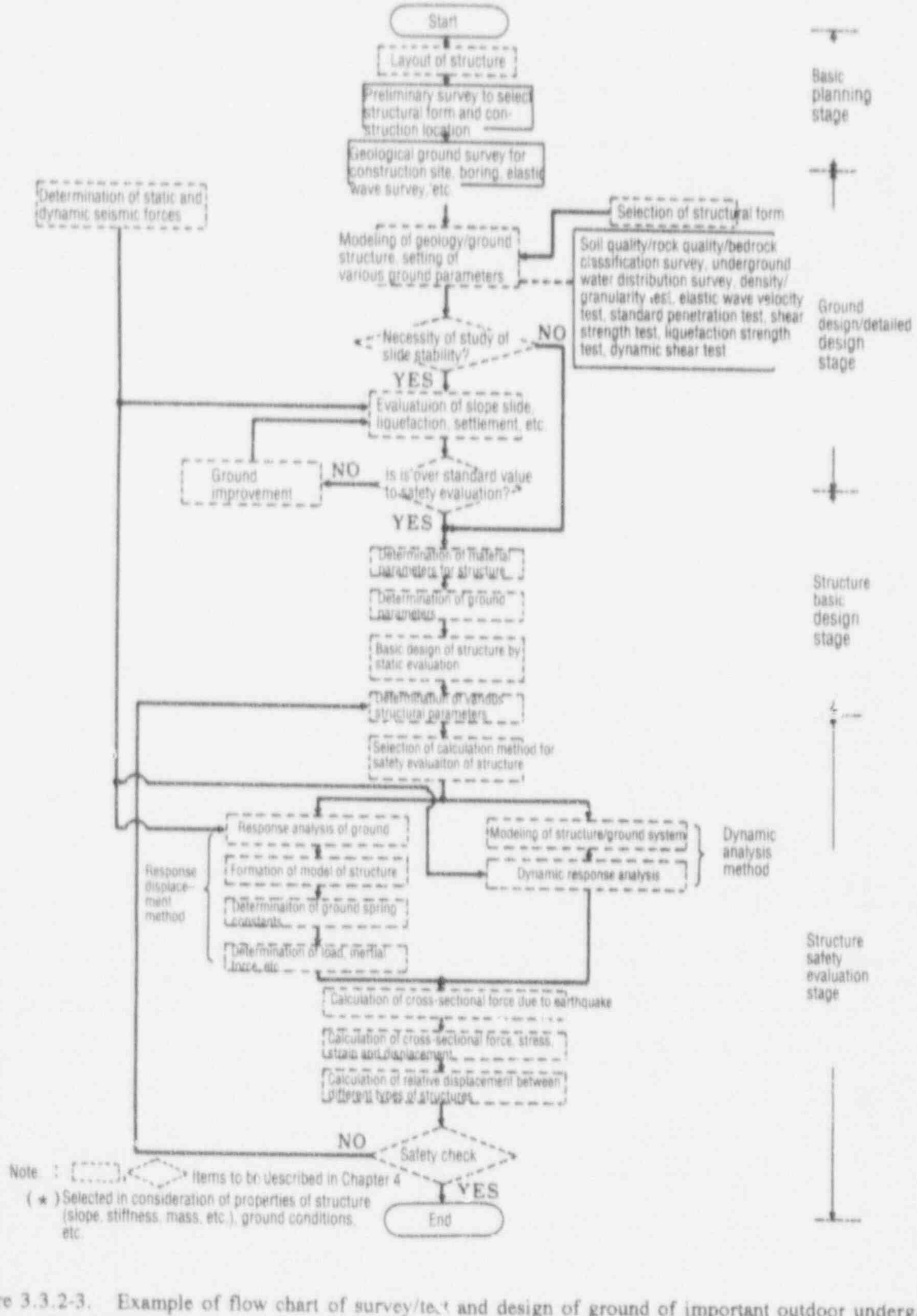


Figure 3.3.2-3. Example of flow chart of survey/test and design of ground of important outdoor underground structure [3.1-1].

b. Survey range

The survey range is determined according to the size and shape of the structure, geological structure, etc. The intake pit (pump chamber) usually has 2-4 spots for boring. For the seawater duct and intake path, boring shall be performed at an appropriate interval. The depth of boring is determined, in principle, to reach the common ground with the foundation of the nuclear reactor building.

3.3.3 Properties needed for stability investigation

The main portion of the stability investigation for the ground is the evaluation of sliding failure. The evaluation of ground bearing capacity and settlement, except conventional methods, may also be included in the evaluation of sliding failure in many cases. In addition, in the case when an important outdoor underground structure is built on sandy ground, it is necessary to evaluate stability with respect to liquefaction. For the seawater intake structure, since it is usually long and large, it is necessary to evaluate the differential displacement during an earthquake. The soil properties which are used to evaluate the stability are mainly as follows:

{1} Weight of unit volume
{2} Static strength characteristics
{3} Static deformation characteristics
{4} Dynamic strength characteristics
{5} Dynamic deformation characteristics
{6} Damping constant

For practical analysis, the properties selected should be most suitable for the characteristics of the ground and the analysis method. Correspondingly, the survey/test combination also depends on the specific case. Tables 3.3.3-1 through 3.3.3-3 list the existing examples of combinations of analysis method, physical properties and survey/tests used. These can be explained as follows.

(1) Static strength characteristics

The static strength is usually represented by the Mohr-Coulomb failure criteria in the form of a simple straight line. For the bedrock, the shear strength and internal friction angle derived in bedrock shear test and 3-axial compression test are used.

(2) Static deformation characteristics

In the static analysis with bedrock as the object, the bedrock is usually taken as an elastic body and E_s (secant elastic modulus) due to bedrock deformation test (plate load test) is used in many cases. In the case of soft rock, in addition to that described above, E_i (initial slope of stress-strain curve) or E_{50} (slope of straight line between the point of 1/2 maximum axial stress on stress-strain curve and origin) is used in some cases.

Table 3.3.3-1. Example of stability evaluation methods, properties needed for evaluation methods, and combinations of tests for determining the properties (foundation ground and slope (rock bed)) [3.1-1].

Analysis method		Properties						Note
		Unit volume weight	Static strength constant		Static elastic modulus	Static Poisson ratio	Ultimate bearing capacity	
		T (tf/m^3)	C, C' (tf/m^2)	ϕ, ϕ' (degree)	E (tf/m^2)	ν		
Conventional method (seismic coefficient method) (slip-surface method)		Boring core sample, block sample	Bedrock shear test	Same as left				
Static analysis (long-term)		Same as above	Bedrock shear test, 3-axial compression test	Same as left	Plate load test (secant stiffness), 3-axial compression test (E_1, E_{50})	Uniaxial compression test, 3-axial compression test		
Static analysis (during earthquake)		Same as above	Same as above	Same as above	Same as above	Same as above		
Dynamic analysis		Same as above	Same as above	Same as above				
Bearing capacity	Conventional method (seismic coefficient method)	Same as above	Same as above	Same as above			Bearing capacity test	In the case of soft rock— long-term: upper limit yield value; in earthquake: the ultimate bearing capacity is used separately
Settlement	Conventional method				Plate load test (secant stiffness)	Uniaxial compression test, 3-axial compression test		Elastic theory (according to Boussinesq equation)

Table 3.3.3-1 *(Cont'd)*. Example of stability evaluation methods, properties needed for evaluation methods, and combinations of tests for determining the properties (foundation ground and slope (rock bed)) [3.1-1].

Analysis method		Properties							Note
		Dynamic strength constant		Dynamic elastic coefficient	Dynamic Poisson ratio	Elastic shear modulus	Damping coefficient	Creep coefficient	
		C_d, C_d' (tf/m^2)	ϕ_d, ϕ_d' (degree)	E_d (tf/m^3)	ν_d	G, G_0 (tf/m^2)	h (%)	α, β	
Conventional method (seismic coefficient method) (slip-surface method)									
Static analysis									
Static analysis (during earthquake)		Static strength is often used	Same as left	Elastic wave velocity test, dynamic shear test, dynamic 3-axial compression test	Elastic wave velocity test (V_P, V_S)	Same as the item of dynamic elastic coefficient			
Dynamic analysis		Same as above	Same as above	Same as above	Same as above	Same as above	Conventionally used values, dynamic 3-axial compression test, dynamic shear test		
Bearing capacity	Conventional method (seismic coefficient method)								In the case of soft rock— long-term: upper limit yield value; in earthquake: the ultimate bearing capacity is used separately
Settlement	Conventional method							In situ creep test	Elastic theory (according to Boussinesq eq.)

Table 3.3.3-2. Example of stability evaluation methods, properties needed for various evaluation methods, and combinations of tests for determining the various properties (foundation ground and slope (fault rupture zone and soil material)) [3.1-1].

Analytical method	Properties					Note
	Unit volume weight	Static strength constant		Static elastic modulus	Static Poisson ratio	
	γ (tf/m^3)	C, C' (tf/m^2)	ϕ, ϕ' (degree)	E (tf/m^2)	ν	
Conventional method (seismic coefficient method) (slip-surface method)	Block sample, in situ density test	3-axial compression test	Same as left			
Static analysis (long-term)	Same as above	3-axial compression test, 1-plane shear test, in situ shear test	Same as left	3-axial compression test (E_i, E_{50}), simple shear test (E_{50})	3-axial compression test, conventionally used values	
Static analysis (in earthquake)	Same as above	Same as above	Same as above			
Dynamic analysis	Same as above	Same as above	Same as above			

Table 3.3.3-2 *(Cont'd)*. Example of stability evaluation methods, properties needed for various evaluation methods, and combinations of tests for determining the various properties (foundation ground and slope (fault rupture zone and soil material)) [3.1-1].

Analytical method	Properties						Note
	Dynamic strength constant		Dynamic elastic modulus	Dynamic Poisson ratio	Elastic shear modulus	Damping constant	
	C_d, C_d' (tf/m^2)	ϕ_d, ϕ_d' (degree)	E_d (tf/m^2)	ν_d	G, G_0 (tf/m^2)	h (%)	
Conventional method (seismic coefficient method) (slip-surface meth-od)							
Static analysis (long-term)							
Static analysis (in earthquake)	Static strength is often used	Same as left	Ultrasonic velocity test, dynamic shear test, dynamic 3-axial compression test	Conventionally used values, ultra-sonic velocity test	Same as the item of dynamic elastic coefficient		
Dynamic analysis	Same as above	Same as above	Same as above	Same as above	Same as above	Conventionally used values, dy-namic shear test, dynamic 3-axial compression test	

Table 3.3.3-3. Example of stability evaluation methods, properties needed for the various evaluation methods, and combinations of tests for determining the various properties (ground of important outdoor underground structure) [3.1-1].

	Properties					
	Unit volume weight	Static strength constant		Dynamic strength constant		
Analytical method	γ (tf/m^3)	C, C' (tf/m^2)	ϕ, ϕ' (degree)	C_d, C_d' (tf/m^2)	ϕ_d, ϕ_d' (degree)	Note
Soil pressure	Block sample, in situ density test	3-axial compression test	Same as left			
Dynamic analysis	Same as above	Same as above	Same as above	Static strength is used in many cases	Same as left	
Response displacement method	Same as above					
Liquefaction	Same as above					

Table 3.3.3-3 *(Cont'd).* Example of stability evaluation methods, properties needed for the various evaluation methods, and combinations of tests for determining the various properties (ground of important outdoor underground structure) [3.1-1].

Analytical method	Properties						Note
	Dynamic elastic modulus	Dynamic Poisson ratio	Elastic shear coefficient	Damping constant	Spring constant	Liquefaction strength	
	E_d (tf/m^2)	ν_d	G, G (tf/m^2)	h (%)	k (kgf/cm)	FI, R, I, R	
Soil pressure							
Dynamic analysis	Elastic wave velocity test, dynamic 3-axial compression test, dynamic shear test	Elastic wave velocity test	Same as the item of dynamic elastic coefficient	Conventionally used values, dynamic 3-axial compression test, dynamic shear test			
Response displacement method	Same as above	Same as above	Same as above	Same as above	Static FEM, manual of design of roads/bridges		
Liquefaction	Same as above	Same as above	Same as above	Same as above		Dynamic 3-axial compression test, dynamic shear test	

In the case of elastic analysis for fault rupture belt or soil material, E_i or E_{50} is used obtained from the 3-axial compression test. In the case when analysis is to be performed in consideration of the nonlinearity of the ground, the deformation coefficient for nonlinear representation as a function of the stress or strain based on the stress-strain curve in the 3-axial compression test may be used.

In the case when the earthquake stability evaluation is performed by static analysis, either static deformation coefficient or dynamic deformation coefficient may be used. However, when the former is used, the deformation cannot be directly evaluated. Hence, appropriate judgment should be made in this case.

(3) Dynamic strength characteristics

In the case of stability evaluation by dynamic analysis, for the weak layers or weak ground with a low strength, the dynamic strength should be applied in principle. However, since many factors influence the dynamic strength, it is difficult to make a general definition of the dynamic strength. Consequently, in dynamic analysis, the static strength is used in many cases as the bedrock strength, so long as it is confirmed that "the dynamic strength is not less than the static strength."

(4) Dynamic deformation characteristics

In the dynamic analysis with bedrock as the object, usually it is possible to ignore the nonlinearity of the bedrock. Hence, with the aid of elastic velocity test or laboratory ultrasonic wave velocity test, the dynamic deformation characteristics are determined from the elastic wave velocity (V_P, V_S). These values can also be determined by vibration test or dynamic load test.

For soft rock ground, when a large strain due to seismic motion is estimated, analyses may be performed using nonlinear properties of the ground.

For the fault rupture zone and soil material, when they are analyzed as elastic bodies, the dynamic deformation characteristics determined by elastic wave velocity test or laboratory ultrasonic wave velocity test can be used. However, since the fault rupture zone and soil material display nonlinear deformation characteristics, analysis is performed with the nonlinear characteristics taken into consideration in many cases. In these cases, the strain-dependent deformation characteristics (G-γ relation) derived by dynamic 3-axial compression test, dynamic shear test, etc., are applied.

(5) Damping characteristics

In the case when bedrock is analyzed and in the case when fault rupture belt and soil material are analyzed as elastic bodies, the values conventionally used for these materials are often used. However, in the case when nonlinear analyses are needed, just as in the above G-γ relationship, the strain-dependent h-γ relationship derived by dynamic 3-axial compression test and dynamic shear test is applied.

3.3.4 Classification of soil and engineering characteristics and evaluation

(1) Soil classification

When the modeling for studying the stability of ground is the primary purpose, grounds can be classified from the engineering point of view into the following types: {1} isotropic ground, {2} anisotropic ground, and {3} heterogeneous ground. For details of the geological classification of ground, please see "Section 3.2.3(3) Bedrock classification."

(2) Engineering characteristics and evaluation of soil

a. Isotopic ground

When the engineering characteristics are evaluated, the plate bearing test and elastic wave velocity test for the deformation characteristics are mainly carried out. If needed, rock test and borehole loading test are also performed. For the strength characteristics, bedrock shear test is mainly performed. If needed, the rock test is also performed. To evaluate the characteristics in the depth direction, classification by observation of boring core, rock test of core, borehole elastic wave velocity test, and borehole loading test are performed to determine the deformation and strength characteristics. In addition, for the foundation rocks which shows local inhomogeneity such as those containing gravel and the foundation rocks with joint system, an evaluation of the results of rock test and selection of the position and size for in situ test should be carried out very carefully. For example, for the size of the loading plates used in the various tests, the diameter or side length of the plate should be 5-6 times the maximum size of the gravel particles.

b. Anisotropic ground

The tests should be performed in consideration of the preferred direction of the joint and the direction of stratification. For in-situ shear and deformation tests of a bedrock, two to three tests are needed with different angles to the plane of stratification of the rock in order to evaluate the strength/deformation anisotropicity due to the angle between the loading direction and the direction of the stratification plane. The following are several examples of measurement results of anisotropicity of bedrock.

(i) Measurement examples of anisotropicity of shear strength

Figures 3.3.4-1 shows an example of the results of a shear test of stratified bedrock. It can be seen that the shear strength depends significantly on the angle of the stratification surface. That is, the strength with a small angle with respect to the stratification (marked by x in the figure) is less than that in the case with a large angle with respect to the stratification (marked by o in the diagram).

(ii) Example of measurement of anisotropicity of elastic modulus

Table 3.3.4-1 lists examples of elastic modulus of the bedrock determined by the plate bearing test. In the case of this foundation rock, the development of schistosity is remarkable and the elastic modulus obtained by the loading parallel to the schistosity plan is larger than that by the loading perpendicular to the schistosity plane. Figure 3.3.4-2 shows an example of anisotropicity caused by the joint surface. This diagram was determined by performing 3-axial compression testing of a rock sample cut out from the bedrock. It can be seen that as the angle between the load direction and joint surface decreases, the elastic modulus increases.

For a ground made of strata of material of different properties, if the strata are relatively thick, for each stratum, the engineering characteristics are derived as isotropic properties. When analysis is to be made, the anisotropicity is taken into consideration. The various test items used for evaluation are the same as those for an isotropic ground.

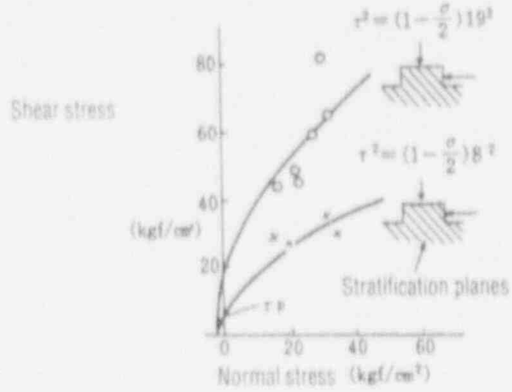


Figure 3.3.4-1. Shear characteristics of anisotropic bedrock (shale) [3.3.4-1].

Table 3.3.4-1. Elastic modulus of anisotropic bedrock (in the case of stratified rock).
(Units: 10^4 kgf/cm², tangential stress under load: 60 kgf/cm²)

Load direction	Bedrock grade*			
	B	C_H	C_M	C_L
Parallel to schistosity	20.5	13.1	4.8	2.3
Perpendicular to schistosity	13.7	7.8	3.8	0.9

*Bedrock classification of Central Power Research Institute.

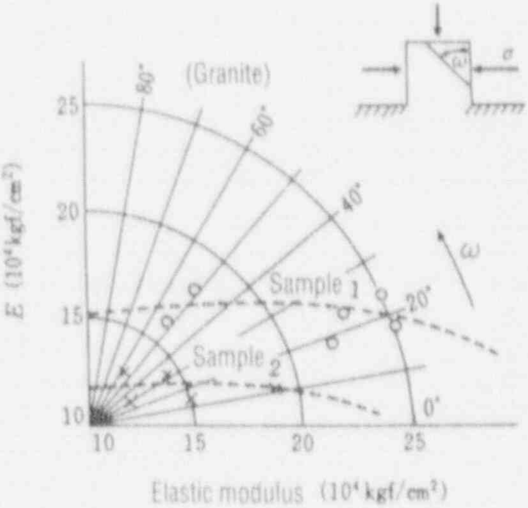


Figure 3.3.4-2. Anisotropicity in elastic modulus of bedrock due to joint surface [3.3.4-3].

c. Heterogeneous foundation rocks

The heterogeneous foundation rocks are characterized as the foundation rocks consisting of different type and class of rocks and/or that consisting of rocks showing locally different extents of weathering and alteration. Consequently, in the case when evaluation is to be made of the engineering characteristics, the ground is zoned according to the geological structure. For each zone, the test methods used to evaluate isotropic ground are adopted, respectively. The heterogeneity can be accounted for by using different properties for each zone in analysis.

(3) Classification of ground constituent materials

a. Classification

The constituent materials of the ground can be classified as follows from the engineering point of view.

{1} Hard rock
{2} Weathered rock, deteriorated rock
{3} Soft rock
{4} Soil
{5} Weak strata (fault rupture zone, etc.)

In some cases, weathered and deteriorated rock are included in the class of soft rocks. However, it may be more convenient to categorize them in separate classes for engineering evaluation. Hence, the above five classes are used.

b. Definitions of various ground constituent materials

{1} Hard rock

Hard rock refers to rock with uniaxial compression strength usually higher than about 500 kgf/cm^2. Also, rock with uniaxial compression strength of about 200-500 kgf/cm^2 which is sometimes referred to as intermediate hard rock is also taken as hard rock in this guideline.

{2} Weathered rock, deteriorated rock

Rock which has been made fragile due to weathering effects is called weathered rock. Rock which has deteriorated due to actions of heat and hot water is called deteriorated rock. Progress in the weathering process depends on the type of rock. For new-period rocks, since there exist few joints, fault rupture zones and other geological separation surface, weathering makes progress from the surface layer. Consequently, the interior portion remains fresh. On the other hand, for old-period rocks, since there are many geological separation surfaces, weathering takes place not only from the surface layer, but also along the geological separation surfaces to the deep interior portion. As a result, the interior may not be as fresh as in the aforementioned case. Chloritized rock, carbonated rock, zeolitized rock, hot-water clay, and other rocks deteriorated by heat or hot water are all weaker than the original rock. On the other hand, hornfels, silicated rock, propylite, etc., are harder and finer than the original rock. They can be treated as hard rocks.

{3} Soft rocks

Soft rocks mainly refer to sedimentary rocks from the Tertiary epoch. The mechanical characteristics of soft rocks are intermediate between soil and hard rocks. The major feature of soft rocks is that they are more consolidated than soil, while their gaps are larger than hard rocks and the soft rocks are weaker in physicochemical function than the hard rocks. The uniaxial compression strength is less than about 200 kgf/cm^2.

{4} Soil

Soil refers to a sediment or loose deposit state of solid particles formed by physical and chemical weathering of rock. In some cases, it may also contain organic substances.

{5} Weak strata (fault rupture zone, etc.)

The fault rupture zones and other weak strata are generated in the tectonic movement, etc. The width of the zone is in the range of several mm to several hundred m. However, since large zones are avoided in the selection of foundation ground, we are concerned only with those with width in the range of several mm to several m.

(4) Engineering characteristics and evaluation of soil constituent materials

a. Hard rock

In many cases, joints are developed in hard rocks; therefore, it is desirable to evaluate the foundation rocks mainly based on in-situ rock tests supplemented by the results of laboratory rock tests. When various bedrock tests are implemented for the bedrock with developed joints, arrangement is made to enable the average joint influence to be taken into consideration in the test.

b. Weathered rock, deteriorated rock

There is no test method dedicated only to weathered rock and deteriorated rock. Hence, the various test methods for hard rock and soil test methods are used in consideration of the hardness of the rock. During the evaluation, it is necessary to perform analysis of the mineralogic observation, geological factors and origins.

c. Soft rocks

As far as formation is concerned, soft rocks cannot be clearly classified from the hard rocks. Cracks caused by schistosity and joints, which are characteristics of hard rocks, can also be observed in soft rocks. However, their influences on the mechanical properties are not as large as in the case of hard rocks. Usually, since the results of rock test are in high agreement with the results of in-situ bedrock test, survey/test of the soft rock is mainly carried out by laboratory test. Of course, if needed, in situ test may also be performed. In some cases it is necessary to determine the creep characteristics.

d. Soil

Soil may be further classified, according to the dimensions of particles contained in it, into gravel, sand, clay (silt, clay), etc. The adhesion among particles is weak in soil. It usually displays a large deformation and may be affected easily by water. In addition, when its structure is disturbed by stirring, its mechanical properties are changed significantly. The engineering characteristics of soil include physical properties, strength characteristics, deformation characteristics, compression characteristics, consolidation characteristics, water permeability, etc. Also, for sandy soil, the liquefaction characteristics usually should be considered.

e. Weak strata (fault rupture zone, etc.)

The fault rupture zone is usually made of fault clay and fault breccia. Usually, fault clay has a lower strength and lower rigidity than fault breccia. Consequently, careful investigation should be performed of the mechanical characteristics of the fault clay. In some cases, evaluation of the mechanical characteristics of fault clay and fault breccia is divided into two portions for survey/test.

As far as survey/test is concerned, for the disturbed sample, the physical test is mainly performed. For the undisturbed sample, the mechanical test is mainly performed. In some cases in-situ tests shall be performed for strength characteristics and deformation characteristics. Table 3.3.4-2 lists the types of tests using undisturbed samples and types of in situ tests. For the disturbed samples, tests shall also be performed according to these methods.

It is very difficult to obtain undisturbed samples by boring. Hence, undisturbed samples collected from the pit are used for laboratory testing. In this case, it is necessary to prevent change in the water content and saturation degree due to boring. For the sample used in laboratory test, if it is forced to saturate, the sample may be disturbed. Hence, test is performed for the saturation degree in the natural state.

For evaluation of the seismic stability of weak strata, when a sufficiently large undisturbed sample can be obtained, the dynamic strength shall be used, or, when it is confirmed that the dynamic strength is not less than the static strength, the static strength can be used. According to the results of the comparison tests performed up to now, in many cases, the dynamic strength is found to be greater than the static strength.

3.3.5 Representation method of properties and application in design

(1) Representation of static strength characteristics

The static strength of a ground is usually represented as a function of normal stress or average principal stress. The strength depends on the type of ground, loading conditions, etc. The following are the major items which should be taken into consideration when the strength characteristics are to be represented.

a. Type of failure criteria

As ground failure is caused by slide, i.e., shear failure, the strength of the ground may be represented as the maximum shear resistance by the ground.

Table 3.3.4-2. Types of tests of undisturbed samples and in situ tests of fault rupture zone and other weak stratum material for evaluation of the bedrock stability [3.1-1].

Subject	Evaluation purpose / Test type					Note
	Long-term stability eval.; stability eval. in earthquake	Long-term stability evaluation		Safety evaluation in earthquake		
Subject	Physical test	Strength test	Deformation test	Strength test	Deformation/ damping test	Note
Weak strata, such as fault rupture zones, etc.	Physical tests (natural water content, saturation degree, plasticity index, density, grain size, etc.)	Single-plane shear, simple shear, 3-axial compression[1]	Simple shear, 3-axial compression, standard consolidation, K_0 consolidation	Dynamic simple shear, dynamic 3-axial compression,[2] or static 1-plane shear, simple shear, 3-axial compression[2]	Dynamic simple shear, dynamic 3-axial compression[2]	Depending on the results of consolidation test, etc., the appropriate test pressure range is determined.
		In some cases, cone penetration, borehole shear, and other in situ tests	In some cases, load in boring hole, and other in situ tests		In some cases, microregion elastic wave velocity test	

[1] Consolidation/drainage test (CD test) is performed; it is also possible to perform consolidation/undrainage test (CU test) with measurement made for the gap hydraulic pressure.

[2] Consolidation/undrainage test (CU test) is performed.

The commonly used failure criteria are as follows:

{1} Mohr-Coulomb failure criteria
{2} Griffith's failure criteria
{3} Modified Griffith's failure criteria
{4} Failure criteria using parabolic representation
{5} Failure criteria using power function representation

Among the above failure criteria, the Mohr-Coulomb criteria is often used in practical applications, since it can be handled in a simple way to evaluate the stability of the ground. In this case, the strength characteristics of the ground can be represented by two strength constants (C, ϕ). In addition, for representation, the range of the stress to be considered should be clarified. Figure 3.3.5-1 illustrates failure criteria {1}-{4}.

b. Total stress representation and effective stress representation

(a) Soil and other ground materials

For a ground material with relatively large porosity, the properties of the material depend on the pressure of water filling the pores. This is particularly important for soil. That is, assuming the total stress applied to saturated soil element is σ, and the porewater pressure is u, then the effective stress σ' can be represented as $\sigma' = \sigma - u$. The effective stress can be used to represent the soil strength in a single defined way independent of the drainage condition and the magnitude of the porewater pressure. However, in the case when the porewater pressure is not clear, representation may be performed using the total stress instead of the effective stress for design and practical application.

As generation and dissipation of the porewater pressure causes variation in the safety factor of the structure or foundation made of the soil, the application ranges are different between the case with total-stress presentation and the effective-stress presentation. Attention should be paid to this feature.

The total stress analysis method based on the undrained shear strength can be used for safety evaluation during the period when the drainage condition of the porewater can be considered as undrained. For example, it

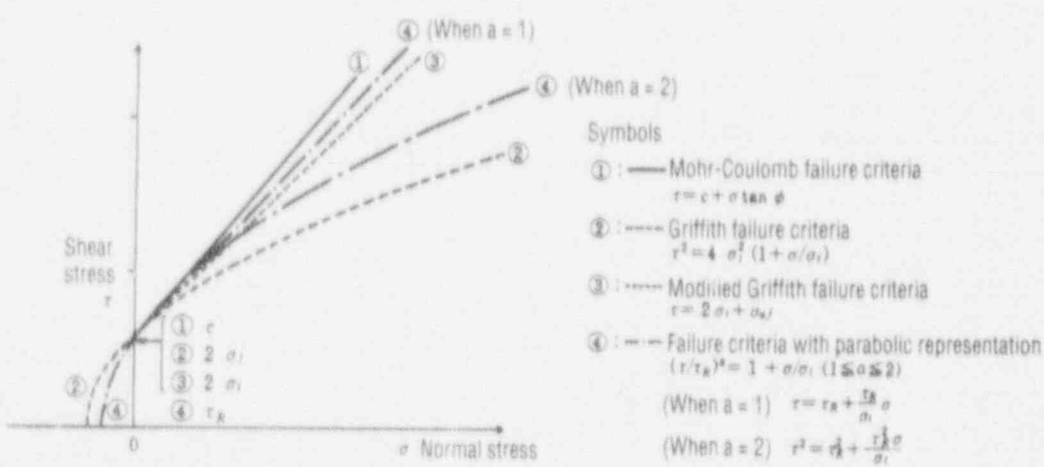


Figure 3.3.5-1. Types and profiles of failure criteria.

can be applied to stability analysis and earthquake analysis for an aquiclude ground under the so-called construction condition in a very short period just after variation in the load conditions (see Tables 3.3.5-1,2).

(b) Hard rock and other ground materials

For the bedrock made of hard rock with fewer pores, the effect of the pore water pressure may be neglected as compared to the level and variation range of the stress under consideration. In this case, the total stress may be used to represent the strength characteristics.

(c) Soft rock

In recent years, it has been found that since soft rock has a relatively high porosity, the concept of effective stress may be applied in some cases.

The behavior of the porewater pressure of soft rock is similar to that of sedimentary clay. Hence, it is necessary to study the long-term stability problem of the slope of saturated soft rock on the basis of the effective stress. However, for soft rock, measurement of the porewater pressure is usually difficult, and the influence of the porewater pressure on the strength deformation characteristics is not as significant as soil when the strength reaches a certain level. As a result, it is necessary to use the concept of total stress and the concept of effective stress respectively in different cases depending on the magnitude of the strength of the soft rock.

Tables 3.3.5-1 and 3.3.5-2 summarize the relations between the type of structure and type of ground as the object of safety evaluation and the test conditions.

c. Short-term strength and long-term strength

In the case when the pore water pressure has a large influence on the strength, as pointed out in section b above, depending on the drainage condition, either the total-stress representation or the effective-stress representation is used for different ranges of application. In this case, variation in the strength caused by generation and dissipation of the pore water pressure is taken into consideration. However, for both the undrainage strength and drainage strength, long-term decrease takes place depending on the stress condition, etc. The decrease pattern can be evaluated by studying the strain ratio effect and the creep strength. For the long-term stability of the slope, Skempton investigated [3.3.5-1] and pointed out the importance of deriving the residual strength. For the short-term strength, please see Section "3.3.5(3) Representation of dynamic strength characteristics."

d. Type of ground and method of deriving local safety factor

The local safety factor is used as an index for determining the potential slide surface from the elements with a small local safety factor, when the safety of the ground is evaluated along the slide surface using the finite element method. For safety evaluation, the safety factor of the entire slide surface can be derived from the safety factors of the elements along the potential slide surface.

As a result, even in the case when the local safety factor is somewhat lower than 1.0, successive calculation can be performed to confirm that no progressive damage takes place. In this way, if it is found that the overall slide safety factor is higher than the prescribed evaluation standard value, the safety of the ground is confirmed.

Stability of the ground should be evaluated for different phases: just after completion [of the structure], long-term, and during earthquake. In this case, as the state of generation of pore water pressure depends on the hardness of the ground as pointed out above, it is necessary to determine the strength of the ground to derive the local safety factor.

Table 3.3.5-1. Types of bedrock and test conditions [3.1-1] (nuclear reactor building foundation ground(1)).

Type of ground(2)			Load condition: Long-term	Load condition: In earthquakes(6)
Soft rock(3)	A(4)	Soft rock in saturated state and with $q_u < 20$ kgf/cm^2	Effective-stress strength is used as the strength; derived from CU test or CD test	Total stress strength is used as the strength; derived from CU test
	B(5)	Soft rock in saturated state with $q_u > 10$ kgf/cm^2, and soft rock in unsaturated state	Total-stress strength is used as the strength; derived from bedrock shear test, uniaxial compression test, 3-axial unconsolidated undrainage test (U[illegible]t), CU test, tensile (pressing cracking) test, etc.	Same as left
Hard rock			Same as above	Same as above

(1)The ground of important outdoor underground structures is defined in Tables 3.3.5-1, 3.3.5-2.

(2)The strength of the rupture zone material is determined accoring to soft rock A.

(3)The classification scheme of the soft rock from the geological point of view is discussed in Section "3.3.2(3) II Bedrock classification." However, this scheme is not adopted here. Instead, the soft rocks are roughly divided into types A and B with regard to the test method (total stress analysis method or effective stress analysis method) used, from the standpoint of test performance.

(4)Soft rocks which are relatively soft and belong to Type II soft rock and Type III soft rock in saturated state.

(5)Soft rocks which belong to Type I soft rock or Type II and Type III soft rocks in unsaturated state or having a relatively large consolidation degree.

(6)It is confirmed that the dynamic strength is not less than static strength.

Table 3.3.5-2. Types of ground and test conditions [3.1.1] (excavation slope, banking slope).

Type of ground			Load condition / Strength needed for stability analysis			
			During and just after building of structure		After construction of structure (long-term and seismic stability)	
			Total-stress strength (long-term) determined by in situ bedrock shear test, UU test, or uniaxial compression test	Total-stress strength (long-term) determined by CU test; use CD strength for sandy soil and gravel	Total-stress strength (both long-term and seismic) determined by in situ bedrock shear test, UU test, or uniaxial compression test	Strength determined by CU or CD test; effectiveness strength for long-term strength, total-stress strength for seismic strength
Sandy soil, gravel				○		○ (including liquefaction)
Clay	Normal consolidation and excessive consolidation		○	CU strength is used in some cases[3]		○ [3]
Clay	Excessive consolidation (dry soil with many cracks)[1]		○	Same as above[3]		○ [3]
Backfill soil material (unsaturated)			○			○
Soft rock[2]	A	$q_u < 20$ kgf/cm^2 and in saturated state	○			○
Soft rock[2]	B	$q_u > 10$ kgf/cm^2 or in unsaturated state	○		○	
Hard rock			○		○	
Note			For the banking slope, this case is usually most dangerous. Hence, investigation is mainly performed in this case.		For the excavation slope, this case is usually most dangerous. Hence, investigation is mainly performed in this case.	

[1]It is necessary to take the influence of cracks into consideration.

[2]See note for classification of soft rocks in Table 3.3.5-1.

[3]In the case of excavation, the strength is determined considering the decrease in strength due to swelling caused by water absorption under long-term loads in some cases.

For grounds made of saturated soil material, fault rupture zone or other weak stratum material, and relatively soft rocks (with $q_u < 20$ kgf/cm^2), since the pore water pressure may rise easily, this feature must be taken into consideration when the local safety factor is to be determined. In this case, in principle, long-term safety evaluation is performed using the strength, deformation coefficients under the consolidation drainage condition. In the conventional 3-axial compression test, the strength in the case of consolidated drainage is higher than the CU strength. In the case when it is difficult to perform the CD test and CU tests for the ground, if there exists a safety margin, the CU strength is to be used to study the sliding safety of the structure foundation ground. However, for a swelling ground, the average principal stress decreases due to cutting and digging, and the strength decreases due to water absorption and swelling. Hence, for a certain stress range, it becomes CD strength < CU strength. For such ground, it is necessary to determine the local safety factor by the strength determined under the CD condition. Safety evaluation during earthquake is usually performed by determining the local safety factor from the strength determined under the CU condition. In this case, the shear strength corresponding to the long-term stress (such as the average principal stress) is believed to be applicable to seismic condition; hence, it is appropriate to determine the local safety factor from the ratio of the above strength to the shear stress during earthquake.

On the other hand, for hard rock and relatively hard soft rock, the generation rate of porewater pressure is low; and there exists a sufficient margin of strength. Consequently, the local safety factor can be determined from the total-stress strength from the in-situ bedrock shear test and uniaxial/3-axial compression test for both long-term and seismic conditions. The same applies to the case of unsaturated ground. For soft ground, however, when the sample is collected, as the stress is released, the state is disturbed manually. In addition, the status of generation of the porewater pressure depends on the saturation degree and confinement pressure in a complicated pattern. Hence, it is desired that the strength under the CU condition similar to the actual ground condition be used.

e. Factors that affect the strength

Usually, the static strength can be represented as a function of normal stress or average principal stress. In some cases, however, representation is made considering strain rate and anisotropicity. Hence, in the case when there are factors that affect the strength in the ground, it is necessary to represent the strength with these factors.

(2) Representation of static deformation characteristics

The static deformation characteristics of the ground is usually represented by the stress-strain relation by regarding the deformation behavior of the ground as a continuous body. Depending on the mechanical model of the continuous body adopted, the stress-strain relation can be represented in various forms (Table 3.3.5-3).

For the nonlinear characteristics of the ground, usually based on the stress-strain relation from the 3-axial compression test, the deformation coefficients and Poisson's ratio are represented as functions of stresses [3.3.5-2-3]. In addition, for soft rock, fault rupture zone, and other weak strata, it is necessary to consider the dependence of the deformation characteristics on the confinement pressure. The dependence of the deformation characteristics on the confinement pressure is usually determined from the stress/strain vs. confinement pressure (overburden load in the case of simple shear test). In the case of soft rock, the long-term deformation may become problematic in some cases. In these cases, it is necessary to evaluate the creep characteristics. The creep characteristics are usually represented using the Voigt-Spring model.

(3) Representation of dynamic strength characteristics

a. Strength

Dynamic strength is defined as the strength under a single impact load or under a repeated load of a certain amplitude at a certain repetition frequency. Its value depends significantly on the magnitude of the confinement pressure and the presence of porewater. Hence, it is necessary to study the dynamic strength of ground and its

Table 3.3.5-3. Stress-strain relation and mechanical model.

Mechanical model	Stress-strain relation	Representation of deformation properties	Features
Linear elastic body	$\{\varepsilon\} = [D]^E \{\sigma\}$ $\{\varepsilon\}$: Strain vector $\{\sigma\}$: Stress vector	$[D]^E$ is a stress-strain matrix, made of two elastic constants (such as elastic modulus E and Poisson's ratio)	It is a basic mechanical model and can be applied easily
Nonlinear elastic body	$d\{\varepsilon\} = [D]^E d\{\sigma\}$	$[D]^E$ is made of two elastic constants expressed as the function of stress and strain	It can represent nonlinear deformation behavior
Elastoplastic body	$d\{\varepsilon\} = d\{\varepsilon\}^E + d\{\varepsilon\}^P$ $d\{\varepsilon\}^E = [D]^E d\{\sigma\}$ $d\{\varepsilon\}^P = h\{(\partial g/\partial \sigma)df\}$ Superscripts E,P represent "elastic" and "plastic," respectively	f, g, and h represent yield function, plastic potential function and hardening function, respectively	It can represent irreversible deformation behavior

representation method with the following factors taken into consideration: loading speed effect, inertial force effect, cyclic loading effect, influence of irregular load, magnitude of confinement pressure, saturation state, liquefaction, etc.

(a) Loading speed effect

According to the past experimental results, the strength of sand does not depend on the loading speed. However, as the confinement pressure increases, even for sand, its strength characteristics become dependent on the strain rate [3.3.5-4]. It is well known that the undrainage strength of clay increases as the time to failure becomes shorter. The relation between the load velocity and increase in the strength depends on the plasticity index, water content, excessive consolidation ratio, etc. For rocks, although a clear strain rate effect can be seen with respect to the peak strength, this effect is not as clear for the residual strength.

(b) Inertial force effect

When an earthquake takes place, the force acting on the ground can be divided into the dynamic stress due to the seismic force acting on the micro elements of the ground and the body force, i.e., inertial force directly acting on individual soil particles. Based on the results of researches on the effect of the magnitude of the body force on the strength of dry sand [3.3.5-5], it was reported that under both vertical vibration and horizontal vibration, as the vibration acceleration is increased, the shear strength of dry sand decreases. However, for the acceleration range related to the engineering analysis, the decrease in the shear strength is not significant.

(c) Cyclic loading effect

According to the results of researches on the effect of the magnitude of the dynamic stress on the strength of the dry sand [3.3.5-5], we have

$$\phi_S = \phi_{Sd} < \phi_d \tag{3.3.5-1}$$

where, ϕ_S: ϕ in static test

ϕ_{Sd}: ϕ in the case when an initial shear stress is applied as a repeated load

ϕ_d: ϕ under repeated load after isotopic consolidation.

It can be seen that ϕ of dry sand in repeated load state is larger than ϕ for static load. In addition, when the repetitive load test is performed for clay, after an initial shear stress with one of various magnitudes is applied to the sample, then the vibration load is applied. This is because the dynamic strength depends on the magnitude of the initial shear stress. For rocks, the strength shows almost no decrease at all for the number of cycles typical of an actual earthquake [3.3.5-6].

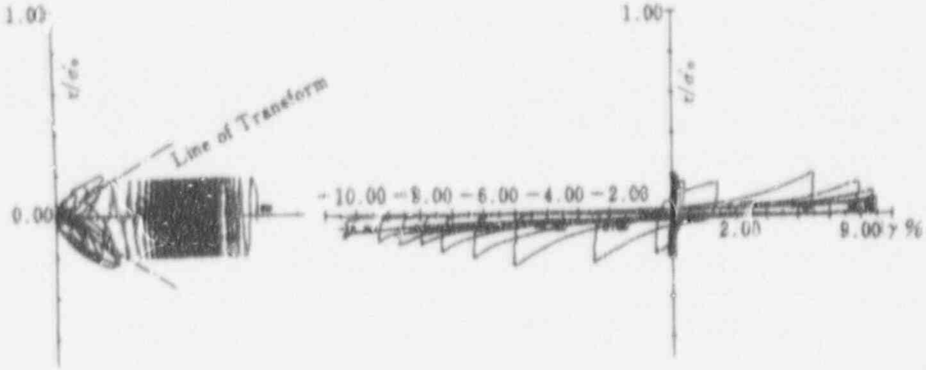
(d) Effect of irregular load

Usually, the dynamic strength under a regular load decreases as the number of cycles increases. For the purpose of studying the difference between the dynamic strength characteristics under a constant stress amplitude and the strength characteristics under an irregular load, the strength-deformation characteristics were studied using a loading time history determined from the actual seismic acceleration waveform of TAFT earthquake (NS component) [3.3.5-6]. As a result, it was found that when an irregular load is applied, the dynamic strength of the soil depends not only on the magnitude of the maximum load, but also significantly on the duration and waveform. On the other hand, according to the test results for rocks, the dynamic strength is not less than the static strength determined by the conventional test.

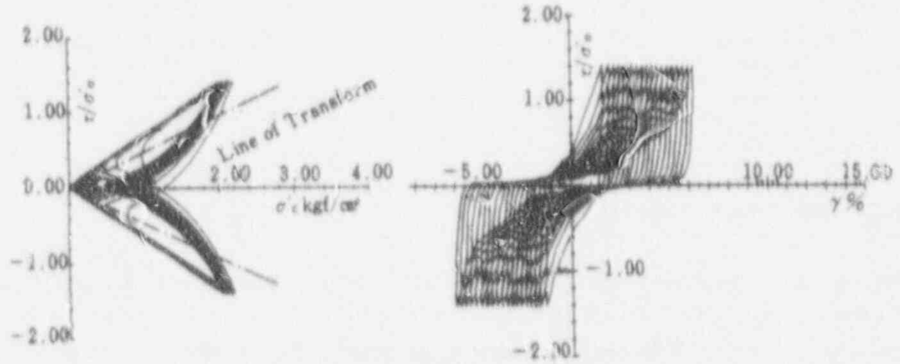
(e) Liquefaction

(i) Loose sand

For saturated sandy ground with a low density, when a repeated shear stress is applied in nearly an undrainage state within a short duration such as an earthquake, as sand particles change relative position to fit each other, the volume tends to decrease and the porewater pressure rises. When the effective stress becomes 0, the shear resistance of the ground is almost totally lost, and a very large strain develops just as in a liquid. This phenomenon is called liquefaction. In addition, due to the upward flow of the underground water caused by the liquefaction of the lower layer, liquefaction is also induced in the upper layer. This is called secondary liquefaction. Figure 3.3.5-2 (1) shows the effective stress path and the stress-strain relationship in the case when a certain stress is applied repeatedly to the loose saturated sand sample [3.3.5-7]. It can be seen that after the stress states passes a certain line (line of transform), the effective stress becomes nearly null as the load is removed. The shear strength is lost and the shear strain increases drastically.



(1) Loose saturated sand sample



(2) Dense saturated sand sample

Figure 3.3.5-2. Effective stress paths (left side) and stress-strain relation (right side) in hollow torsion shear tests for loose sand and dense sand.

(ii) Dense sand

For sandy ground with a high density (for example, with N-value in the standard penetration test of 20-30 or higher), there are few examples of liquefaction caused by past earthquakes. However, according to laboratory tests, even for a dense sand layer with a relative density of about 80% or greater, it is found that the porewater pressure rises and the shear rigidity decreases due to the repeated shear action.

Figure 3.3.5-2(2) shows the effective shear path and stress-strain relation of a dense saturated sand sample under repeated action of a certain stress [3.3.5-7]. It can be seen that even when the stress state passes the line of transformation, a large strain is not generated in the sand, and the shear rigidity is lost only within a limited strain range.

In order to distinguish it from the liquefaction of loose sand, this phenomenon generated in dense sand is usually called cyclic mobility. In consideration of these differences from loose sand, there is no need to be afraid of the danger of complete loss of the shear strength by liquefaction in dense sand. Instead, it is important to evaluate the strain range where the shear rigidity is lost (limit strain), i.e., evaluate the degree of strain generated in the ground due to earthquake.

(iii) Factors that influence liquefaction strength

Factors that influence liquefaction strength of saturated sand include density, confinement pressure, particle size and content of fine particle, consolidation shear loading history, particle's microscopic structure and cementation, initial shear stress, etc. However, it is clear that the frequency has almost no influence on the liquefaction strength of sand itself [3 [illegible]]. On the other hand, since the frequency content of a seismic motion has a large influence [illegible] stress in sandy ground, the frequency content should be considered as one factor for evaluating the possibility of liquefaction.

(iv) Judgment of liquefaction

{1} Method for predicting liquefaction

There are various methods of predicting liquefaction, ranging from empirical methods to laboratory tests and in situ liquefaction tests. Table 3.3.5-4 lists the types of prediction methods and the necessary survey items, tests and analyses. Among these methods, data obtained by standard penetration test, granularity test, and other conventional soil investigation methods are usually used as the simple methods for predicting ground liquefaction. In this method, various standards and guidelines published by various institutions are summarized. As liquefaction of a dense sand ground is evaluated, it is desirable that investigation be performed according to the flow chart shown in Figure 3.3.5-3 with full consideration of the differences compared to loose sandy ground.

{2} Methods of determining liquefaction

For loose sandy ground and dense sandy ground, the liquefaction determining methods illustrated in Figure 3.3.5-4 can be applied. [illegible] liquefaction stress ratio R can be represented by the following formula:

Table 3.3.5-4. Types of liquefaction prediction method and necessary survey items [3.1-1].

Liquefaction predicting method	Necessary survey, test, analysis
Preliminary study based on records of past earthquakes	Collection of earthquake damage records, survey of related information
Preliminary study based on topography and geology	Microtopographic classification, formation of geological section maps
Various standards (Road/Bridge Seismic Design Guidelines, Building Foundation Structure Design Standards, Bay Facility Technical Standards, Building Design Standards and Commentary, Regulations Concerning Dangerous Materials, Road/Bridge Guidelines, Land Reform Business Program Design Standards) and various simple prediction methods	Boring, standard penetration test, sampling granularity test, determination of seismic force
Detailed prediction methods using liquefaction test and seismic response analysis (including th[illegible] which takes the accumulation and dissipation of excessive pore water pressure into consideration)	Boring, standard penetration test, sampling PS detection, granularity test, (maximum/minimum density test), liquefaction test (dynamic 3-axial compression test, etc.), (test for deriving dynamic deformation coefficient), determination of seismic force, seismic response analysis (stability analysis with excessive pore water pressure taken into consideration)
Methods which perform model test (vibration table, etc.), in situ test, etc.	Vibration table test, stake table test, test using explosion

Note: Items within ( ) are not needed in some cases.

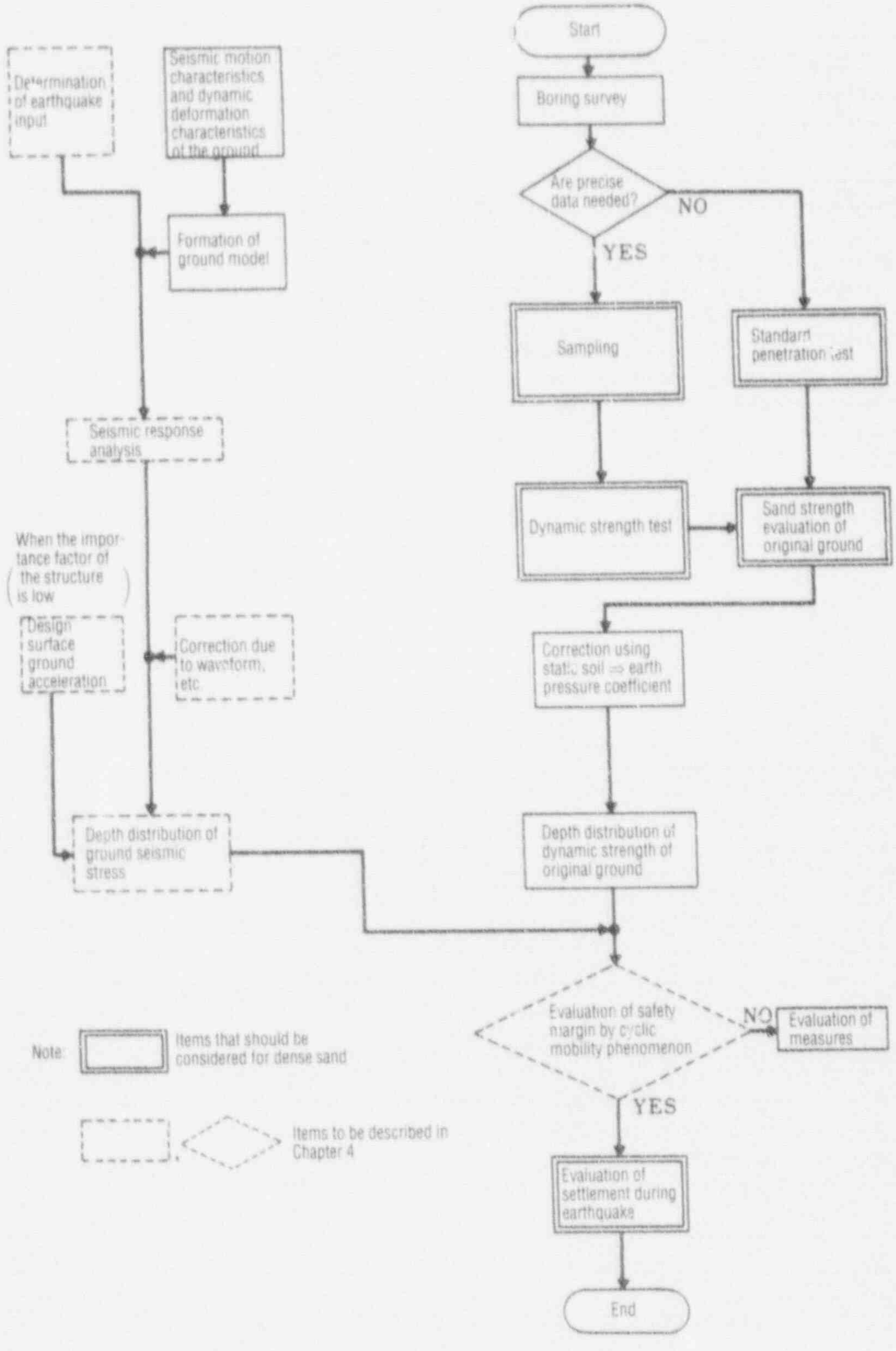


Figure 3.3.5-3. Flow chart of liquefaction judgment method for dense sandy ground [3.1-1].

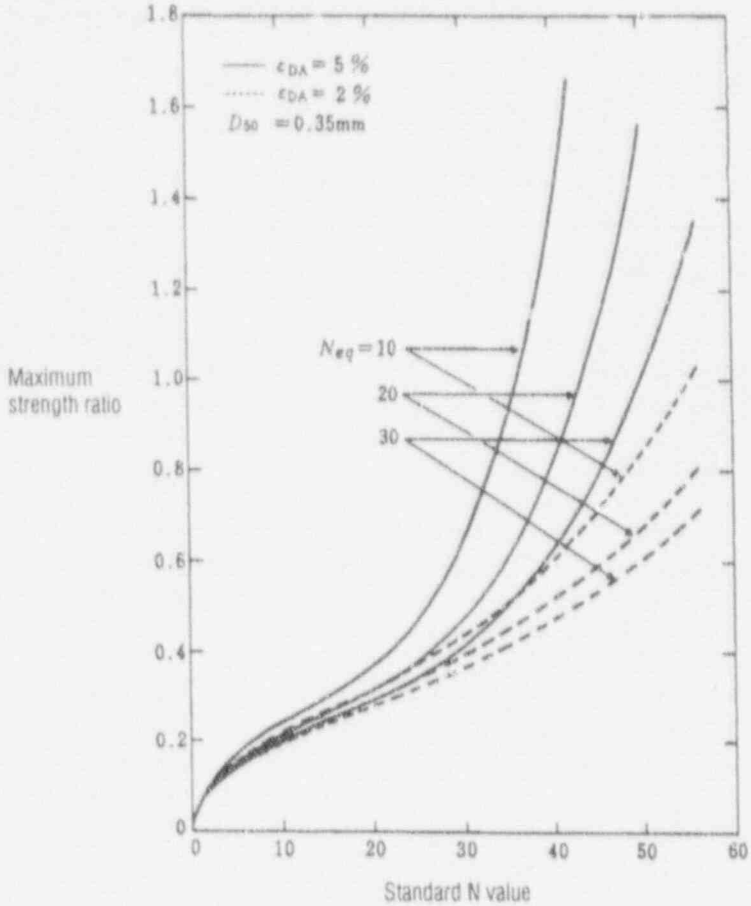


Figure 3.3.5-4. Liquefaction determining curve [3.3.5-9].

$$R = a\left\{N_1^{0.5} + (bN_1)^n + c - 14.8 \cdot f(D_{50})\right\}$$

$$\begin{cases} f(D_{50}) = 0.255\log(D_{50}/0.35) & : \ 0.04 \le D_{50} \le 0.6 \text{ mm} \\ f(D_{50}) = 0.05 & : \ 0.6 \le D_{50} \le 1.5 \text{ mm} \end{cases} \qquad (3.3.5\text{-}1)$$

where a, b, c, and n are constants, which are determined according to reference (3.3.5-9); D_{50} is the average particle size (mm). For dense sand, the error is small even when c = 0.

(4) Representation of dynamic deformation and damping characteristics

a. Representation method of dynamic deformation and damping characteristics

The dynamic deformation and damping characteristics of ground are used to evaluate the behavior of the ground during earthquake. The dynamic deformation and damping characteristics are often represented by elastic shear modulus, Poisson's ratio, and damping constant. They usually depend on the magnitude of the strain generated and the confinement pressure. Hence, for dynamic deformation and damping characteristics as well as their representation method, evaluation should be performed considering the magnitude of strain and the influence of confinement pressure.

Also, in order to study the soil-structure interaction during earthquake or to design the structure considering the ground reaction force, spring constants are used.

(a) Shear modulus of elasticity

(i) Deformation characteristics of soil using linear viscoelastic model

In the viscoelastic model, usually a dashpot which displays a resistance proportional to the velocity is used as the model to represent the damping mechanism. Various combinations of the springs representing the rigidity of the system and the dashpot have been proposed such as Voight type and Maxwell type models.

(ii) Nonlinear deformation characteristics of soil

The relationship between stress and strain of the soil usually exhibits nonlinearity. When the strain generated during an earthquake is sufficiently small, the behavior of the ground can be fully analyzed with elastic assumption. On the other hand, when the strain becomes larger, it is necessary to evaluate the change in the deformation characteristics, i.e., a nonlinear behavior, is needed.

In many cases, the nonlinear dynamic stress vs strain relation of soil is usually represented by the stress-strain relation when a virgin load is applied on the soil (skeleton curve) and the stress-strain loops (hysteresis curves) obtained under prescribed repeated loads on the soil. In the equivalent linear model, the skeleton curve is used to determine the shear elastic modulus, and the hysteresis loops for the damping characteristics. For seismic response analysis, the Hardin-Drnevich model and the Ramberg-Osgood model are often used [3.3.5-10].

(b) Damping constant

(i) Internal damping (material damping)

{1} Damping characteristics of soil according to linear viscoelastic model

Figure 3.3.5-5 shows the relations between damping constant h and radial frequency for the Voight model, the Maxwell model, and the nonviscous damping model. It can be seen that the damping constant increases proportional to the radial frequency according to the Voight model; it is inversely proportional to the radial frequency according to the Maxwell model, opposite to the Voight model. In the case of the nonviscous damping model, the damping constant is a constant independent of the radial frequency. Careful evaluation should be performed when the damping model is selected for the relation between the stress and strain of the soil. Usually, the damping constant of the ground material is taken as constant irrespective of the frequency in the normal frequency range.

{2} Nonlinear damping characteristics of soil

As the nonlinear modeling of soil with stiffness determined from the skeleton curve and damping characteristics from the hysteresis loops, there are the Hardin-Drnevich model (referred to as "H-D model" hereinafter) and Ramberg-Osgood model (referred to as "R-O Model" hereinafter) [3.3.5-10]. For both models, their formulas are frequently used for representing the nonlinear characteristics of the soil material.

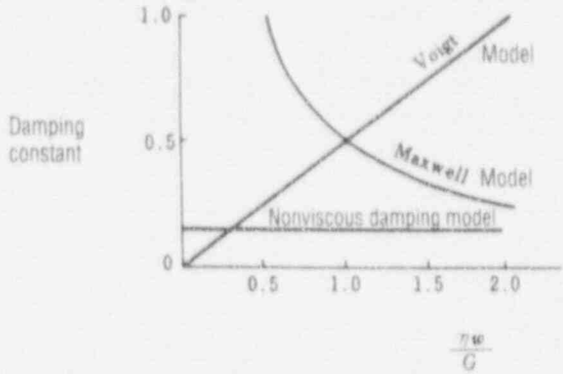


Figure. 3.3.5-5. Damping model and damping constant.

However, these models have some disadvantages. In the R-O model, the strength of the soil is not fully taken into consideration, and it is difficult to determine the model constants. In the H-D model, only part of the necessary constants can be determined experimentally, and the nonlinear characteristics of soil cannot be fully represented. For both the R-O model and the H-D model, several modified models have been proposed. However, the R-O model was originally proposed for metallic materials; and the H-D model was originally proposed for sand or other soft material. Hence, these models cannot provide a sufficient representation for the mechanical characteristics of rock material. In addition, it is difficult to apply these models in the case when the deformation of soil is significantly large. It is also possible to use the experimental data directly for analysis without using formula representation.

When seismic response analysis is performed using an equivalent linear modeling, it is possible to use the shear elastic coefficient and damping constant as functions of the magnitude of strain. However, in the case of time history integration analyses, the stress-strain curve at each time point is needed. Usually, due to the ease in formula representation and the simplicity of the response analysis, the H-D model is used more frequently than the R-O model.

(ii) Dissipation damping of soil

Due to radiation of wave motion from the structure back into the ground, the vibration of the structure can be damped out. This is called dissipation damping of soil or radiation damping. Usually, in the case when the dynamic interaction between the foundation and ground is taken into consideration, the damping is considered as being composed of the internal damping of the ground material and the dissipation damping of soil. In the range where the shear strain is small, the contribution of the dissipation damping of soil is believed to be larger. Given the same ground, the dissipation damping of soil increases as the structure becomes stiffer. Given the same structure, the dissipation damping of soil increases as the ground becomes softer.

(c) Spring constants

The spring constants depend not only on the stiffness and distribution of the ground, but also on the dimensions of the foundation. The following methods are mainly used to determine them. The spring constants include vertical spring constant, horizontal spring constant, rotational spring constant, shear spring constant, etc., which are defined corresponding to the vibration states of the structure-foundation-ground system. Depending on the theory used, they can be roughly divided into the static spring constant and dynamic spring constant.

(i) Schemes for deriving static spring constant

{1} Method defined in Road/Bridge Substructure Design Guideline

According to this guideline, the static spring constants for calculating the ground reaction force for vertical load, horizontal load and bending moment are determined.

{2} Method based on the theories on foundation slab on soil

For the static spring constants for the foundation slab on a semi-infinite elastic ground, there are theories by Taijimi [3.3.5-11], Barkan [3.3.5-12], and Pauw [3.3.5-13].

{3} Method using numerical analysis such as finite element method, etc.

For a complicated ground structure, the spring constants can be derived from the relations between applied forces and displacements by using the finite element method, etc.

(ii) Dynamic spring constant

Generally speaking, the following two methods have been proposed as theoretical and numerical analysis methods, respectively.

{1} Dynamic theory of foundation on semi-infinite elastic ground

There are Tajimi's vibration admittance theory based on the theory of foundation vibration on semi-infinite elastic ground [3.3.5-14], Kohori's grand compliance theory [3.3.5-15], and other methods.

{2} Numerical analysis methods

There are numerical methods using substructuring technique and FEM to account for the ground and boundary conditions.

(5) Evaluation method of scatter in soil properties

a. Basic ideas

Scatter in the soil properties affects the results of design and evaluation. It is thus necessary to evaluate the scatter and reflect it in the design by using appropriate methods considering the cause of scatter, its variation amplitude, survey/test method, design method, evaluation method, treatment of design safety factor, safety evaluation standard value, etc. Figure 3.3.5-6 shows the basic flow sheet for evaluating the scatter of the soil properties. In the following, the flow sheet will be explained with reference to the sequential numbers in the diagram.

{1} Survey/test for evaluating the ground

Survey/test of the geological state of the foundation ground is implemented.

{2} Engineering judgment on the basis of survey/test results

In the case of a ground (such as hard rock) which is expected to have sufficient stability according to the experience of both the survey/test in Item {1} and past examples, there is no need to consider scatter in the properties of the ground.

{3} Extraction of properties needed for safety evaluation using schematic design method

The basic items for safety evaluation of the foundation ground and peripheral slope of structure include the safety factor against sliding failure and deformation/settlement amount. The following properties of the ground are needed for the evaluation.

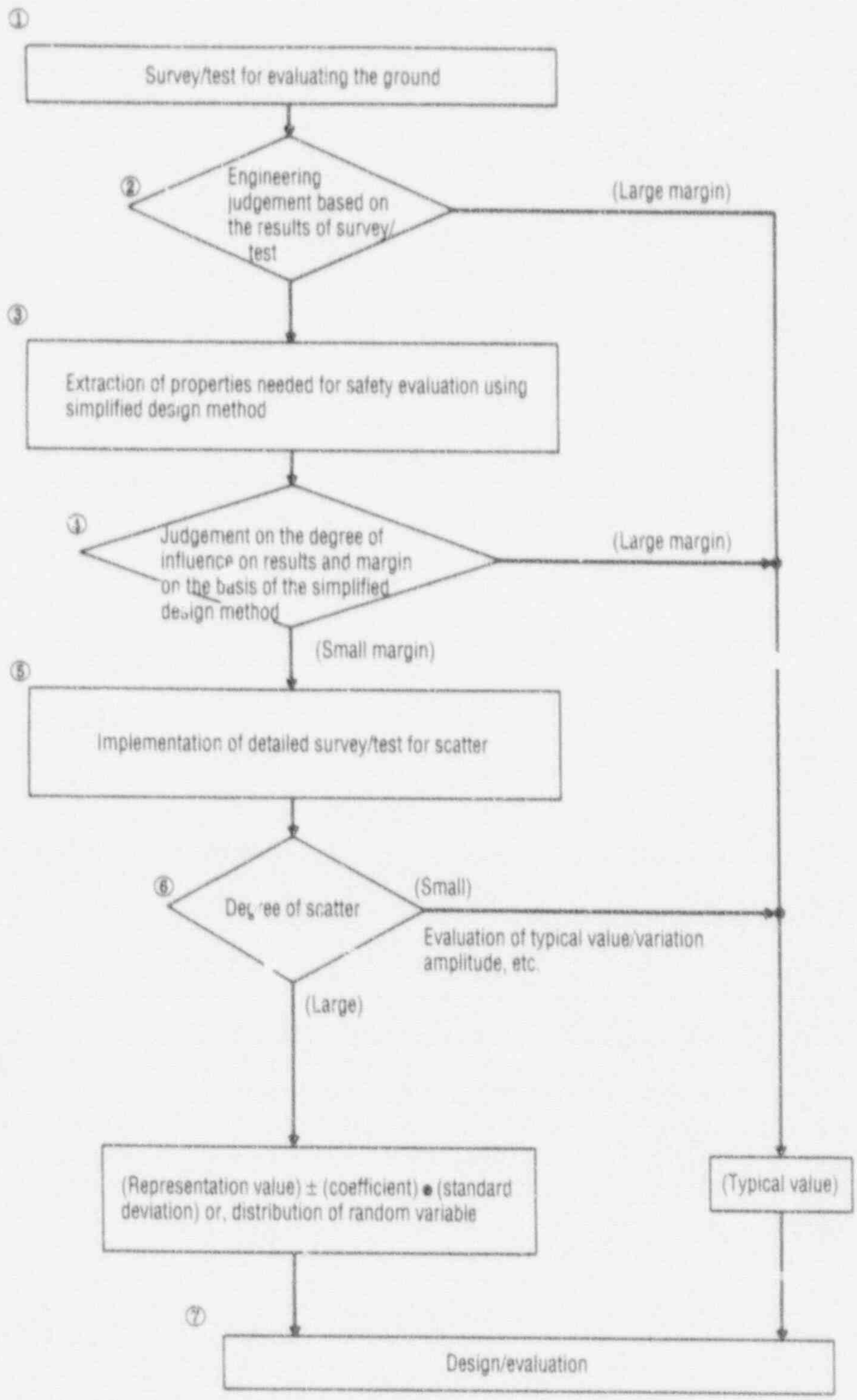


Figure 3.3.5-6. Basic flow sheet for evaluating scatter of ground properties.

Safety factor against sliding failure: cohesion (C), internal frictional angle (ϕ), weight per unit volume (γ_t), etc.

Deformation/settlement amount: elastic coefficient (E), Poisson's ratio (ν), etc.

Among these properties, C and ϕ are the dominant factors for the safety factor against sliding failure; E is the dominant factor for the deformation/settlement amount. As a result, it is important to make appropriate evaluation of the scatter of these properties of ground.

{4} Determination of the influence of scatter on the results of schematic design and the margin

Whether or not a detailed survey/test of scatter should be performed is determined by studying the degree of influence of the properties of the ground extracted in {3} using a simplified design method (sensitivity analysis). In the case when the degree of influence is low and there is a large margin with respect to the evaluation criterion value, the representative values can be used for the properties of the ground. On the other hand, in the case when the degree of influence is high or, although the degree of influence is low, the margin is also small, a detailed survey/test should be made of the properties of the ground.

{5} Implementation of detailed survey/test of scatter

In the case when the influence of the property values of the ground on the stability evaluation is large, or, although the influence is small, the margin of the evaluation results is also small, a detailed survey/test of the ground should be implemented in order to evaluate the degree of scatter in the properties of the ground. In order to determine the necessary data number, i.e., the so-called appropriate sample size, needed for evaluating the scatter characteristics, it is important to make an engineering judgment in addition to the statistical method.

{6} Degree of scatter

The results of the detailed survey/test are assorted to evaluate the degree of scatter. For the properties of ground with a small scatter, a representative value is selected based on engineering judgment. On the other hand, for the properties of ground with a large scatter, it is necessary to select the representative value and distribution type based on the observed scatter.

{7} Design/evaluation

Design and safety evaluation are performed in consideration of the scatter. The factors that cause the scatter in the ground properties include both intrinsic factors and human factors. When the influence of the scatter is to be evaluated, the latter factor should be excluded as much as possible.

b. Design methods in consideration of scatter

The design/evaluation methods in consideration of the scatter of the properties of ground include {1} deterministic methods and {2} probabilistic methods. As far as method {1} is concerned, the overall representative values (average values or most likely values) are used as the design properties of the ground; or, the values adjusted appropriately from the representative values in consideration of the variation amplitude of the properties are used. On the other hand, for method {2}, the design properties of the ground are taken as random variables and are input into the design formulas as a distribution function. The evaluation results are also taken in terms of probability. Method {2} is more tedious than method {1} in calculation; in addition, it is difficult to evaluate the societal and economical aspects at present. Consequently, it may be considered as a design method for the future.

From the aforementioned point of view, at present, as shown by the references [3.3.5-16], the deterministic method {1} is adopted as the design method with scatter in the properties of the ground taken into consideration. In this case, design is implemented by adjusting by the amount of kσ for the representative value μ of the properties of ground for various designs. Here, σ is the standard deviation of the property value, and k is the engineering coefficient. The value of k should be determined appropriately in consideration of the degree of scatter in the ground properties, degree of influence on the results, reliability of the evaluation criteria, precision of the design method, etc., so that the evaluation can be determined to the safe side.

3.4 Examples of survey and test programs

Survey/test should be performed in a rational way according to the basic planning stage, design stage, and detailed design stage, as shown in Figure 3.4-1.

For the nuclear reactor building foundation ground, nuclear reactor building peripheral slope and importance outdoor underground structure ground, the following seven ground models are selected, with examples of their survey/test programs illustrated.

{1} Homogeneous ground (hard rock)
{2} Homogeneous ground (soft rock)
{3} Jointed ground
{4} Layered ground
{5} Ground containing fault rupture zone and other weak strata
{6} Soil ground
{7} Backfilled ground

Tables 3.4-1-7 illustrate examples of survey/test. Among them, Tables 3.4-1-5 illustrate examples of the foundation grounds of nuclear reactor buildings. However, they may also be applied within the necessary ranges to the peripheral slope of nuclear reactor buildings and the foundation ground of important outdoor underground structures. Table 3.4-5 describes the survey/test of the fault rupture zone and other weak layer.

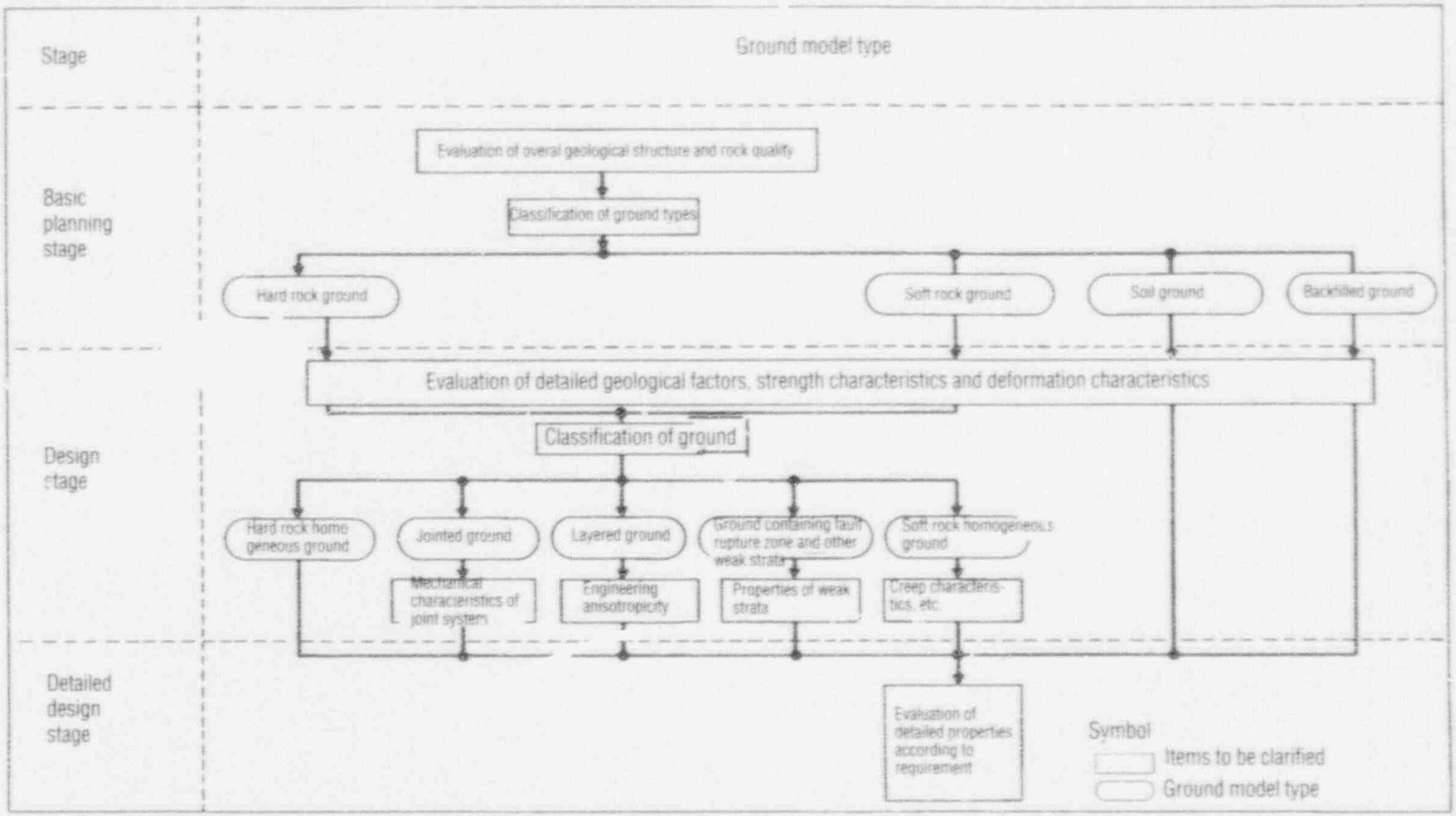


Figure 3.4-1. Survey/test stages and ground model types.

Table 3.4-1. Survey/test example of foundation ground of nuclear reactor building [3.1-1] (for hard-rock homogeneous ground).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined, etc.	Survey/test range and amount	Relation with design/analysis method	Note
Basic planning stage	General geological structure in the site, rock type/rock grade distribution, fault rupture zone, and other properties are surveyed, and the basic construction program of the nuclear reactor facility is set up	1. Survey of geological structure and rock type/rock grade					(4) For the hard rock, since the rock type, rock grade, weathering level, crack distribution, etc., are more complicated than those in the case of soft rock, the mesh for boring survey is usually made finer than that for soft rock
		(1) Reference survey (2) Ground surface geological survey		Fault survey, bedrock distribution, presence/absence of fault	Within site		
		(3) Ground surface elastic wave survey		Bedrock depth, V_p	Periphery of the nuclear reactor site		
		(4) Boring survey		Rock type/rock grade distribution	Same as above, 50-100 m mesh		
		2. Evaluation of schematic properties (1) Physical test (2) Uniaxial compression test		γ_t, e, w, q_u, ν	Periphery of the nuclear reactor site		It is performed to a level at which the genral characteristics of the rock can be evaluated
Design stage	The geological structure of the foundation ground, mechanical properties, wave propagation characteristics, etc., are surveyed, safety evaluation and structural design of the nuclear reactor facility are performed	1. Evaluation of geological structure (1) Boring survey		Rock type/rock grade distribution	5 or more holes with depth greater than the foundation width, under the ground surface		
		(2) Pit survey	Near foundation bottom surface		2 or more pits crossing each other with [length] about the foundation width		

Table 3.4-1 *(Cont'd)*. Survey/test example of foundation ground of nuclear reactor building [3.1-1] (for hard-rock homogeneous ground).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined, etc.	Survey/test range and amount	Relation with design/analysis method	Note
Design stage		2. Evaluation of properties of bedrock					
		(1) Shear test of bedrock	Bedrock shear, or block shear	C, ϕ	Rock type, rock grade	Seismic stability evaluation using conventional methods (slide, bearing capacity, settlement)	
		(2) Bedrock deformation test - Deformation characteristics - Bearing strength	Vertical direction (ϕ=30,80 cm)	E Bearing capacity	About 2 sites for each rock type/rock grade		(2) It is necessary to make sure that no failure takes place when the load becomes about twice the long-term load on the foundation bedrock
		(3) Elastic wave velocity test (in pit)	Refractive method Direct method	V_P , V_S (E_d , ν_d)	Pit side wall, pit bottom, space between pits	Determination of ground constant for dynamic aseismic design	(3) Evaluation of scatter of properties on different sites, with data used for evaluation of anisotropicity
		(4) Boring in-hole test (PS logging, in-hole load test, etc.)		V_P , V_S E ~ Depth	Pit side wall, pit bottom, space between pits	Evaluation of degree of property variation in the depth direction	
		3. Evaluation of rock properties (1) Physical test (2) Uniaxial compression, 3-axial compression, tensile (crushing) test	Boring core or block sample	γ_t , e, w, q_u , ν, E, C, ϕ, σ_t	For each rock type/rock grade, it is performed to a level which enables evalulation of rock properties		(2) Mechanical test of weathered rock with many cracks is usually difficult to implement. Also, the mechanical test results usually are not directly used for design.

Table 3.4-1 *(Cont'd)*. Survey/test example of foundation ground of nuclear reactor building [3.1-1] (for hard-rock homogeneous ground).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined, etc.	Survey/test range and amount	Relation with design/analysis method	Note
Detailed design stage	Depending on the requirement, for the ground evaluation of 3-dimensional property distribution, evaluation of scatter, evaluation of detailed properties, etc., are performed; detailed design is carried out	1. Evaluation of initial stress of foundation bedrock (1) Measurement of initial stress		Initial stress	Implement according to necessity	Static/dynamic analysis	
		2. Evaluation of detailed properties of foundation bedrock (1) Boring in-hole test (PS logging, in-hole load test, etc.) (2) In situ creep test		V_P, V_S, ν_d E ~ Depth G_0 ~ Depth Creep coefficient (α, β)	The test [illegible] mented in the design stage [illegible] utilized	[illegible] basic [illegible]ysis, dy-[illegible] linear analysis, evaluation of scatter of properties at the site	(1) For weathered rock and cracked bedrock, nonlinear evaluation is needed (2) Loaded with self weight of nuclear reactor building and other long-erm load. Usually, the Voight-Spring three-element model is used to derive the coefficients.
		3. Evaluation of dynamic characteristics of foundation bedrock (1) Dynamic deformation test		E_d	Pit	Determination of ground constants for dynamic aseismic design	Depending on the test method, the deformation characteristics may vary significantly in some cases. Hence, care should be taken in determining the design constants.

Table 3.4-1 *(Cont'd)*. Survey/test example of foundation ground of nuclear reactor building [3.1-1] (for hard-rock homogeneous ground).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined, etc.	Survey/test range and amount	Relation with design/analysis method	Note
Detailed design stage		4. Evaluation of scatter of ground properties (1) Elastic wave velocity test (2) In-hole load test (3) Schmidt rock hammer test					The scatter of properties is evaluated on the basis of the various survey results

Table 3.4-2. Example of survey/test of foundation ground for nuclear reactor building [3.1.1] (for soft-rock homogeneous ground).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined	Survey/test range amount	Relation with design/analysis method	Note
Basic planning stage	The general geological structure in the site, distribution of rock type/rock grade, and properties of fault rupture belt, etc., are surveyed and the data are used to establish the basic planning of the nuclear reactor facility	1. Survey of geological structure and rock type/rock grade (1) Reference survey (2) Ground surface geological survey (3) Ground surface elastic wave survey (4) Boring surface		Fault survey Bedrock distribution, presence/absence of fault Bedrock depth V_P Rock type/rock grade distribution	In site Periphery of nuclear reactor site		(*) For soft rock ground, a layered structure is usually developed. Since the major purpose is to evaluate its continuity, the boring interval can be made larger than that of the hard rock ground. When the bedrock distribution is to be identified, for soft rock ground, the elastic wave velocity usually increases gradually as the depth increases; hence, boring survey is more effective than ground surface elastic wave survey.
		2. Evaluation of general properties (1) Physical test (2) Uniaxial compression test		γ_t, e, w, q_u, ν	Periphery of nuclear reactor site		

Table 3.4-2 *(Cont'd)*. Example of survey/test of foundation ground for nuclear reactor building [3.1.1] (for soft-rock homogeneous ground).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined	Survey/test range amount	Relation with design/analysis method	Note
Design stage	The geological structure of the foundation ground, mechanical properties, wave propagation characteristics, etc., are surveyed and the data are used for the structural design of the nuclear reactor facility.	1. Evaluation of the geological structure of foundation bedrock (1) All-core boring (2) Pit survey		Rock type/rock grade distribution, presence/absence of fault Rock type/rock grade distribution, presence/absence of fault	In a range with radius of about 200 m, 5 or more holes with depth greater than the foundation width 2 pits cr[illegible]ing each other with length about the foundation width		(1) For soft rock ground, usually a homogeneous stratified structure is displayed. Hence, it is possible to have a l[illegible]er boring interval.
		2. Evaluation of rock properties (1) Physical test (2) Uniaxial compression, 3-axial compression, tensile (crushing) test	UU (CU) boring core or in-pit sample	γ_t, e, w, q_u, C, ϕ, E, ν, σ_t	For each rock type/rock grade, survey/test is performed to a level which enables evaluation of rock properties		For soft rock, usually there are few joints and the tissue is homogeneous; hence, in many cases, the bedrock properties can be evaluated using the rock properties. (2) For a soft rock with a low consolidation degree, q_u may be very small in some cases; hence, a 3-axial compression test with the actual design load is desirable. In addition, in

Table 3.4-2 *(Cont'd)*. Example of survey/test of foundation ground for nuclear reactor building [3]) (for soft-rock homogeneous ground).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined	Survey/test range amount	Relation with design/analysis method	Note
Design stage		(3) Consolidation test		P_y			the case when the variation in properties in the depth direction is to be surveyed, it is preferred that the 3-axial compression test be performed under a confinement pressure corresponding to the thickness of the overburden soil. Usually, the drainage condition in the 3-axial compression test may be made according to unconsolidated undraining condition (UU). However, for soft rock with q_u of 10-20 kgf/cm^2 or lower, a consolidated undrainage condition (CU) is desirable. (3) Even for soft rock, usually, the consolidation yield stress (P_y) is much higher than the conventional load of the nuclear reactor. For a quantitative

Table 3.4-2 *(Cont'd)*. Example of survey/test of foundation ground for nuclear reactor building [3.1.1] (for soft-rock homogeneous ground).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined	Survey/test range amount	Relation with design/analysis method	Note
Design stage							evaluation, however, it should be implemented for relatively soft rock.
		(4) Uniaxial (3-axial) creep deformation test	In-pit sample	Creep coefficient (α,β)		Evaluation of settlement using conventional method	(4) In practical application, the coefficients can be derived using the Voigt-Spring three-element model
		(5) Uniaxial (3-axial) creep strength test	In-pit sample	Creep strength		Evaluation of long-term bearing capacity using conventional method	(5) Even in soft rock, for rock with a high strength, this is not needed even when the decrease in strength due to creep is taken into consideration Evaluation may be made by using the step-up creep test proposed by Murayama
		3. Evaluation of bedrock properties (1) Elastic wave velocity test	In boring hole	V_P , V_S , E_d , ν_d	Depending on requirement, boring hole in 1.(1)	Determination of ground constants for dynamic aseismic design	(1) For soft rock ground, usually the velocity difference is small. In addition, significant anisotropicity is not displayed. Hence, it is usually
			Pit wall, refractive method, direct method	V_P , V_S , E_d , ν_d	Depending on requirement, within pit in 1.(2)		

Table 3.4-2 *(Cont'd)*. Example of survey/test of foundation ground for nuclear reactor building [3.1.1] (for soft-rock homogeneous ground).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined	Survey/test range amount	Relation with design/analysis method	Note
Design stage							possible to omit the elastic wave velocity test by performing the direct method
		(2) Plate load (bearing strength) test		Bearing capacity	In pit, for each rock type/rock grade	Evaluation of bearing capacity using conventional method	(2) It is necessary to confirm that the bedrock is not failed when the load becomes three times the long-term load or twice the short-term load
		(3) Plate load (deformation) test		E	In pit, for each rock type/rock grade	Evaluation of settlement using conventional meth d	(3) Within the short-term load amplitude range, cyclic loading is applied, and the deformation coefficient, secant elastic modulus, and tangential elastic modulus are determined according to the requirement. It is possible to implement it concurrently as the bearing strength test.
		(4) Plate loading (creep) test		Creep coefficient (α,β)	In pit, for each rock type/rock grade	Evaluation of settlement using conventional methods	(4) Long-term load is used for load test over a long period. The period is in the range of 1-3 months.

Table 3.4-2 *(Cont'd)*. Example of survey/test of foundation ground for nuclear reactor building [3.1.1] (for soft-rock homogeneous ground).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined	Survey/test range amount	Relation with design/analysis method	Note
Design stage							For practical application, the Voigt-Spring three-element model is used to derive the coefficients
		(5) Shear test	Bedrock shear or block shear	C, ϕ	In pit, for each rock type/rock grade	Evaluation of slide using conventional methods	(5) The vertical loads should be three or more types
		(6) Borehole loading test		E	Implemented if necessary	Evaluation of settlement using conventional methods	(6) It is effective in the case when there exist sandrock with a low consolidation degree and other layer dificult for boring in the deep portion of the foundation bedrock. In the bedrock with deformation coefficient E less than about 10,000 kgf/cm^2, the value is usually smaller than the value of E obtained from the results of the plate loading test. Hence, attention should be paid to this feature during evaluation.

Table 3.4-2 *(Cont'd)*. Example of survey/test of foundation ground for nuclear reactor building [3.1.1] (for soft-rock homogeneous ground).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined	Survey/test range amount	Relation with design/analysis method	Note
Detailed design stage	If needed, evaluation of 3-dimensional property distribution of the soil, evaluation of scatter in detailed properties, etc., are performed, and the data are used for detailed design	1. Evaluation of initial stress of foundation bedrock (1) Initial stress measurement		Initial stress	Implemented if necessary	Static/dynamic analysis	(1) For a soft rock ground with relatively flat ground surface, usually the stress state is isotropic. However, measurement is needed in the case when it is determined that the topography and geological structure are complicated and the initial stress has a large influence on the analysis results.
		2. Evaluation of the detailed properties of foundation bedrock (1) Physical test (2) Uniaxial compression, 3-axial compression, tensile (crushing) test	UU(CU)	γ_t, e, w, q_u, E, ν, C, ϕ, σ_t $E \sim \varepsilon$ $\nu \sim \varepsilon$	Test implemented in the design stage is utilized; added depending on the necessity	Static/dynamic analysis	(2) For the bedrock made of relatively soft rock compared to other soft rocks, in order to perform static analysis in consideration of the nonlinearity, it

Table 3.4-2 *(Cont'd)*. Example of survey/test of foundation ground for nuclear reactor building [3.1.1] (for soft-rock homogeneous ground).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined	Survey/test range amount	Relation with design/analysis method	Note
Detailed design stage							is necessary to evaluate the nonlinear characteristics
		3. Evaluation of dynamic characteristics of foundation bedrock (1) Dynamic 3-axial compression test (dynamic shear test)	UU(CU) In-pit sample	$G \sim \gamma$, $h \sim \gamma$ C_d , ϕ_d		Dynamic analysis	(1) For relatively hard soft rock, depending on the characteristics of the input seismic motion, the influence of strain dependence is usually small. In this case, it is possible to use the dynamic deformation characteristics obtained from the elastic wave velocity as a substitute. For the dynamic strength characteristics, if a sufficient margin can be obtained when the static strength is used for evaluation, there is no need to make a special evaluation. In addition, in the case when the dynamic strength characteristics are

Table 3.4-2 *(Cont'd)*. **Example of survey/test of foundation ground for nuclear reactor building** [3.1.1] **(for soft-rock homogeneous ground).**

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined	Survey/test range amount	Relation with design/analysis method	Note
Detailed design stage		(2) Elastic velocity test	In boring hole	V_P, V_S, E_d, ν_d	Test prformed in the design stage is utilized; added according to necessity	Dynamic analysis	needed, if it is found that the dynamic strength is not less than the static strength, the static strength may be used.
		4. Evaluation of scatter of ground properties					Evaluation of scatter of properties is performed in consideration of the various survey results
		(1) Uniaxial compression test	In-pit sample				
		(2) 3-axial compression test	In-pit sample				
		(3) Elastic wave velocity test	In pit				
		(4) Borehole loading test	In boring hole				
		(5) Schmidt rock hammer test	In pit				

Table 3.4-3. Example of survey/test of foundation ground of nuclear reactor building [3.1-1] (for ground with joints).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined	Survey/test range and amount	Relation with design/analysis	Note
Basic planning stage	The general geological structure of the site, rock type/rock grade distribution, fault rupture zone, and other properties are surveyed, and the data are used for the basic layout planning of the nuclear reactor facility	Survey of geological structure and rock type/rock grade (1) Reference survey		Fault survey	On site		Ground with joints is usually hard ground (4) Hard rock has more complicated distributions of rock type/rock grade, weathering degree, cracks, etc., than soft rock. Hence, the mesh used for boring survey is usually finer than that for soft rock.
		(2) Ground surface geological survey		Bedrock distribution, presence/absence of fault			
		(3) Ground surface elastic wave survey		Bedrock depth, V_P	Periphery of nuclear reactor site		
		(4) Boring survey		Rock type/rock grade distribution	Same as above, 50-100 m mesh		
		2. Evaluation of general properties (1) Physical test (2) Uniaxial compression test		γ_t, e, w, q_u, ν	Vicinity of nuclear reactor site		It is performed to a level which enables evaluation of the genral characteristics of rock
Design stage	The geological structure of the foundation ground, mechanical properties, wave propagation characteristics, and other characteristics are surveyed, and the data are used for the structural design of nuclear reactor facility.	1. Understanding of geological structure (1) Boring survey		Rock type/rock grade distribution	Beneath foundation surface, 5 or more holes with depth greater than foundation width		Statistical processing of joint distribution is needed
		(2) Pit survey (including joint survey)	Near foundation bottom surface		2 or more pits crossing each other with length about the foundation depth		

Table 3.4-3 *(Cont'd)*. Example of survey/test of foundation ground of nuclear reactor building [3.1-1] (for ground with joints).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined	Survey/test range and amount	Relation with design/analysis	Note
Design stage		2. Evaluation of bedrock properties					
		(1) Bedrock shear test	Bedrock shear or block shear	C, ϕ	For each rock type/rock grade (in consideration of flow mesh and setting mesh)	Seismic stability evaluation using conventional method (sliding failure, bearing strength, settlement), smaller strength value is usually used	(2) It is necessary to confirm that no failure takes place under a load three times the stationary load acting on the foundation bedrock or twice the short-term load.
		(2) Bedrock deformation test Deformation characteristics Bearing strength	Vertical direction (ϕ=30,80 cm)	E Bearing strength	For each rock type/rock grade, 2 sites for each of 3 different directions		The deformation property is tested in the vertical direction and the directions perpendicular/parallel to the joints; in this way, anisotropicity is evaluated.
		(3) Elastic wave velocity test (in pit)	Refraction method	V_P , V_S (E_d , ν_d)	Pit side wall, pit bottom	Determination of ground constants for dynamic aseismic design	(3) Scatter of properties in different locations is evaluated, and the data are used for evaluation of anisotropicity.
			Direct method		Interval between pits		
		(4) Test in boring hole (PS logging, borehole load test)		V_P , V_S , E ~ Depth	Deep boring	Evaluation of degree of change in the properties in the depth direction	Generally speaking, the strength of the ground containing joints is believed to be reflected in the results of bedrock test.

Table 3.4-3 *(Cont'd)*. Example of survey/test of foundation ground of nuclear reactor building [3.1-1] (for ground with joints).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined	Survey/test range and amount	Relation with design/analysis	Note
Design stage		3. Evaluation of the rock properties (1) Physical test (2) Mechanical unit Uniaxial compression test, 3-axial compression, tensile (crushing) test	Boring core or block sample	γ_t, e, w, q_u, ν, E, C, ϕ, σ_t	For such rock type/rock grade, it is performed to a degree at which evaluation can be made of properties	When it is possible to form a sample containing joints, test for anisotropicity is implemented	(2) For weathered rock with many cracks, it is usually difficult to implement mechanical tests. Also, the mechanical test results can almost not be used directly in design.
Detailed design stage	If needed, evaluation of 3-dimensional property distribution of the soil, evaluation of scatter in detailed properties, etc., are performed, and the data are used for detailed design	1. Evaluation of initial stress of foundation ground (1) Initial stress measurement		Initial stress	Implemented depending on the requirement	Static/dynamic analysis	
		2. Evaluation of detailed properties of foundation bedrock (1) Boring Hole (PS logging, borehole loading test) (2) In situ creep test		V_P, V_S, ν_d, G_0 ~ Depth Creep coefficient (α,β *etc.*)	Test implemented in the design stage is utilized	Static/dynamic analysis Evaluation of scatter in properties in different places	(1) In the case when weathered rock and cracked bedrock exist, a nonlinear evaluation may be needed (2) Loaded with long-term load of self-weight of nuclear reactor building, etc. Usually, Voigt-Spring three-element model is used to determine the coefficients

Table 3.4-3 *(Cont'd)*. Example of survey/test of foundation ground of nuclear reactor building [3.1-1] (for ground with joints).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined	Survey/test range and amount	Relation with design/analysis	Note
Detailed design stage		3. Evaluation of dynamic characteristics of foundation bedrock (1) Dynamic deformation test		E_d		Determination of ground constants for dynamic aseismic design	(1) Depending on the test method, the deformation characteristics may vary significantly in some cases. Hence, care should be taken in determining the design constants.
		4. Evaluation of scatter of ground properties (1) Elastic wave velocity test (2) Borehole loading test (3) Schmidt rock hammer test					Scatter in the properties is evaluated on the base of the results of the various surveys

Table 3.4-4. Example of foundation ground survey/test of nuclear reactor building [3.1-1] (for layered ground).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined	Survey/test range and amount	Relation to design/analysis method	Note
Basic planning stage	The general geological structure of the site, distribution of rock type/rock grade, fault rupture zone, and other properties are surveyed, and the data are used for the basic layout planning of nuclear reactor facility	1. Survey of geological structure and rock type/rock grade (1) Survey of ground surface geography (2) Ground surface elastic wave test (3) Boring survey		Bedrock distribution, presence/absence of fault Bedrock depth, V_p Rock type/rock grade distribution	In site Periphery of nuclear reactor site 50-100 m mesh in site		(Note) Layered ground may be made of hard rock or soft rock. Here, mainly hard rock is described. For layered ground made of soft rock, please see the table on soft rock ground.
		2. Evaluation of general properties (1) Physical test (2) Uniaxial compression test		γ_t, e, w, q_u, ν	Periphery of nuclear reactor site		
Design stage	Geological structure of foundation ground, mechanical characteristics, wave propagation characteristics, etc., are surveyed, and the data are used for design of the nuclear reactor facility	1. Evaluation of geological structure (1) Boring survey		Rock type, rock grade, thickness of each layer, layer slope	Beneath foundation surface, 5 or more holes, with depth greater than the foundation width; at the foundation periphery, foundation width ×2		Thickness and layer slope distribution of each layer are studied and are reflected in bedrock property survey and test program
		(2) Pit survey			2 or more pits crossing each other near the foundation bottom		

Table 3.4-4 *(Cont'd)*. Example of foundation ground survey/test of nuclear reactor building [3.1-1] (for layered ground).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined	Survey/test range and amount	Relation to design/analysis method	Note
Design stage		2. Estimation of bedrock properties (1) Bedrock shear test (2) Bedrock deformation test (3) Borehole loading test (4) Elastic wave speed test	Flow mesh, insertion mesh, vertical, horizontal, etc., 3 directions Pit side wall, pit bottom, region between pits	C, ϕ E, Bearing strength V_P, V_S E_d, ν_d	For each rock type/rock grade	Evaluation of seismic stability using conventional methods (sliding failure, bearing strength, settlement) Determination of ground constants for dynamic seismic design	Anisotropicity of strength and deformation are investigated. In this stage, for the anisotropicity, the properties on the safe side are used to evaluate sliding failure, bearing strength, settlement, etc., so that the margin of safety can be evaluated.
		3. Evaluation of rock properties (1) Physical tests (2) Mechanical tests Uniaxial compression, 3-axial compression, tensile (crushing) test	Boring core or block sample	γ_t, e, w q_u, E, ν, C, ϕ, σ_t	For each rock type/rock grade of layered ground		(2) In the case of saturated soft rock with q_u < 10-20 kgf/cm^2, CU test may be performed. In the case of soft rock, creep test and, if needed, initial stress measurement are performed.
Detailed design stage	Depending on the requirement, anisotropicity is considered in evaluation of ground safety and the data are used to determine the ground processing needed for measures to ensure ground stability	1. Evaluation of dynamic characteristics of foundation bedrock (1) Dynamic deformation test (2) Dynamic 3-axial compression test (simple shear test, torsional shear test, etc.)	 UU (CU)	G, h ~ γ G, h ~ γ	Anisotropic three-directions Boundary between different layers	For anisotropic nonlinear ground model, seismic coefficient method is used for static FEM analysis or dynamic analysis	

Table 3.4-5. Example of foundation ground survey/test of nuclear reactor building [3.1-1] (for ground containing fault rupture zone and other weak layers).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined	Survey/test range and amount	Relation with design/analysis method	Note
Basic planning stage	The general geological structure of the site, distribution of rock type/rock grade, fault rupture zone, and other properties are evaluated, and the data are used for the basic layout planning of the nuclear reactor facility	1. Evaluation of general distribution of fault rupture zone (1) Reference survey (2) Ground surface geological survey (3) Boring survey			Periphery of site		It is necessary to study the favorable layout planning from the distribution and spread of fault, rupture zone, etc.
Design stage	The distribution and mechanical properties of the fault rupture zone, etc., in the foundation ground are evaluated, and the data are used for safety evaluation of the nuclear reactor facility and the structural design of the nuclear reactor facility	1. Evaluation of detailed distribution and properties of fault rupture zone (1) Ground surface geological survey (2) Pit survey (3) Shaft (4) Boring survey			Range needed for safety investigation of major fault	Formation of ground model for design of ground containing fault rupture zone and other weak layers	(5) For example, survey is performed [in a range] twice the foundation width with a depth about the foundation width
		2. Evaluation of properties (1) Physical tests (2) 3-axial compression test, single-plane shear test (3) Bedrock shear test		γ, e, w C, ϕ, E ~ ε C, ϕ		Evaluation of stability of ground in stationary state and in earthquake using conventional method, calculation of slide stability, support force, settlement, etc.	Depending on the thickness and properties of the rupture zone, appropriate test is performed to evaluate the properties. (2) The maximum gravael size is about 1/6 the size of the sample. As the test conditions, UU or

Table 3.4-5 *(Cont'd)*. Example of foundation ground survey/test of nuclear reactor building [3.1-1] (for ground containing fault rupture zone and other weak layers).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined	Survey/test range and amount	Relation with design/analysis method	Note
Design stage		(4) In situ deformation test (5) Elastic wave velocity test		E V_P, V_S, ν_d, G_0			CU for $q_u > 10$ kgf/cm^2 CU for $q_u < 20$ kgf/cm^2 (see Table 3.3.5-1). (4) In the case of thick soft rupture belt and in the case of a large clay content, consolidation test is performed to calculate the settlement amount and to determine the shear test conditions.
Detailed design stage	Depending on the necessity, processing of ground [data] is performed as required for the detailed design and determination of ground safety measures	1. Evaluation of detailed properties of foundation bedrock (1) Borehole load test (when the depth is large and the layer thickness is large) (2) Dynamic 3-axial compression test (simple shear test) (3) Ultrasonic elastic wave test	CU	E, P_y $G \sim \gamma$, $h \sim \gamma$ C_d, ϕ_d V_P, V_S, ν_d, G_0		Static FEM analysis or dynamic analysis in consideration of nonlinearity	For the weak layer, the mechanical characteristics are generally evaluated in an average way. Distribution of strength/deformation in the depth direction is confirmed. When the fault has a large width, Schmidt rock hammer test (in pit) is used to evaluate the homogeneity of the ruptured zone.

Table 3.4-6. Example of survey/test (for soil ground).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined	Survey/test range and amount	Relation to design/analysis method	Note
Basic p[illegible]ning stage	The general geological structure of the site and the properties of the ground soil are surveyed, and the data are used to determine the basic layout planning of the peripheral slope of the nuclear reactor building, etc.	1. Survey of geological structure (1) Ground surface geological survey (2) Ground surface elastic wave survey (3) Boring survey		Layer thickness of ground soil, type and distribution status of constituent materials V_P N value			
		2. Evaluation of general properties (1) Physical tests (2) Uniaxial compression test		γ_t , e, w, q_u , ν			
Design stage	The geological structure, mechanical characteristics, wave propagation characteristics, etc., are surveyed, and the data are used for design of the peripheral slope of nuclear reactor building, etc.	1. Evaluation of geological structure (1) Boring survey		Geological distribution	Implemented upon requirement in normal direction to the object slope		
		2. Evaluation of ground properties (1) 3-axial compression test (2) Water permeation test (3) Physical test (4) Standard penetration test (5) Consolidation test (6) Underground water level observation (7) Elastic wave velocity test (including borehole test)	UU, CU, CD	C, ϕ, E ($E \sim \varepsilon$) k γ_t , e, w etc. N value P_y V_P , V_S , E_d , ν_d		Determination of liquefaction Reference for determination of initial stress	

Table 3.4-6 *(Cont'd)*. Example of survey/test (for soil ground).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined	Survey/test range and amount	Relation to design/analysis method	Note
Detailed design stage	Depending on the requirements, the dynamic mechanical characteristics of the ground are surveyed, and the data are used for detailed design	1. Evaluation of dynamic properties of ground (1) Elastic wave velocity test (including borehole test) (2) Dynamic 3-axial compression test (simple shear test, hollow torsional test, etc.) (3) Test of liquefaction (dynamic 3-axial compression test)		V_p, V_s, E_d, ν_d $G \sim \gamma$, $h \sim \gamma$ C_d, ϕ_d Liquefaction strength			

Table 3.4-7. Example of survey/test (for backfilled ground).

	Purpose	Survey/test items	Survey/test conditions	Properties to be determined	Survey/test range and amount	Relation to design/analysis method	Note
Basic planning stage	The properties of backfilled ground are surveyed, and the data are used for ground stability evaluation	1. Evaluation of properties of backfilled ground (1) Physical test (2) Consolidation test		γ_t , e, w, Granularity Consolidation test		Judgement of liquefaction	
Design stage	The physical/mechanical characteristics of the backfilled ground are surveyed, and the data are used for the basic design of important underground civil structures	1. Evaluation of properties of backfilled ground (1) Pressure test (2) Elastic wave velocity test (at the location of the above test) (3) Plate load test (4) 3-axial compression test (5) Liquefaction test (dynamic 3-axial compression test)		γ_t , Granularity distribution V_P , V_S , E_d , ν_d E C, ϕ Liquefaction strength		Static analysis of backfilled ground and its application in static design of structure	
Detailed design stage	Depending on the requirement, the dynamic characteristics of the backfilled ground are investigated, and the data are used for seismic response analysis of the structure	1. Evaluation of dynamic properties of backfilled ground (1) Elastic wave velocity test (at pressure test sites) (2) Dynamic 3-axial compression test (3) Elastic wave velocity test (backfilled ground)		V_P , V_S , E_d , ν_d $G-\gamma$, $h-\gamma$ C_d , ϕ_d V_P , V_S , G_0 E_d , ν_d		Dynamic design of structure (response analysis) and, if needed, application in dynamic analysis of backfilled ground	From the elastic wave velocity test, the properties of the actual backfilled ground are surveyed, and the appropriateness of the properties used in analysis are studied in some cases

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Chapter 4. Safety evaluation of ground and aseismic design of underground structures

4.1 Basic guidelines of aseismic design

4.1.1 Evaluation of aseismic importance of ground and civil structures

According to "Technical Guidelines of Aseismic Design of Nuclear Power Plants: Volume of Importance Classification/Allowable Stress, JEAG-4601 Supplement-1984" by the Electrical Society of Japan, for the evaluation of the support function of indirect support structures and influence between equipment, it is necessary to confirm that there exists no safety problem against the ground motions used for the aseismic design of the related equipment.

Table 4.1.1-1 shows examples of the ground and underground structures of nuclear power plants. Among the underground structures, for example, the support structures (such as seawater pump foundation, seawater pipe duct, etc.) which support the emergency water intake equipment (nuclear reactor auxiliary cooling seawater equipment, etc.) are classified as indirect support structures. When subjected to the ground motions appropriate for the aseismic design of the equipment supported by them, it is necessary to confirm that the function for supporting the aforementioned equipment is not degraded.

On the other hand, the structures related to the emergency water intake equipment in the range from the sea to the pump chamber, such as intake inlet, water channel, etc., are classified as Class As. Since they have various different structural forms, it is necessary to determine the design guideline for each specific structural form in the aseismic design. Usually, they are handled as indirect support structures for the safest judgment.

The foundation bedrock of the nuclear reactor building supports the buildings and structures containing Class As structures. Hence, when evaluation of the seismic stability of the ground is to be performed, it is appropriate to handle it as indirect support structure.

As far as the peripheral slope of the nuclear reactor building is concerned, as the peripheral slope itself does not contain radioactive substances, nor does it directly support facilities containing radioactive substances, it is not a structure with possible problem of influence on the environment by radioactive substances. Also, even if a slope failure is assumed to occur during an earthquake, so long as it does not directly affect the nuclear reactor building and the function of the nuclear reactor facility can still be maintained, there exist no safety problems. Therefore, when the seismic safety of the peripheral slope of the nuclear reactor building is to be evaluated, it is necessary to confirm that its collapse does not affect the nuclear reactor facility, etc.

In addition to the items described above, the nuclear power plant also has various other structures, such as circulation cooling water inlet/outlet facility, etc., as listed in Table 4.1.1-1. They will be explained in section 4.5 "Other civil structures."

4.1.2 Guideline of consideration of design seismic force

For nuclear reactor building foundation bedrock, nuclear reactor building peripheral slope, and important outdoor underground structures, any of them should not degrade the function of the Class A and Class As buildings and structures, neither should they have secondary influence on the ability to maintain the function of buildings and structures. Hence, in the aseismic design of these ground and underground structures, the seismic force used is that based on the basic earthquake ground motion S_1 or S_2. The method for determining the basic earthquake ground motion is described in Chapter 2.

For the seismic force used for evaluating the safety, a detailed description will be presented in section 4.2 and later for different types of ground and structures.

Table 4.1.1-1. Ground and underground structures (examples).

Classification	Major equipment
Nuclear reactor building foundation bedrock	
Nuclear reactor building peripheral slope	
Important outdoor underground structures	Underground structures related to emergency cooling facility: Seawater pump foundation (intake pit) Seawater pipe duct Water inlet Water channel
Other civil structures	Condenser cooling water inlet/outlet equipment (Water inlet, water channel, exhaust channel, exhaust outlet)
	Tank foundation (Raw water tank foundation, pure water tank foundation, etc.)
	Foundations of electrical equipment, machines, piping (Foundation of substation equipment, cable duct, etc.)
	Harbor facilities (Breakwater, pier, dike, etc.)
	Road, bridge, tunnel, retaining wall, etc.

4.1.3 Basic guidelines of safety evaluation

When safety evaluation is to be performed for the nuclear reactor building foundation ground and nuclear reactor building peripheral slope, the results of ground survey and testing are used to determine the appropriate ground model; then, analysis is performed by using the sliding-plane method or other conventional method, or the finite element method is used to implement the static analysis and dynamic analysis.

In principle, the nuclear reactor building foundation bedrock is selected as a stable bedrock with sufficient bearing strength; hence, there is usually little problem related to safety. However, in the case when prominent anisotropicity or significant nonhomogeneity is found, uneven stress may be generated. As a result, it is necessary to perform detailed investigation of such factors as ground slip along the weak layer, bearing strength, settlement, etc.

On the other hand, for the peripheral slope, it is necessary first to determine the range of the safety evaluation to account for the distance from the nuclear reactor building, size of the slope, etc. In this respect, as pointed out in section 3.2.3 "Survey on site," based on the results of the past cases of slope failure, the slope to be considered is usually that which has a distance between its tail and the nuclear reactor building shorter than about

50 m, or shorter than about 1.4 times the slope height. There are several types of slopes, such as bedrock slope, earth slope, banking slope, etc. When safety evaluation is to be performed, it is important to have a good knowledge on the characteristics of these constituent materials and to select an appropriate method of analysis.

Figure 4.1.3-1 shows the flow chart of basic consideration on the safety evaluation of the foundation bed and peripheral slopes. Aseismic safety evaluation of ground is performed by analysis using the sliding-plane method or other conventional method which has been actually used in the past design and can be handled easily, as well as by the static and dynamic analysis using finite element method, etc., which can treat more complicated conditions. In this case, as shown in Figure 4.1.3-1, basically, investigation is performed in the sequence of analyses using conventional method, static analysis, and dynamic analysis. In each analysis stage, if the prescribed safety evaluation standard value can be satisfied, there is usually no need to perform analysis with higher precision. The safety evaluation standard values for the soil are listed in Secti n 4.2 "Foundation ground of nuclear reactor building" and Section 4.3 "Peripheral slope of nuclear reactor building."

For the important outdoor underground structures, there are the following features in aseismic design.

(1) They are mostly built under the ground.
(2) They are large in dimensions.

The safety of underground structures against earthquake depends significantly on the safety of the peripheral ground. Factors related to safety of ground include sliding failure of ground due to slope, etc., liquefaction of saturated sandy soil, significant uneven settlement caused by liquefaction or slide. On the other hand, the major factors that may degrade the safety of underground structures include underground water, as well as buoyancy and uplift due to liquefaction during an earthquake. If the safety of the ground is not degraded during an earthquake, and the safety against buoyancy and uplift can be ensured, the seismic safety of the overall structure is believed to be ensured, then the importance of investigation of slide, overturning, etc., of the structure is decreased. In this case, the major purposes of the aseismic design include calculation of the appropriate structural cross section of underground structures, evaluation of the safety function supporting the equipment system, and evaluation of the seismic force on the equipment.

As pointed out above, the earthquake response of underground structure fully depends on the response of the peripheral soil, and therefore an independent response is less important. Hence, the appropriate evaluation of the earthquake response of soil is important in safety evaluation.

In addition, for a long structure, differential displacement takes place for the various portions of the structure during an earthquake. In particular, large differential displacement may take place easily at the portion where the soil condition changes drastically and at the joint portion between two structures having different rigidities. In this case, evaluation of the differential displacement is needed.

Figure 4.1.3-2 shows the flow chart of basic consideration on the safety evaluation of important outdoor underground structures.

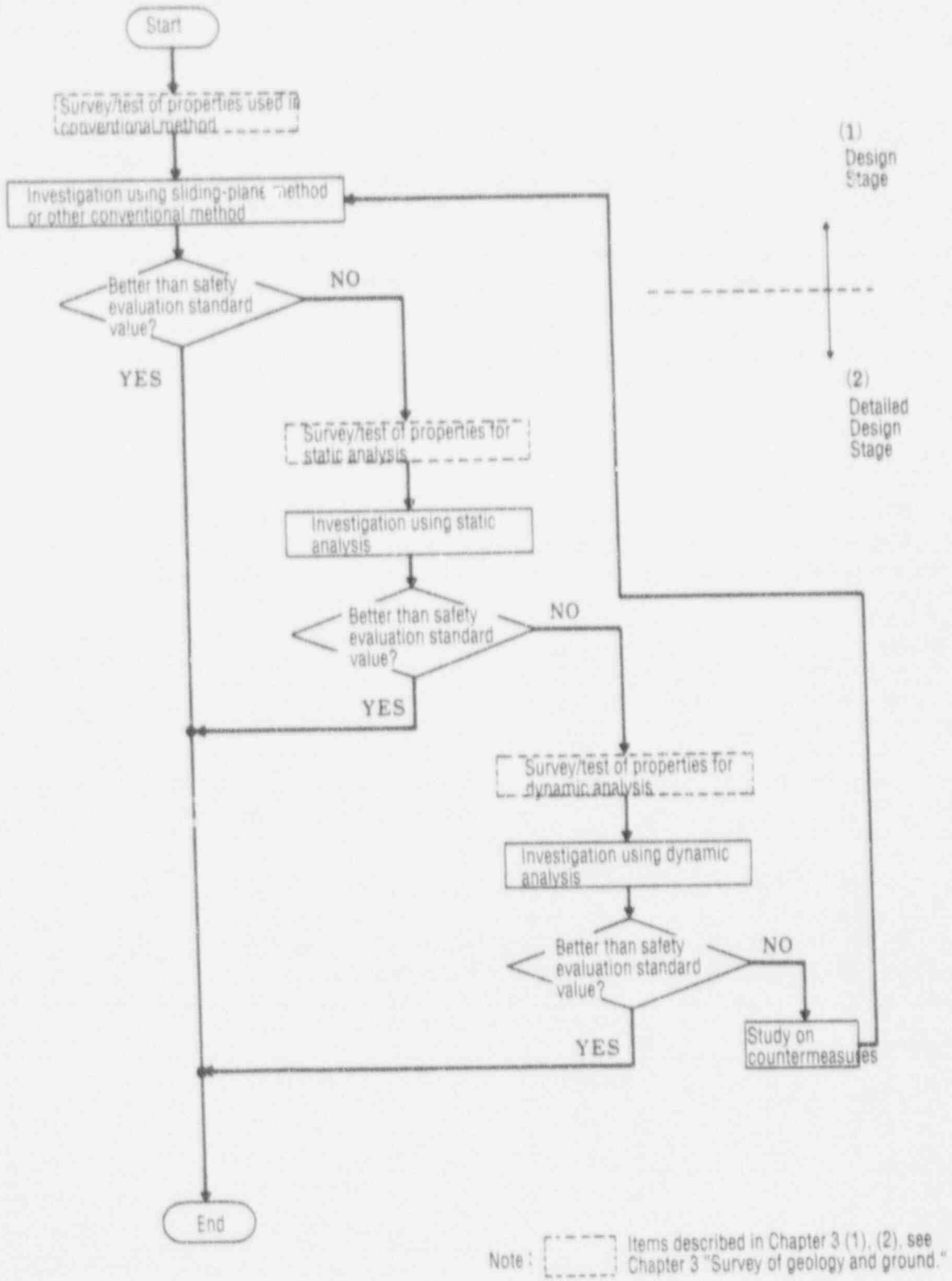


Figure 4.1.3-1. Basic flow chart of safety evaluation of bedrock [4.1.1].

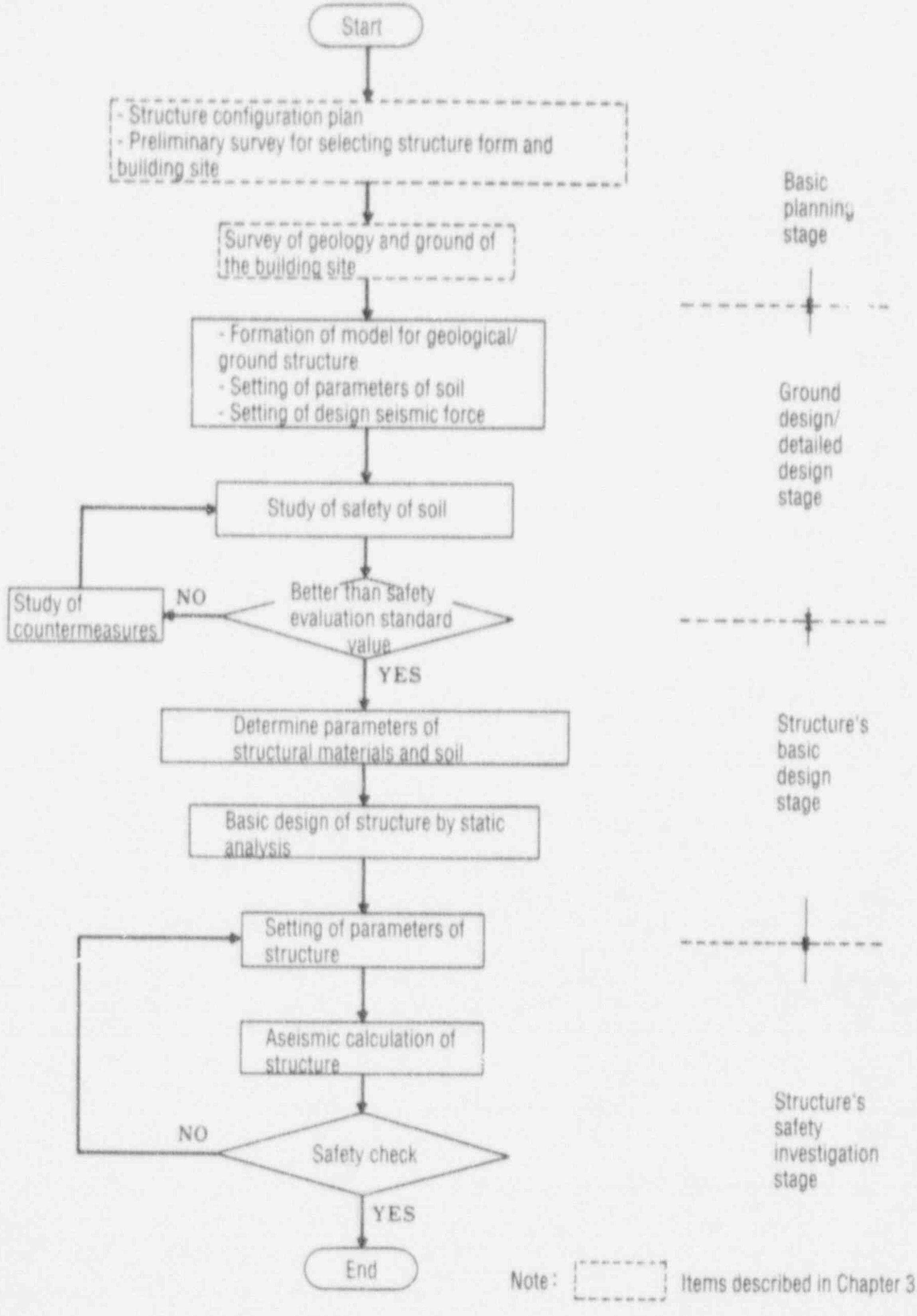


Figure 4.1.3-2. Basic flow chart of safety evaluation of important outdoor underground structures [4.1-1]

4.2 Foundation ground of nuclear reactor building

4.2.1 Modeling of ground

(1) Survey and classification of foundation ground

The foundation ground can be classified as isotropic, anisotropic, and heterogeneous grounds on the basis of the results of investigation of geology/ground. Then, the safety is evaluated according to the classification. The analytical model is determined appropriately on the basis of geology, rock grade, and property distribution. Also, the investigation range, boundary conditions, etc., are determined appropriately according to the relative location between the nuclear reactor building and the weak layer (fault rupture zone, etc.), inclination of weak layer, as well as the analysis (static or dynamic analysis).

(2) Properties

The safety analysis of the foundation ground is performed by superimposing the stress state due to seismic loads with the stress state under the long-term load condition. The analytical methods include sliding-plane method and other conventional methods, finite element method and other static analysis and dynamic analysis methods. Hence, the properties used in the safety analysis should be determined according to the specific conditions and analytical scheme. Table 4.2.1-1 summarizes the correspondence between analytical methods and properties.

Table 4.2.1-1. Correspondence between analytical methods and properties.

Analytical methods	Properties							
	Specific gravity	Static strength	Static modulus of elastic-ity	Static Poisson's ratio	Dynamic strength	Dynamic modulus of elasticity	Dynamic Poisson's ratio	Damping constant
Sliding-plane method	○	○(1)	—	—	—	—	—	—
Static analysis	○	○	○	○	—	○(2)	○(2)	—
Dynamic analysis	○	○(3)	—	—	○([illegible]	○	○	○

(1)To evaluate the slide stability of the building foundation bottom when a portion of the building foundation uplifts and is separated from the ground due to the seismic force (overturning moment from building), it is preferred that the residual strength of the ground be used as the slide resistant.
(2)When it is necessary to determine the deformation amount using a simplified static analysis, it is preferred that the dynamic properties be used.
(3)For the strength used in dynamic analysis, see section 3.3.3 (3) "Dynamic strength characteristics."

4.2.2 Seismic design force

(1) Seismic force used for static evaluation

a. Seismic coefficient for ground

In principle, the design horizontal seismic coefficient (K_H) at the ground surface is calculated using the following formula, or, it is taken as the equivalent seismic coefficient derived in consideration of the seismic characteristics of the ground on the basis of basic earthquake ground motion S_2.

$$K_H = n_1 \cdot K_0 \qquad (4.2.2\text{-}1)$$

where K_0: standard design horizontal seismic coefficient, taken as 0.2
n_1: correction factor depending on the region, taken as 1.0.
For the design vertical seismic coefficient (K_V), it is taken as $K_V = K_H/2$.

When the said seismic coefficient is used, attention should be paid to the following items.

(a) The standard design horizontal seismic coefficient is taken as 0.2. This is determined in consideration of the results attained in the past which indicate that for the bedrock with an S wave speed higher than about 500 m/s, the maximum acceleration of basic earthquake ground motion S_2 lower than about 500 Gal can be obtained as the rule of thumb. However, since there are various types of grounds, when this is applied, sufficient care should be exercised.

(b) The seismic force determined by the seismic coefficient method is static. Therefore, the stability is evaluated assuming the duration of loading is infinite. On the other hand, the inertia force actually acting during the earthquake varies both in magnitude and direction. That is, according to the seismic coefficient method, the inertia force is continuously applied. This assumption is much more severe than the case when an instantaneous load is applied. Suppose the static load in the seismic coefficient method is regarded as a normal sinusoidal acceleration wave with the seismic coefficient as the amplitude, in order to obtain the same acceleration response spectrum as that of the seismic motion with a random waveform, the amplitude of the constant sinusoidal wave should be reduced to 40-60% of the maximum acceleration amplitude of the typical seismic motions [4.2.2-1]. In this sense, the value set as the design seismic coefficient is believed to correspond to a rather large peak acceleration value.

(c) When the stability of the foundation soil is evaluated with the aid of the static seismic force, as the conventionally adopted ground seismic coefficient, the horizontal seismic coefficient (K_H) is taken as 0.2 and the vertical seismic coefficient (K_V) is taken as 0.1. As shown in Table 4.2.2-1, according to the results of past investigation of the design seismic forces, the analysis using the seismic coefficient approach usually yields the equivalent results of the dynamic analysis using basic earthquake ground motion S_2. As indicated by the analysis examples in section 4.6.1 "Foundation ground of nuclear reactor building," the horizontal seismic coefficient of 0.2 almost envelopes the following two types of seismic coefficients: equivalent seismic coefficient derived by superimposing contributions by slip planes at various depths in the foundation soil, and the equivalent seismic coefficient corresponding to the maximum horizontal shear stress distribution caused by the response of the soil to basic earthquake ground motion S_2, etc. Correction factor, n_1, for the region is a factor corresponding to $\Delta 1$ in the "New aseismic design method (draft)" [4.2.2-2]. In this case, however, $n_1 = 1.0$ is adopted following "Evaluation guidelines of seismic design of nuclear reactors power plants: Nuclear Power Safety Committee, July 20, 1981" [4.2.2-3] (referred to as "Evaluation Guidelines" hereinafter).

Table 4.2.2-1. Sliding safety factor of foundation soil (examples of investigation).

Site	Analytical method			
	Sliding-plane method	Static analysis	Dynamic analysis	Basic earthquake ground motion S_2 (max. acceleration) (Gal)
A	4.9	5.6	8.9	338
B	2.6	2.3	4.0	450
C	5.8	5.0	6.1	380
D	8.0	10.1	12.3	360
E[1]	1.9	2.9	2.0	600
F	7.0	7.3	7.6	370

[1]Examples for over 500 Gal, and outside the range where horizontal seismic intensity 0.2 is applicable.

(d) The design horizontal seismic coefficient of the ground adopted here corresponds to the upper-limit value of the important structures other than the nuclear power plant facilities, such as dams, etc. [4.2.2-4]. In addition, it is larger by 30% than the standard seismic intensity of 0.15 specified for the upper surface of the layer with N value in standard penetration test over 50 and S wave speed over 300 m/s according to other aseismic standards and guidelines [4.2.2-5].

(e) Usually, the seismic motion is amplified in the process of propagation from underground to ground surface; hence, the value of the acceleration amplitude on the ground surface is larger than that in the ground. Hence, the underground seismic coefficient can be taken as the same value as the value defined on the ground surface regardless of the depth. In addition, in the case when the underground seismic coefficient distribution is taken into consideration, the distribution profile should be determined by using an appropriate method corresponding to the specific ground.

b. Seismic force acting on ground due to building vibration

Because a building vibrates due to earthquake, the inertia force of the building acts on the ground as a seismic force. The horizontal seismic force acting from the building on the ground is taken as the static seismic force based on the "Evaluation Guidelines" [4.2.2-3] or the seismic force caused by basic earthquake ground motion S_2, whichever larger.

On the other hand, the vertical seismic force acting from the building on the ground is calculated by applying a uniformly distributed vertical seismic coefficient derived using the standard value of 0.3 and with the vibration characteristics of the building/structure taken into consideration (1/2 the maximum horizontal acceleration amplitude if the basic earthquake ground motion S_2 is used).

In the case when dynamic analysis is also performed, it is possible to omit the static discussion using the seismic force due to basic earthquake ground motion S_2.

c. Direction of action of vertical seismic coefficient

The vertical seismic coefficient is combined with the horizontal seismic coefficients in the unfavorable direction, which is usually taken as the upward direction.

(2) Seismic motion used in the dynamic evaluation

The horizontal seismic motion used in the dynamic analysis is obtained from the basic earthquake ground motion S_2 defined at the outcrop of the site by a deconvolution analysis to the level of the lower boundary of the analytical model.

It is supposed that the vertical seismic intensity acts in combination with the horizontal seismic motion, both acting in the unfavorable directions at the same time. The vertical seismic coefficient used in this case is determined on the basis of the seismic force used for static investigation.

4.2.3 Aseismic design methods

The following analyses may be used according to the requirement for safety evaluation of the foundation ground.

{1} Analysis using sliding-plane method and other conventional methods
{2} Static analysis
{3} Dynamic analysis

Usually, when a more detailed analytical method is adopted, the obtained results have a higher reliability. Hence, as shown in Figure 4.2.3-1, when the safety evaluation of the foundation ground is performed, investigation is performed in the following sequence with increasing reliability: sliding-plane method and other conventional methods, static analysis, and dynamic analysis. If in any of these analytical stages, it is found that the safety evaluation standard value described in section 4.2.4 (2) "Evaluation standard value" is satisfied, more detailed analyses can usually be omitted without any problem.

(1) Analysis using sliding-plane method and other conventional methods

a. There are the following types of sliding-plane method [4.2.3-1 and 2].

{1} Circular sliding-plane method
{2} Plane sliding-plane method
{3} Composite sliding-plane method

The specific scheme to be adopted is determined according to the shape of the sliding plane, which is determined in the survey on the geological conditions and topography of the ground concerned.

For an isotropic soil, usually, the sliding stability of the building foundation bottom is investigated. If the safety rate is good enough, no further detailed analysis is needed.

For an anisotropic soil or a heterogeneous soil, in addition to the investigation of the sliding stability of the building foundation bottom, it is also necessary to perform investigation of the sliding stability along a weaker layer. If a high enough safety rate is found, there is no need to perform a more detailed analysis on the sliding safety, just as in the case of an isotropic soil.

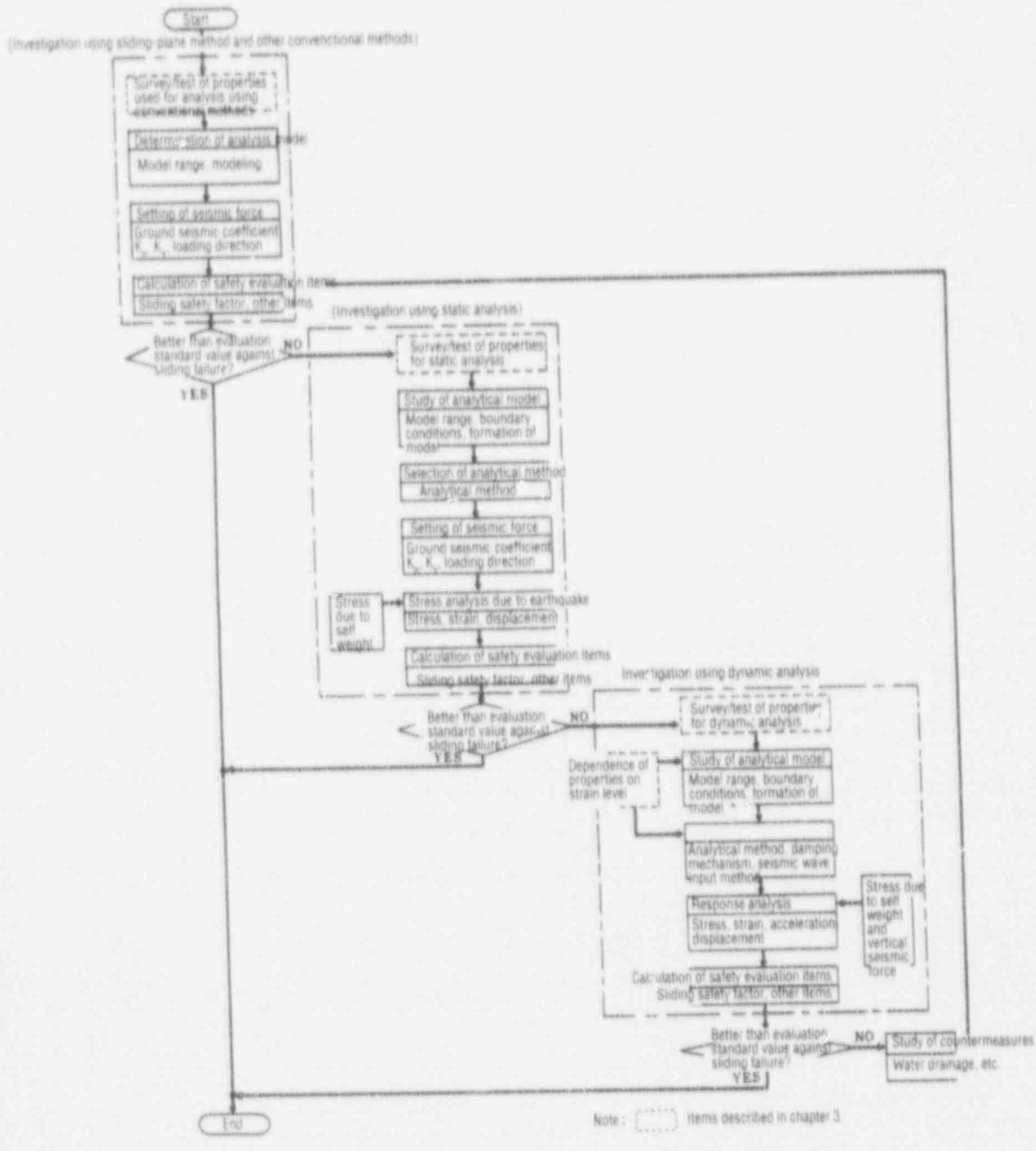


Figure 4.2.3-1. Flow sheet of safety investigation of nuclear reactor building foundation bedrock [4.1-1].

b. With regard to the bearing strength, the bearing strength of the ground obtained in plate loading test or using bearing strength formula is compared with the contact pressure generated during earthquake for evaluating the stability. As far as the deformation is concerned, if needed, the bedrock is considered as an elastic body for an investigation using an elastic theory.

(2) Static analysis

In the static analysis, the finite element method or other method are used to determine the stress distribution, displacement distribution, etc., in the ground; the results are used for evaluating the stability.

a. According to the scheme for determining the mechanical characteristics of the soil material, the analytical schemes can be generally divided into the following two types:

{1} Linear analysis (elastic analysis)
{2} Nonlinear analysis (nonlinear elastic/elastoplastic/viscoelastic analysis, No-tension method, etc.)

In a nonlinear analysis, the stress distribution in the soil is rearranged according to the nonlinear characteristics of the properties. Hence, the sliding safety factor tends to be increased somewhat than in the case of linear analysis.

b. While the cross section and the analytical model range are selected appropriately based on the results of the sliding-plane method, the material properties, boundary conditions and element division of the model also should be selected appropriately in consideration of the geological conditions and the building location.

c. The boundaries of the analysis model are determined by considering the topography of the site such that the effect of the boundary conditions do not affect the stress in the soil under investigation. For the model width, a satisfactory result can usually be obtained by extending about 2.5 times the foundation width to the both sides from the center of the building.

d. As far as the boundary conditions of the model are concerned, usually, in the case of long-term load and vertical seismic force load, the lower boundary is fixed and the side boundaries are modeled as vertical roller; in the case of horizontal seismic force load, the lower boundary is fixed and the side boundaries are modeled as horizontal roller.

e. In the case when a significant nonlinearity in the mechanical characteristics of the soil material is found and a significant influence on the stability evaluation is judged, the analytical method which reflects the nonlinear characteristics of the material should be adopted.

f. The aseismic stability evaluation is usually performed using the following analytical sequence:

{1} Calculation of stress in the soil due to the self weight.
{2} Calculation of stress in the soil by seismic force.
{3} Calculation of the stress in the soil as the stresses in {1} and {2} are combined.
{4} Stability evaluation of the soil using the stress obtained in {3}.
{5} If needed, stability evaluation of the soil is performed using the displacement obtained from the results of {2}.

(3) Dynamic analysis

In the dynamic analysis, finite-element method or other methods are used to find the stress distribution, displacement distribution, etc., in the ground, and the results are used to evaluate the stability.

a. The analysis methods can be classified depending on the solution of the equations of motion.

{1} Mode superposing method (model analysis method)
{2} Direct integration method
{3} Complex value analysis method (Fourier transform method).

These analytical schemes may also be divided into the following two types of analytical methods according to the method of handling the mechanical characteristics of the ground material:

{1} Linear analysis (elastic analysis)
{2} Nonlinear analysis (equivalent linear analysis, step-by-step nonlinear analysis)

Usually, in nonlinear analysis, as the strain level increases, the damping constant of the soil material increases. Hence, the response value can be suppressed lower than that of the linear analysis, and the safety factor against sliding failure to increase.

The results of analysis using these techniques are closely related to handling of boundary conditions and soil properties. Hence, when dynamic analysis is implemented, an appropriate method, that can fully displey the stress distribution and displacement distribution in the soil, should be used on the basis of a suitable engineering judgment on the geological conditions and building layout.

b. The bottom of the dynamic analysis model is called the base ground for analysis. Its depth is usually set at the position where there is no significant change in the maximum amplitude of the incident wave. However, as long as the dissipation wave motion due to the influence of topography and the building is negligible as compared to the magnitude of incident wave motion, the basement for analysis can be made even shallower when the boundary conditions are modeled such that the dissipation energy is absorbed by them.

In addition, recently, the boundary element method (BEM) has been used to treat the ground problem in more and more cases. Its basic solution can meet the Sommerferd radiation conditions and the infinity of the ground can be evaluated unconditionally [4.2.3-3]. Hence, it is believed to be a powerful means for performing dynamic analysis in the future.

c. The analysis range in the horizontal direction of the model for dynamic analysis is determined considering the features of the analysis method. In principle, the width is selected to ensure that the response spectrum is similar to the response spectrum of the free-field ground. For a homogeneous ground, the anelysis boundary is usually set at points separated [from the building] in the vibration direction of the building by a distance over 2.5 times the width of the building [4.2.3-4,5,6]. However, by determination a nonreflective boundary on the side, it is possible to reduce the analysis range.

d. The dynamic soil properties are used for the dynamic analysis. The dynamic deformation/damping characteristics are usually represented by the shear stiffness, Poisson's ratio, and damping constant. These, however, usually depend on the magnitude of the strain generated and the confinement pressure. Since no significant strain is generated in the bedrock, in many cases, it is assumed that there is no change in the properties during the earthquake. In the case when nonlinear mechanical characteristics are necessary for the surface ground and weak layer, the static analysis results are used to evaluate the corresponding properties. In the case of equivalent linear analysis, step-by-step calculation is performed on the basis of the prescribed relation between strain and properties until convergence is realized.

e. The seismic stability evaluation is usually performed in the following analysis sequence:

{1} Calculation of the underground stress generated by self weight.
{2} Calculation of underground stress due to the action of the static vertical seismic coefficient.
{3} Calculation of the response values (underground stress, acceleration, displacement, etc.) due to the action of the horizontal seismic motions.
{4} Calculation of the stress in the ground due to the sum of stresses of {1} {2} and {3}.
{5} Stability evaluation of the ground using the stress obtained in {4}.
{6} If needed, the response characteristics derived in {3} are used for safety evaluation of the ground and the evaluation of presence/absence of the unique dynamic characteristics of the ground.

(4) Others

a. If needed, three-dimensional analysis is performed. However, as long as there is no geologically weak surface, the three-dimensional analysis results usually have higher stability than the two-dimensional analysis results. Hence, in the conventional case, two-dimensional analysis is believed to be sufficient.

b. In the case when the foundation ground contains weak layers, or there are locations with extremely different stiffness values, if the mechanical model of a continuous body is used, a tensile stress region, or region with local safety factor below 1.0 (see section "4.2.2. Safety evaluation"), may be generated in the foundation ground. In the case when it is determined that the results affect the stability of the foundation ground, it may be necessary to study the seismic stability using more detailed methods, such as nonlinear elastic analysis, no-tension method, etc., so that the redistribution of stress is taken into consideration [4.2.3-7]. However, since these methods are for analysis of a continuous body, for hard ground soil with joint cracks and discontinuous surface, although they are effective analysis methods for approximate prediction of the degree of deformation, they are nevertheless inappropriate for studying the precise behavior of the discontinuous surfaces.

In the case when the tensile stress along weak layer is significantly developed so that a continuous plane is formed or in the case when the behavior of the discontinuous plane is the main factor in determining the stability of the foundation ground, the seismic stability is studied by using a joint model and other mechanical models of a discontinuous body [4.2.3-8,9]. When these mechanical models of a discontinuous body are used, evaluation of properties of the model should be performed and the appropriateness should be suitably confirmed.

c. Depending on the method of treating the pore water pressure, the aforementioned analysis methods can be divided into total stress analysis and effective stress analysis. Usually, safety evaluation is performed by total stress analysis. In the case when it is possible to make an appropriate evaluation of the pore water pressure generated, the effective stress method may also be used to evaluate the stability. For further information concerning ground type and total stress representation/effective stress representation, please see section "3.3.5. Representation method of properties and application in design."

d. In the case when the underground water level is higher than the bottom of the building foundation and an uplifting pressure acts on the building, the sliding stability of the building foundation bottom surface is evaluated with the uplifting force taken into consideration.

4.2.4 Safety evaluation

(1) Evaluation items

a. Study of sliding failure

The stability of the foundation is evaluated using the "sliding failure safety factor."

(a) The sliding failure safety factor used for sliding plane method is determined considering the equilibrium of the shear force or moment with respect to the sliding plane, using the following formulas.

(Safety factor F_m due to equilibrium of moments)

$$F_m = \frac{\text{Moment of force that can resist the sliding}}{\text{Moment of force that tries to cause the sliding}} \qquad (4.2.4\text{-}1)$$

(Safety factor F_s due to equilibrium of shear forces)

$$F_s = \frac{\text{Sum of shear resistance forces on the sliding plane}}{\text{Sum of shear forces on the sliding plane}}$$

(b) For the static and dynamic analyses when the foundation ground is modeled using the finite element method, etc., the distribution of stresses on the sliding plane are used to calculate the sliding safety factors by formula (4.2.4-1), etc. Also, in this case, the sliding planes necessary to evaluate are those for which it is determined that a more detailed study is needed based on the results of sliding-plane method or those for which it is determined a new evaluation should be performed to account for the distribution of local safety factor, direction of the potential sliding plane (mobilized plane), etc. For the ground type and methods used to determine the local safety factor, a detailed description is presented in section "3.3.5. Representation method of properties and application in design."

b. Others

Depending on the foundation ground at the point of evaluation, in some cases, there may be evaluation items other than the sliding failure which can affect the stability of the nuclear reactor facility on the basis of engineering judgment on geology/geological structure, ground properties, etc. For example, in the case of heterogeneous ground and anisotropic ground, depending on the nature of the heterogeneity and anisotropicity, it is necessary to investigate the following items:

(a) In the case when a ground has soft rock and hard rock distributed in an irregular way so that there exist very large differences in the local ground stiffness, or in the case when the ground has significantly deteriorated portions due to weathering, underground water, rupturing effect, hot-water deteriorating effect, etc., it is necessary to study the stress concentration in the ground and the different degrees of settlement.

(b) For the aforementioned heterogeneous ground or anisotropic ground, in addition to the study performed on the deformation caused by the weak strata, there are some areas in which it is necessary to study the response acceleration, response spectrum and stress to evaluate the presence/absence of unique vibration characteristics during earthquake.

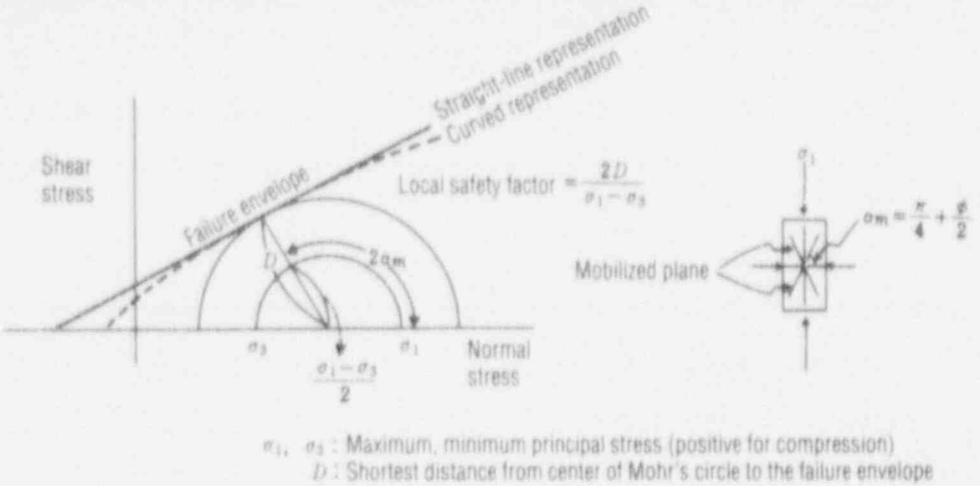


Figure 4.2.4-1. Definition of local safety factor (example).

(2) Evaluation standard values

a. Evaluation of sliding

The sliding-plane method and static analysis method are based on the traditional seismic coefficient method, in which the earthquake is regarded as a static phenomenon. In this analysis, the standard value for evaluating safety is usually taken as about 2.0, independent of the type of foundation ground of the nuclear power plant.

In the case of isotropic ground, evaluation is usually limited to the sliding of the foundation bottom surface. Consequently, only the properties at the bottom of the foundation need to be evaluated, and the analysis accuracies of both the sliding-plane method and the static analysis are considered to be about the same. That is, for isotropic ground, it is possible to assume that the sliding-plane method and static analysis have the same level of general accuracy. For these two analysis methods, the evaluation standard value is taken as 2.0 as in the conventional scheme.

For heterogeneous ground or an anisotropic ground, it is necessary to study the stability of the sliding plane along the weak strata in addition to the sliding of the foundation bottom surface. The sliding-plane method differs from static analysis in that the stiffness and the nonlinear characteristics of the ground cannot be considered. As a result, in certain respects, it is impossible to say that the ground characteristics are reflected fully in evaluating the stability against sliding. Therefore it is desirable that a certain margin over the aforementioned evaluation standard value of 2.0 be considered for sliding-plane method.

For static analysis, it is possible to adjust the model corresponding to the characteristics of the ground. Consequently, the analysis precision can be taken as the same for both isotropic ground and heterogeneous ground. Hence, the evaluation standard value used for nonhomogeneous ground and anisotropic ground can be taken as identical to that of isotropic ground, i.e., 2.0.

Also, the aforementioned evaluation standard values in the various stages are used for the following judgment: if the obtained safety factor is greater than the evaluation standard value, the sliding stability is taken as fully guaranteed, and the ground does not need further detailed evaluation.

Dynamic analysis is a method having much higher precision than the seismic coefficient method with respect to both property evaluation and analysis accuracy. As the evaluation standard value for sliding stability, 1.5 is used as the instantaneous sliding safety factor of dynamic analysis due to basic earthquake ground motion S_2. In consideration of the fact that the sliding safety factor of fill-type dam and other important conventional public facilities is taken as 1.2 for sliding safety evaluation [1.2.2-4], selection of the evaluation standard value for the

Table 4.2.4-1. Evaluation standard values for sliding of foundation ground of nuclear reactor building.[1]

Sliding-plane method	Static analysis	Dynamic analysis
2.0	2.0	1.5

[1]Reference values for seismic evaluation of the foundation of nuclear reactor building with respect to sliding safety factor.

instantaneous sliding safety factor derived from the dynamic analysis of 1.5 provides a stricter condition for the safety evaluation of the ground which directly supports the nuclear reactor building.

b. Evaluation of items in section "4.2.4(1) Others"

Evaluation is performed for each item in section "Others."

4.3 Peripheral slope of nuclear reactor building

4.3.1 Formation of soil model

(1) Slope as object for stability evaluation

The peripheral slope of a nuclear reactor building refers to the slope for which the distance between the toe of the slope and the nuclear reactor building is less than 50 m, or less than about 1.4 times the height of the slope.

(2) Properties

The properties used for stability analysis of the peripheral slope of a nuclear reactor building can be determined in the same way as in section "4.2.1(2) Properties."

(3) Other conditions that should be taken into consideration

In order to investigate the stability of the peripheral slope of the nuclear reactor building, it is necessary to study the following items in addition to the seismic force.

a. Underground water

Usually, for slope stability analysis, the soil below the underground water level is taken as being in saturated state. For the banking slope and ground made of soil material in saturated state, excessive pore water pressure is generated due to the effect of the shear stress during earthquake, and the stability against sliding is decreased. Hence, an effective measure for maintaining stability of the slope is to actively lower the underground water level using drain holes and drainage tunnels.

b. Initial stress mode

When the stability in earthquake is to be studied, the stress in the slope caused by excavation or banking is regarded as the initial stress. Its evaluation is usually performed by an analysis in consideration of the topography and geological state. When there exists a large-scale fault or distorted layering structure in the periphery of the site, the initial stress should be measured in order to confirm its influence.

c. Liquefaction of soil

In the case when the banking slope or the soil just beneath the banking soil are made of sandy earth or other earth materials, and also when the soil is below the underground water level, study of liquefaction should be performed according to section "3.3.5(3) Representation of dynamic strength characteristics."

4.3.2 Design seismic force

(1) Seismic force for static evaluation

In principle, the seismic force for static seismic evaluation is determined according to the following formula or according to the equivalent seismic coefficient determined in consideration of the seismic characteristics of the soil based on the basic earthquake ground motion S_2.

$$K_H = n_1 \cdot n_2 \cdot K_0 \qquad (4.3.2\text{-}1)$$

where K_0: standard design seismic coefficient, taken as 0.2
n_1: correction coefficient of the site, taken as 1.0
n_2: additional response coefficient depending on site conditions, slope shape, etc., usually taken as 1.5.

In principle, the vertical seismic coefficient is taken as 1/2 the horizontal seismic coefficient. They are assumed to act in the unfavorable directions at the same time. When the aforementioned seismic coefficients are used, the following items are taken into consideration.

a. For the basis of $K_0 = 0.2$, please see section "4.2.2(1) Seismic force for static evaluation."

b. According to above item (a) and the analysis evaluation of the equivalent seismic coefficient and sliding safety factor (see section "4.6.2. Peripheral slope of nuclear reactor building"), the horizontal seismic coefficient of 0.3 nearly corresponds to the seismic force caused by basic earthquake ground motion S_2. The response characteristics of the slope depend significantly on the slope shape and the seismic characteristics of the soil that forms the slope. Hence, it is necessary to make sufficient consideration when they are used. As a rule of thumb at present, the slopes to which horizontal seismic coefficient $K_H = 0.3$ can be applied include slopes which have thin surface soil and thin talus and are made of soil having V_S greater than 300 m/s, with an average slope gentler than 1:1.2 and with a height less than 150 m. Also, the maximum acceleration of the basic earthquake ground motion S_2 is lower than 500 Gal.

c. In the case when it is predicted that the seismic coefficient of the slope is significantly different from the horizontal seismic coefficient (K_H) determined using the aforementioned formula, it is possible to use the equivalent seismic coefficient derived in consideration of the seismic characteristics based on the basic earthquake ground motion S_2 or a modified equivalent seismic coefficient which is expressed as a function of the height.

d. Depending on the soil conditions, slope shape, and other states, the unfavorable direction of action of the vertical seismic coefficient may be either upward or downward as determined by the relation between the sliding force due to earthquake and the resistance force. In many cases that have been studied up to now, the unfavorable direction is usually downward when the sliding plane is deep while it is usually upward when the sliding plane is shallow.

(2) Seismic motion for dynamic evaluation

The seismic motion used for dynamic analysis is determined according to section "4.2.2(2) Seismic motion used in dynamic evaluation."

4.3.3 Aseismic design method

Stability evaluation of the peripheral slope of the nuclear reactor building is performed according to section "4.2.3 Aseismic design method."

4.3.4 Evaluation of stability

(1) Evaluation items

a. Consideration of sliding

In principle, safety evaluation of a slope during earthquake is performed using the sliding safety factor in consideration of the sliding along weak surface or in the unstable region, etc.

(a) The sliding safety factor on the sliding plane is determined from the definition in Equation (4.2.4-1).

(b) The safety factor in static and dynamic analysis is also determined from the definition in Equation (4.2.4-1).

In addition, the sliding planes to be evaluated are selected according to section "4.2.4(1) Evaluation items."

b. Others

In addition to the aforementioned evaluation items, there are also cases when it is necessary to study the distribution of local safety factor, expansion of tensile stress region, and deformation of earth slope during earthquake. It should be noted that the local safety factor (see Figure 4.2.4-1) is only one of indices for the local damage of the individual elements in the finite-element method. As long as the elements with local safety factor smaller than 1.0 do not form a continuous sliding plane, it [the local safety factor] is not directly related to the sliding failure of the whole slope. However, it is an effective index for evaluating the local stability of the slope.

(2) Evaluation standard values

a. Evaluation of sliding

When the seismic stability of the peripheral slope of the nuclear reactor building is evaluated it is only necessary to make sure that collapse of the slope would not have secondary effect on the nuclear reactor building. Hence, except in certain special cases, there is no need to tackle the problem of deformation of the slope. Consequently, as long as only the safety evaluation of slide is taken as the subject, the slip-surface method and the static analysis method are believed to have the same analysis precision.

Table 4.3.4-1. Evaluation standard values with respect to the peripheral slope of nuclear reactor building.[1]

Sliding-plane method	Static analysis	Dynamic analysis
1.5	1.5	1.2

[1]Values as index for seismic evaluation of the peripheral slope for sliding safety factor

According to section "4.3.2(1) Seismic force for static evaluation," the design horizontal seismic coefficient K_H is taken as 0.3, which is about 1.2-3.0 times that for a filled-type dam, etc. [4.2.2-4], and is believed to be a rather large seismic force. On the other hand, for the standard value to evaluate the stability using the sliding-plane method and excavation analysis method, a value of 1.2 to 1.5 has been used traditionally regardless of the type of the slope, such as banking slope, excavation slope, natural slope, etc. In consideration of the aforementioned features, the evaluation standard value for the safety evaluation using the sliding-plane method and static analysis is determined to be 1.5.

The dynamic analysis has a much higher accuracy than the aforementioned two analysis methods based on the seismic coefficient scheme. The evaluation standard value for sliding stability is determined to be 1.2. This value is equal to the sliding safety factor of 1.2 taken for filled-type dam in sliding stability evaluation using the seismic coefficient method. However, since this sliding safety factor for dynamic analysis is used to evaluate the instantaneous sliding condition, the safety evaluation of the slope is rather strict.

b. Evaluation of items in section "4.3.4(1) Others"

Evaluation is performed of each item in "Others."

4.4 Important outdoor underground structures

4.4.1 Basic times

(1) Scope of objective structures

The important outdoor underground structures refer to the structures related to emergency cooling facilities, such as water inlet, water channel, water pit (pump chamber), seawater duct, and other seawater piping support structures. They form a very long structure from the water inlet to the nuclear reactor building, and may be affected easily by such conditions as topography of the site, geology, structure layout planning, etc. Consequently, sufficient care should be exercised in their seismic design. Figure 4.4.1-1 illustrates an example of the emergency cooling facility.

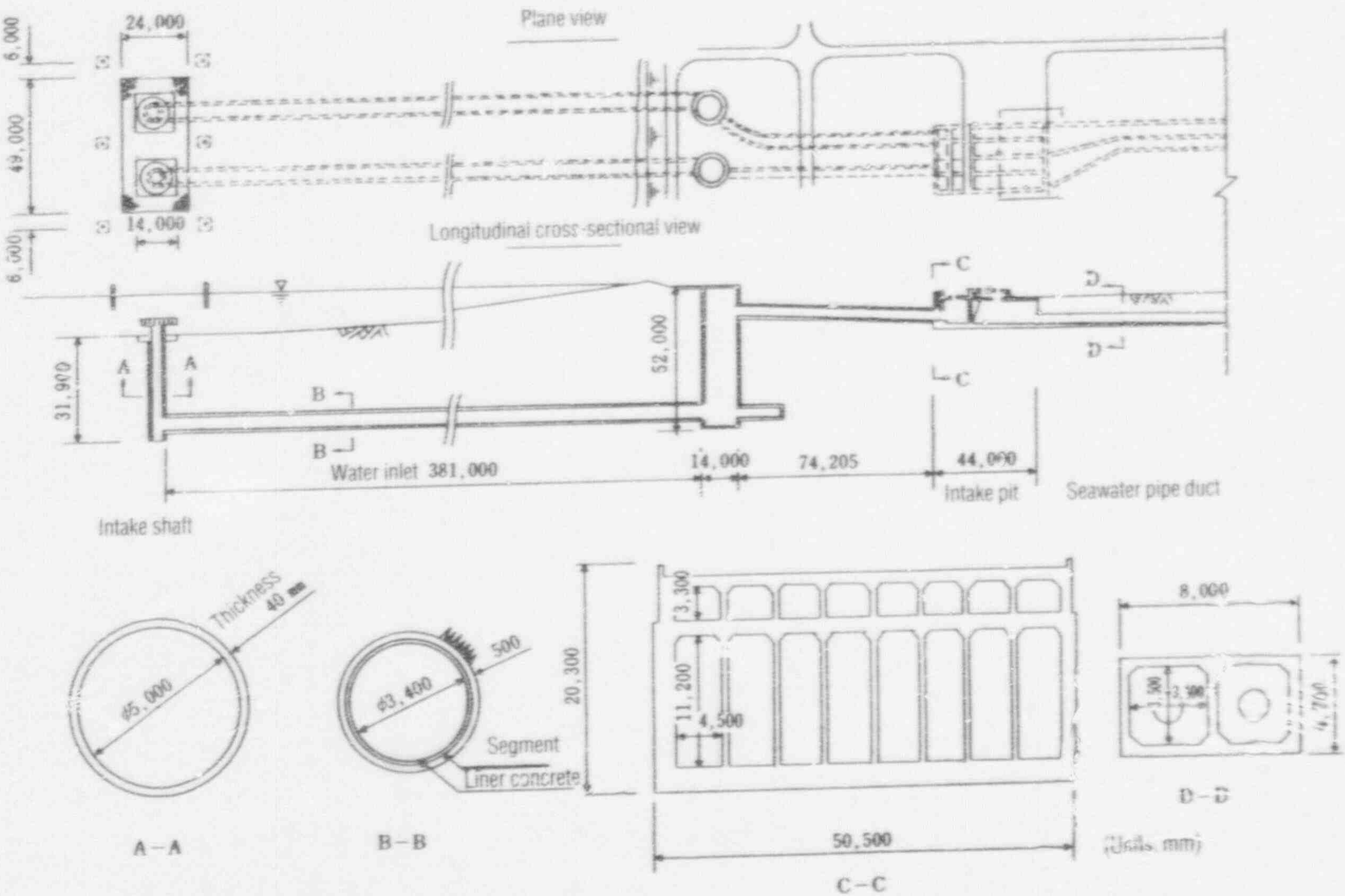


Figure 4.4.1-1. Conceptual diagram (example) of emergency cooling facility.

(2) Functions needed

For the seismic design of the emergency cooling facilities, the following functions must be maintained even under the basic earthquake ground motion S_1 or S_2.

{1} It should be able to take seawater with the prescribed flow rate through the water inlet and send it to the residual heat removal system after passing through the water channel, pump chamber, and seawater pipe. That is, it should be able to maintain the water transporting function of the water channel, the support of the pump, and the support of the seawater pipe.

{2} Even when the nuclear reactor is totally shut down after an earthquake, the aforementioned functions still should be maintained to maintain the safe shut-down state.

4.4.2 Items that should be taken into consideration

(1) Effects of earthquake

In the seismic design, structures to be evaluated include the following types:

{1} Structures that support machines with high importance
{2} Structures that are mainly underground
{3} Long and large structures.

For these structures, the following earthquake influences must usually be taken into consideration.

a. Stability of soil in the periphery of the structure during earthquake

The structures as the subjects are usually underground structures. For underground structures, the seismic safety strongly depends on the seismic stability of the surrounding soil. Consequently, depending on the degree of seismic stability of the surrounding soil, it is necessary to make drastic change for the ideas of the aseismic design of the structure. As a result, sufficient care should be exercised in its evaluation

b. Deformation of soil or earth pressure during earthquake

The behavior of the underground structure during earthquake depends on the motion of the surrounding soil. Therefore, the influence of earthquake on the structure is mainly due to the deformation of surrounding soil. Hence, the seismic safety of the structure should be evaluated mainly for the stress and deformation generated on the structure caused by soil deformation.

c. Inertial force caused by dead and live loads, reaction force by machine

Although the influence of earthquake on the underground structure mainly comes from soil deformation, the effect of the inertial force cannot be neglected for some structures, while for other structures, this effect may be neglected. The live load comes from pumps, pipes, etc., which have different seismic inertial force patterns due to these different vibration characteristics. As a result, it is necessary to determine the inertial force according to these characteristics, and the reaction force on the support structures due to vibration of the machines should be considered appropriately.

d. Differential displacement

In the regions between water inlet and water channel, between water channel and pump chamber, between pump chamber and seawater pipe duct, between seawater pipe duct and nuclear reactor building, and in the adj[illegible] spans of water channel, seawater pipe duct, and other long and large structures, the behaviors during earthquake depend on the structure state, topography, and soil conditions. Therefore it is necessary to study the effects of differential displacement between structures.

e. Dynamic hydraulic pressure during earthquake

For water inlet, water channel, water pit (pump chamber), etc., as their interior is full of seawater, the inertial force of the internal water must be taken into consideration. In addition, for the water tower, the effect of the external seawater should be taken into consideration as dynamic hydraulic pressure or additional mass.

(2) Properties

The properties which should be evaluated are determined appropriately according to the items to be studied and the means used. In the case when soil improvement measures are applied to improve the stability of the surrounding soil of the structure, appropriate testing method should be adopted to confirm the improvement effect.

Table 4.4.2-1 lists the general relationship between the test methods and the seismic evaluation methods. For details of these survey/test methods, please see section "3.3 Survey and soil test."

4.4.3 Design seismic force

Dynamic analysis and its simplified form (response displacement method) are usually used for underground structure, instead of the seismic coefficients method. However, because of past experience and simplicity, the seismic coefficient method is used as a means of rough evaluation of the structure's cross section in the preliminary design stage or as a means of making relative comparison with the conventional design. Because of these reasons, the seismic coefficients for underground structures are determined in the following item-(2).

(1) Seismic motion for dynamic evaluation

The horizontal seismic motion for dynamic evaluation is determined on the basis of the basic earthquake ground motions S_1 or S_2 at the rock outcrop of the site. The basic earthquake ground motion S_1 or S_2 is determined at the rock outcrop of the site according to "Evaluation Guidelines" [4.2.2-3]. The vertical seismic coefficient is set at 1/2 of the maximum acceleration amplitude. The methods of evaluation of design seismic motion at the structural site are different for the following two cases:

{1} In the case when the structure is set on ground identical to the rock outcrop of the site, the design seismic motion is based on the basic earthquake ground motion S_1 or S_2 determined at the rock outcrop.

{2} In the case when the foundation of the structure is set on ground different from the rock outcrop of the site, the design seismic motion is determined appropriately on the basis of the basic earthquake ground motion at the rock outcrop in consideration of the seismic characteristics of the ground on which the structure is set.

Table 4.4.2-1. Relation between survey/test methods and seismic evaluation method [4.1-1].

Survey/ test item	Test method and soil constants to be determined	Seismic character. of ground	Stability of soil during earthquake		Seismic analysis of structures		
		Dynamic analysis	Static analysis	Dynamic analysis	Seismic coefficient method	Response displacement method	Dynamic analysis
Geological survey	Geological soil structure (including water table level, etc.)	○	○	○	○	○	○
Microtremor	Geological soil structure	□		□		□	□
Elastic wave test	PS logging, refractive method; geological soil structure; V_P, V_S, ν_d	○		○		○	○
In situ test	Penetration test; N, etc.	○	○	○	○	○	○
	Borehole loading; K_h, E_s, P_y			□	□	○	
	Plate loading; K_v, E_s, P_y				○	○	
	Bedrock shear; C, ϕ		○	○	○		
Laboratory test	Physical test; ρ, particle size, granularity distribution, consistency	○	○	○	○	○	○
	Uniaxial; q_u		□	□	□		
	Consolidation; P_y, etc. (for clay only)		○	○			
	Static triaxial, etc.; C, ϕ, E_s, ν_s, etc.	□	○	○	○		
	Dynamic triaxial, etc.; τ_f, R_l, E_d, G, h, etc.	○	○	○		○	○

(1) ○ indicates close relation
□ indicates a certain relation

(2) Soil constant symbols

ρ	:	Density
ν_s, ν_d	:	Static and dynamic Poisson's ratios
N	:	Penetration resistance value
K_h, K_v	:	Horizontal and vertical reaction coefficients of soil
E_s, E_d	:	Static and dynamic modulus of elasticity
G	:	Shear modulus of elasticity
C, ϕ	:	Cohesive force, internal frictional angle
τ_f	:	Dynamic shear strength
h	:	Damping constant
V_P, V_S	:	P-, S-wave velocities
P_y	:	Consolidation yield stress
q_u	:	Uniaxial compression strength (C = 2C)
R_l	:	Liquefaction strength

Table 4.4.3-1. Correction coefficient n_2 for different ground conditions.

Ground type	n_2
Ground almost identical to the foundation ground of the nuclear reactor building	1.0
Ground which is softer than the foundation ground of the nuclear reactor building and is expected to amplify the seismic coefficient	1.5

(2) Seismic force for static evaluation

a. Design horizontal seismic coefficient

Design horizontal seismic coefficient (K_H) at the ground surface can be determined by the following equation:

$$K_H = n_1 \cdot n_2 \cdot n_3 \cdot K_0 \tag{4.4.3-1}$$

where K_0: standard design seismic coefficient, taken as 0.2.
n_1: correction coefficient at the site, usually taken as 1.0.
n_2: correction coefficient according to ground conditions, with values listed in Table 4.4.3-1.
n_3: coefficient due to factors other than those described above, usually 1.0.

In the case when the seismic coefficient is believed to be different from the standard seismic coefficients shown here in consideration of past design cases, the equivalent seismic coefficient is determined based on the seismic characteristics of the ground.

b. Underground seismic coefficient

The aforementioned K_H is used as the underground seismic coefficient. However, a lower value can be used on the basis of dynamic analysis of ground or other appropriate method.

c. Vertical seismic coefficient

In the case when the design vertical seismic coefficient (K_V) is considered, in principle, $K_V = K_H/2$, which is the value used for both the portion above ground and the underground portion. For the vertical seismic coefficient, in principle, there is no decrease in the depth direction. However, if there is a decrease due to the site conditions, the decrease pattern may be adopted.

4.4.4 Aseismic design method of structures

(1) Aseismic design sequence

The aseismic design of important outdoor underground structure is carried out according to the following sequence (an example of the design sequence is shown in Figure 4.4.4-1).

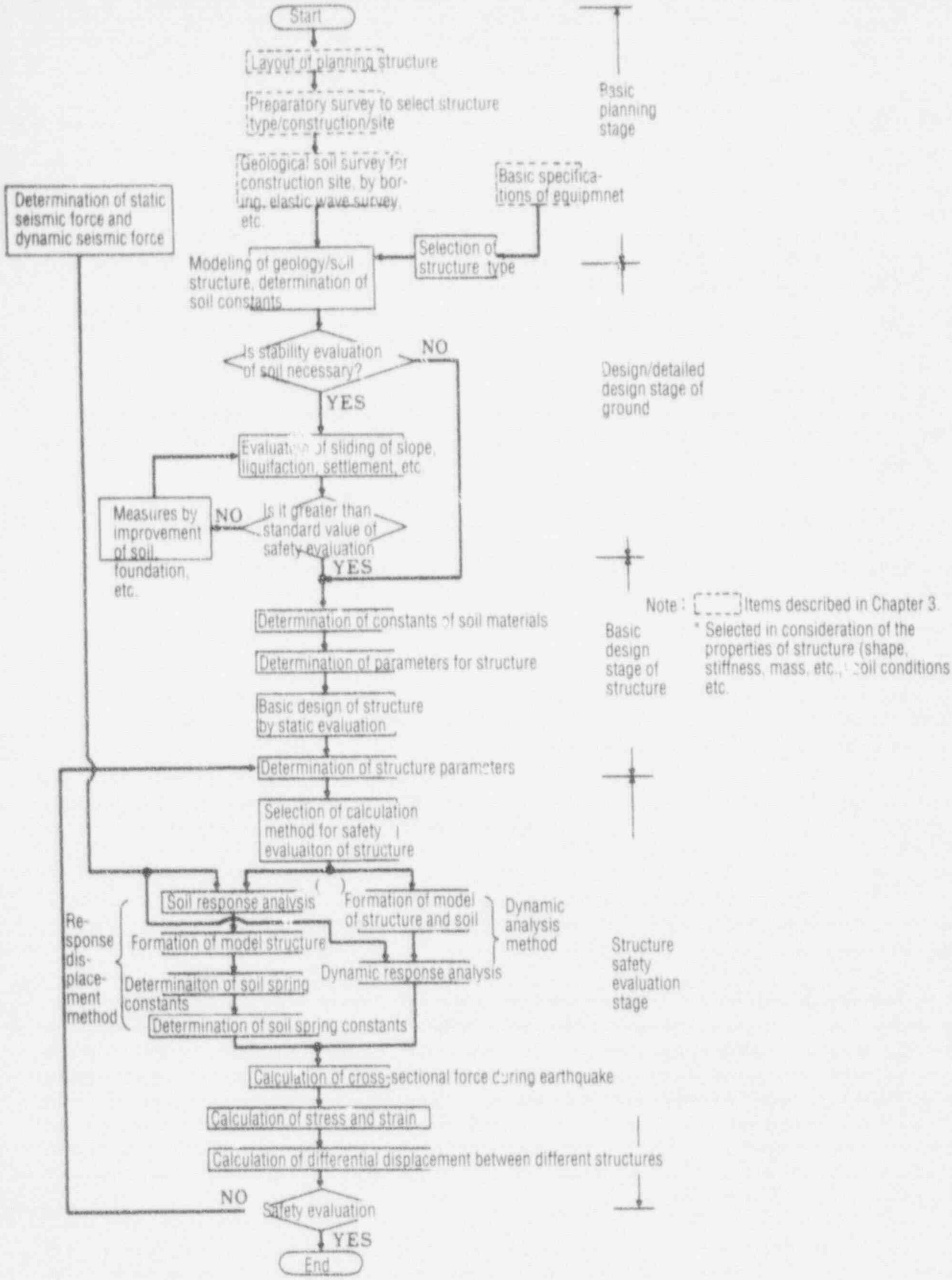


Figure 4.4.4-1. An example of aseismic design sequence of important outdoor underground structure [4.1-1].

{1} Determination of basic requirements
{2} Evaluation of soil stability
{3} Design of structure

a. Evaluation of soil stability

The aseismic properties of the underground structure are closely related to the stability of the surrounding soil. Hence, before the aseismic design of the structure itself, it is necessary to perform safety evaluation of the sliding of the peripheral slope and liquefaction of sandy soil, etc., according to the requirement.

b. Design of structures

{1} Basic design
In the basic design stage of the structure main body, the seismic force (seismic coefficient) used in the static evaluation described in section "4.4.3 Design seismic force" is used for evaluation of the structure cross section (first draft), etc.

{2} Detailed design
In the safety study stage of detailed design, the dynamic evaluation method described in section "4.4.3 Design seismic force" is used for evaluating the safety by "response displacement method" or "dynamic analysis method." If needed, the earthquake loading to the machine is determined by "dynamic analysis method."

(2) Seismic coefficient method

In the seismic calculation using seismic coefficient method, the inertial force caused by the self-weight of the structure and the load, the earth pressure during earthquake, and the dynamic hydraulic pressure during earthquake are taken into consideration in calculating the member forces.

{1} When the inertial force during earthquake used in the seismic design by the seismic coefficient method is to be calculated, the inertial force is calculated by multiplying the self-weight and live load by the design seismic coefficient.

{2} The design horizontal seismic coefficient (K_H) and vertical seismic coefficient (K_V) used in the seismic calculation by the seismic coefficient method are derived using the method shown in section "4.4.3 Design seismic force."

{3} Either the deformation of the peripheral soil or the earth pressure is considered as the earthquake effects received by the structure from the peripheral soil. In the seismic calculation by the seismic coefficient method, the earth pressure during earthquake is taken into consideration. When the earth pressure during earthquake is to be calculated, the conventional calculation formula may be used. When the conventional earth pressure equation is used to design the water pit, seawater channel, and other underground structures, the structure's shape and stiffness, the surrounding soil's behavior, etc., should be taken into consideration; the magnitude and distribution profile of the earth pressure during earthquake should be taken into full consideration; also, the method of combining the soil pressures acting on the left and right sides of the underground structure and the way to apply bottom shear force should be selected carefully.

{4} The overburden pressure during earthquake is calculated from the weight of the overburden soil multiplied by ($1 \pm K_V$). Upward (-) or downward (+) is selected to ensure the safer side depending on the conditions.

(3) Response displacement method

In the response displacement method, first, the free-field spectrum is determined; then, the spectrum is input to the structure through the soil springs, and the member forces are calculated. At the same time, effects of the initial force due to the self-weight of the structure, dynamic hydraulic pressure, etc., must be taken into consideration, with these effects superimposed with the effect of the soil displacement to ensure a design on the safe side. In order to calculate the displacement of the soil during earthquake, the type of seismic wave, propagation path, soil conditions, etc., should be taken into consideration, with the conditions selected as most suitable for the seismic evaluation of the structure. The following wave propagation types are considered in the response displacement method:

{1} Wave motion transmitted vertically in the soil

When the soil conditions are simple, usually only the primary shear vibration mode of the soil is taken into consideration. However, when the changes in physical properties of ground in the depth direction are obvious from the results of ground survery, the displacement amplitude distribution of the soil is determined by using an appropriate analysis method, such as the multiple reflection theory.

{2} Wave motion transmitted horizontally on the ground surface

In order to calculate the displacement of soil when the seismic waves are assumed to travel horizontally, it is necessary to wait until more data are accumulated. At present, a simple method, in which the wave motion transmitted along the ground surface is represented by a sinusoidal wave, is used for soft ground.

{3} More complicated wave propagation due to soil conditions, etc.

In the case when the soil is nonhomogeneous, or when the soil layers are inclined, etc., propagation of the wave motion becomes complicated. In this case, the soil is represented by a discrete-mass model or a finite element model. The displacement of the soil can be calculated using dynamic analysis.

In principle, the displacement of the soil is applied to the structure through soil spring. The soil spring constants are determined appropriate from the results of ground survey, soil test, etc., with the shape and stiffness of the structure taken into full consideration. The soil spring constants depend primarily on the properties of the soil. In addition, they also depend significantly on the shape, stiffness, displacement pattern, etc., of the structure. Also, when the nonlinearity of the soil properties during a strong earthquake is significant, they are also affected by this nonlinearity. The optimum soil spring constants for the specific problem to be handled should be determined on the basis of a good understanding of these features. For the calculation methods of the soil spring constants, please see section "3.3.5 Representation method of properties and application in design."

(4) Dynamic analysis method

In the seismic calculation using dynamic analysis, the structure and soil are represented by an appropriate dynamic model to account for the vibration characteristics of the structure, and the displacements and forces of the structure are evaluated.

{1} When a model is to be determined for the structure and soil, the soil conditions, structure, structural characteristics, functional characteristics, as well as analysis purpose, characteristics of input seismic motion, etc., must be taken into full consideration. Methods for modeling structures include discrete-mass model, finite-element model, etc. For the soil model, in addition to the aforementioned two models, the wave propagation may also be calculated by using the one-dimensional multiple reflection model.

{2} The various constants of the soil and structure needed for the dynamic analysis must be determined from the results of the various material tests and ground survey with a full consideration of the analysis. In particular, in the case of a strong earthquake, the soil materials display nonlinearity, which must also be taken into consideration.

{3} When the finite element model or one-dimensional multiple reflection model is used, the stiffness and damping constant of each portion of the model may be evaluated directly from the properties of the material. On the other hand, when a discrete-mass model is used, since the soil spring constants and damping constants are closely related to the analysis method and the characteristics of the analysis model, evaluation should be performed with these factors taken into full consideration. For details of the soil spring constants and damping constants, please see Section "3.3.5 Representation method of properties and application in design."

{4} Methods for calculating the vibration response using these models include mode superposition method, complex response analysis method, direct integration method, etc. It is important to select the method that fits the conditions of the structure and soil and the analysis purpose.

4.4.5 Safety evaluation

Safety evaluation should be performed for all of the items related to the seismic safety of the important outdoor underground structures. The major items include stability of peripheral soil, safety of components, and differential displacement.

(1) Stability of soil

a. Sliding of peripheral soil of structure

In the case when there exist a slope or a shore-protection adjacent to the structures and when the safety of the structure during earthquake is predicted to be affected by the sliding failure of the ground, it is necessary to examine the measures including the planning of layout of the slopes and the important outdoor structures. For detailed description of the method of evaluating the slide stability, please see section "4.2.3 Aseismic design methods."

b. Liquefaction and settlement

For sandy soil, when design is to be made of a structure, it is necessary to confirm the presence/absence of liquefaction of the soil. When liquefaction during earthquake is expected, it is necessary to take the following factors into consideration for the aseismic design: static/dynamic soil hydraulic pressures, increase in buoyancy by the liquefied soil, settlement of the soil, etc. For a soil with a low concentration and a not fully consolidated reclaimed ground, a possibility of a significant differential settlement and its effect on the structure should be studied carefully. For details about liquefaction of sandy soil, please see section "3.3.5 Representation of properties and application in design."

(2) Inspection of safety of structural components

Inspection of seismic safety of structural components is performed in principle by checking if the state of the components subjected to the seismic motion in the dynamic evaluation described in section "4.4.3 Design seismic force" is below the "limit state."

a. Concrete structural components

Although several schemes have been proposed to calculate the limit values corresponding to the ultimate limit state or functional limit state [4.4.5-1], etc., for the reinforced concrete and prestressed concrete structural components, a standard method has not yet been established. It is yet to be developed in the future. Hence, when it is difficult to define the "limit state" and the corresponding limit value, the safety is usually checked according to the allowable stress design method. On the other hand, if the "limit state" at which the support function of the structure can be maintained is clearly defined, and also the corresponding strength capacity, deformation limit, crack width and other limit values are defined appropriately, then, it is possible to check the safety by using the design method based on this "limit state."

b. Steel structural components

The safety of water shaft and water channel made of steel can be inspected by checking if the strain of the component concerned is below the strain corresponding to the "limit state." When the allowable strain is determined, past research works and technical manuals [4.4.5-2,3] may be used as references. The strain of the component is calculated using the properties of the material appropriate for the magnitude of the strain generated.

(3) Differential displacement

The water channel, seawater pipe duct, and other linear structural components in the horizontal direction are affected by the differential displacement of soil due to the spacious distribution of the seismic motion. On the other hand, the water shaft and other vertical linear structural components are affected by the differential displacement of soil in the vertical direction. When these structural components are designed and it is found that the stress and strain in the structural components during earthquake become greater than the allowable limit, the shape/dimensions of the part should be changed or a flexible joint should be arranged at an appropriate position. In this case, it is necessary to make sure that the differential displacement at the point is not greater than the allowable limit.

For the joint portion between different types of structural parts with different vibration performances, such as between seawater pipeduct and water pit, differential displacement may take place during earthquake. In this case, the vibration characteristics of the various structural parts should be taken into consideration to make sure that the differential displacement at the joint portion can be absorbed. Usually, a flexible joint is arranged at the joint portion.

In principle, the response values of the soil and structure system for which the differential displacement needs to be evaluated are determined by dynamic analysis. However, in some cases, only the dynamic response of the soil is calculated, and the response of the structure is evaluated statically on the basis of the soil responses.

4.5 Other civil structures

In addition to the important outdoor underground structures described in the above section, the nuclear plant also has various other civil structures, such as seaport facility, recirculating cooling water inter/outlet facility, bridge, road, tank foundation, electrical/equipment/piping equipment foundations, water drainage route, retaining wall, etc. The aseismic design of these civil structures can be performed according to the standards and guidelines listed in Table 4.5-1. In addition, the design should ensure that these civil structures do not cause trouble for the adjacent important structures during an earthquake. Careful evaluation should be made against landslide, flood, and other natural disasters.

4.6 Analysis of problems related to aseismic design and examples of aseismic design

In this section, we will mainly discuss the items for attention in the aseismic design of the foundation ground of nuclear reactor building, and important outdoor underground structures described in the above, as well as the items for further evaluations regrading the seismic coefficient of the ground described in sections 4.2 and 4.3. All of the cases described in the following are cited from the numerous studies performed by the Japan Society of Civil Engineers (JSCE). For details, please see reference [4.1-1].

4.6.1 Foundation soil of nuclear reactor building

(1) Analysis items

{1} Width of soil model in static analysis
{2} Ground depth for base motion input in dynamic analysis
{3} Relation between seismic motion and equivalent seismic coefficient

For item {1}, parametric study is performed on the influence of the soil model width on the analysis results in static finite element analysis, and the standard soil model width is evaluated.

For item {2}, parametric study is performed on the influence of the ground depth for base motion input on the analysis results in dynamic finite element analysis, and the standard ground depth for base motion input is evaluated.

For item {3}, in the case when stability evaluation of the foundation soil is performed using the sliding-plane method and static analysis, the seismic coefficient is used. Regarding the concept of the equivalent seismic coefficient of the soil, a comparison is made between the soil's design horizontal seismic coefficient $K_H = 0.2$ and standard seismic motion S_2.

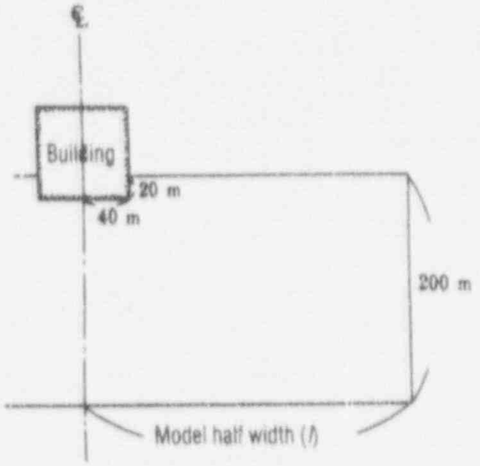
(2) Analysis models

a. Soil model for analysis and parameters of nuclear reactor building

Figures 4.6.1-1-3 illustrate the analysis model. In this case, the building is a standard BWR MARK-II nuclear reactor embedded in the soil 20 m below the ground surface.

Table 4.5-1. Standards/guidelines of aseismic design of underground structures.

	Standards/guidelines	Publishers
1	Guidelines of Aseismic Design of Underground Tunnel (Draft) (published in 1976)	Japan Society of Civil Engineers
2	Specifications of Concrete Standards/Commentary (published in 1980)	Japan Society of Civil Engineers
3	Reinforced Concrete Structure Calculation Standards/ Commentary	Architectural Institute of Japan
4	Building Foundation Structural Design Standards/Commentary (published in 1974)	Architectural Institute of Japan
5	Road/Bridge Specifications/Commentary I. Section of common features II. Section of steel bridges (published in 1981)	Japan Road Association
	Road/Bridge Specifications/Commentary I. Section of common features III. Section of concrete bridges (published in 1981)	Japan Road Association
	Road/Bridge Specifications/Commentary I. Section of common features IV. Section of foundation structures (published in 1981)	Japan Road Association
	Road/Bridge Specifications/Commentary I. Section of common features V. Section of aseismic design (published in 1981)	Japan Road Association
6	Technical Standards/Commentary of Port Facilities (published in 1980)	Japan Port Association
7	Guidelines/Commentary of Aseismic Engineering of Aqueduct Facilities (published in 1979)	Japan Aqueduct Association
8	Explanation of Aseismic Design Guidelines (Draft) (published in 1979)	Japan Railway Facility Association
9	Second Amended Edition of Dam Design Standards (published in 1978)	Japan Major Dam Council
10	New Aseismic Design Method (Draft) (published in 1977)	Ministry of Construction
11	Technical Standards and Official Procedure for Hydraulic Equipment for Power Generation (published in 1974)	Ministry of International Trade and Industry
12	Standards of Application of Structure of Oil Retaining Wall (published in 1977)	Fire Defense Agency



(Analysis conditions)

(1) Side boundary : Horizontal roller

(2) Lower boundary : Fixed

(3) Soil : Homogeneous soil
(V_s = 800 m/s)

(4) Depth of analysis model : 200 m

Model width (2l)	Model width/ building bottom width	Model Name
200 m	2.5	Model – a
400 m	5.0	Model – b
800 m	10.0	Model – c
1600 m	20.0	Model – d

Figure 4.6.1-1. Analysis model (soil model width for static analysis).

(a) Soil model width in static analysis

A study was performed regarding the standard model width by static analysis on the four types of soil model width shown in Figure 4.6.1-1 (the soil model single-side widths measuring from the building center are 1.25 times, 2.5 times, 5 times, and 10 times the width of the building bottom, respectively). In this case, it is assumed that the soil is a homogeneous soil without weak layers (with V_S = 800 m/s constant). Based on past analysis examples, the depth of the soil model is taken as 200 m.

(b) Ground depth for base motion input in dynamic analysis

As shown in Figure 4.6.1-2, the standard ground depth for base motion input is evaluated by performing response analysis for the soil model having four types of depths (0.75 times, 1.5 times, 2 times, and 2.5 times the width of the building foundation, respectively) as the input base ground. In this case, the soil is assumed to be a layered ground without weak layers. The soil model width is taken as 240 m.

(c) Relation between seismic motion and equivalent seismic coefficient

For the model shown in Figure 4.6.1-3, static analysis and dynamic analysis are performed, and comparison is made on the relation between the dynamic seismic motion and the equivalent seismic coefficient. The analysis model has a width of 800 m and a depth of 200 m to account for the distribution of weak layers.

b. Properties of soil model

Table 4.6.1-1 lists the various constants of the soil model.

c. Seismic forces for evaluation

(a) Static seismic force (sliding-plane method, static analysis)

Independent of the depth, the following seismic coefficients are assumed to act on the soil at the same time in the unfavorable directions:

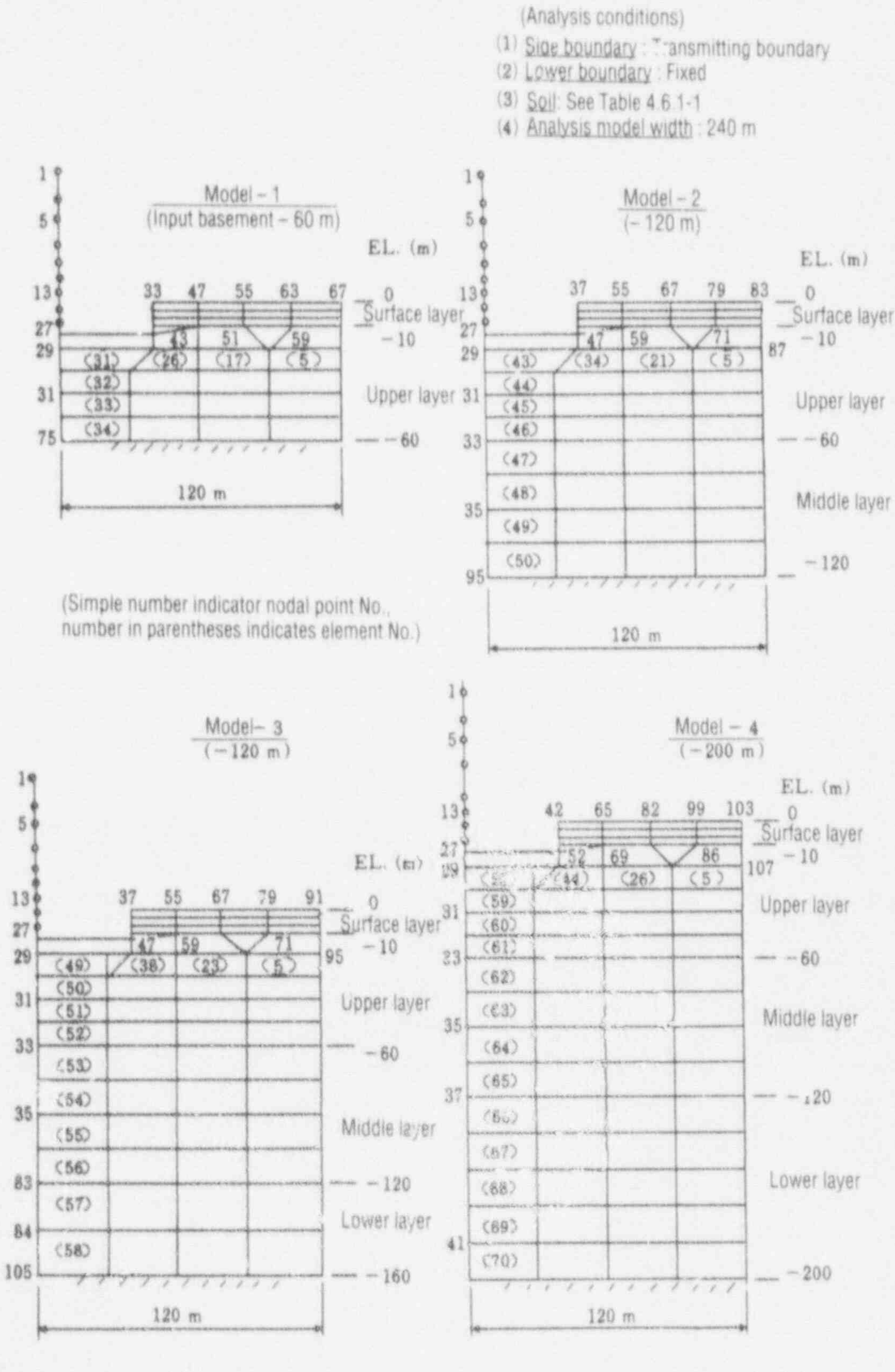


Figure 4.6.1-2. Analysis model (depth for base motion input in dynamic analysis).

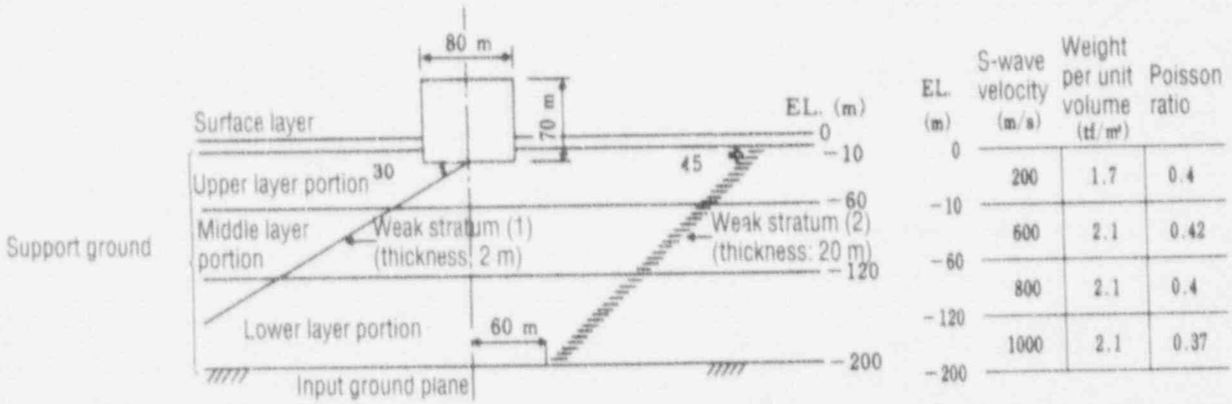


Figure 4.6.1-3. Analysis model (relation between seismic motion and equivalent seismic coefficient).

Table 4.6.1-1. Properties of the model soil used in evaluation.

Item	Units	Surface soil	Weak layer 1 (ruptured belt)	Weak layer 2 (hetero-geneous portion)	Bedrock			Note
					Upper layer portion	Middle layer portion	Lower layer portion	
Layer thickness	m	10	(2)	(20)	50	60	80	
Static modulus of elasticity (E_s)	kgf/cm^2	1,940	2,350	9,400	22,000	38,400	58,700	The value is made identical to that of the dynamic modulus of elasticity
Static Poisson's ratio (ν_s)	—	0.4	0.4	0.4	0.26	0.26	0.26	
Weight per unit volume	tf/m^3	1.7	2.0	2.0	2.1	2.1	2.1	
Cohesion (C)	kgf/cm^2	0	0.4	0.4	5	5	5	
Internal friction angle (ϕ)	degrees	35	25	25	40	40	40	
S-wave velocity (V_s)	m/s	200	200	400	600	800	1,000	
Shear modulus of elasticity (G)	kgf/cm^2	690	820	3,270	7,700	13,700	21,400	
Dynamic modulus of elasticity (E_d)	kgf/cm^2	1,940	2,350	9,400	22,000	38,400	58,700	
Dynamic Poisson's ratio (ν_d)	—	0.4	0.44	0.44	0.42	0.40	0.37	
Damping constant (h)	%	2 ~ 20	2 ~ 20	5	5	5	5	
Note		Dependence of G, h on shear strain is considered	Dependence of G, h on shear strain is considered					

Horizontal seismic coefficient: 0.2
Vertical seismic coefficient: 0.1

On the other hand, the seismic force acting from the building to the soil is calculated using the shear force distribution coefficient A_i in the height direction (value obtained from the dynamic response analysis).

(b) Dynamic seismic force (dynamic analysis)

Among the seismic motions listed in Table 4.6.1-2, five types of seismic motions (simulated seismic waves No. 2, No. 6, and No. 7, TAFT (EW), and Kaihokukyo (TR)) are defined and used at the rock outcrop surface (with S-wave velocity greater than 700 m/s, location of EL. -60 m for this study). The input seismic motion for analysis is obtained by converting the seismic motions defined at the rock outcrop surface to the level of the input ground plane of the analysis model through the one-dimensional deconvolution technique.

(3) Analysis results

a. Ground model width in static analysis

Figure 4.6.1-4 shows the relation between the stress in the ground obtained in the analysis and the ground model width for four types of ground models. It can be seen from these results that if the single-side width of the model is greater than 2.5 times the width of the building bottom surface, there is almost no difference for the calculated stress value in the ground. That is, the effect of the seismic force from the building on the ground can be properly evaluated if the model's boundary is set at a distance 2.5 times the building bottom width from the building center.

b Ground depth for base motion input in dynamic analysis

Figures 4.6.1-5, 6 illustrate the distributions of response acceleration and shear stress along the central axis of the building. As can be seen from these figures, when the input base ground depth is taken as 1.5 times the building bottom width, i.e., 120 m, the effects of the input base ground depth on the response characteristics of both the building and soil are not as sensitive for the analysis results. Consequently, good enough results can be obtained when the input base ground depth of the soil model used for safety evaluation of the soil studied is taken as 1.5-2 times the building bottom width.

c. Relation between seismic motion and equivalent seismic coefficient

Generally speaking, there are the following two methods for determining the equivalent seismic coefficient of soil: determination from the acceleration response values of the dynamic analysis, and determination from the maximum shear stress distribution.

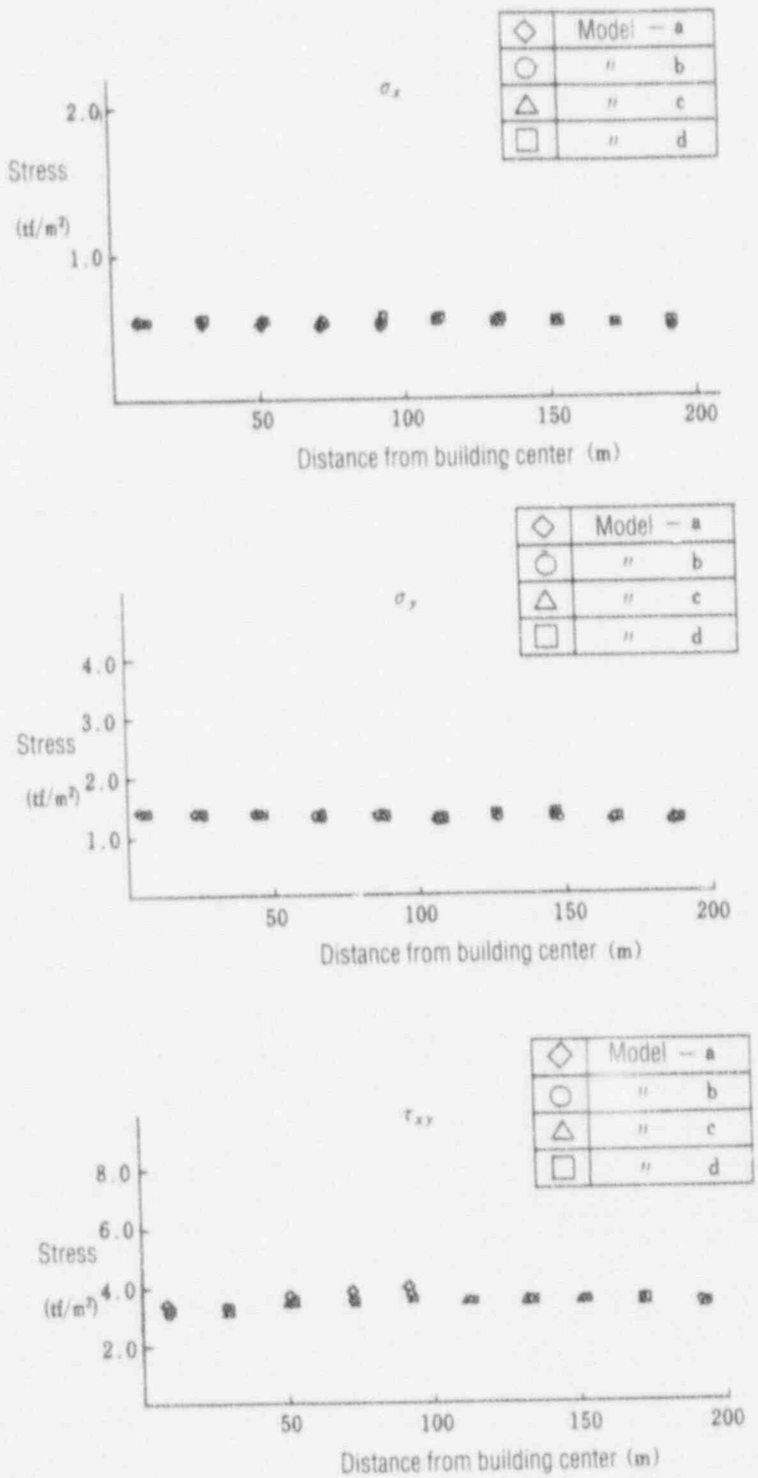
The equivalent seismic coefficient determined from the acceleration response is defined from the peak time history value of the equivalent acceleration defined by the following equation:

$$\text{Equivalent acceleration} = \frac{\sum \text{(product of mass of the soil element assumed to slide and the response acceleration of the element)}}{\sum \text{(mass of the soil element assumed to slide)}} \quad (4.6.1\text{-}1)$$

Table 4.6.1-2. Seismic motions used in evaluation.

Seismic wave name		Max. acceleration (Gal)	Magnitude	Note
TAFT (EW)[3]		147	7.7	The maximum acceleration was adjusted for analysis (200, 300, 400, 500 Gal)
Simulated seismic waves[1]	No. 2	340	6.5	Shallow-focus earthquakes (S_2)
	No. 3	353	8.0	Distant earthquake (S_2)
	No. 4	267	7.0	Near earthquake (S_1)
	No.5	286	8.4	Distant earthquake (S_1)
	No. 6[3]	388	7.5	Near earthquake (S_2)
	No. 7	407	8.5	Distant earthquake (S_2)
Kaihokukyo[2]	TR	287	7.4	—
	LG	193	7.4	—

[1]Basic earthquake ground motion in reference (H.K-2).
[2]Records at Kaihokukyo (TR, LG) in Miyagikenoki earthquake in 1978.
[3]Seismic motion mainly used in stability evaluation of slope.



Note: Verticle seismic coefficient is upward

Figure 4.6.1-4. Relation between soil model width and stress in ground in static analysis (stress value at a depth of 45 m from the ground surface (EL. −45 m)).

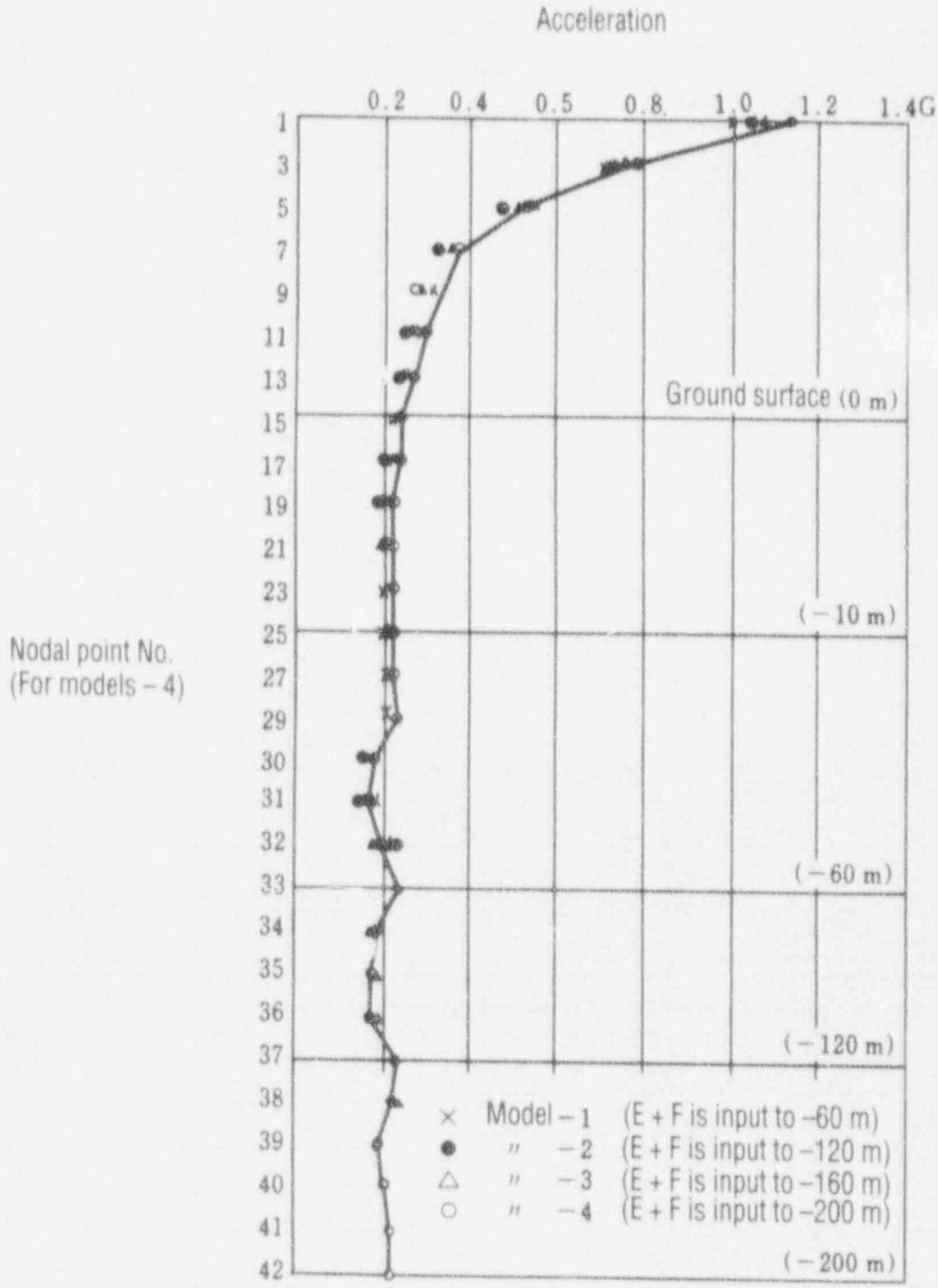


Figure 4.6.1-5. Results of evaluation of depth of input ground plane in dynamic analysis (distribution of acceleration along the central axis of the building (simulated seismic wave No. 2)).

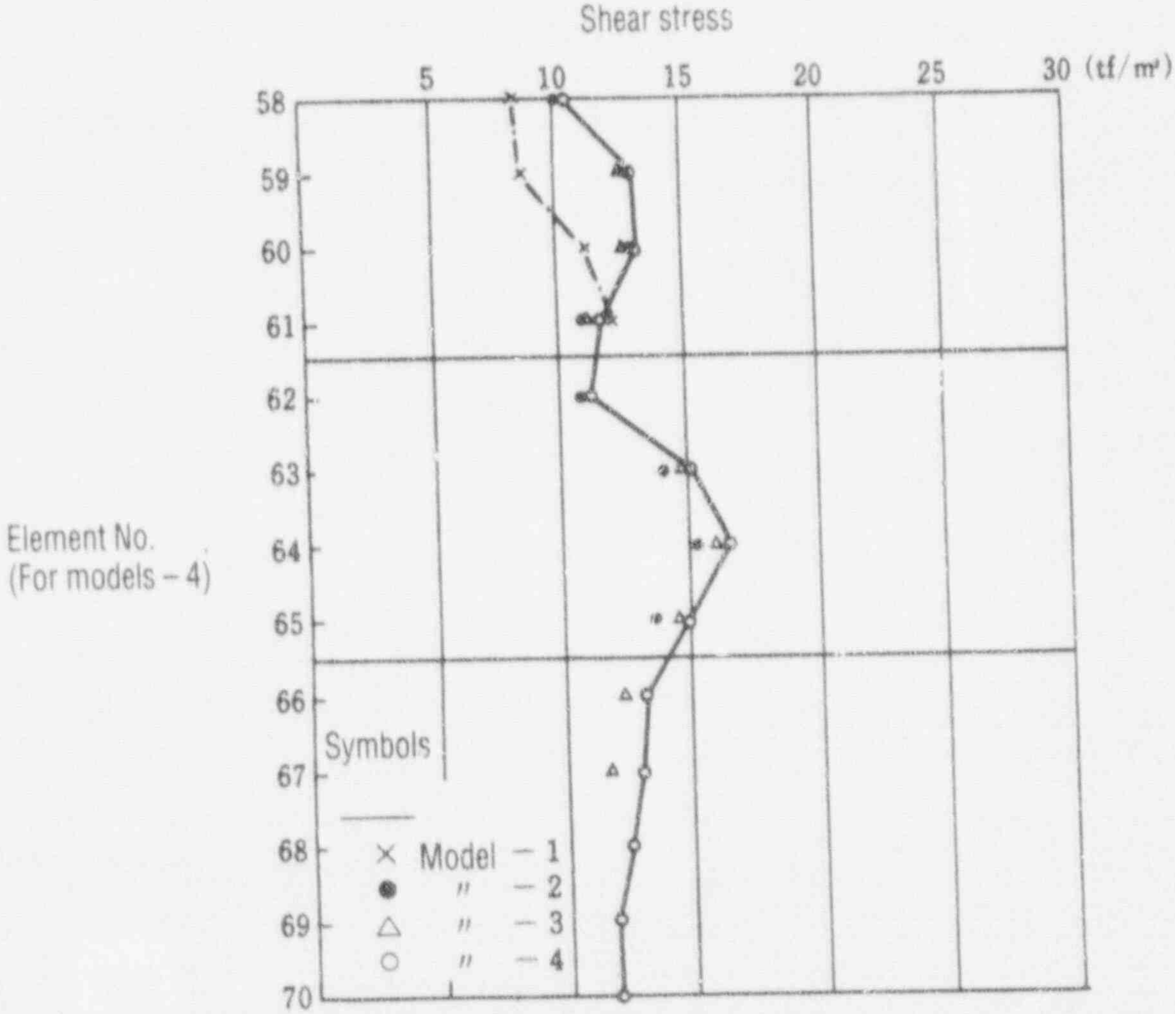


Figure 4.6.1-6. Results of evaluation of depth of input ground plane in dynamic analysis (distribution of shear stress along the central axis of the building (simulated seismic wave No. 2)).

The equivalent seismic coefficient [4.6.1-1] determined from the distribution of the maximum shear stress is defined by the following equation:

$$K_{Hi} = \frac{2(|\tau_i|_{max} - |\tau_{i-1}|_{max})}{h_{i-1} \cdot \rho_{i-1} + h_i \cdot \rho_i} \tag{4.6.1-2}$$

where K_{Hi}: seismic coefficient at i-th layer
$|\tau_i|_{max}$: maximum shear stress at i-th layer
ρ_i: unit-volume weight at i-th layer
h_i: thickness of i-th layer

(a) Equivalent seismic coefficient determined from acceleration response value

Figure 4.6.1-8 illustrates the equivalent seismic coefficients obtained from the results of dynamic analysis for the sliding plane as shown in Figure 4.6.1-7. It can be seen from the figure that, except for the simulated seismic wave No. 7, the equivalent seismic coefficient is less than 0.2. On the other hand, as shown in Figure 4.6.1-9, the equivalent acceleration for simulated seismic wave No. 7 becomes greater than 0.2 only in a fraction of a second.

(b) Equivalent seismic coefficient determined from the maximum shear stress distribution

As can be seen from Figure 4.6.1-10, for the equivalent seismic coefficient (K_{Hi}) determined from the distribution of the maximum shear stress in the depth direction using the one-dimensional wave theory, 0.2 is the upper limit except for the surface layer portion. The results are in agreement with the equivalent seismic coefficient determined from the acceleration response.

(c) Results of evaluation of design horizontal seismic coefficient based on sliding safety factor comparison

By comparing the sliding safety factors derived from the static analysis performed using a design horizontal seismic coefficient of 0.2 with that from dynamic analysis performed using the basic earthquake ground motion S_2, an evaluation can be made on the design horizontal seismic coefficient of 0.2. Table 4.6.1-3 lists the results of the sliding safety factors determined using various analysis methods. Figure 4.6.1-11 shows the relation between the ratio of the safety factor of dynamic analysis to the safety factor of static analysis listed in this table and the maximum acceleration of the basic earthquake ground motion S_2. This figure also includes the results of evaluation for a few existing sites. It can be seen from this figure that the maximum acceleration range of the basic earthquake ground motion S_2, in which the ratio of the safety factors of the dynamic analysis to the static analysis is greater than 1.0, is roughly below 500 Gal. As a result, the design horizontal seismic coefficient of 0.2 is believed to correspond to the basic earthquake ground motion S_2 with maximum acceleration of about 500 Gal. In addition, the S-wave velocity of the ground soil in the model for this evaluation is greater than 600 m/s, and the S-wave velocity of the existing sites is greater than about 500 m/s. Based on the aforementioned analysis results, it can be said that the design horizontal seismic coefficient of 0.2 for the ground is a value that roughly envelops the seismic forces determined based on the basic earthquake ground motion.

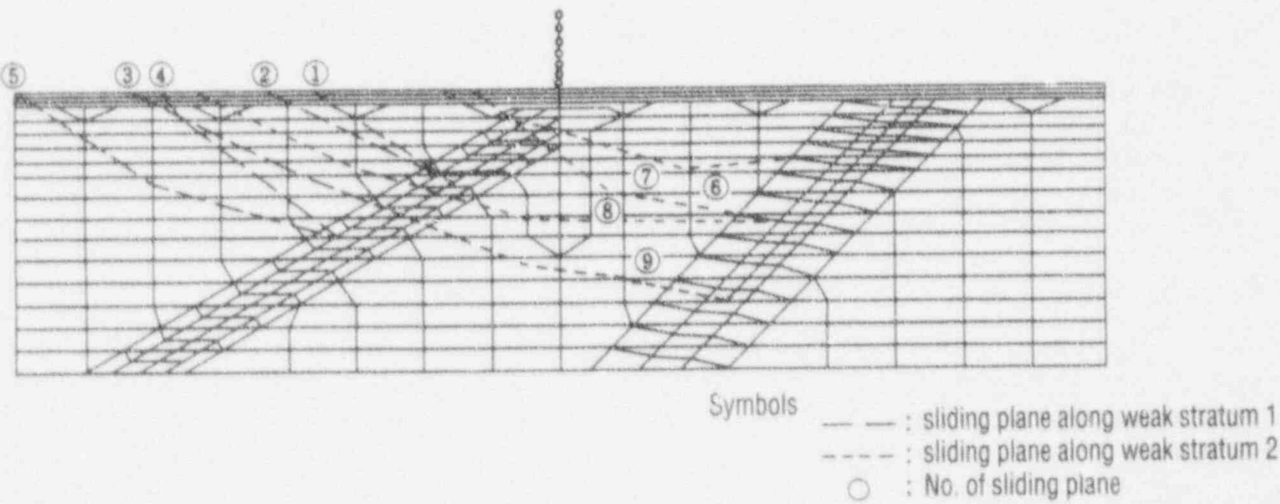


Figure 4.6.1-7. Sliding planes for analysis (relation between seismic motion and equivalent seismic coefficient).

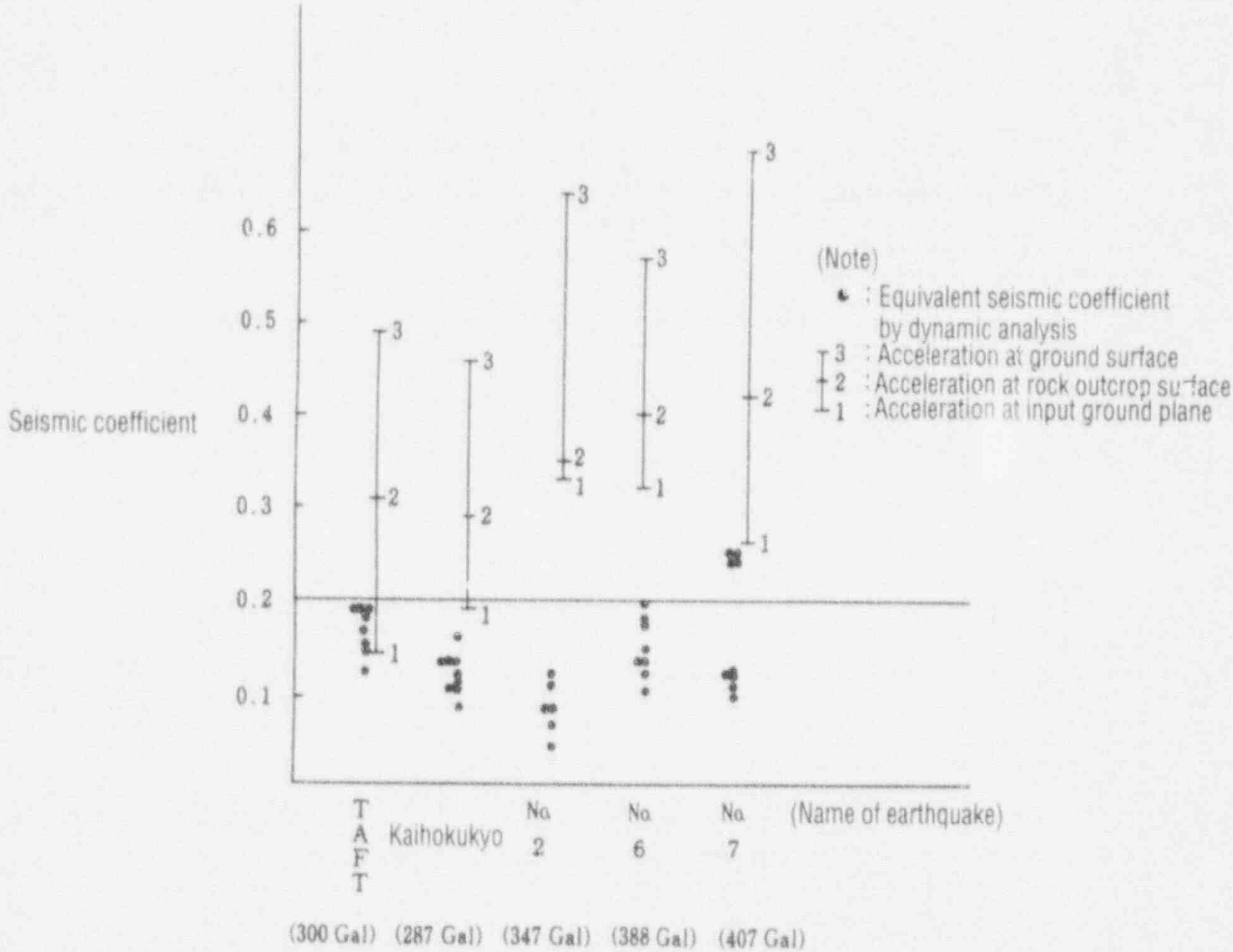


Figure 4.6.1-8. Comparison between equivalent seismic coefficient, response acceleration (represented in seismic intensity) calculated in dynamic analysis and design horizontal seismic coefficient of 0.2.

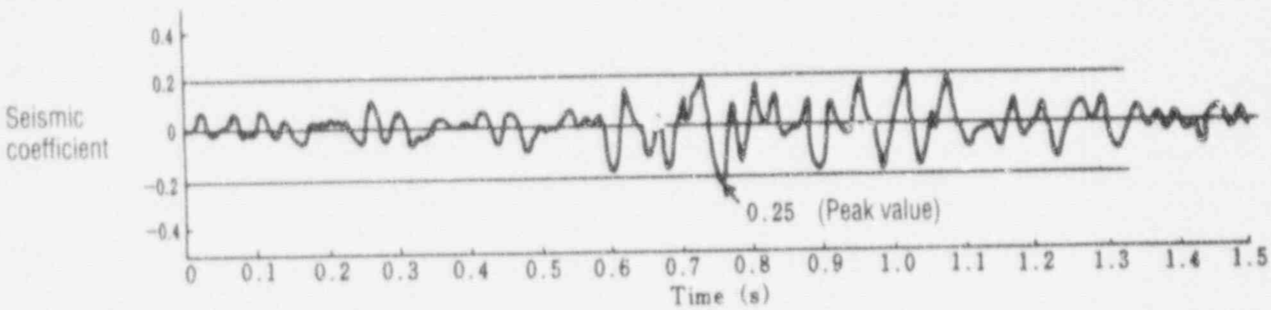


Figure 4.6.1-9. Time function of equivalent acceleration with respect to simulated seismic wave No. 7 (representation of seismic coefficient, (results with respect to sliding plane (1)).

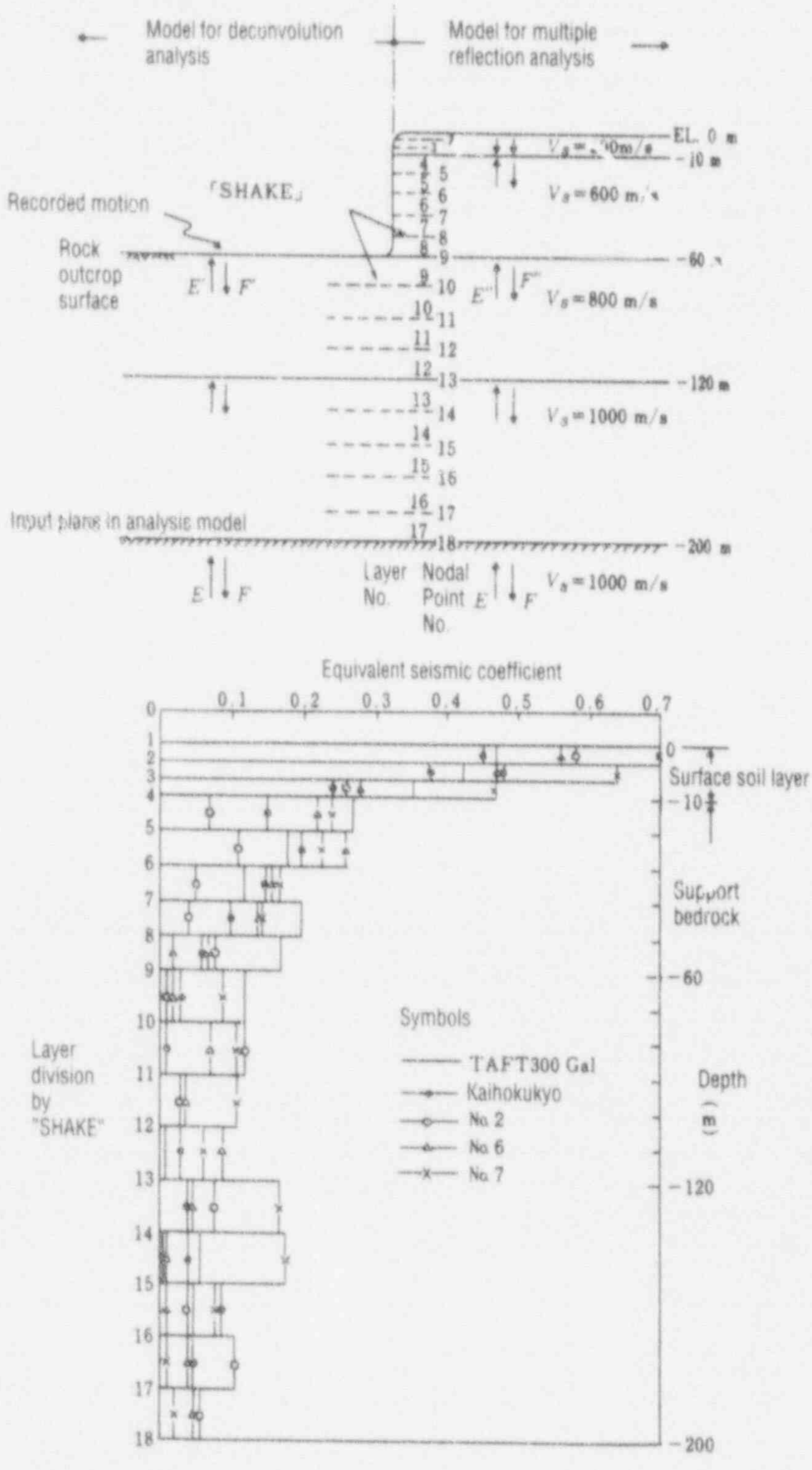


Figure 4.6.1-10. Distribution of equivalent seismic coefficient determined from the distribution of the maximum shear stress along the depth direction.

Table 4.6.1-3. Comparison of sliding safety factors obtained using different analysis methods.

Sliding-plane No.	Analysis method						
	Sliding-plane method	Static analysis	Dynamic analysis				
			TAFT 300 Gal	Kaihokukyo 287 Gal	No. 2 340 Gal	No. 6 388 Gal	No. 7 407 Gal
1	4.60	5.81	7.47 (6.05)	9.57	13.47	10.94	5.87
2	4.68	5.03	6.16 (4.93)	7.70	10.25	8.44	4.86
3	5.02	4.95	5.99 (4.73)	7.45	11.09	9.44	4.92
4	4.79	6.94	8.96 (7.22)	13.74	17.21	11.24	9.35
5	4.58	5.14	7.38 (5.61)	9.83	21.40	8.47	7.14
6	5.39	5.89	6.18 (4.70)	7.57	10.49	6.37	5.69
7	4.96	6.12	8.93 (6.77)	11.37	23.09	10.41	8.43
8	5.33	5.24	7.66 (5.84)	10.15	19.97	9.45	7.52
9	4.88	5.10	7.64 (6.13)	10.01	19.60	10.03	9.35

Note: Values in parentheses refer to the case when it is 400 Gal at the rock outcrop surface.

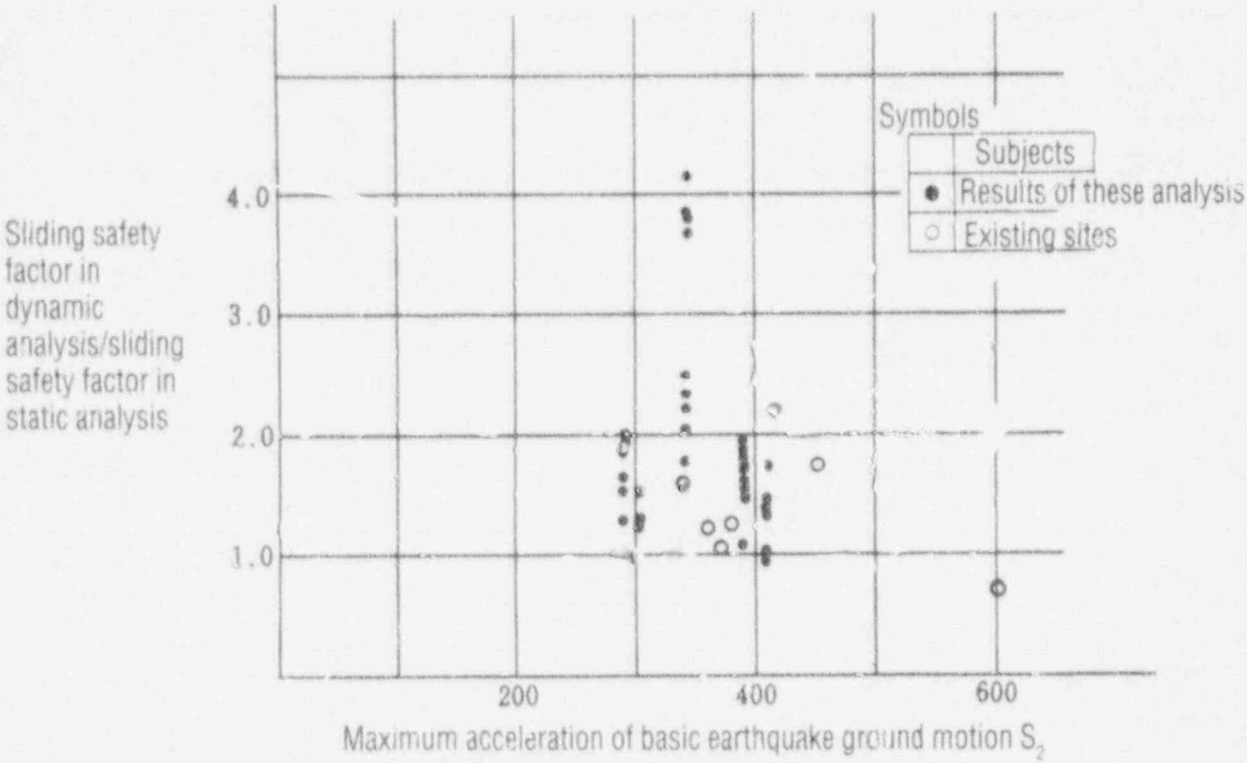


Figure 4.6.1-11. Relation between maximum acceleration of basic earthquake ground motion S_2 and (sliding safety factor in dynamic analysis)/(sliding safety factor in static analysis).

4.6.2 Peripheral slope of nuclear reactor building

(1) Analysis items

For four typical types of slopes made of soft rock and hard rock, the sliding-plane method, static analysis, and dynamic analysis are used to investigate the following items:

{1} Relation of safety factors using different analysis methods
{2} Relation between seismic motion and equivalent seismic coefficient

In item {1}, evaluation is made of the degree of difference among the sliding safety factors derived using various different analysis methods.

In item {2}, in the case when the slope stability is evaluated using sliding-plane method and static analysis, the concept of the equivalent seismic coefficient is evaluated using the relationship between the slope's design horizontal seismic coefficient $K_H = 0.3$ and the basic earthquake ground motion S_2.

(2) Analysis model

a. Profile/dimensions of model slope for analysis

With the slope profile, topography, and geological structure of the existing sites as reference, the models for analysis representing soft rock and hard rock are set up (see Figure 4.6.2-1). In addition, when it is necessary to determine the region for evaluation, the existing site examples and analysis conditions (boundary conditions, load conditions) are taken into consideration.

b. Properties of analysis model

With examples of existing sites as references, the various constants of the soils that form the model slopes are determined as listed in Table 4.6.2-1. Among these properties, the static/dynamic elastic moduli, Poisson ratio and shear modulus of elasticity are calculated on the basis of the elastic wave velocity. The strength constants (C, ϕ) are determined with the results of in situ testing and triaxial compression test taken as reference. The damping constant (h), etc., are determined with the past experimental data and results of past analyses. In addition, for the surface soil, sandy mudstone, Class D bedrock, fractured zone, etc., the nonlinear deformation characteristics are taken into account [4.1-1].

c. Seismic force for analyses

(a) Static seismic force (sliding-plane method, static analysis)

The seismic force used in static evaluation should be determined by accounting for the slope's dynamic characteristics, such as the amplification effect of the seismic motion observed in the actual seismic motion. In this evaluation, however, the same seismic coefficient is used for all the analysis cases. The vertical seismic coefficient is taken as 1/2 of the horizontal seismic coefficient. In the analysis, the downward vertical direction is taken as the unfavorable direction, based on the results of the preliminary analyses.

(b) Dynamic seismic force (dynamic analysis)

The seismic motions listed in Table 4.6.1-2 are used as the dynamic seismic force used in the stability evaluation of the slope. Among them, simulated seismic wave No. 6 and TAFT (EW) waveform, which are the

Table 4.6.2-1. Properties of slope models.

Model	Analysis method	Soil classification	Properties: γ_t (tf/m^3)	E (tf/m^2)	ν	c (tf/m^2)	ϕ (degrees)	h
Soft rock slope	Sliding-plane method	Surface soil	1.8	—	—	12.5	15	—
		Sandy mudstone	1.9	—	—	35.0	25	—
		Mudstone	2.3	—	—	75.0	30	—
	Static analysis	Surface soil	1.8	27,000	0.35	12.5	15	—
		Sandy mudstone	1.9	85,000	0.35	35.0	25	—
		Mudstone	2.3	370,000	0.25	75.0	30	—
	Dynamic analysis	Surface soil	1.8	27,000	0.35	12.5	15	0.10
		Sandy mudstone	1.9	85,000	0.35	35.0	25	0.05
		Mudstone	2.3	370,000	0.25	75.0	30	0.05
Hard rock slope	Sliding-plane method	D	2.0	—	—	20.0	20	—
		C_L	2.1	—	—	100.0	30	—
		C_M	2.4	—	—	300.0	40	—
		C_H	2.6	—	—	500.0	50	—
		Fractured zone	2.0	—	—	4.0	25	—
	Static analysis	D	2.0	50,000	0.30	20.0	20	—
		C_L	2.1	150,000	0.20	100.0	30	—
		C_M	2.4	400,000	0.20	300.0	40	—
		C_H	2.6	600,000	0.15	500.0	50	—
		Fractured zone	2.0	20,000	0.40	4.0	25	—
	Dynamic analysis	D	2.0	50,000	0.30	20.0	20	0.10
		C_L	2.1	150,000	0.20	100.0	30	0.05
		C_M	2.4	400,000	0.20	300.0	40	0.05
		C_H	2.6	600,000	0.15	500.0	50	0.05
		Fractured zone	2.0	20,000	0.40	4.0	25	0.15

representative seismic motions for the near earthquake and distant earthquake, are used as the major seismic forces for evaluation. Also, in the stability evaluation, the horizontal plane at the toe of the slope is assumed as the rock outcrop surface of the seismic motion.

d. Types of analysis

The evaluation of the slope stability during earthquake is performed using sliding-plane method, static analysis and dynamic analysis. Methods used in the sliding-plane method include the simple scheme (modified Fellenius method) and Janbu method. Static analysis is performed by elastic as well as nonlinear elastic finite element methods. As the dynamic analysis, Complex response analysis method is applied to linear as well as equivalent linear finite element models.

(3) Analysis results

a. Relation among safety factors obtained using different analysis methods

Figure 4.6.2-1 illustrates the predetermined sliding planes. Figure 4.6.2-2(a), (b) compare the sliding safety factors obtained using different analysis methods. In these figures, sliding planes (A-A) and (B-B) are arc and composite sliding plane profiles which indicate the minimum sliding safety factors of the slopes obtained using the sliding-plane method; sliding plane (C-C) is selected arbitrarily for comparison with sliding plane (A-A). In the figure, the seismic coefficient of dynamic analysis is the value obtained by dividing the maximum acceleration amplitude at the rock outcrop surface of the input seismic motion by gravitational acceleration. From these results, the following features can be found:

{1} The safety factor by the sliding-plane method is the smallest.
{2} The safety factor by the dynamic analysis is the largest.

b. Relation between seismic motion and equivalent seismic coefficient

In order to study the relation between the seismic force corresponding to basic earthquake ground motion S_2 and the seismic coefficient as the static seismic force, various analyses were performed according to the scheme shown in Figure 4.6.2-3 for the seismic force corresponding to a seismic coefficient $K_H = 0.3$. The appropriateness of the results is discussed. In the following, the results of the evaluation will be presented. $K_H = 0.3$ has been selected based on a comparison between the sliding safety factors of the dynamic analysis and the [static] analysis and the seismic coefficient shown in Figure 4.6.2-2.

Seismic motion and acceleration response characteristics

Figures 4.6.2-3(a), (b) illustrate the distribution of the peak accelerations for the soft rock slope model when TAFT (EW) (200 Gal, 500 Gal at the horizontal plane of the toe of slope, which is the rock outcrop surface) and No 6 (388 Gal at the rock outcrop surface) spectra are used, respectively. As can be seen from these results, the maximum acceleration depends significantly on the material that form the slope and the profile of the slope. In any case, [the maximum acceleration] along the slope increases as a function of the height, as it is magnified from the acceleration value of the input seismic motion at the rock outcrop surface. In the model used in this analysis, except for a portion of the surface layer, the responses are magnified by about 1.2–1.5 times that at the rock outcrop surface.

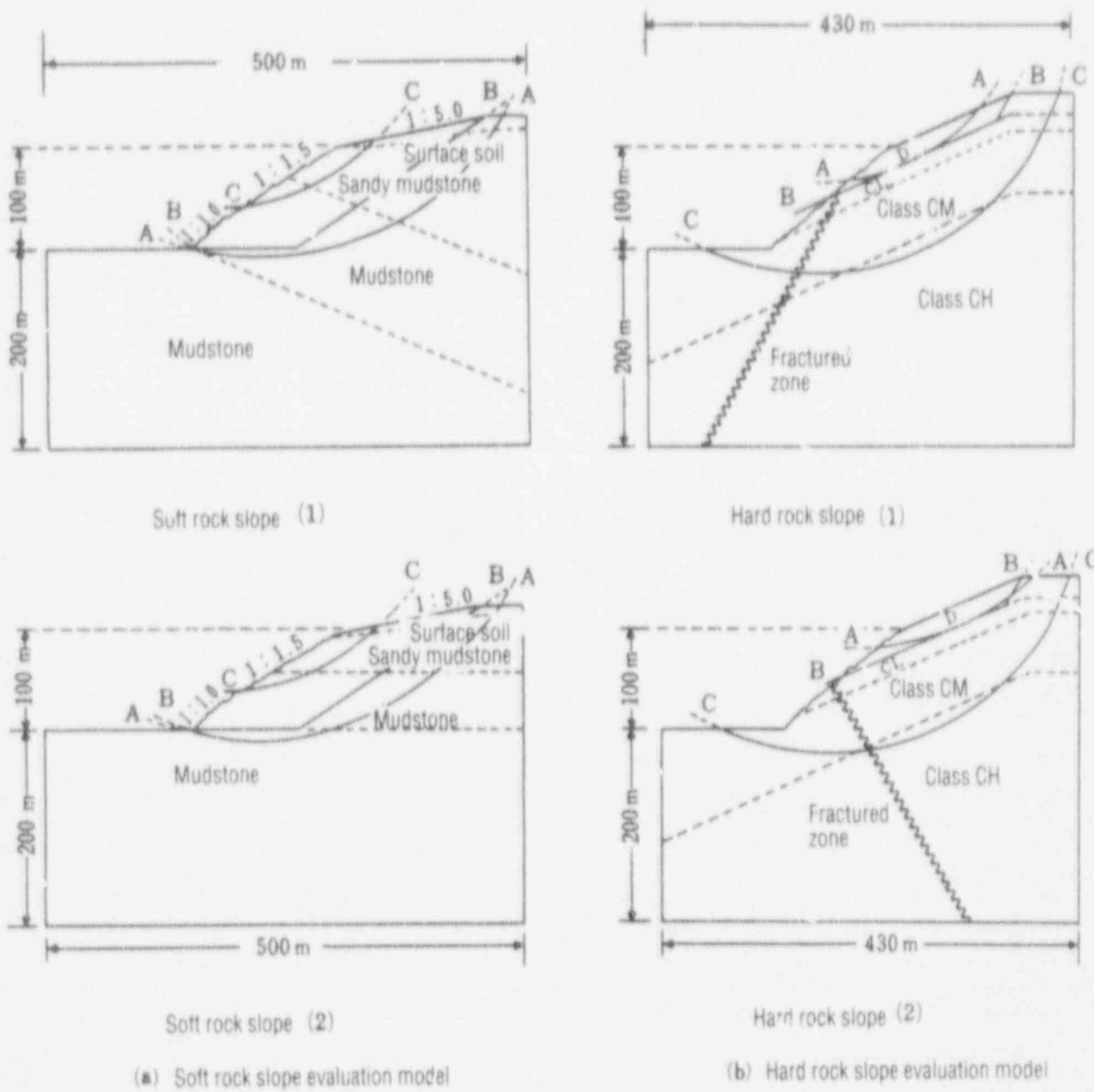


Figure 4.6.2-1. Models used for evaluating slopes and profiles of assumed sliding planes.

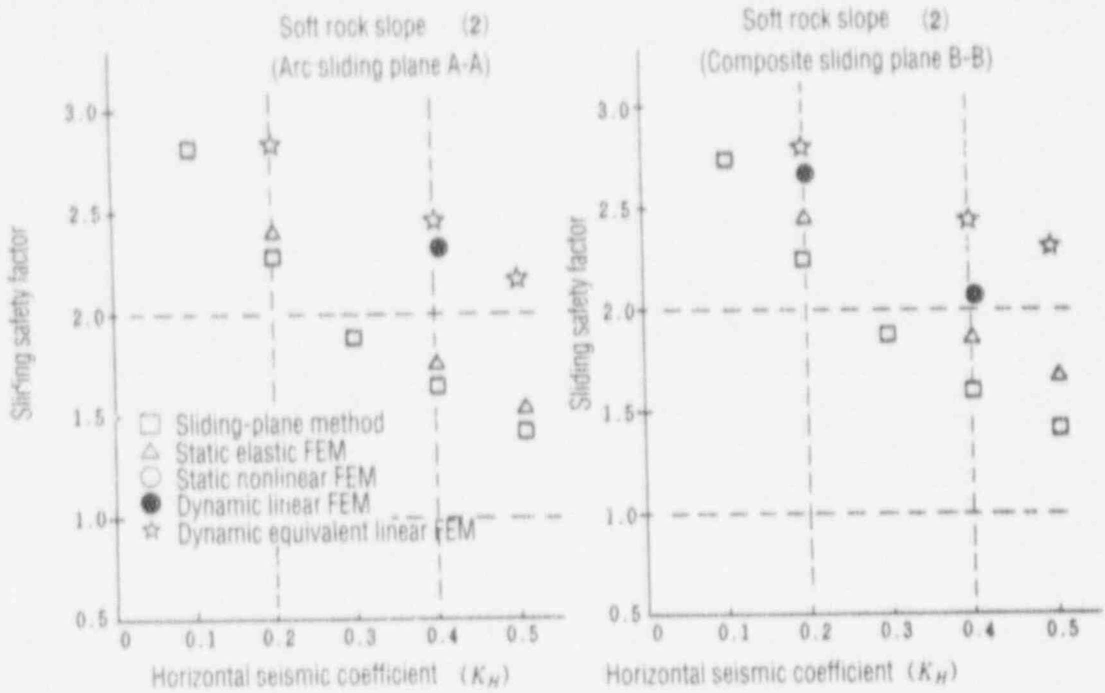


Figure 4.6.2-2(a). Comparison of sliding safety factors derived using different analysis methods.

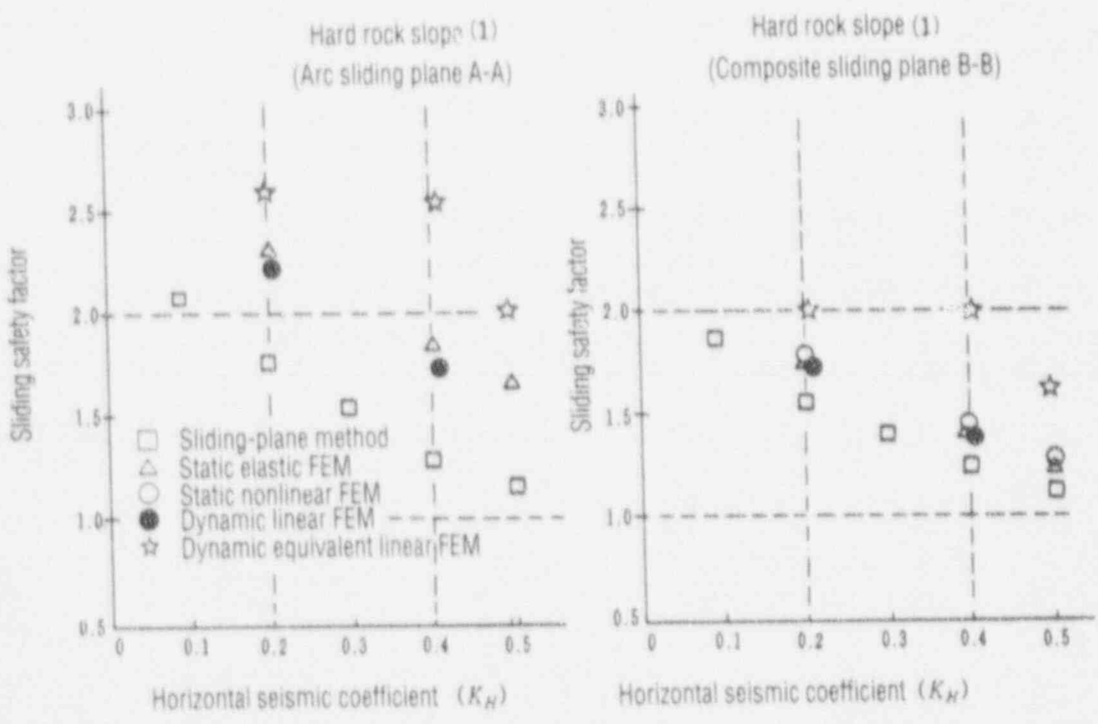


Figure 4.6.2-2(b). Comparison of sliding safety factors derived using different analysis methods.

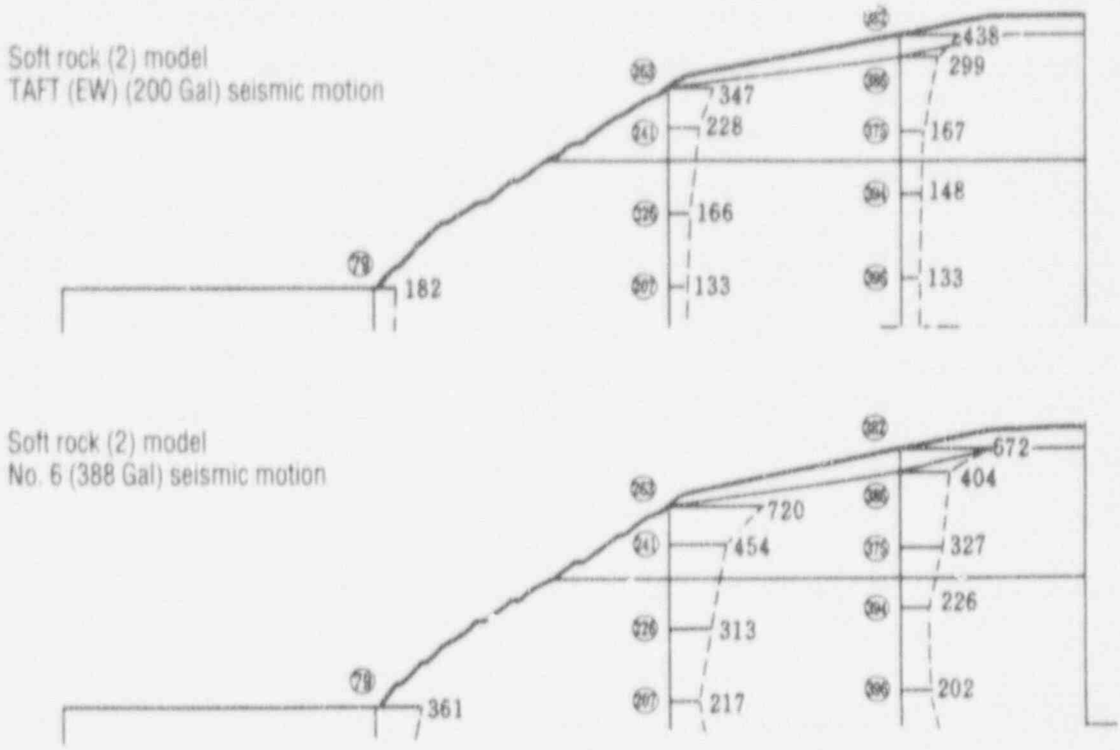


Figure 4.6.2-3(a). Distribution of maximum acceleration of slope in linear dynamic analysis.

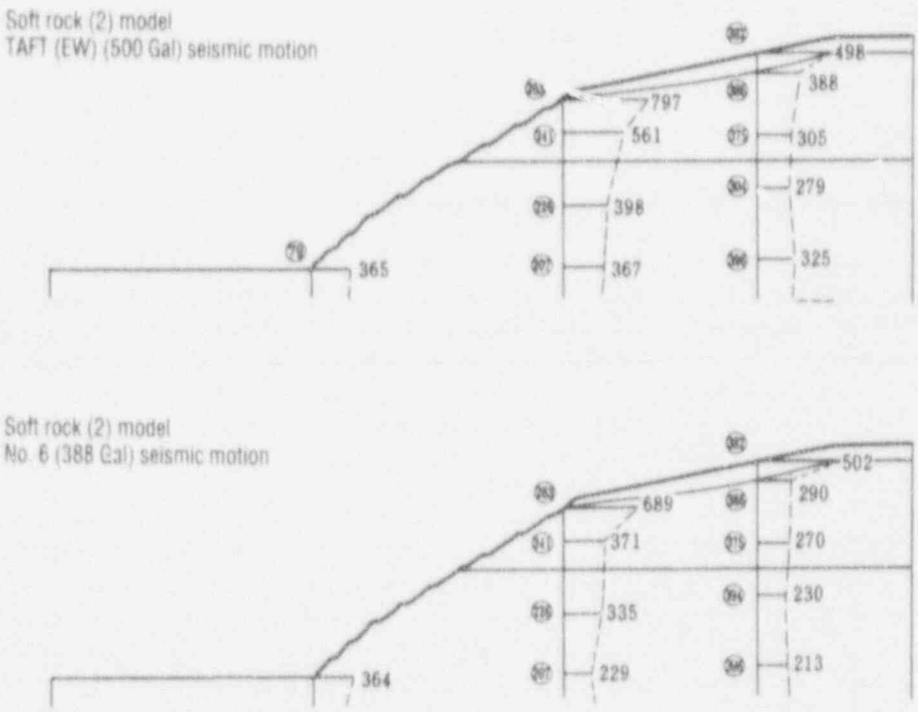


Figure 4.6.2-3(b). Distribution of maximum acceleration of slope in equivalent linear dynamic analysis.

Table 4.6.2-2. Equivalent seismic coefficients on sliding planes selected.

Analysis	Seismic motion	Shape of sliding plane	Soft rock slope (1)	Soft rock slope (2)	Hard rock slope (1)	Hard rock slope(2)
Dynamic linear FEM	TAFT (EW) 200 Gal	Arc A-A	0.12	0.14	0.28	0.21
		Composite B-B	0.14	0.14	0.23	0.20
		Arc C-C	0.17	0.19	0.14	0.14
	No. 6 388 Gal	Arc A-A	0.21	0.26	0.37	0.38
		Composite B-B	0.22	0.26	0.30	0.31
		Arc C-C	0.29	0.36	0.17	0.20
Dynamic equivalent linear FEM	TAFT (EW) 200 Gal	Arc A-A	—	0.11	0.17	—
		Composite B-B	—	0.13	0.15	—
		Arc C-C	—	0.17	0.11	—
	No. 6 388 Gal	Arc A-A	—	0.21	0.21	—
		Composite B-B	—	0.23	0.19	—
		Arc C-C	—	0.36	0.17	—
	TAFT (EW) 500 Gal	Arc A-A	—	0.26	0.41	—
		Composite B-B	—	0.29	0.36	—
		Arc C-C	—	0.40	0.27	—

(b) Comparison between magnitude of equivalent seismic coefficient determined from the acceleration response values and design horizontal seismic coefficient $K_H = 0.3$

Table 4.6.2-2 lists the values of the equivalent seismic coefficient acting on the soil masses along the sliding planes described in Figure 4.6.2-1(a), (b). Judging from these results, when the sliding plane with the smallest sliding safety factor is considered, the equivalent seismic coefficient is generally smaller than the value of the seismic coefficient calculated by converting the maximum acceleration at the rock outcrop surface, and is 50-97% in this evaluation. Also, in some cases, on the sliding plane near the surface layer, the equivalent seismic coefficient may be larger than the seismic coefficient of 3. In this case, the exceedence occurs only for a fraction of time in the dynamic analysis.

It has been shown in the previous studies [4.2.2-1] that if the static seismic coefficient (i.e., 0.3) is regarded as the acceleration amplitude for a stationary sinusoidal wave, in order to obtain acceleration responses identical to those obtained using a typical random waveform, it is needed to set the amplitude of the stationary sinusoidal wave as 40-60% of the maximum acceleration amplitude of the typical random motion. Based on the aforementioned viewpoint, the seismic coefficient of 0.3, used as the static design seismic force, is believed to be a seismic force of nearly the same strength as the basic earthquake ground motion.

(c) Comparison between modified equivalent seismic coefficient for slopes and seismic coefficient $K_H = 0.3$

In order to evaluate the distribution of equivalent seismic coefficients along the height of a slope for the basic earthquake ground motion S_2, with the horizontal plane at the toe of the slope taken as the standard level, the slope was sliced horizontally with a 20 m interval; the distribution of the equivalent seismic coefficient acting on the sliced soil masses (referred to as "modified equivalent seismic coefficient" hereinafter) is determined using equation (4.6.1-1) based on the acceleration distribution of the linear/equivalent linear analyses determined, and the results are compared with the constant seismic coefficient of 0.3.

Figure 4.6.2-4 illustrates the distribution of the modified equivalent seismic coefficient for a mudstone homogeneous slope model shown in the figure. On the other hand, Figure 4.6.2-5 illustrstes the distribution of modified equivalent seismic coefficient obtained using No. 6 seismic motion for the slope model shown in Figure 4.6.2-1.

Judging from these results, it can be seen that, except for the surface layer portion on the top of the slope, in all cases, the modified equivalent seismic coefficient is less than the seismic coefficient calculated using the maximum acceleration at the rock outcrop surface. In addition, in almost all the cases with different seismic motions, slope shapes, and material characteristics, the values of the modified equivalent seismic coefficient are less than 0.3. At the surface layer portion of the top of the slope, although the modified equivalent seismic coefficient is greater than 0.3, it may be considered to be enveloped by the seismic coefficient of 0.3 if the duration of the equivalent seismic coefficient as the static seismic motion is taken into account.

(d) Results of evaluation of design horizontal seismic coefficient based on comparison of sliding safety factors

The sliding safety factors determined using the sliding-plane method with design horizontal seismic coefficient of 0.3 and using dynamic analysis for the basic earthquake ground motion are compared with each other. In this way, evaluation is performed of the design horizontal seismic coefficient of 0.3.

Tables 4.6.2-3 and 4.6.2-4 list the sliding safety factors obtained from various methods and the equivalent seismic coefficients by dynamic analysis. Figure 4.6.2-6 shows the relation between the ratio of the dynamic analysis safety factors to those obtained by the sliding-plane method and the maximum acceleration of the basic earthquake ground motion.

Judging from these results, it can be seen that although the equivalent seismic coefficient on the soil mass along the sliding plane determined in the dynamic analysis is greater than $K_H = 0.3$, which is used in the sliding-plane method; however, the safety factor determined using the sliding-plane method with a uniform seismic coefficient $K_H = 0.3$ is usually less than those obtained by a dynamic analysis.

Judging from the results of items (a)-(d) in the above, in the case when the seismic force is substituted as static force on the basis of the seismic evaluation of the slope, the design horizontal seismic coefficient of 0.3 for a slope is believed to be a value which almost envelopes the seismic force corresponding to the basic earthquake ground motion.

Based on the aforementioned analysis results, it is believed that the horizontal seismic coefficient K_H = 0.3 which is set as 50% higher than standard design seismic coefficient $K_0 = 0.2$, which in turn roughly corresponds to the upper limit (see section "4.2.2 Design seismic force") of the basic earthquake ground motion S_2 set by the Light Water Reactor Improved Type Standardization Aseismic Design Subcommittee, is a value with an appropriate margin corresponding to the maximum acceleration of up to 500 Gal at the toe of the slope, and it is considered to be the upper limit of the static design seismic force even when the response variabilities due to slope shape and material properties are accounted for.

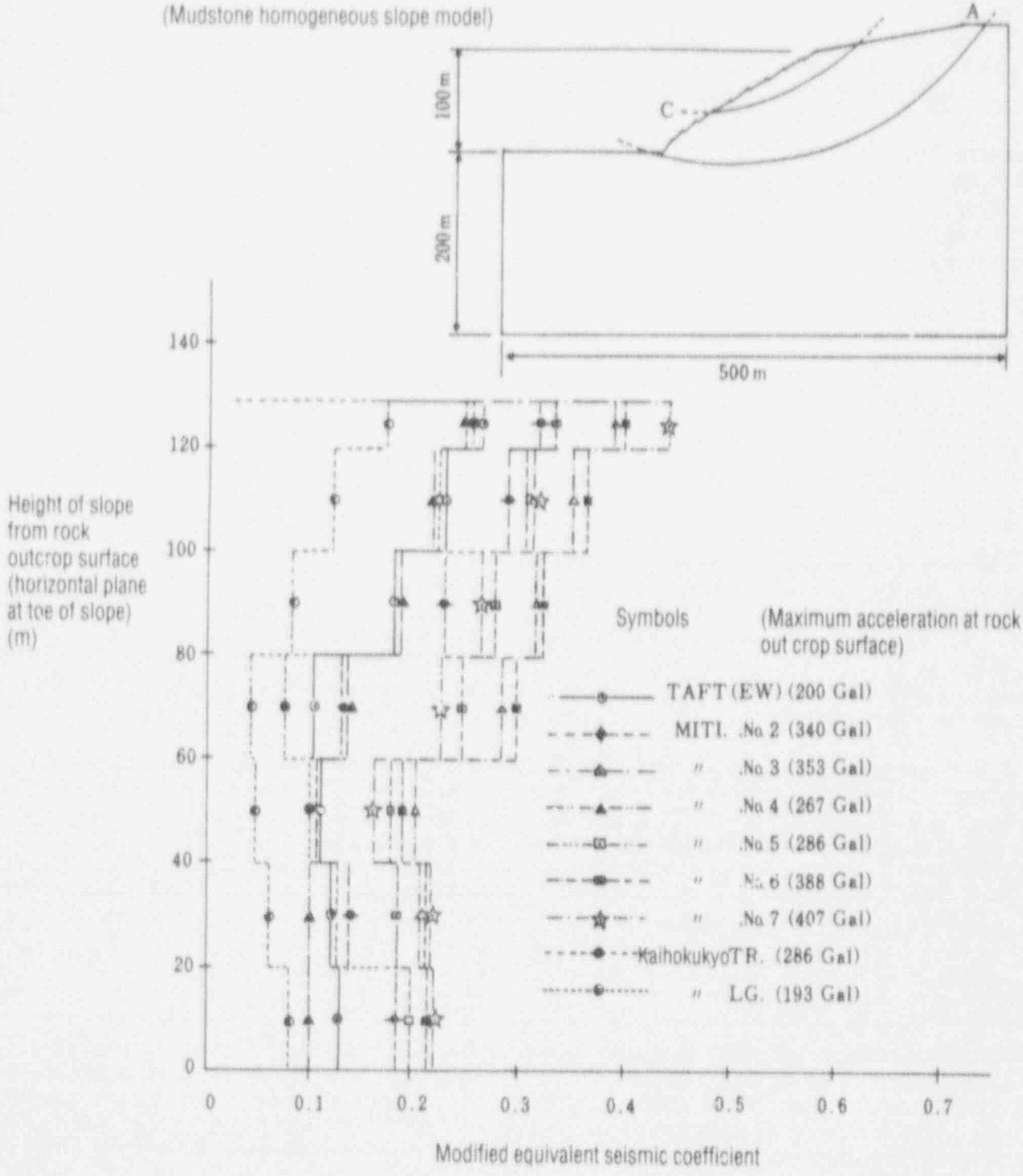


Figure 4.6.2-4. Distribution of modified equivalent seismic coefficient of slope due to various seismic motions.

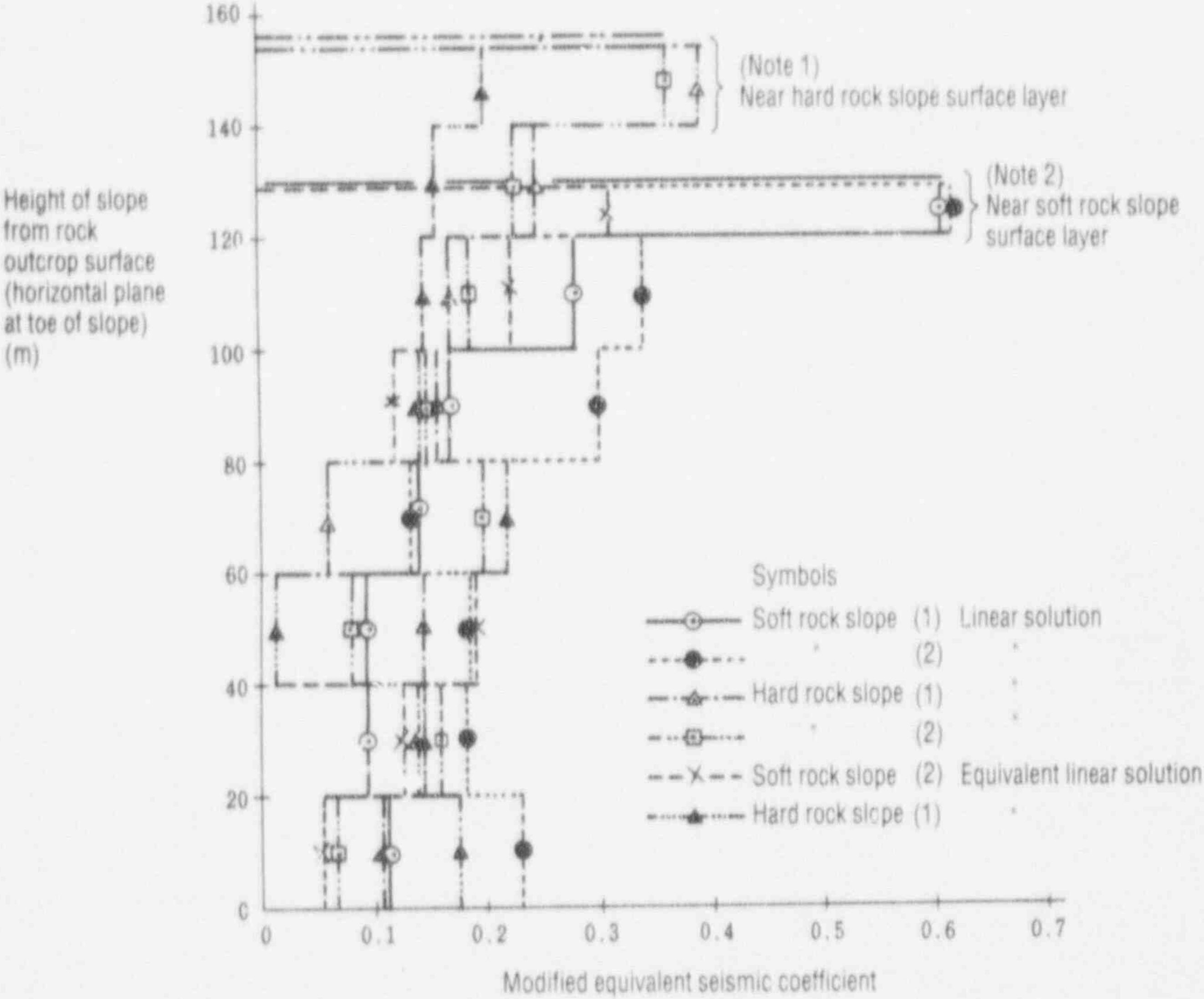


Figure 4.6.2-5. Distribution of modified equivalent seismic coefficient of slope due to No. 6 input seismic motion (388 Gal).

Table 4.6.2-3. Correspondence between equivalent seismic coefficient and sliding safety factor.

Solution method	Seismic force and safety factor		Sliding plane							
			Soft rock slope (1)		Soft rock slope (2)		Hard rock slope (1)		Hard rock slope (2)	
			A-A	B-B	A-A	B-B	A-A	B-B	A-A	B-B
Dynamic analysis (linear)	TAFT (EW) 200 Gal	Equivalent seismic coefficient	0.12	0.14	0.14	0.14	0.28	0.23	0.21	0.20
		Sliding safety factor	2.76	2.68	2.86	2.71	2.26	1.76	2.32	2.07
	No. 6 388 Gal	Equivalent seismic coefficient	0.21	0.22	0.26	0.26	0.37	0.30	0.38	0.31
		Sliding safety factor	2.34	2.19	2.32	2.10	1.72	1.35	1.69	1.55
Sliding-plane method	$K_H = 0.3$	Sliding safety factor	1.81	1.83	1.91	1.91	1.50	1.40	1.41	1.39
Dynamic analysis (equivalent linear)	TAFT (EW) 200 Gal	Equivalent seismic coefficient	—	—	0.11	0.13	0.17	0.15	—	—
		Sliding safety factor	—	—	2.86	2.83	2.62	2.01	—	—
	TAFT (EW) 500 Gal	Equivalent seismic coefficient	—	—	0.26	0.29	0.41	0.36	—	—
		Sliding safety factor	—	—	2.15	2.34	2.02	1.53	—	—
	No. 6 388 Gal	Equivalent seismic coefficient	—	—	0.21	0.23	0.21	0.19	—	—
		Sliding safety factor	—	—	2.41	2.44	2.53	1.98	—	—
Sliding plane method	$K_H = 0.3$	Sliding safety factor	—	—	1.91	1.91	1.50	1.40	—	—

Table 4.6.2-4. Correspondence between equivalent seismic coefficient and sliding safety factor on mudstone homogeneous slope.

Analysis model			Mudstone homogeneous slope Seismic motion / Profile of sliding plane						
			No. 2 340 Gal	No. 3 353 Gal	No. 4 267 Gal	No. 5 286 Gal	No. 6 388 Gal	No. 7 407 Gal	Kaihokukyo (TR) 286 Gal
			A-A	A-A	A-A	A-A	A-A	A-A	A-A
Dynamic a[illegible]ysis (linear)		Equivalent seismic coefficient	0.16	0.24	0.11	0.21	0.26	0.21	0.15
		Sliding safety fa[illegible]r	3.08	2.43	2.95	2.52	2.38	2.41	2.88
Sliding-plane analysis method	$K_H = 0.3$	Sliding safety factor	Slid[illegible] 1.95						

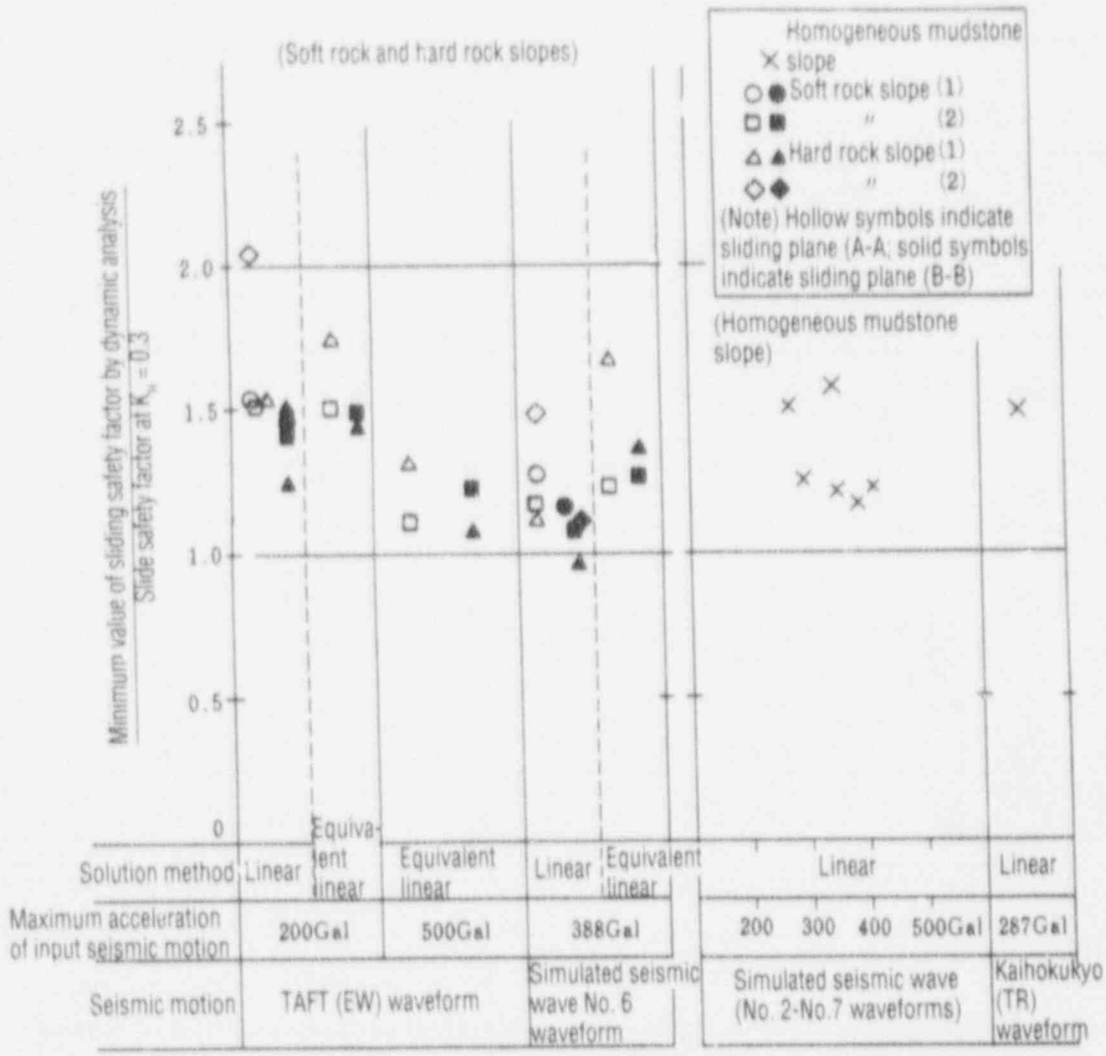


Figure 4.6.2-6. Maximum acceleration of input seismic motion vs. (sliding safety factor using dynamic analysis)/(sliding safety factor using sliding-plane method).

4.6.3 Important outdoor underground structures

(1) Evaluation items

In this section, several typical structures among the various emergency water intake equipment, such as water channel, water pit, and seawater duct, are selected for evaluation. The dependency of the calculation results is evaluated regarding the analysis method, soil spring constant, application scheme of soil pressure during earthquake in the seismic coefficient method, etc.

(2) Analysis models

a. Parameters of analysis models

(a) Water channel

The water channel is a steel structure with an inner diameter of 4 m and a length of 200 m, buried horizontally in soil at a depth of 10 m. The surrounding soil consists of sandy layer (I) and bedrock. The bedrock surface is inclined at an angle of 15° from the water channel end (EL. -5.0 m), and becomes level after EL. -35.0 m (Figure 4.6.3-1).

(b) Water pit

The water pit is a reinforced concrete structure with a width of 50.5 m, a height of 20.3 m, and a length of 70.0 m. It has 8 sets of water inlets and is buried in sandy layer (II). It is directly supported on the bedrock (Figure 4.6.3-2). Table 4.6.3-1 lists the long-term load.

(c) Seawater duct

The seawater duct is a two-story reinforced concrete structure with a width of 8.80 m and a height of 4.70 m. It is buried in sandy layer (II) (Figure 4.6.3-3). Table 4.6.3-2 lists the long-term load.

b. Properties of ground and materials of analysis models

The ground studied in this case consists of upper sandy layer (I) (assumed to be alluvium), sandy layer (II) (assumed to be diluvium), and bedrock. Their properties are listed in Table 4.6.3-3. In addition, the properties of the materials of reinforced concrete, concrete, and steel are listed in Table 4.6.3-4.

c. Seismic force for analysis

(a) Dynamic seismic force (dynamic analysis)

Simulated seismic wave No. 6 is used. The vertical seismic coefficient is taken as 1/2 the maximum acceleration amplitude of simulated seismic wave No. 6.

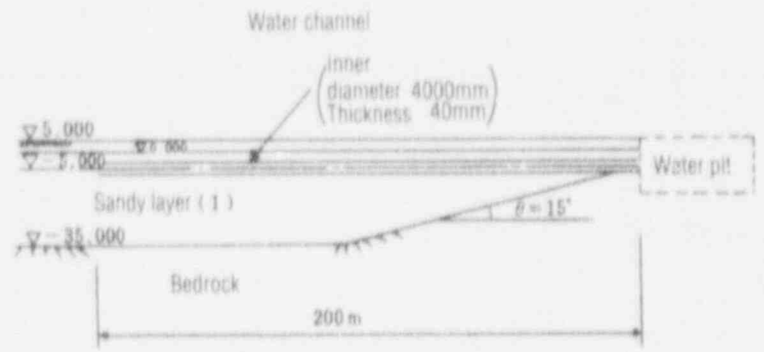


Figure 4.6.3-1. Structure of water channel and ground configuration.

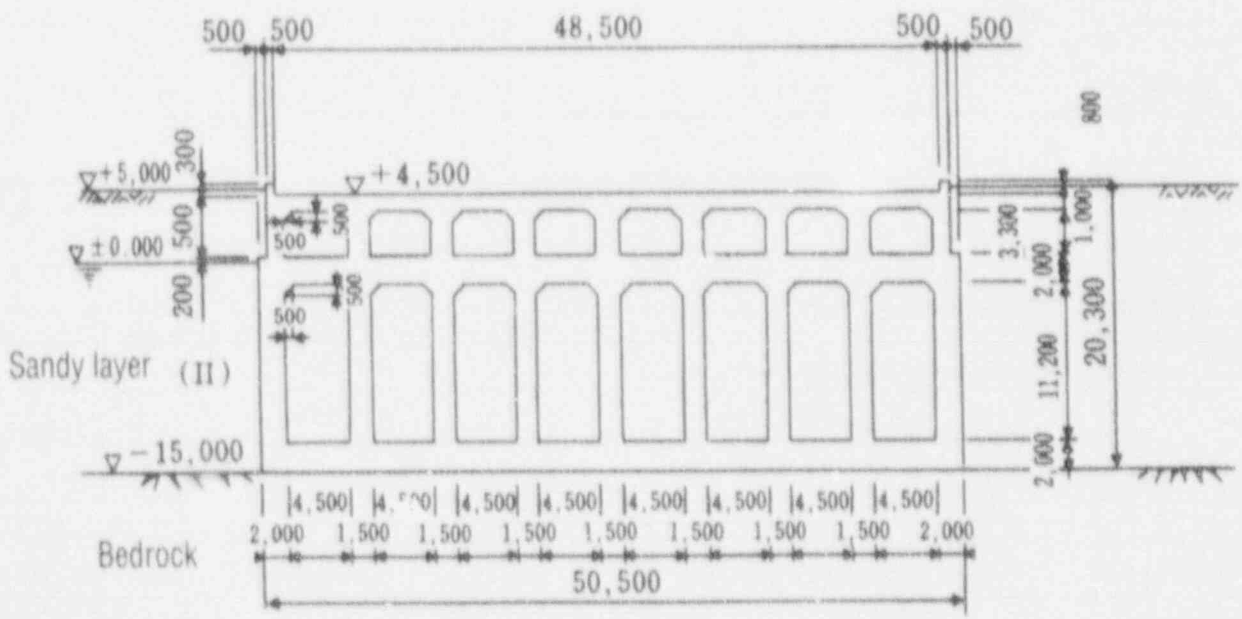


Figure 4.6.3-2. Structure of water pit and ground configurati n.

Table 4.6.3-1. Long-term load (water pit).

Item of condition	Design value
Self-weight	The weigth per unit volume is determined as 2.4 tf/m^3 for reinforced concrete.
Water content	It is full of seawater (specific gravity 1.03). (Seawater level EL. $\pm$0.000)
Live load	The load due to equipment, pipes, etc., is taken as 1 tf/m^2 on the ceiling plate and center floor plate.
Overburden load on ground surface	It is taken as 1 tf/m^2 on the surface of the surrounding sand.
Underground hydraulic pressure and buoyancy	With the groundwater level determined as EL. $\pm$0.00, the hydrostatic pressure on the side wall and the buoyancy on the bottom plate are considered.
Long-term earth pressure	The static earth pressure coefficient is determined as K_0 = 0.5.

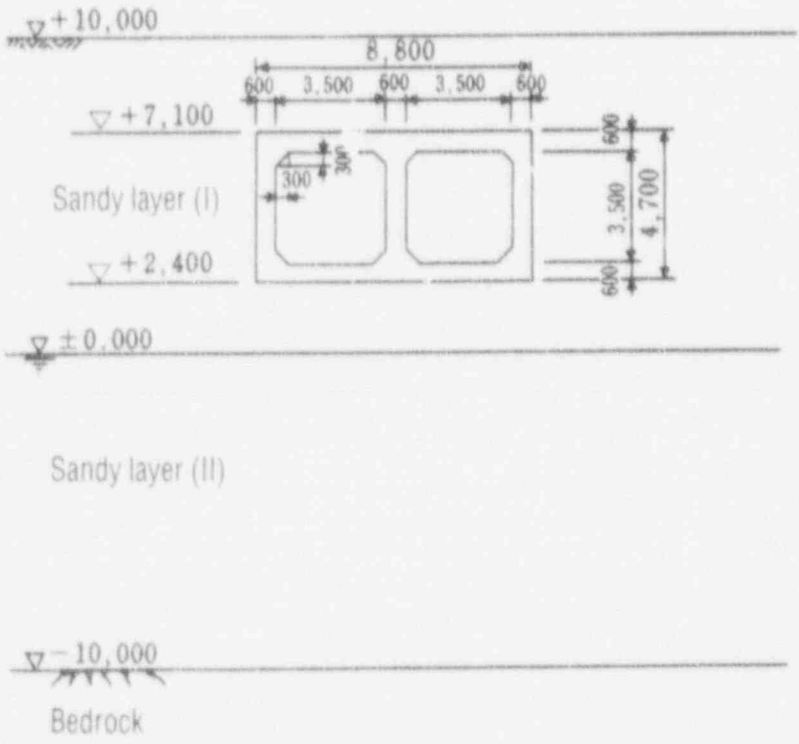


Figure 4.6.3-3. Structure of seawater duct and ground configuration.

Table 4.6.3-2. Long-term load (seawater duct).

Item of condition	Design value
Self-weight	The weigth per unit volume is set as 2.4 tf/m^3 for reinforced concrete.
Pipe load	The pipe load is taken as 1 tf/m.
Overburden load on ground surface	It is taken as 1 tf/m^2 on the surface of the surrounding soil.
Long-term earth pressure	The static earth pressure coefficient is taken as $K_0 = 0.5$.
Overburden earth pressure	It is taken as the product of the unit-volume weight of the upper soil and the thickness of the upper soil layer.

Table 4.6.3-3. Properties of model soil.

		Sandy layer (I)	Sandy layer (II)	Bedrock
Cohesive force C (kgf/cm^2)		0.0	0.0	5.0
Internal friction angle ϕ (degrees)		30	38	40
Weight per unit volume (tf/m^3)	Wet weight γ_t	1.8	1.8	—
	Saturated weight γ_{sat}	2.0	2.0	2.0
	In-water weight γ'	1.0	1.0	—
Shear wave velocity V_S (m/s)		150	300	700
Damping constant h (%)		Strain dependence is considered		2.0
Poisson's ratio (ν)	In air	0.45	0.40	—
	In water	0.48	0.48	0.33
N value		15	35	—

Note: In this case, for the shear modulus of elasticity (G) and damping constant (h) of sandy layers (I) and (II), the strain dependence shown in the following figure should be taken into consideration.

Table 4.6.3-4. Properties of materials.

Material	Item of condition	Design value
Reinforced concrete	Weight per unit volume	2.4 tf/m^3
Concrete σ_{ck} = 240 kgf/cm^2	Young's modulus for calculating cross-sectional force	2.7 × 10^6 tf/m^2
	Shear modulus of elasticity for calculating cross-sectional force	1.17 × 10^6 tf/m^2
	Damping constant	5 %
Steel	Weight per unit volume	7.85 tf/m^3
	Young's modulus	2.1 × 10^7 tf/m^2
	Shear modulus of elasticity	8.1 × 10^6 tf/m^2
	Damping constant	3 %

(3) Analysis results

a. Comparison of analysir methods

(a) Water channel

(i) Analysis methods and analysis conditions

The following three types of analysis methods are used to perform soil response calculation.

{1} One-dimensional multiple reflection (referred to as "multiple reflection" hereafter).
{2} Two-dimensional FEM complex response analysis [4.6.3-1] (referred to as "FEM" hereinafter).
{3} Buried tunnel method [4.6.3-2] (referred to as "burying" hereinafter).

Table 4.6.3-5 lists the analysis models and analysis conditions.

(ii) Evaluation items

The results obtained by using the aforementioned three methods are used to perform the soil-structure response calculation using multi-input response analysis [4.6.3-3]. Evaluation of the following three items is performed.

{1} Maximum response acceleration
{2} Maximum response displacement
{3} Maximum member forces

(iii) Comparison of analysis results

{1} Figure 4.6.3-4 illustrates the maximum response acceleration distributions determined using different analysis methods. Figure 4.6.3-5 illustrates the maximum response displacement distribution.

Table 4.6.3-5. Analysis models and analysis conditions (water channel).

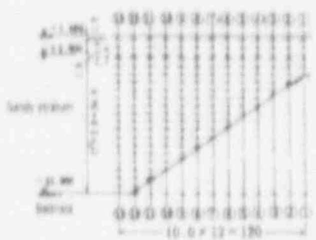
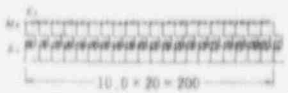
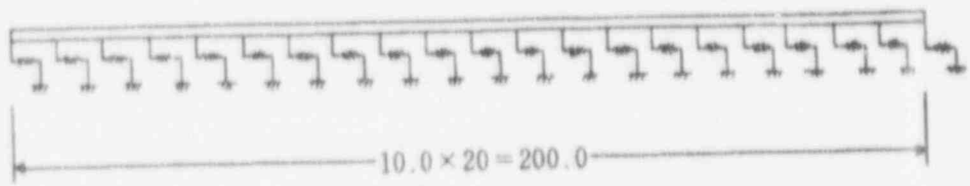
	Method	"One-dimensional multiple reflection"	"FEM"	"Buried tunnel method"
Soil response analysis	Analysis models and analysis conditions	 - The soil model is determined by dividing the soil into soil columns (1)-(13). - The strain dependence of the sandy layer is considered using the equivalent linear method. - The soil model is taken as semi-infinite layered ground and the soil response is calculated using the one-dimensional multiple reflection theory.	 - A two-dimensional FEM model is formed for the soil (EL. +5.000-EL. −35.00, width 200 m) (boundary conditions: transfer boundary for the side surface, and viscous boundary for the lower surface). - The shear modulus of elasticity G′ and damping constant h′ obtained in "multiple reflection" are used as the soil properties. - The soil response is calculated by complex response analysis with the transfer function determined up to 25 Hz.	 - G′ and h′ obtained in "multiple reflection" are used to form a discrete-mass soil model. {1} Calculation of each soil column using discrete-mass model (soil springs K_3, equivalent mass, equivalent damping). {2} Soil springs K_2 for connecting soil columns. - Damping is assumed to be proportional to the strain energy. - The soil response is calculated by using mode superposition method considering fundamental mode (0.623 Hz) to 10th mode (3.74 Hz).
	Input conditions	- For each soil column, the design seismic motion shown in section 4.6.3(2) is incident on the assumed input basement surface (EL. −35.000)	- The design seismic motion is incident on the input basement via a viscous boundary as shown in section 4.6.3(2).	- The acceleration spectrum (E+F) at position EL. −35.000 of soil column {13} obtained in "multiple reflection" is applied to the foundation.

Table 4.6.3-5 *(Cont'd)*. Analysis models and analysis conditions (water channel).

<table>
<tr><td rowspan="4">Calculation of soil-structure response</td><td>Method</td><td colspan="3">Multi-input response analysis</td></tr>
<tr><td>Analysis models and analysis conditions</td><td colspan="3">
- In the model, the water channel is represented by a beam and the soil is represented by discrete shear springs (free only in the axial direction of the pipe).
- The water channel self-weight and the water contained (seawater = 1.03) are taken into consideration as inertial forces.
- Strain energy proportional-type damping is adopted (3% for piping, 5% for soil).
- The response and member force of the piping are calculated using the mode superposition method for fundamental vibration mode (7.51 Hz) to 10th vibration mode (51.9 Hz).</td></tr>
<tr><td>Soil spring constant</td><td colspan="3">According to Item 8.4.3 in the "Road/Bridge Guidelines" (Japan Road Association, May 1980), the axial shear spring constant, which represents interaction between the piping and the surrounding soil, is determined. The discrete shear spring constants at the various input points are calculated from this axial shear spring constant.</td></tr>
<tr><td>Input conditions</td><td>- The response of the piping burying level (EL. −5.000) of each soil column is input via the soil spring.</td><td>- The response at the position corresponding to the piping input point is input via the soil spring.</td><td>- The response of the piping burying level (EL. −5.000) derived according to the mode ratio of each soil column from the response of each discrete mass is input via the soil spring.</td></tr>
</table>

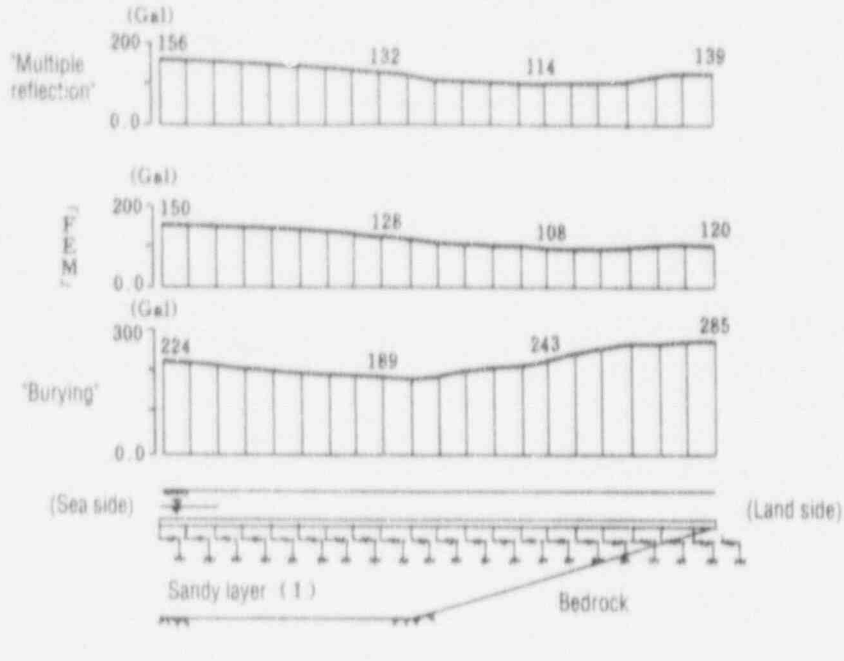


Figure 4.6.3-4. Distribution of maximum response acceleration of water channel.

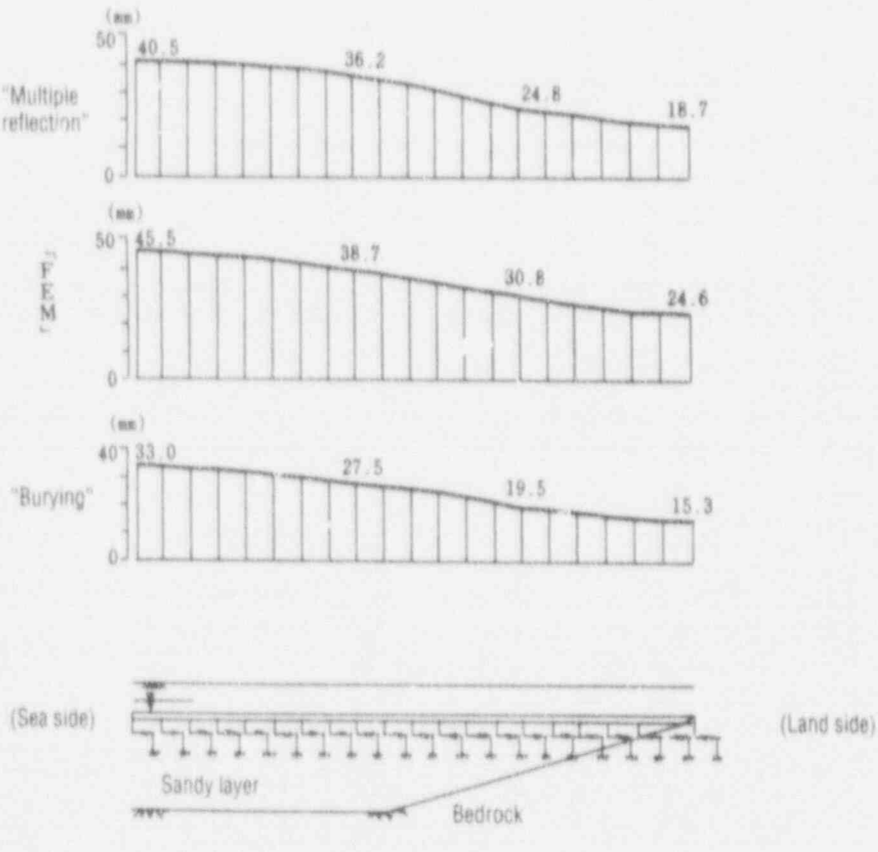


Figure 4.6.3-5. Distribution of maximum response displacement of water channel.

For the maximum response acceleration distribution, it can be seen that the "burying" method gives a relatively larger result compared with the other two methods. As far as the maximum response displacement distribution is concerned, there is a tendency for the value to be larger to the sea side and smaller to the land side. This is a common feature among the three methods. Quantitatively speaking, the "burying" gives the highest response value, followed by "multiple reflection" and "FEM."

{2} Figure 4.6.3-6 illustrates the distribution of the maximum member forces (axial force). As far as the distribution of the maximum member forces (axial force) is concerned, for all three methods, the distribution patterns are nearly the same, with the maximum response located offset from the center to the land side. Quantitatively speaking, the "multiple reflection" method gives the largest value of 2640 tf, followed by 2480 tf for the "burying" method, and 2170 tf for "FEM." It is rather predictable that the "multiple reflection" method gives the largest responses for member forces as the continuity of soil properties between layers is not considered in the analysis scheme. What is noteworthy is that, although there exists certain difference in the response acceleration and response displacement, the member forces computed by the three methods are almost the same.

(b) Water pit

(i) Analysis methods and analysis condition

The following three analysis methods are used for calculation:

{1} FEM [4.6.3-1]
{2} Multi-input response analysis
{3} Response displacement method [4.6.3-3]

Table 4.6.3-6 lists the analysis models and analysis conditions. To determine the earthquake response of the ground, the one-dimensional multiple reflection theory is used to compute the responses of the various layers in the ground. The strain dependency of the sandy layer is evaluated using the equivalent linear method. The location of the seismic motion input is the assumed input ground plane (EL. -35.0 m).

(ii) Items for comparative evaluation

Calculation is performed using the aforementioned three methods. Evaluation of the following two items is performed.

{1} Maximum response acceleration
{2} Maximum member forces

(iii) Comparison of analysis results

{1} Figure 4.6.3-7 shows the results of the maximum response acceleration distribution derived using various analysis methods. It can be seen that for the "multi-input response analysis" and "FEM" there exists a difference between the relative amplitude of the acceleration response of the ground and the water pit ("FEM": ground ≥ water pit; "multi-input response analysis": ground ≤ water pit). This might be due to the difference in the model formation of the dynamic interaction between the soil and structure. For the "multi-input response analysis," due to the relation between the predominant period of ground motion and the natural period of the coupled vibration system, the vibration might be significantly amplified in the system.

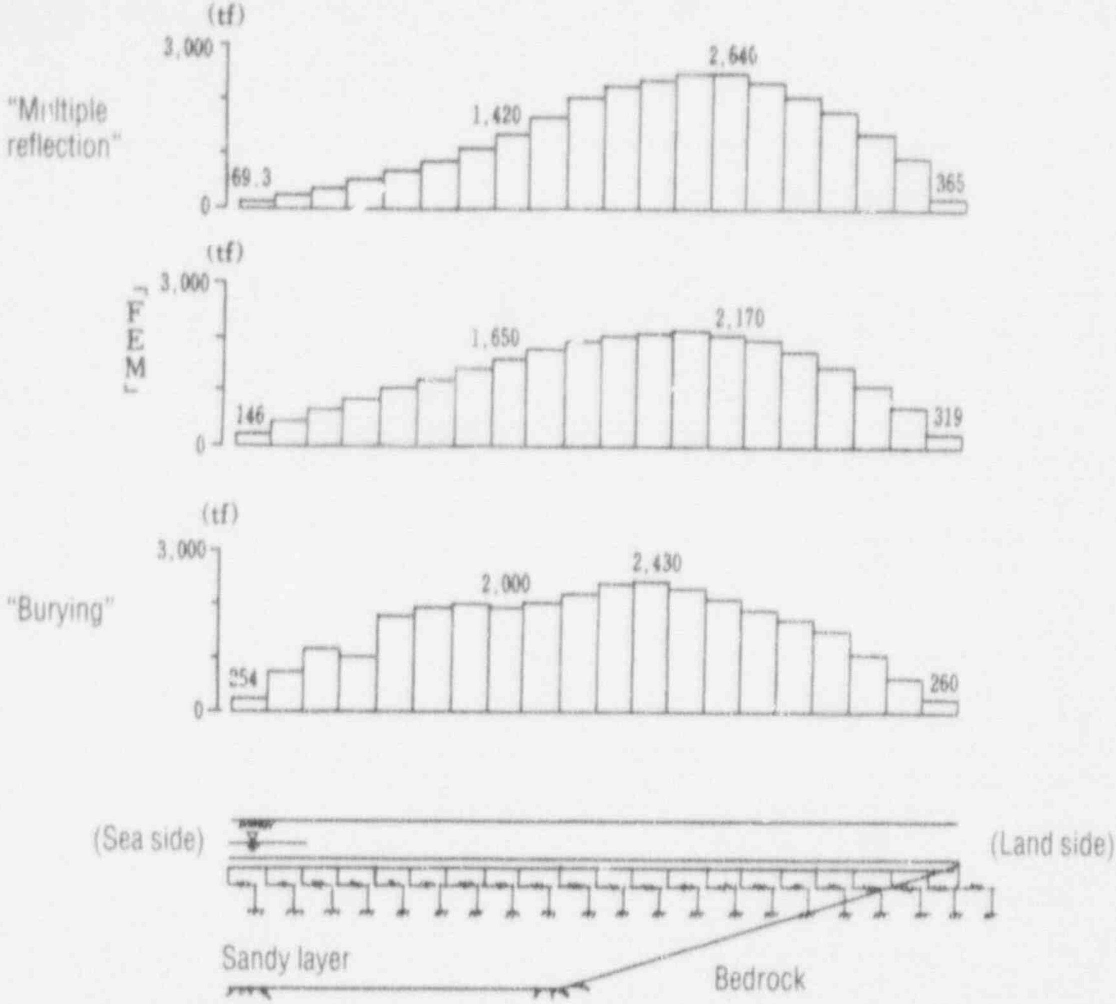


Figure 4.6.3-6. Distribution of maximum member forces (axial force) of water channel.

Table 4.6.3-6. Analysis models and analysis conditions (water pit).

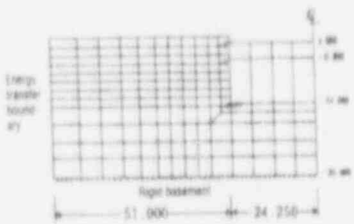
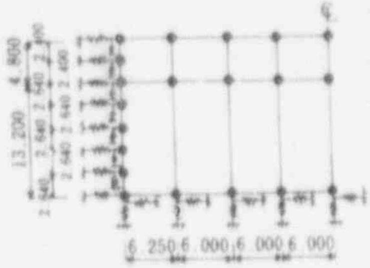
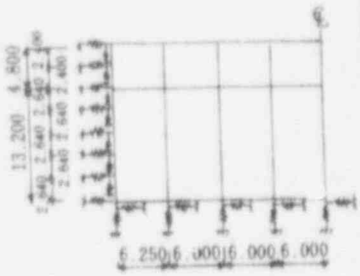
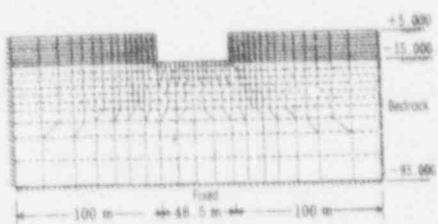
	"FEM"	"Multi-input response analysis"	"Response displacement method"
Analysis models and analysis conditions	 - In the model, the pit is represented by beam elements, and the soil is represented by two-dimensional FEM. - The side boundary is taken as energy transfer boundary, the lower boundary is taken as rigid foundation. - In addition to the self weight of the pit, the loads carried on the top/medium floor plates and the water contained are also taken into consideration as additional mass. - The converted values of G and h obtained in the soil response calculation are used as the values of the soil properties. - The complex response method is used for calculation in the frequency range up to 25 Hz.	 - In the model, the pit is represented by beam elements, and the soil is represented by discrete springs (lower surface: normal springs/shear springs; side surface: normal springs/shear springs). - In addition to the self weight of the pit, the loads carried on the top/medium floor plates and the water contained are also taken into consideration as additional mass. - Analysis is performed using the mode superposition method, with fundamental vibration mode (4.01) through 10th vibration mode (41.6) superimposed. - Damping is taken as the strain energy proportional type (pit: 5%, soil spring on side surface: 5%, soil spring on bottom: 2%).	 In the model, the pit is represented by beam elements, and the soil is represented by discrete springs (bottom surface: normal/shear springs; side surface: normal/shear springs).
Soil spring		 The soil spring constant is calculated by performing static FEM analysis. - The analysis model is shown in the diagram above, with the pit portion excluded. The boundaries are fixed both on the sides and on the bottom. - The converged value of G obtained in the soil response calculation is used as the value of the soil property.	

Table 4.6.3-6 *(Cont'd)*. Analysis models and analysis conditions (water pit).

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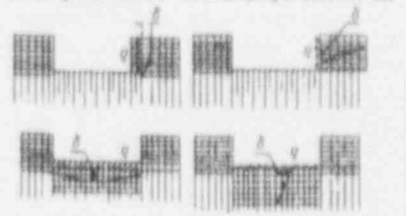
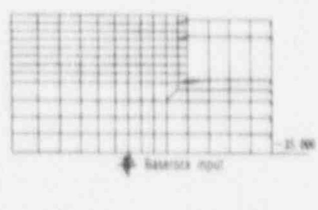
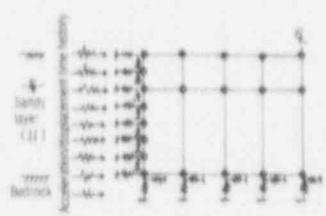
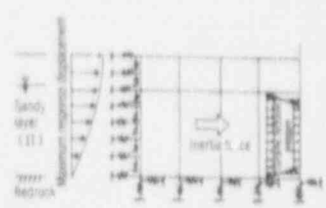
<td colspan="2">

- The soil reaction coefficient is calculated using the displacement distribution δ due to a uniformly distributed load q in the direction of each spring.

$$k = q/\delta$$
- The soil spring constant is calculated from the distribution of the soil reaction force coefficient.</td>
</tr>
<tr>
<td>Input conditions</td>
<td>

(EL. −35.000)
The response (acceleration time history) on the assumed seismic input basement surface (EL. −35.000) obtained from the soil response calculation is input from the rigid foundation.</td>
<td>

The responses (acceleration/displacement time history) of the various layers obtained from the soil response calculation are input via side-surface normal springs and bottom surface shear springs.</td>
<td>

From the maximum response value obtained in soil response calculation,
1. The maximum response displacement is input in the same direction via the normal springs on the two side walls.
2. From the maximum response acceleration distribution, the horizontal seismic coefficient is determined, and the inertial force due to structure and overburden load on floor plate.
3. The average value of the horizontal seismic coefficient set in 2. is used, dynamic hydraulic pressure according to "Guidelines of Aseismic Work of Water Facilities."</td>
</tr>
</table>

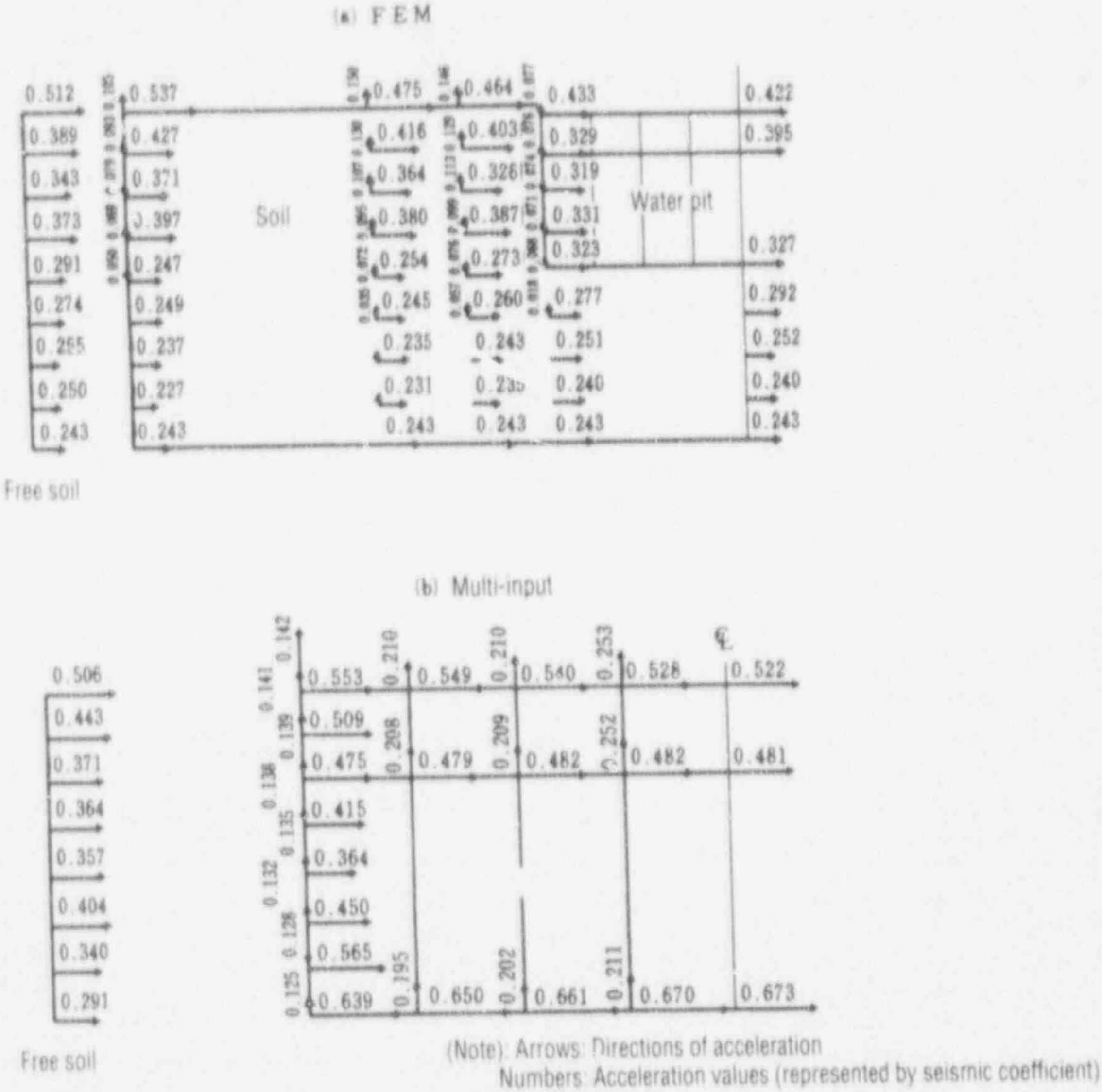


Figure 4.6.3-7. Distribution of maximum response acceleration (units: G) using different analysis methods (water pit)

{2} Figure 4.6.3-8 illustrates the distribution of the maximum member forces (bending moment). It can be seen that the three methods provide nearly the same results with respect to the location of the maximum member forces, distribution pattern, and magnitude.

(c) Seawater duct

(i) Analysis methods and analysis conditions

The following two analysis methods were used for analysis.

{1} FEM [4.6.3-1]
{2} Response displacement method [4.6.3-2]

Table 4.6.3-7 lists the analysis models and analysis conditions. T determine the earthquake response of ground, the one-dimensional multiple reflection theory is applied on the soil model to compute the responses of the various layers of the ground. The strain dependency of the sand layer is evaluated using the equivalent linear method. The location of the seismic motion input is at the assumed input ground plane (EL -20.0 m).

(ii) Evaluation items

The analyses using the above two methods were performed and the following items were evaluated:

{1} Maximum response acceleration
{2} Maximum member forces (bending moment, axial force, shear force)

(iii) Comparison of analysis results

{1} Figure 4.6.3-9 illustrates the distribution of the maximum response acceleration (horizonal acceleration only) obtained from "FEM." It can be seen that although the magnitude of response is a little larger at the duct ceiling plate than the surrounding soil, generally speaking, the vibration of the seawater duct is similar to that of the surrounding soil. This is because the effective weight of the duct is less than the surrounding soil; hence, there is no self-vibration of the seawater duct.

{2} Figure 4.6.3-10 shows the distribution of the maximum member forces obtained using the two methods. It can be seen that these two methods provide similar distribution shape and magnitude. Generally speaking, a comparison of member force amplitudes gives the following tendency: "FEM" ≥ "response spectrum method."

(d) Summary

From the aforementioned analysis results, it can be seen that for the maximum response acceleration and the maximum response displacement, although there are differences in some cases between different analysis methods, there is no significant difference in member forces that is large enough to affect the cross-sectional design.

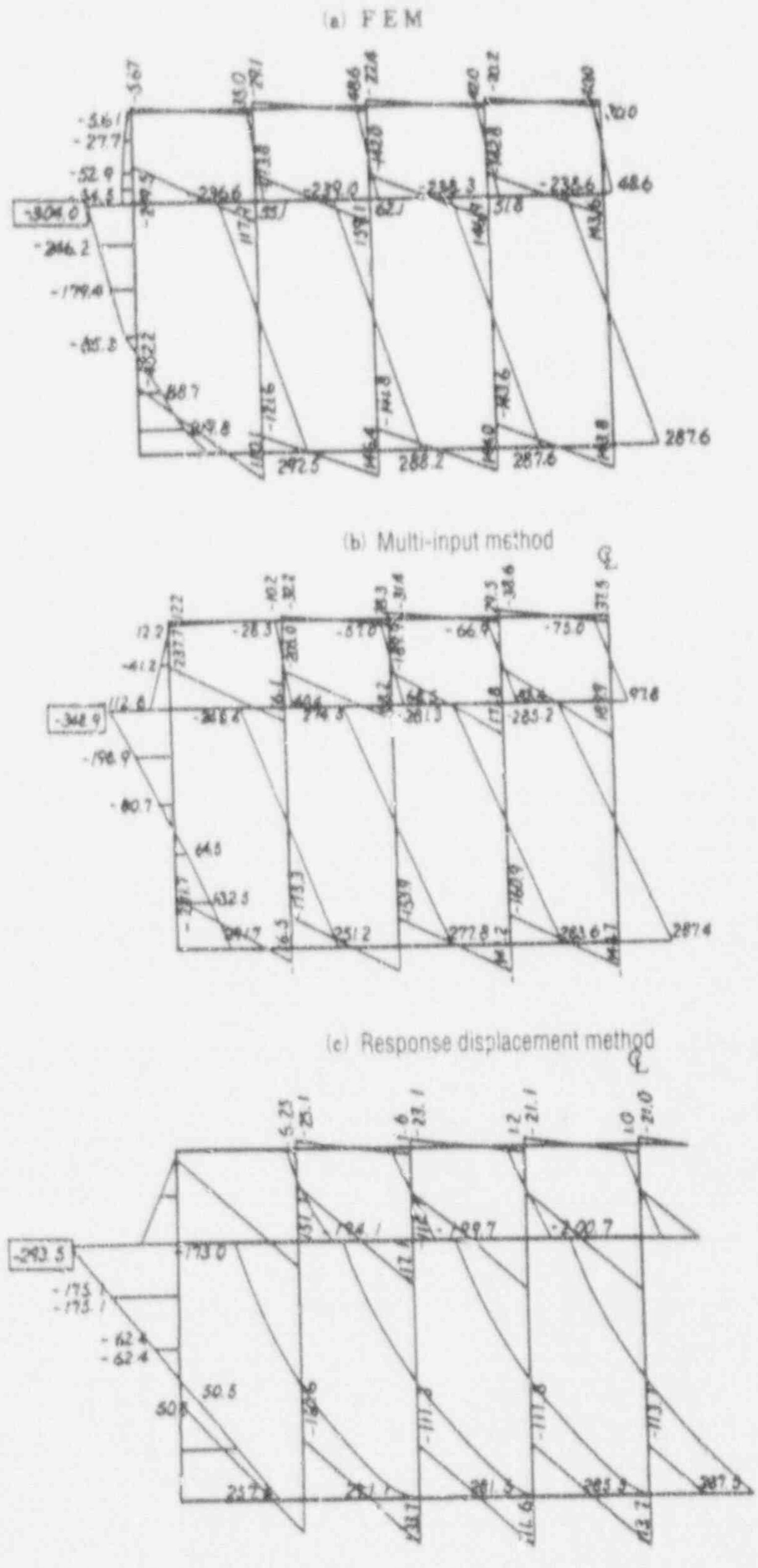
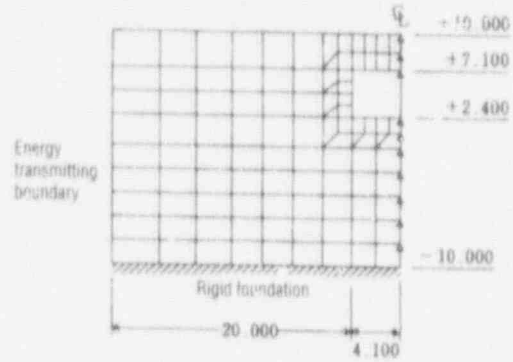
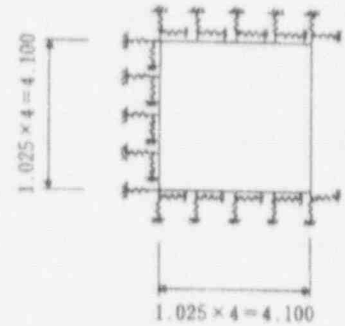
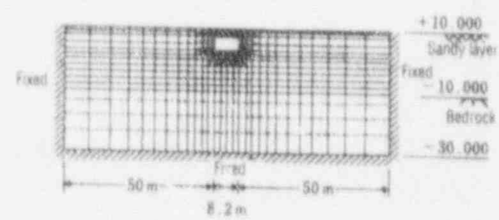


Figure 4.6.3-8. Distribution of member forces (bending moment) using different analysis methods (water pit).

Table 4.6.3-7. Analysis models and analysis conditions (seawater duct).

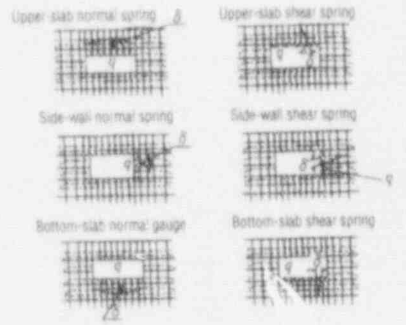
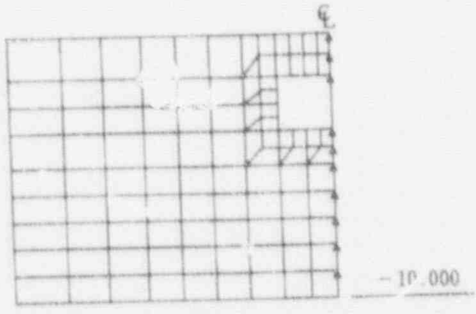
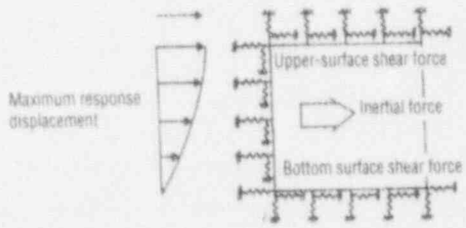
<table>
<tr><th></th><th>"FEM"</th><th>"Response displacement method"</th></tr>
<tr><td>Analysis models and analysis conditions</td><td>

- In the model, the duct is represented by beam elements, and the soil is represented by two-dimensional FEM.
- The side boundary is taken as the energy transfer boundary, the lower boundary is rigid foundation.
- In addition to the self wight of the duct, the weight of the pipe is also taken into consideration as additional mass.
- The converged values of G and h obtained in the soil response calculation are used as the values of the soil properties.
- The complex response method is used for calculation in the frequency range of up to 25 Hz.</td><td>

- In the model, the duct is represented by beam elements, and the soil is represented by discrete springs (upper surface: normal/shear springs; side surface: normal/shear springs; bottom surface: normal/shear springs).</td></tr>
<tr><td>Soil spring (part 1)</td><td></td><td>

- The soil spring constant is calculated using static FEM analysis.
- The analysis model shown in the diagram above has fixed boundaries on both the side walls and lower wall.
- The converged value of G obtained in the soil response calculation is used as the value of the soil property.</td></tr>
</table>

Table 4.6.3-7 *(Cont'd)*. Analysis models and analysis conditions (seawater duct).

Soil spring (part 2)		 - The soil reaction coefficient is calculated using the displacement distribution δ under action of a uniformly distributed load q in the direction of each spring $k = q/\delta$ - The soil spring constant is calculated from the distribution in the soil reaction force coefficient.
Input conditions	 Ground input The response (acceleration time history) on the virtual seismic input base ground surface (EL. −10.000) obtained from the soil response calculation is input from the rigid foundation.	 From the maximum response value obtained in the soil response calculation, 1. The maximum response displacement (differential displacement from the bottom surface) is input in the same direction through the normal springs on the two side walls and shear springs on the upper surface. 2. From the distribution of the maximum response acceleration distribution, the horizontal seismic coefficient is determined, the inertial force due to the body and piping load. 3. From the maximum shear stress distribution, the shear stress of the upper-surface level is applied on the upper surface, the horizontal stress equal to the sum of the upper-surface shear force and inertial force is applied on the bottom surface.

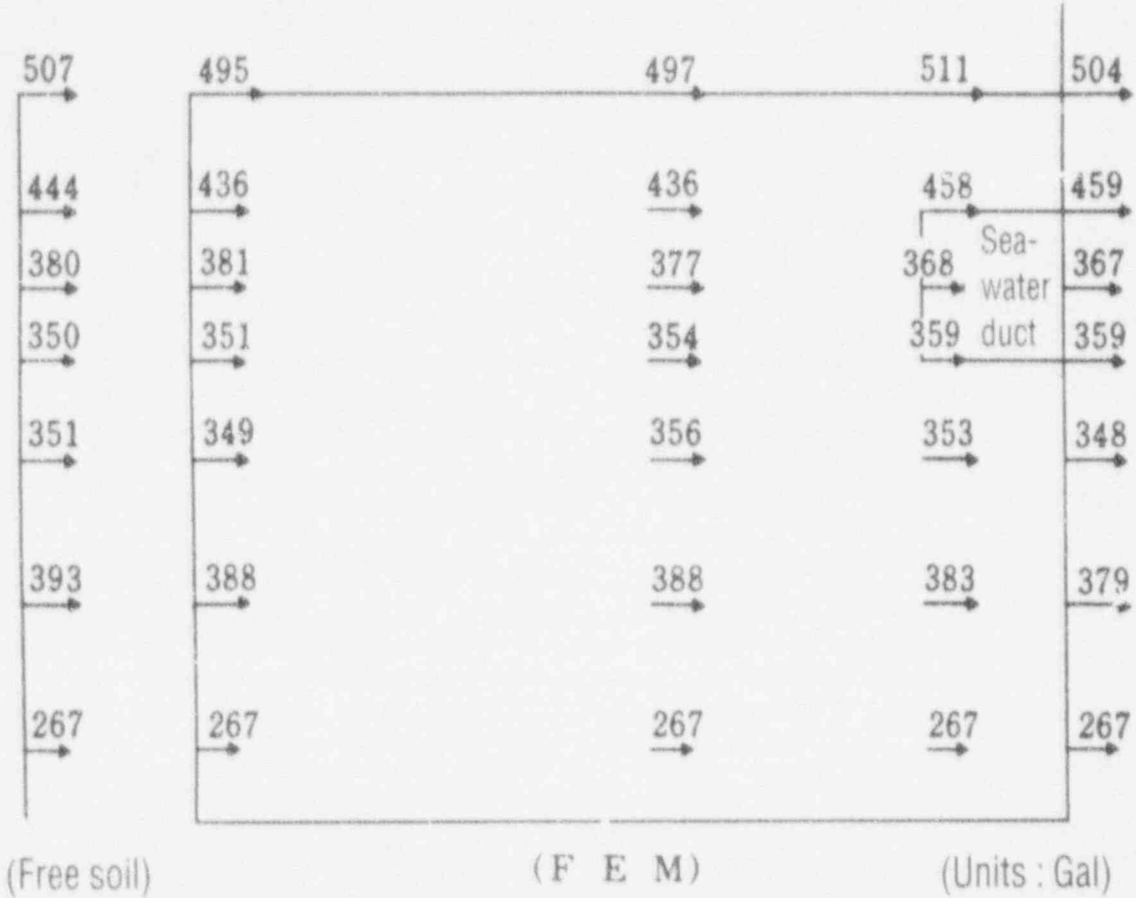


Figure 4.6.3-9. Distribution of maximum response acceleration determined using FEM (seawater duct).

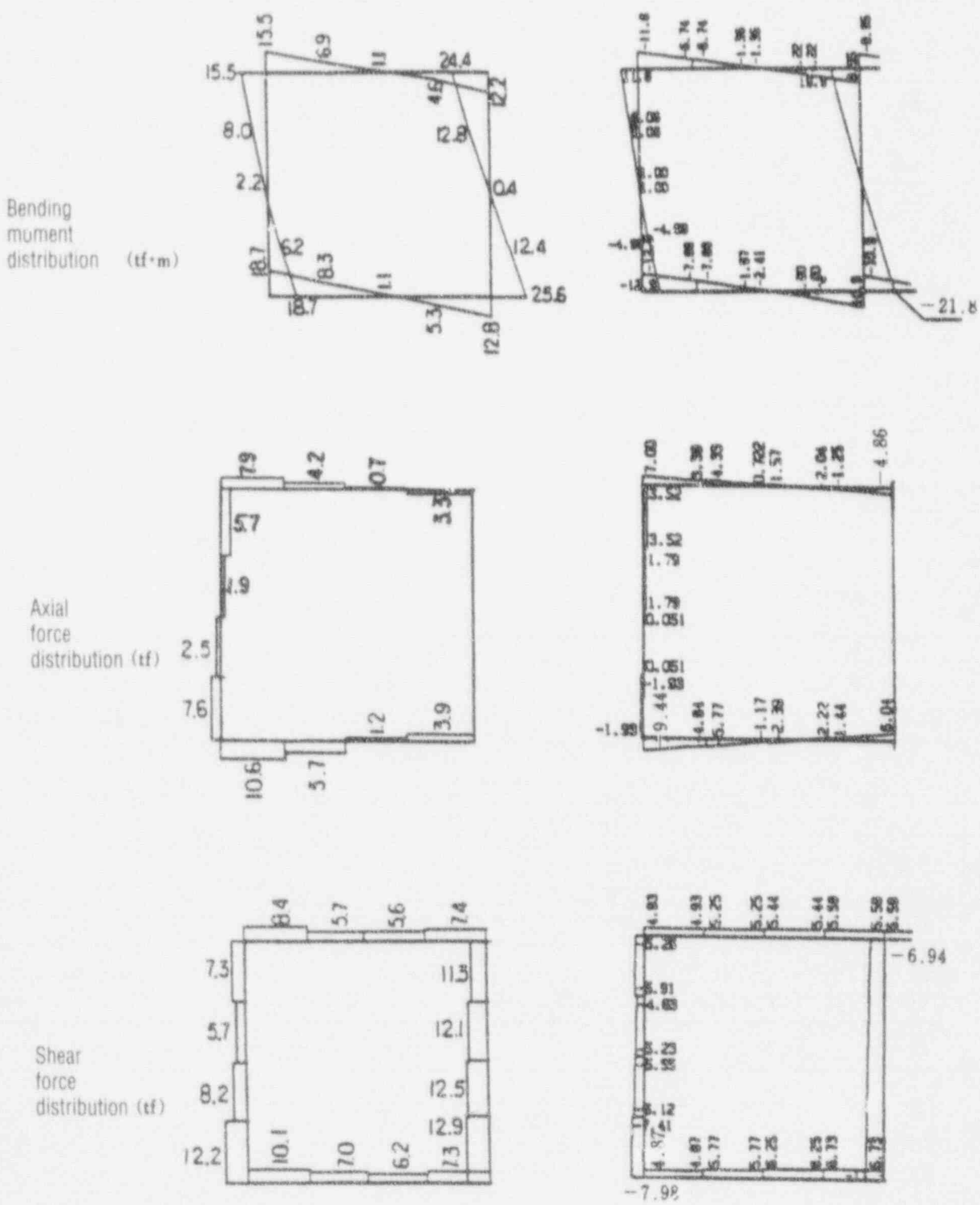


Figure 4.6.3-10. Distribution of maximum member forces using different analysis methods (seawater duct).

b. Evaluation of soil spring constant

(a) Water channel

(i) Calculation methods of soil spring constant used in evaluation

{1} Method defined in Guidelines of Road and Bridge [4.6.3-4]
{2} Method defined in Guidelines and Commentary of Aseismic Work of Water Facilities [4.6.3-5]

(ii) Evaluation items

{1} Distribution of soil spring constants
{2} Maximum member forces calculated by multi-input response analysis
{3} Effect of soil spring constant on maximum member forces.

Here, the member forces are calculated using the response displacement method.

(iii) Comparison of analysis results

{1} Figure 4.6.3-11 shows the distribution of soil spring constants calculated using two different methods. In the figure, the "spring constant based on Guidelines of Road and Bridge" is calculated using N-value, and is a constant value of 3,840 tf/m^2; the "spring constant based on Guidelines of Aseismic Work of Water Facilities" is calculated using the shear modulus of elasticity of the surface layer of soil (here, the shear modulus of elasticity is the converging value calculated by equivalent linear soil response calculation method by the multiple reflection analysis) is 6,270 tf/m^2 in the thick portion of the surface [illegible] the sea side, and it decreases as the bedrock surface becomes shallower towards the land side; it becom[illegible] 65 [illegible]f/m^2 at the end portion.

{2} Figure 4.6.3-12 shows the member forces (axial forces) calculated using two types of spring constants. It can be seen that the maximum value obtained form "the spring constant based on Guidelines of Road and Bridge" is 2,630 tf, and the maximum value obtained from "the spring constant based on Guidelines of Aseismic Work of Water Facilities" is 2,900 tf. Both the analysis results show the peaks at the same location where the bedrock plane is inclined on the land side. Also, their distribution patterns are almost identical.

{3} Figure 4.6.3-13 shows the member forces (axial force) of the water channel when the values of the soil spring constants are changed. Since the stiffness of the structure is higher than the stiffness of soil, the axial direction of the water channel may be easily affected by the soil spring constant.

(b) Water pit

(i) Calculation methods of soil spring constants

{1} Method using static FEM analysis
{2} Method using the Guidelines of Road and Bridge and elastic theoretical solution [4.6.3-6] (solution by Tajimi on rectangular foundation)

According to method {2}, the normal springs and shear springs determined at the bottom surface of the water pit are calculated using the elastic theoretical solution for bedrock; the normal springs and shear springs determined on the side surfaces of the water pit are calculated on the basis of the Guidelines of Road and Bridge. In this case, the depth of the water pit is taken as 70 m. (Shear spring constant = 1/3x (normal spring constant). Also, both the shear spring constants and the normal spring constants are assumed to be distributed uniformly in the depth direction.)

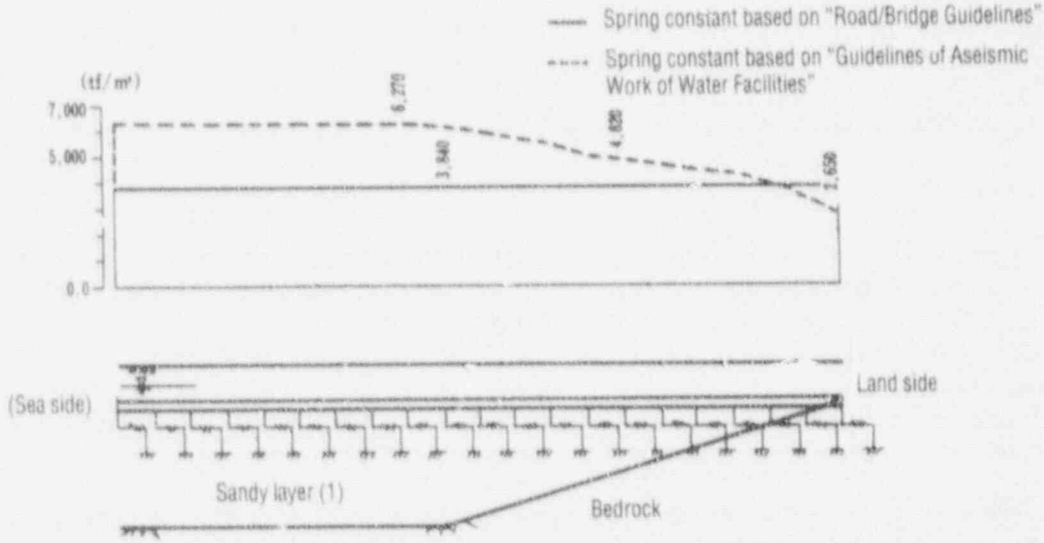


Figure 4.6.3-11. Distribution of soil spring constant (water channel).

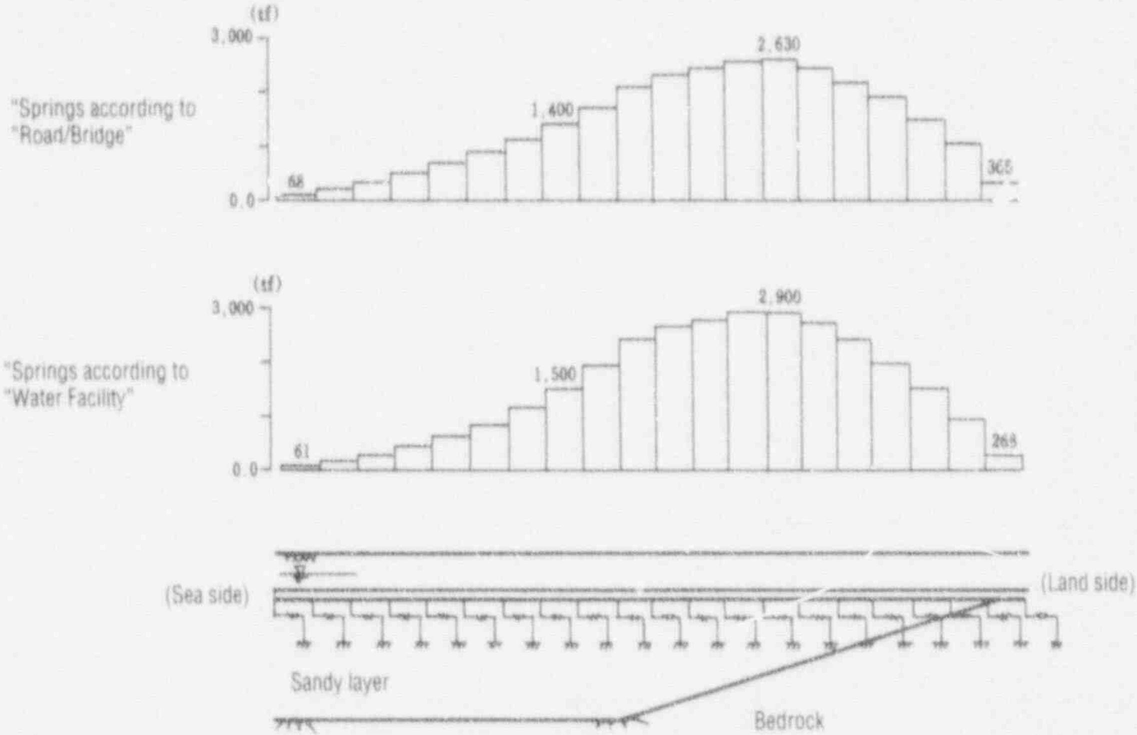


Figure 4.6.3-12. Distribution of maximum member forces (axial force) (water channel).

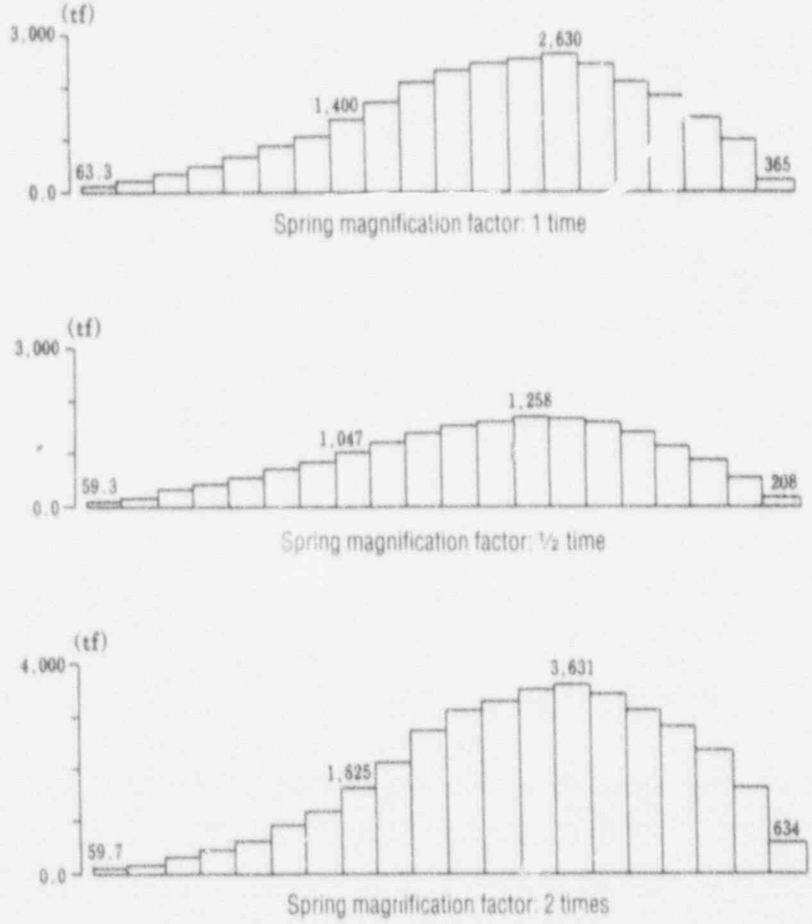


Figure 4.6.3-13. Effect of magnitude of soil spring constant on axial force of water channel.

(ii) Evaluation items

{1} Dependency of the soil reaction force coefficient and member forces on calculation methods of soil spring constant
{2} Effects of depth of lower boundary of static FEM analysis on soil reaction force coefficient and member forces
{3} Effect of presence/absence of shear spring on pit side surface (static FEM analysis) on the maximum member forces
{4} Effects of the values of soil spring constant on the maximum member forces

Here, calculation of the member forces is performed by using the response displacement method.

(iii) Comparison of analysis results

{1} Figure 4.6.3-14 shows the distribution of soil reaction force coefficient obtained using two calculation methods. Generally speaking, the magnitude determined by the "method using static FEM analysis" is larger. Also, Figure 4.6.3-15 illustrates the distribution of the member forces (bending moment). It can be seen that the distribution profiles obtained by the above two methods are similar to each other and coincide with the deformation shape of the whole structure due to shear deformation. The magnitude of the member forces determined from "the spring constant using static FEM analysis" is about 0-20% larger.

{2} Figure 4.6.3-16 shows the distribution of soil reaction force coefficient in the case when the depth of the lower boundary in the static FEM analysis is changed. It can be seen that the location of the lower boundary has the largest influence on the normal springs on the bottom surface. In addition, Figure 4.6.3-17 shows the distribution of the member forces (bending moment, shear force). Although there is no significant difference in the overall pattern, for the bending moment and shear force at the corner between the side wall and bottom plate, the values obtained for a shallow boundary (EL. -35 m) are about 20% larger than those obtained for a deeper boundary (EL. -95 m).

{3} Figure 4.6.3-18 shows the distribution of member forces (bending moment) in the presence/absence of the side-wall shear springs. It can be seen that there exists a certain difference in the bending moment on the side wall. The value obtained "without springs" is smaller by about 15% than the value obtained "with springs."

{4} Figure 4.6.3-19 shows the member forces (bending moment, axial force) of the water pit when the values of the soil spring constant are changed. In this case, the effect of the magnitude of the spring constant on the member forces is small.

(c) Seawater duct

(i) Calculation methods of soil spring constant used for analysis

{1} Method using static FEM analysis
{2} Method according to Guidelines of Road/Bridge [4.6.3-7]

Shear spring constant = 1/3x (normal spring constant). Both the shear spring constant and the normal spring constant are assumed to be distributed uniformly.

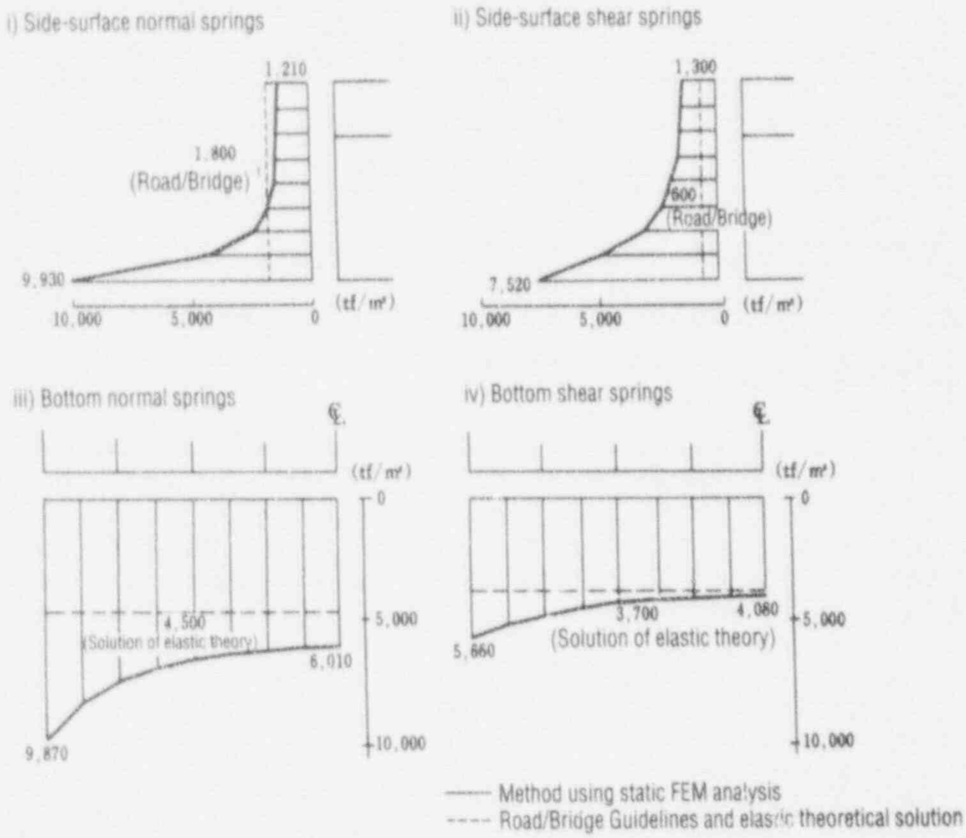


Figure 4.6.3-14. Distribution of soil reaction force coefficient according to different calculation methods of soil spring constant (water channel).

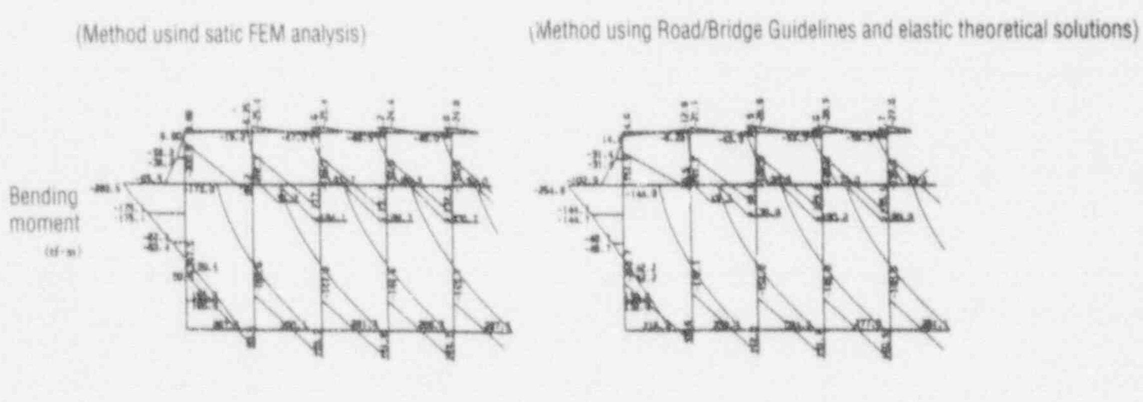


Figure 4.6.3-15. Member forces determined using different methods of calculating soil spring constants (water channel).

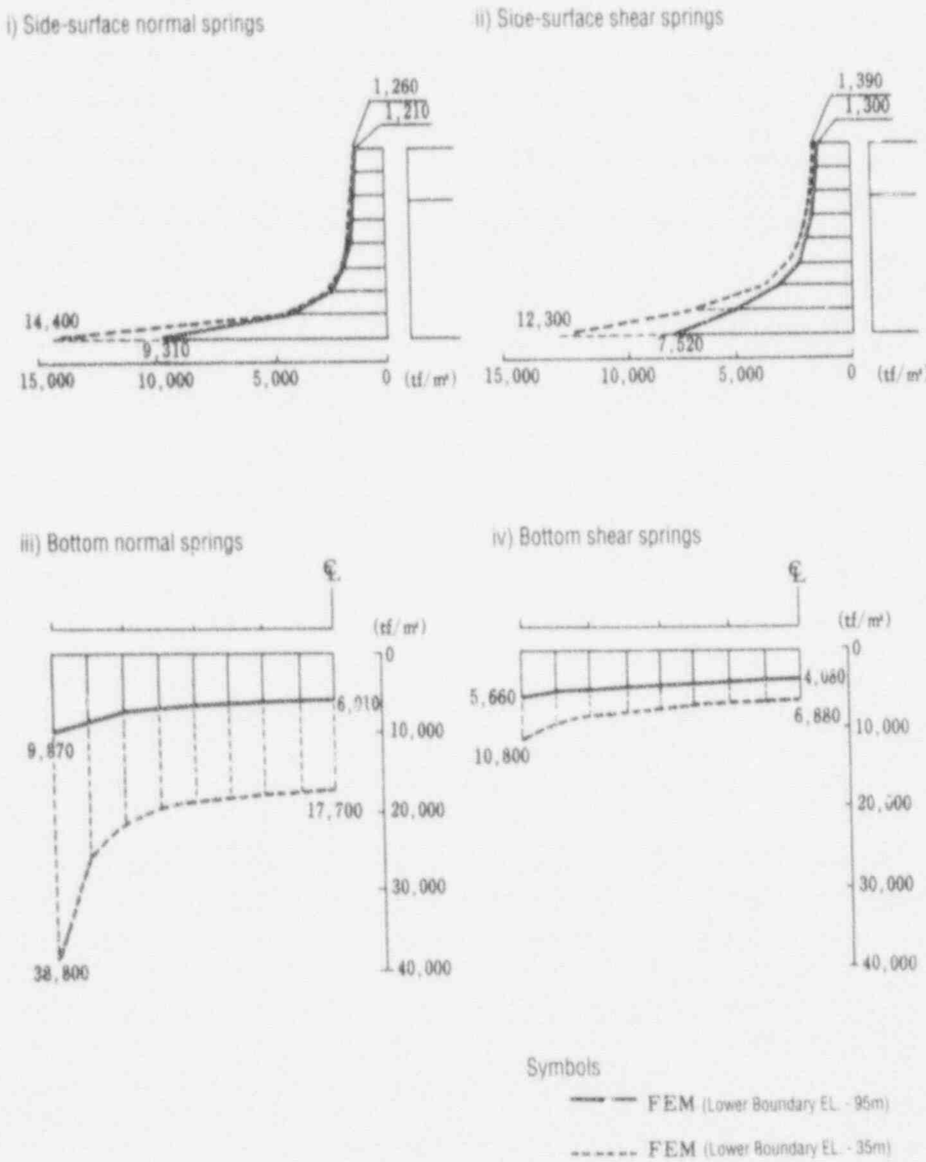


Figure 4.6.3-16. Distribution of soil reaction force coefficient for different lower boundary locations in static FEM **analysis used** for calculating the soil spring constant (water channel).

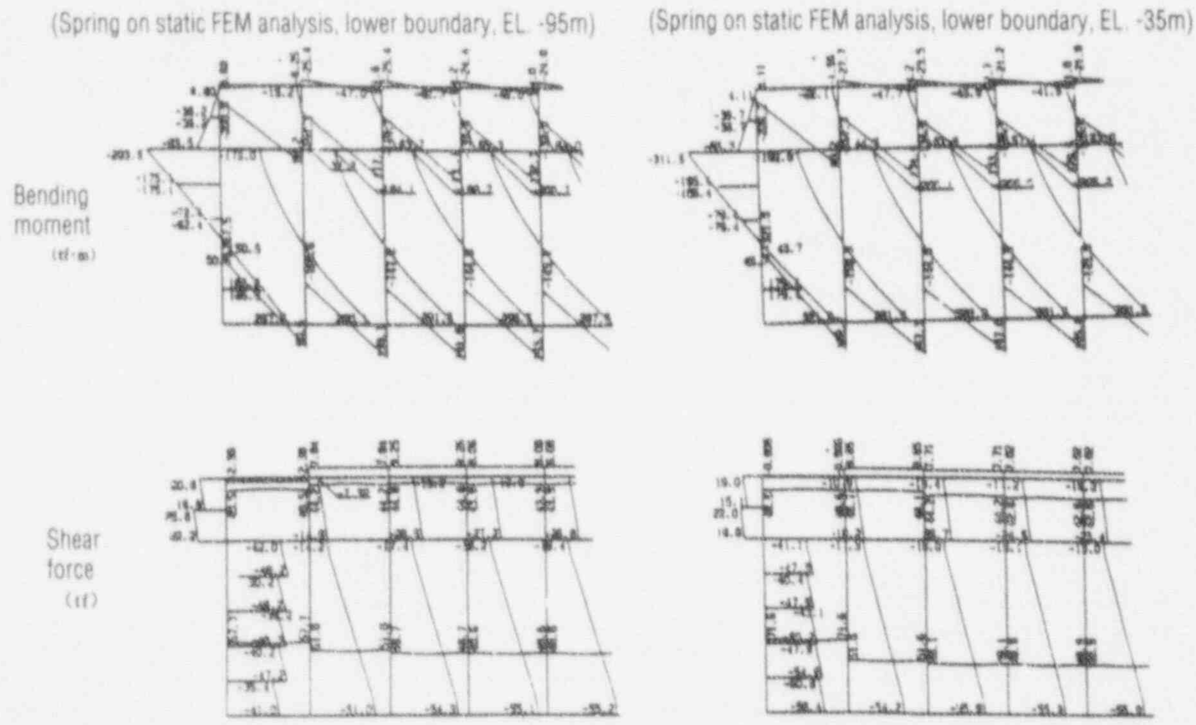


Figure 4.6.3-17. Member forces for different lower boundary positions in static FEM analysis for calculating the soil spring constant (water channel).

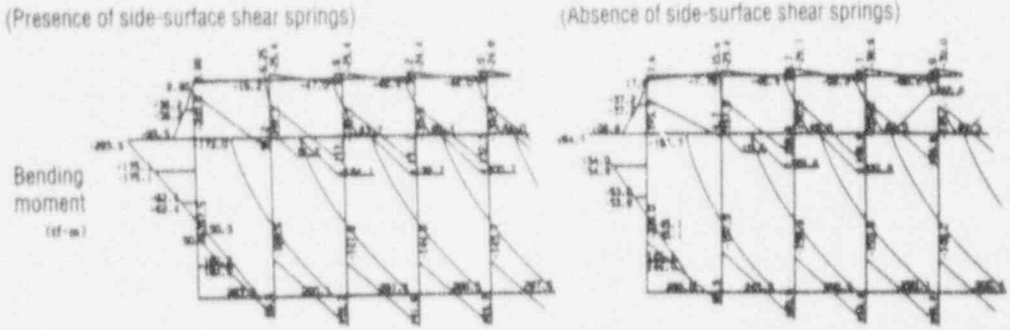


Figure 4.6.3-18. Comparison of member forces (bending moment) between presence/absence of side-surface shear springs (water channel).

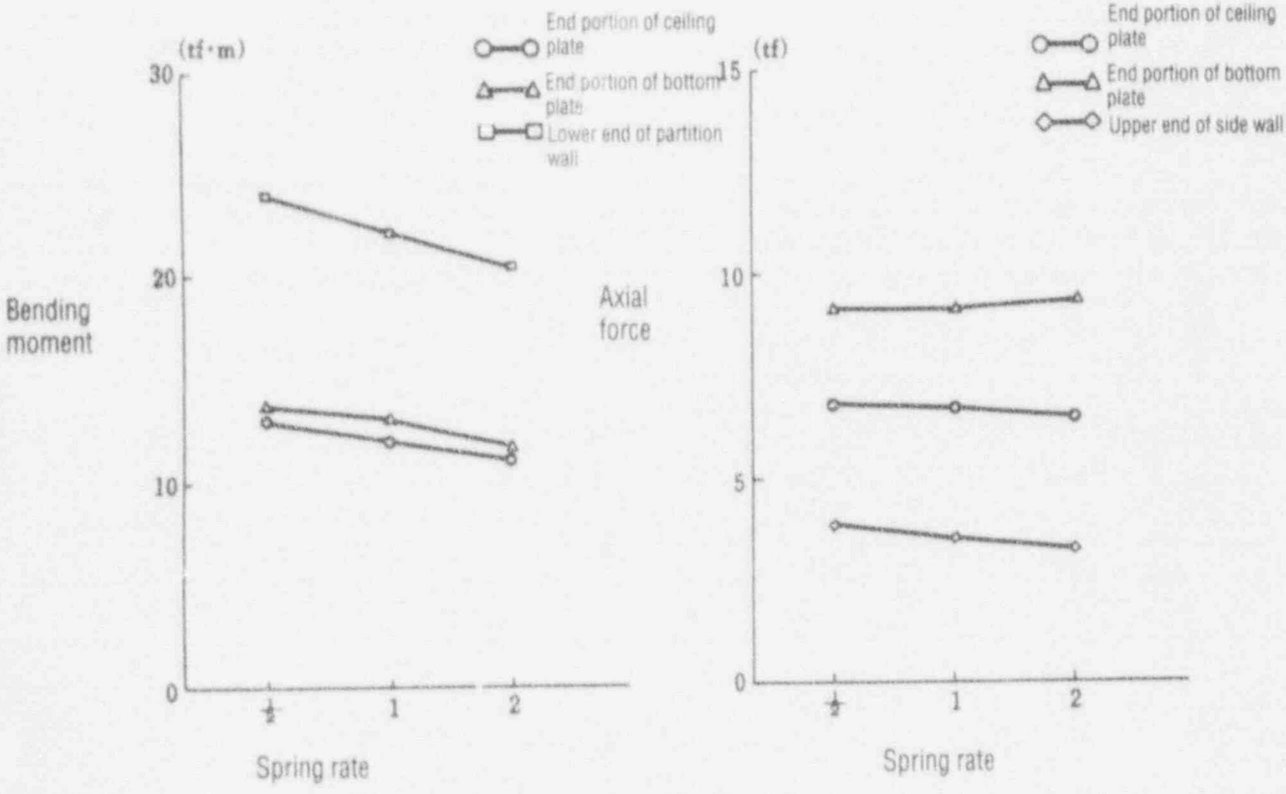


Figure 4.6.3-19. Effect of magnitude of spring constant on member forces (bending moment, axial force) of water pit.

(ii) Evaluation items

{1} Maximum displacement and member forces (bending moment) according to different calculation methods of soil spring constants
{2} Effect of magnitude of soil spring constant on the maximum member forces

Here, calculation of the displacement and member forces is performed using the response displacement method.

(iii) Comparison of analysis results

{1} Figures 4.6.3-20, 21 show the distribution of displacement and member forces (bending moment) determined using the two calculation methods. It can be seen that the difference between these two cases is small for both the displacement distribution and the member forces.

{2} Figure 4.6.3-22 shows the member forces of a seawater duct (bending moment) when the magnitude of the soil spring constant is changed. In this case, the influence of the magnitude of the spring constant on the member forces is small.

(d) Summary

(i) Although the distribution of the soil spring constant may depend significantly on the method of calculation in some cases, this difference nevertheless has little influence on the member forces.

(ii) For an underground structure with a stiffness similar to or less than that of the soil, the member forces do not depend significantly on the variation of the soil spring constant. However, attention should be paid to the fact that this does not apply when the stiffness is large, such as in the axial direction of the water channel.

c. Consideration on earth pressure during earthquake using seismic coefficient method

For the water pit, the effects of using different loading schemes of the earth pressure during earthquake on the seismic calculation results are evaluated when the seismic coefficient method is used. The results are compared with those obtained by the response displacement method which uses dynamic seismic force. A horizontal seismic coefficient of 0.3 and a vertical seismic coefficient of 0.15 are used as the static seismic force.

(a) Loading methods of earth pressure during earthquake in analysis

{1} Active earth pressure + active earth pressure
{2} Active earth pressure + passive resistant earth pressure
{3} Active earth pressure + static earth pressure
{4} Active earth pressure + earth springs (uniform distribution)
{5} Active earth pressure + earth springs (triangular distribution)

Table 4.6.3-8 shows the combination of loading pattern in seismic coefficient method. Table 4.6.3-9 lists the kind of load in the seismic coefficient method.

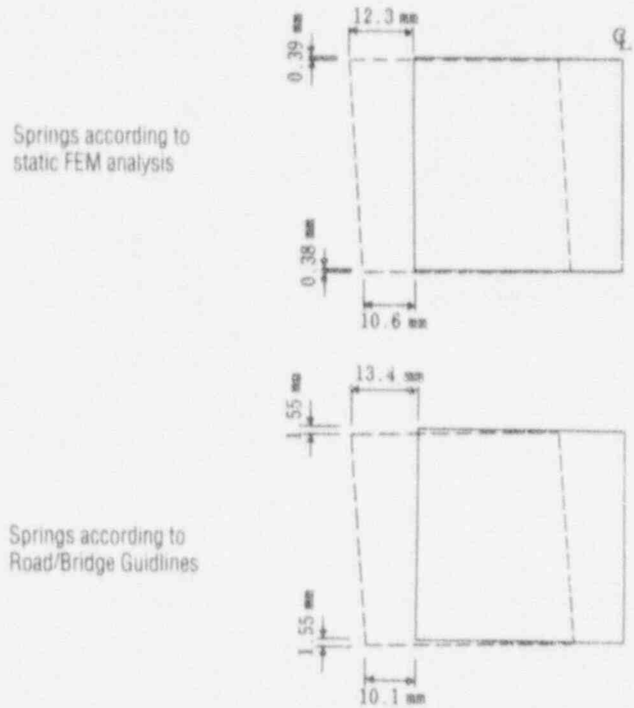


Figure 4.6.3-20. Displacements for different calculation methods of soil spring constant (seawater duct).

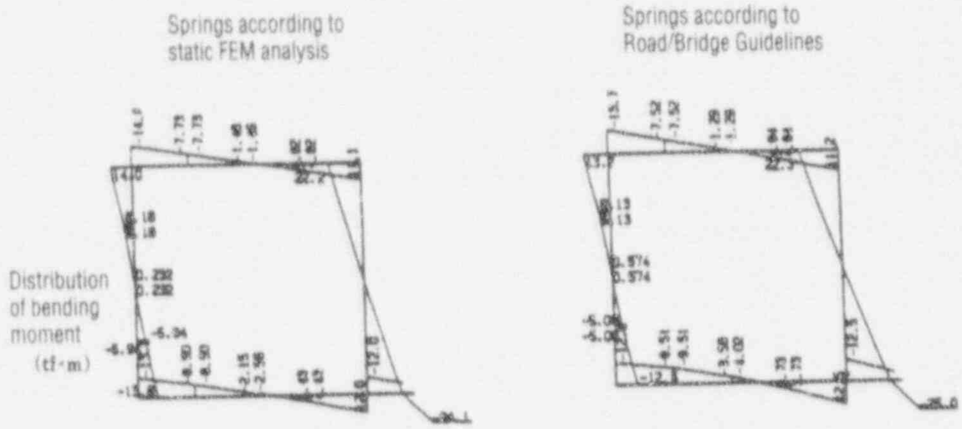


Figure 4.6.3-21. Member forces (bending moment) for different calculation methods of soil spring constant (seawater duct).

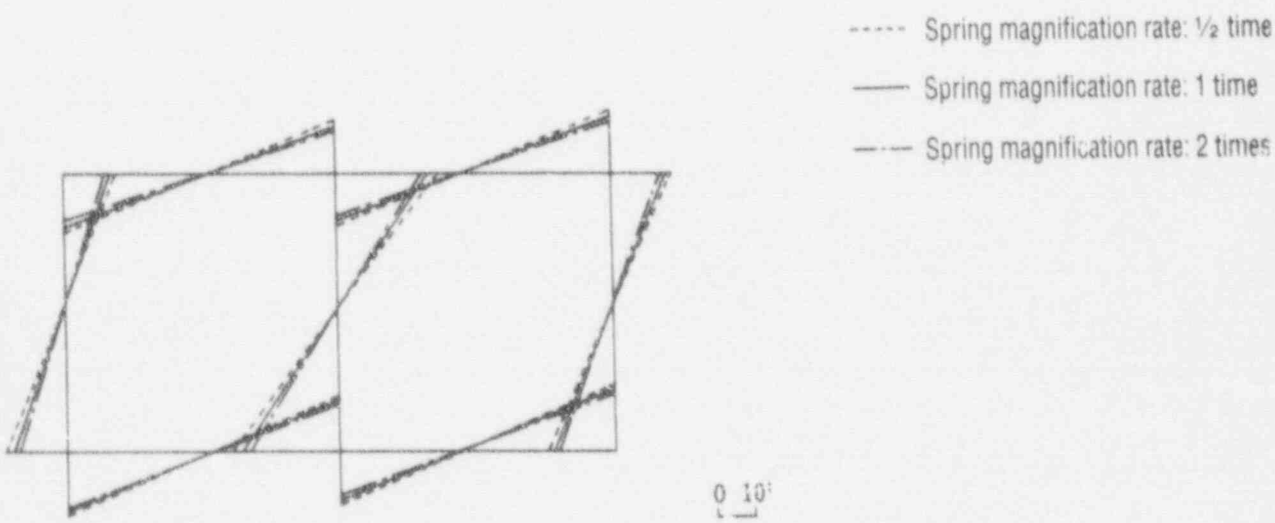
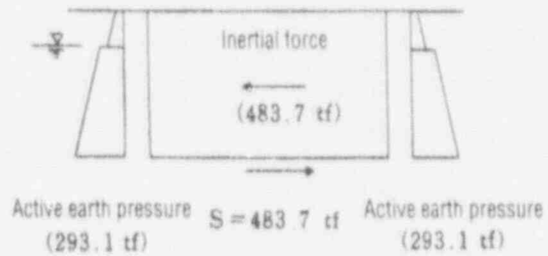
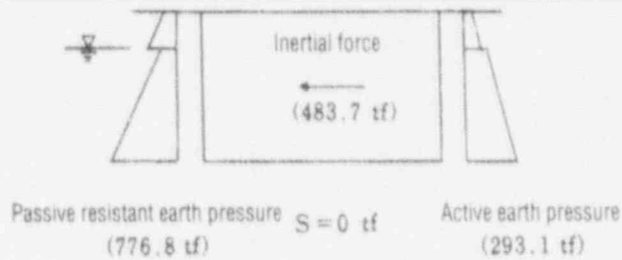
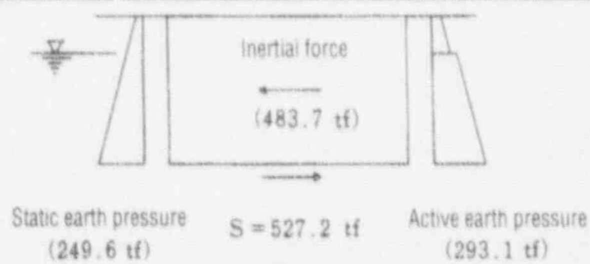
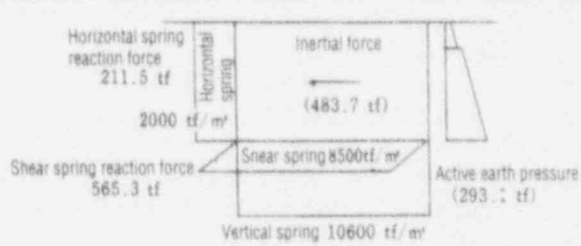
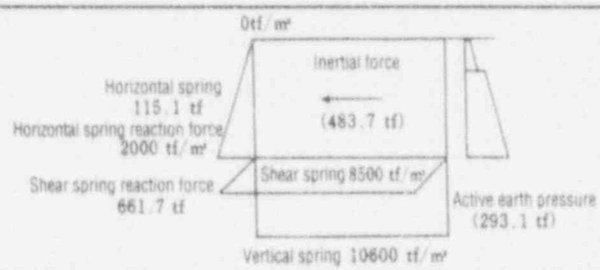


Figure 4.6.3-22. Effect of magnitude of soil spring constant on the member forces (bending moment) of seawater duct (horizontal component during earthquake).

Table 4.6.3-8. Load diagram for evaluation of the application scheme of earth pressure during earthquake according to the seismic coefficient method (water pit).

<table>
<tr><th></th><th>Load diagram</th></tr>
<tr><td>Active earth pressure
+
Active earth pressure</td><td>
</td></tr>
<tr><td>Passive resistant earth pressure
+
Active earth pressure</td><td>
</td></tr>
<tr><td>Static earth pressure
+
Active earth pressure</td><td>
</td></tr>
<tr><td>Soil spring (uniform distribution)
+
Active earth pressure</td><td>
</td></tr>
<tr><td>Soil spring (triangular distribution)
+
Active earth pressure</td><td>
</td></tr>
</table>

Note: The inertia forces in the figure include the inertia forces for the self weights, load, and contained water.

Table 4.6.3-9. Evaluation conditions used for evaluating earth pressure during earthquake.

Item of condition	Design value
Self-weight	Taken as horizontal inertial force and vertical inertial force.
Live load	
Water contained	The dynamic hydraulic pressure is evaluated using Westergard's formula ("Guidelines of Aseismic Works of Water Facilities").
Soil load	According to the earth pressure formula during earthquake given by Mononobe and Okabe. (K_H = 0.3, K_V = 0.15 (upwards)) Active earth pressure coefficient K_{EA} = 0.477 (in air), 1.054 (in water). Passive earth pressure coefficient K_{EP} = 3.413 (in air), 2.285 (in water). Wall-surface frictional angle $\delta = 0°$.

(b) Evaluation items

{1} Effects of the aforementioned five types of earth pressure loading methods on the member forces in the seismic coefficient method

{2} Comparison between member forces according to the seismic coefficient method and member forces according to the response displacement method

(c) Results of evaluation

Figure 4.6.3-23 shows the distributions of the bending moment using the seismic coefficient method and the response displacement method.

{1} The distribution in the case of "active earth pressure + passive resistant earth pressure" is different from the distributions in the other cases. This is because a resistant earth pressure is determined to be equal to the sum of the total inertial force and active earth pressure; as a result, the bottom shear force becomes zero, and the entire water pit is subjected to a load state without shear deformation.

{2} The results of the response displacement method is similar to the results except in the case of "active earth pressure + passive resistant earth pressure." In particular, in this case, the results are very similar to those of the case of "active earth pressure + static earth pressure."

The same results are obtained for the evaluation performed on the seawater duct.

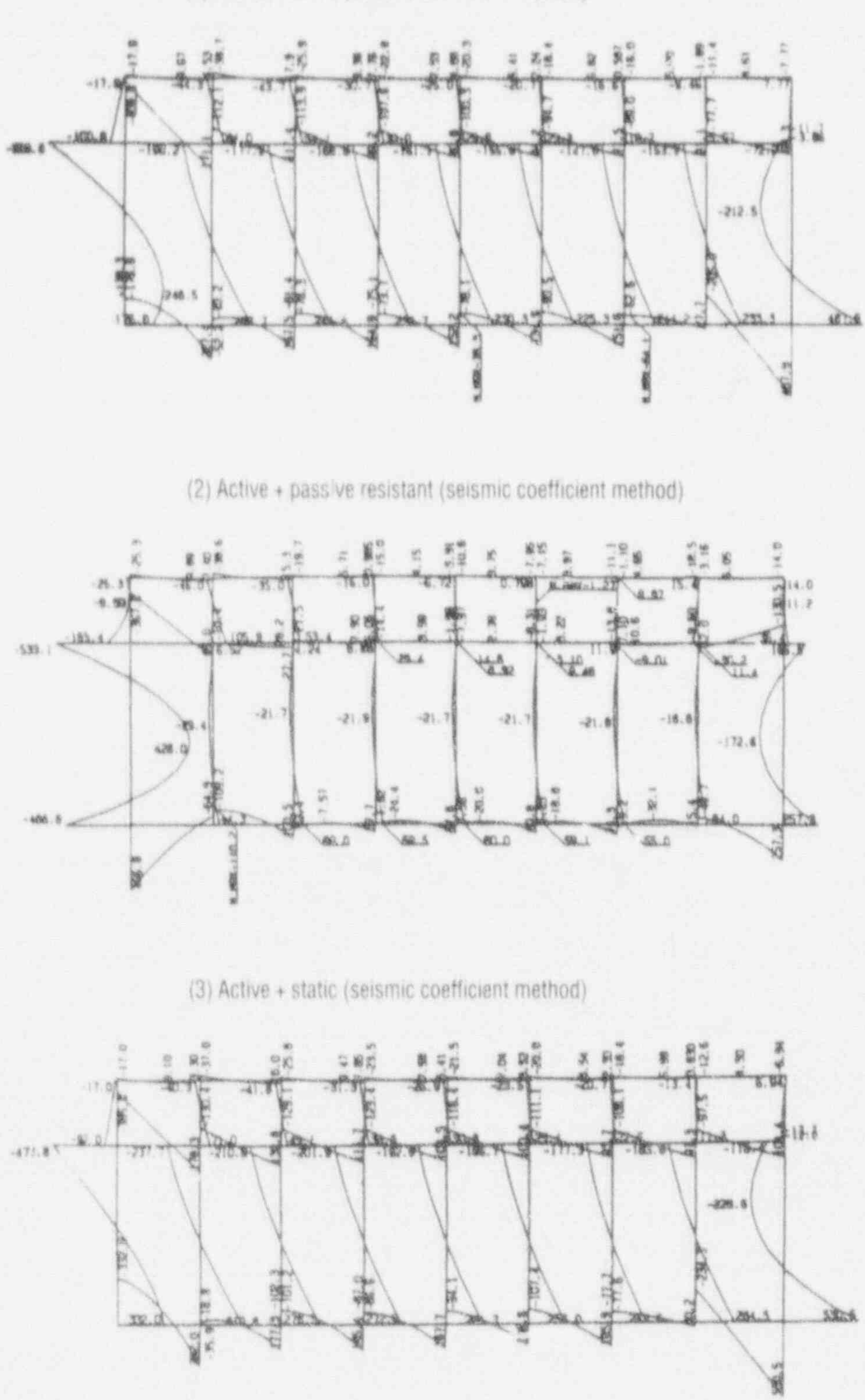
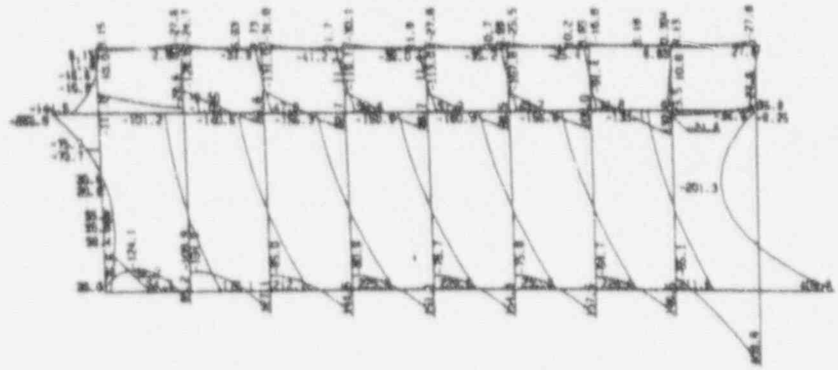
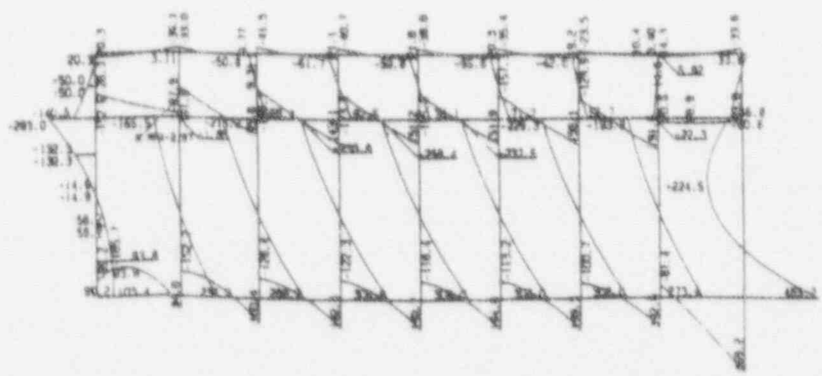


Figure 4.6.3-23(a). Distribution of bending moment using seismic coefficient method and response displacement method.

(4) Active + spring (uniform distribution) (seismic coefficient method)



(5) Active + spring (triangular distribution) (seismic coefficient method)



(6) Response displacement method

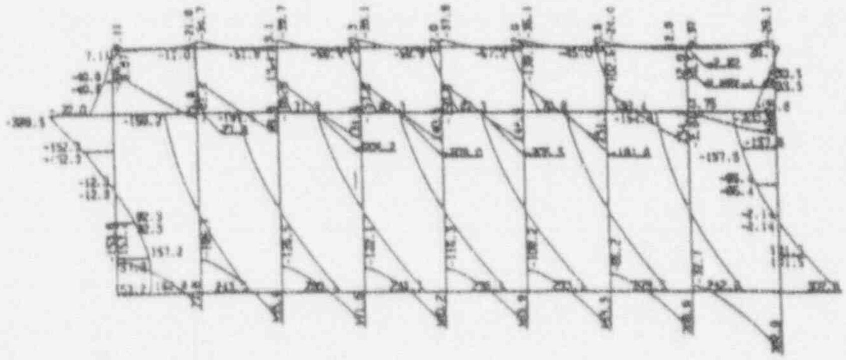


Figure 4.6.3-23(b). Distribution of bending moment according to seismic coefficient method and response displacement method.

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Chapter 5. Aseismic Design of Building Structures

5.1 Basic Items

5.1.1 Basic guidelines of aseismic design

Any nuclear reactor facility should have high enough earthquake strength to ensure that in any anticipated earthquake the damage to the nuclear reactor facility does not become the cause of a major accident. For this purpose, aseismic design should be carried out according to "Guidelines for evaluation of aseismic design of nuclear reactor facilities in nuclear power plants: Nuclear Power Safety Committee, July 20, 1981" (referred to as "Evaluation Guidelines" hereinafter).

The basic guidelines for the aseismic design are as follows:

{1} In principle, the building and facilities should be a rigid structure.

{2} In principle, the reactor buildings and other important facilities should be supported on bedrock.

{3} The degree of importance of the earthquake strength of a nuclear reactor facility can be classified as Class A, Class B, or Class C from the viewpoint of the effect on the environment of the radioactivity that might be released during an earthquake. Aseismic design should be performed according to this importance.

{4} The facilities of Classes A, B, and C should be designed to resist the seismic force on the basis of the base shear coefficient which is determined according to the respective importance.

{5} For Class A facilities, design should be made to resist the seismic force determined from the dynamic analysis performed on the basis of the basic earthquake ground motion S1. Among the Class A facilities, the particularly important facilities are called Class As facilities. For these facilities, design is performed to ensure the ability to maintain safety function against the seismic force determined from the dynamic analysis performed on the basis of the basic earthquake ground motion S2. Also, dynamic analysis should also be performed for Class B equipment and piping, if there exists the possibility of resonance.

{6} For Class A facilities, in addition to the horizontal seismic force, the vertical motion should also be taken into consideration. The unfavorable direction should be assumed for the vertical motion.

{7} During the process for drafting the structural design and layout plan for a nuclear reactor facility, consideration should be made to reduce the possible effects of earthquake.

5.1.2 Classification of importance in aseismic design

Facilities of aseismic Classes As, A, B and C are as follows according to the functions of the facilities.

Class As Parts, damage of which may cause loss of coolant; parts which are required for emergency shutdown of the nuclear reactor and are needed to maintain the shutdown state of the reactor in a safe state; facility for storage of spent fuel; and nuclear reactor containment.

Class A Parts, which are needed to protect the public from the radioactive hazard in the case of a nuclear reactor accident, and parts, malfunction of which may cause radioactive hazard to the public, but are not classified as Class As.

Class B Parts, which are related to the highly radioactive substance, but are not classified as Class As or Class A.

Class C Facilities, which are related to the radioactive substance but are not classified in the above aseismic classes, and facilities not related to radioactive safety.

Table 5.1.2-1 lists definitions of aseismic importance and facilities of the various classes. Based on their functions, nuclear reactor facilities can be divided into the following groups: primary equipment, auxiliary equipment, direct support structures, indirect support structures, and equipment for which the interaction of equipment must be taken into consideration. Table 5.1.2-1 lists the facilities of various classes as the primary equipment and indirect support structures for building structures.

The primary equipment refers to the system equipment that is related directly to safety functions. For aseismic Class As facilities of the primary equipment, design should be performed with respect to the seismic force determined by basic earthquake ground motions S1 and S2. For aseismic Class A facilities, design should be performed with respect to the seismic force determined by basic earthquake ground motion S1.

The direct support structures refer to the support structures on which the primary equipment and auxiliary equipment are directly mounted, as well as the support structures which directly bear the loads of this equipment. On the other hand, the reinforced concrete structures, steel structures, etc. (building structures) which bear the loads transmitted from the direct support structures are called indirect support structures.

For the indirect support structures, although they do not have safety functions themselves, they are required not to hamper the safety functions of the equipment supported by them. Hence, although there is no definition for the aseismic importance from the viewpoint of safety functions, it is nevertheless required to confirm that there is no safety problem with respect to the earthquake motion appropriate to the equipment being supported.

A detailed description on the aseismic importance is presented in "Technical guidelines of aseismic designs of nuclear power plants: Importance classification, allowable stresses, JEAG 4601, HO-1984" (referred to as "JEAG 4601, HO-1984" hereinafter); the said guidelines should be used over similar guidelines.

5.1.3 Methods of calculating seismic force

The design seismic force of Class As and Class A facilities is the static or dynamic seismic force, whichever is the larger, determined using the following calculation methods. For design of Class B and Class C facilities, the static seismic force is applied.

(1) Static seismic force

Depending on the importance classification of the reactor facility, the horizontal seismic force is calculated by multiplying the following story shear coefficients with the weight above the story concerned.

Class As and Class A	story shear coefficient	$3.0\ C_I$
Class B	story shear coefficient	$1.5\ C_I$
Class C	story shear coefficient	$1.0\ C_I$

Here, the value of the story shear coefficient C_I is determined by taking 0.2 as the basic story shear coefficient and by taking the dynamic characteristics of the structure, type of ground, etc., into consideration.

Table 5.1.2-1(a). Definitions of aseismic importance and facilities of various classes.[1]

Classification and definition of aseismic importance	Classification of functions	Primary equipment Application range	Indirect support structures Application range	Earthquake motion to be used
Class As: Parts, damage of which may cause loss of coolant; parts which are required for emergency shutdown of the nuclear reactor and are needed to maintain the shutdown state of the reactor in a safe state; facility for storage of spent fuel; and nuclear reactor containment	(i) Piping and equipment that form the "pressure boundary of nuclear reactor coolant" (as defined in "Guidelines of safety design in evaluation of light water reactor facilities for power generation")	{1} Pressure containment of nuclear reactor (B) {2} Containment of nuclear reactor (P) {3} Containments, piping, pumps, and valves belonging to the pressure boundary of nuclear reactor coolant	(1) Reactor building (2) Control building (3) Pedestal of pressure containment of nuclear reactor (B) (4) Internal concrete (P) (5) Auxiliary building (P)	S_2
	(ii) Equipment for storage of spent fuel	{1} Spent fuel pool (B) {2} Spent fuel storage rack (B) {3} Spent fuel pit (P) {4} Spent fuel rack (P)	(1) Reactor building (2) Auxiliary building (P) (3) Fuel handling building (P)	S_2
	(iii) Equipment used for applying rapid negative reactivity for emergency shutdown of nuclear reactor, and equipment for maintaining the shutdown state of the nuclear reactor	{1} Control rods, control rod driving unit, and control rod driving hydraulic system (the portion related to the scram function) (B) {2} Control rod cluster and control rod driving unit (the portion related to the scram function) (P) {3} Boric acid injecting unit (transfer system) (P)	(1) Reactor building (2) Internal concrete (P) (3) Auxiliary building (P) (4) Control building (P) (5) Diesel building (P)	S_2
	(iv) Equipment for removal of decay heat from the reactor core after shutdown of the nuclear reactor	{1} Cooling system for isolating nuclear reactor (B) {2} High-pressure reactor core spray system (B) {3} Residual heat removal system (equipment required for cooling mode operation in shutdown state) (B) {4} Suppression pool as cooling water source (B)	(1) Reactor building (2) Control building (3) Foundation of seawater pump, and other structures for supporting the seawater system (for emergency cases) (B) (4) Internal concrete (P) (5) Auxiliary building (P) (6) Diesel building (P) (7) Foundation of seawater pump and other structures for supporting the seawater system (P)	S_2

Table 5.1.2-1 *(Cont'd)*. Definitions of aseismic importance and facilities of various classes [1]

Classification and definition of aseismic importance	Classification of functions	Primary equipment	Indirect support structures	
		Application range	Application range	Earthquake motion to be used
		{5} Main steam feedwater system (from primary feedwater check valve, through secondary side of steam generator, to main steam isolating valve) (P) {6} Auxiliary feedwater system (P) {7} Condensate water tank (P) {8} Residual heat removal system (P)		
	(v) Equipment which becomes a pressure barrier for preventing direct discharge of radioactive substances in case of accidental rupture of the coolant pressure boundary of the nuclear reactor	{1} Containment structure[2] {2} Piping and valves belonging to the containment boundary of the nuclear reactor	(1) Reactor building (2) Auxiliary building (P)	S_2
			(1) Control building (2) Diesel building (P) (3) Reactor building	S_1
Class A: Parts, which are needed to protect the public from the radioactive hazard in the case of a nuclear reactor accident, and parts, malfunction of which may cause radioactive hazard to the public, but are not classified as Class As	(i) Equipment required for removing decay heat from reactor core after accidental rupture of the coolant pressure boundary of the nuclear reactor	{1} Emergency core cooling system (B) 1) High-pressure core spray system 2) Low-pressure core spray system 3) Residual heat removal system (equipment required for operation in the low-pressure core water injection mode) 4) Automatic pressure relief system {2} Suppression pool as cooling water source (B) {3} Safety injection system (P) {4} Emergency core cooling system (ECCS) (P) {5} Water tank for exchange of fuel (P)	(1) Reactor building (2) Control building (3) Foundation of seawater pump and other structures for supporting the seawater system (for emergency use) (B) (4) Auxiliary building (P) (5) Diesel building (P) (6) Foundation of seawater pump and other structures for supporting the seawater system (P)	S_1

Table 5.1.2-1 *(Cont'd)*. Definitions of aseismic importance and facilities of various classes.[1]

Classification and definition of aseismic importance	Classification of functions	Primary equipment	Indirect support structures	
		Application range	Application range	Earthquake motion to be used
	(ii) Equipment, not included in aseismic Class As (V), for preventing release of radioactive substances to the outside in an accident accompanied by leakage of radioactive substances	{1} Residual heat removal system (equipment required for cooling containment and for operation in spray mode) (B) {2} Reactor building (B) {3} Combustible gas concentration control system (B) {4} Emergency gas treatment system and exhaust port (B) {5} Nuclear reactor containment pressure suppressing equipment (diaphragm floor, vent pipe) (B) {6} Main steam separating valve leakage control system (B) {7} Suppression pool as cooling water source (B) {8} Containment spray system (P) {9} Water tank for replacement of fuel (P) {10} Annulus seal (P) {11} Annulus air cleaner (P) {12} Containment gas exhaust pipe (P) {13} HVAC for auxiliary safety equipment room (P) (including engineering safety facilities)	(1) Reactor building (2) Control building (3) Foundation of seawater pump and other structures for supporting seawater system (for emergency use) (B) (4) Primary exhaust pipe (B) (In case of support of exhaust port of emergency gas treatment system) (5) Auxiliary building (P) (6) Reactor containment vessel (P) (7) External shield (P) (8) Diesel building (P) (9) Foundation of seawater pump and other structures supporting the seawater system (P)	S_1
	(iii) Others	{1} Fuel pool water feed equipment (for emergency use) (B) {2} Boric acid solution injecting system (B) {3} Spent fuel pool feed equipment (for emergency use) (P)	(1) Reactor building (2) Auxiliary building (P) (3) Fuel handling building (P)	S_1
		Internal structures of reactor	(1) Reactor pressure containment pedestal (B) (2) Reactor building (B)	S_2

Table 5.1.2-1(b). Definitions of aseismic importance and facilities of various classes.[1]

Classification and definition of aseismic importance	Classification of functions	Primary equipment	Indirect support structures	
		Application range	Application range	Earthquake motion to be used
Class B: Parts, which are related to the highly radioactive substance, but are not classified as Class As and Class A	(i) Equipment that contains or can contain primary coolant in direct contact with coolant pressure boundary of nuclear reactor	Main steam system (from outside main steam isolation valve to turbine primary blockage value) (B)[4]	(1) Reactor building (2) Turbine building (portion for supporting the piping and valves from outside main steam isolation valve to primary blockage valve) (B)	S_1
		(1) Main steam system and feedwater system (B) (2) Reactor coolant purification system (B) (3) Extraction system and residue extraction in chemical volume control system (P)	(1) Reactor building (2) Turbine building (B) (3) Auxiliary building (P) (4) Internal concrete (P)	S_B[5]
	(ii) Equipment for containing radioactive waste, excluding those which have a small content or a special storage method, therefore possess a smaller radioactive effect to the public in case of rupture than the annual exposure dose allowable outside the peripheral monitoring region	Equipment for processing wastes, excluding that belonging to Class C	(1) Waste treatment building (2) Reactor building (P) (3) Auxiliary building (P)	S_B
	(iii) Equipment which is related to radioactive substances other than the radioactive waste, and the rupture of which may cause an excessive radioactive exposure to the public and employees	(1) Shields with significant effect in reducing the radiation level (2) Steam turbine, condenser, feedwater heater, and major piping (B) (3) Condensing/desalting equipment (B) (4) Condensate storage tank (B) (5) Fuel pool purifying system (B) (6) Control rod drive hydraulic system (the portion containing radioactive fluid) (B)	(1) Reactor building (2) Turbine building (B) (3) Turbine pedestal (B) (4) Internal concrete (P) (5) Auxiliary building (P)	S_B

Table 5.1.2-1 *(Cont'd)*. Definition of aseismic importance and facilities of various classes.[1]

Classification and definition of aseismic importance	Classification of functions	Primary equipment	Indirect support structures	
		Application range	Application range	Earthquake motion to be used
		(7) Reactor building crane (B) (8) Fuel handling equipment (B) (9) Control rod storage rack (B) (10) Spent fuel pool purifying system (P) (11) Parts other than Class C in the chemical volume control system (P) (12) Auxiliary building crane (P) (13) Spent fuel pool crane (P) (14) Fuel exchange crane (P) (15) Fuel transfer equipment (P)		
	(iv) Equipment for cooling spent fuel	(1) Fuel pool cooling system (B) (2) Spent fuel pool-cooling system (P)	1 Reactor building 2 Auxiliary building (P) 3 Fuel handling building (P) 4 Foundation of seawater pump and other structures supporting the seawater system (P)	S_B
	(v) Equipment which do not belong to aseismic Class As and Class A, and is used to suppress dissipation of radioactive substances to the outside when the radioactive substances are released			S_B

[1]Courtesy "JEAG 4601, Supplement-1984", with the contents reorganized in this table.

[2]In principle, there is no need to perform evaluation using basic earthquake ground motion S2. However, as it is the final barrier for preventing dissipation of the radioactive substance, only the reactor containment boundary is taken as aseismic Class As. For the isolating value, the requirement is that it should maintain as isolated state after basic earthquake ground motion S2 takes place.

[3]The CAD scheme is also included.

[4]Although it belongs to aseismic Class B, analysis should be performed to ensure no failure after basic earthquake ground motion S1.

[5]S_B is the seismic input to be applied for aseismic Class B equipment.

*Others (B): BWR; (P): PWR; no mark: BWR, PWR common.

For Class A facilities, the vertical seismic force is also considered, and it is assumed that the horizontal seismic force and the vertical seismic force ... in combination at the same time in unfavorable direction. The vertical seismic force is calculated by takin, 0.3 as the basic seismic coefficient and is determined by considering the vibrational characteristics of building structures, type of foundation, etc. The vertical seismic coefficient is assumed to be constant in the height direction. In addition, for a building or structure, its horizontal strength capacity should be confirmed to have an appropriate safety margin, corresponding to its importance, with respect to the required horizontal strength capacity.

The static seismic force calculation method is defined in "Evaluation guidelines," which are based on the Standard Building Code regarding the seismic strength calculations of structures. In addition, in recent years, investigations have been conducted on the schemes for specifically applying the "Evaluation guidelines" for the nuclear reactor facilities in nuclear power plants. These schemes may be briefly outlined as follows:

a. For dynamic analysis of a reactor building, which is significantly different from conventional buildings with respect to weight and stiffness distribution, the fundamental period (T) of the building is determined by the eigenvalue analysis and is taken as the reference value; the story shear distribution coefficients (Ai) are derived using modal analysis.

b. For the dynamic characteristic coefficients (R_t), based on the results of special investigation or research, they can be modified within the range no less than 0.7.

c. As the static seismic force is calculated, the ground surface is taken as the reference surface in principle; the horizontal seismic in ity of the underground portion can be calculated by performing corrections corresponding to the transverse wave speed of the bedrock and the embedment depth. For the embedment effect on the horizontal seismic intensity of the underground portion, when it is proved to be appropriate by reference surveys, analysis, etc., the horizontal seismic intensity determined by analysis may be used.

(2) Dynamic seismic force

The horizontal seismic forces caused by the maximum design earthquake and the extreme design earthquake (referred to as "dynamic horizontal seismic forces" hereinafter) are calculated by dynamic analysis from basic earthquake ground motions S1 and S2, respectively. For Class As and Class A facilities, the dynamic horizontal earthquake force is calculated from the basic earthquake ground motion S1. In addition, for Class As facilities, it is required that their safety function be maintained against the seismic force due to basic earthquake ground motion S2. The vertical seismic force in combination with the said dynamic horizontal seismic force is determined by taking half the value of the maximum acceleration amplitude of the basic earthquake ground motion as the vertical seismic coefficient. Here, the vertical seismic coefficient is taken as a constant static seismic force in the height direction.

Among Class B equipment and piping, for equipment which might resonate with the vibration of the support structures, a dynamic analysis for the structure is performed by taking half the amplitude of the basic earthquake ground motion S1, applicable for said Class A facilities, as the input. The dynamic seismic force calculated in this way is then used for the subsequent analysis of Class B equipment and piping. (See Table 5.1.3-1.)

5.1.4 Load combinations and allowable limits

(1) Load combinations

a. Types of loads

(a) Loads that always act irrespective of the state of the nuclear reactor, such as dead load, live load, earth pressure, water pressure, and load depending on conventional meterological conditions.

Table 5.1.3-1. Seismic forces which should be taken into consideration for buildings in nuclear power plants.

Class	Static seismic force	Dynamic seismic force	
		Basic earthquake ground motion S1	Basic earthquake ground motion S2
As	• Horizontal seismic force; calculated from $3C_I$ • Vertical seismic force; calculated from C_V	• The horizontal seismic force is the seismic force on the building due to basic earthquake ground motion S1 • The vertical seismic force is calculated by taking half of the maximum horizontal acceleration amplitude of the basic earthquake ground motion as the vertical seismic coefficient(1)	• The horizontal seismic force is the seismic force on the building due to basic earthquake ground motion S2 • The vertical seismic force is calculated by taking half of the maximum horizontal acceleration amplitude of the basic earthquake ground motion as the vertical seismic coefficient(1)
A	• Horizontal seismic force; calculated from $3C_I$ • Vertical seismic force; calculated from C_V	• The horizontal seismic force is the seismic force on the building due to basic earthquake ground motion S1 • The vertical seismic force is calculated by taking half of the maximum horizontal acceleration amplitude of the basic earthquake ground motion as the vertical seismic coefficient(1)	—
B	• Horizontal seismic force; calculated from $1.5C_I$	Not taken into consideration (investigation is conducted for equipment and piping with the possibility of resonance)	—
C	• Horizontal seismic force; calculated from C_I	—	—

C_I (story shear coefficient): Value determined with 0.2 taken as the basic shear coefficient and by taking the dynamic characteristics of the structure, type of ground, etc., into consideration.

C_V (vertical seismic coefficient): Value determined with 0.3 taken as the basic value, and the dynamic characteristics of the structure, type of ground, etc., are considered.

(1)Both horizontal seismic force and vertical seismic force take place simultaneously combined in unfavorable directions. The vertical seismic force is considered to be constant in the height direction.

(b) Loads that act on the facility depending on the state of operation.

State of operation: The operation state of the nuclear reactor under the conventional normal, natural conditions. The operation state includes the conventional operation state and the abnormal transient periods of the operation.

(c) Loads that act on the facilities in the state of accident.

State of accident: The state when the nuclear reactor facility is in an accident.

(d) Seismic force, wind load, snow load.

The loads during operation and during accident includes the loads acting from the equipment and piping system; the seismic force includes the loads caused by the earth pressure during an earthquake, the reaction forces from the equipment and piping system, sloshing, etc. The wind load is defined in the Standard Building Code. However, it is not required to combine the seismic force and wind load. In snowy regions, the seismic force should be combined with the snow load.

b. Load combinations

The combinations between seismic force and other loads are as follows.

(a) The seismic force is combined with the loads that always act and the loads that act on the facilities during operation (conventional operation period, abnormal transition period of operation).

(b) The seismic force due to basic earthquake ground motion S1 is combined with the loads that always act and the accident loads that act continuously over a long period of time.

Attention should be paid to the following items with respect to the load combinations:

(i) For Class A facilities, it should be assumed that the horizontal seismic force and the vertical seismic force may act simultaneously in unfavorable directions.
(ii) A certain load combination can be omitted from the consideration if other load combinations obviously cause higher stresses.
(iii) When it becomes obvious that the time points of the peaks of loads acting at the same time do not overlap each other, it is not always required to superimpose the peak stresses.
(iv) To evaluate the supporting function of the structure that supports different aseismic class of facilities, depending on the specific aseismic class of the facility, the seismic force is combined with the loads that always act, loads that act on the facility during operation, and other required loads.

(2) Allowable limits

The allowable limits for the combination states of seismic force and other loads are as follows:

a. Class As structures

(a) Allowable limits for combination with the seismic force due to basic earthquake ground motion S1 or static seismic force.

The allowable stress according to the appropriate regulations and standards is taken as the allowable limit. When combining the loads due to accident, the allowable limit defined by (b) should be applied.

Table 5.1.4-1. Load combinations and allowable limits.

Class	Structure: Load combinations	Structure: Allowable limits	Remarks
As	(3)S1* + Accident loads(1) and (2)S2 + Long-term loads + Loads in operation	Should have safety margin with respect to the ultimate strength capacity	(1) Even for a phenomenon that may not be caused by earthquake, if the phenomenon lasts over the long period of time when an accident takes place, the load due to this phenomenon should be combined. (2) S2 represents the dynamic seismic force based on basic earthquake ground motion S2. (3) S1* represents the dynamic seismic force and static seismic force (for Class A) based on basic earthquake ground motion S1.
As and A	(3)S1* + Long-term loads + Loads in operation	Short-term allowable of Standard Building Code	
B	Static seismic force (for Class B) + Long-term loads + Loads in operation	Same as above	
C	Static seismic force (for Class C) + Long-term loads + Loads in operation	Same as above	

(b) Allowable limits for the combination with the seismic force caused by basic earthquake ground motion S2.

The structure should have a sufficient deformation ability (ductility) as an overall structure, it also should have a safety margin with respect to the ultimate strength capacity.

The ultimate strength capacity refers to the maximum strength limit at which the deformation or strain of a structure increases significantly under monotonically increased loadings. In addition to the available empirical formulas and results of model experiments, if needed, tests are also performed using models in consideration of the structural characteristics of the corresponding portion. On the basis of these results, the ultimate strength capacity is determined appropriately.

b. Class A structures

The allowable stress limits determined according to the above section a(a) are taken as the allowable limits.

c. Class B, Class C structures

The loads that always act and the loads that act on the facilities during operation are combined with the static seismic force. For the stress determined due to this load combinations, the allowable stress limits determined from the regulations and standards believed to be appropriate for safety are taken as the allowable limits.

d. Structures supporting facilities of different aseismic classes

The structure should have a sufficient deformation ability as an overall structure, it also should have a safety margin with respect to the ultimate strength capacity. In addition, for a structure supporting a facility of different aseismic class, the functions of the facility should not be degraded due to the deformation of the supporting structure.

e. Horizontal strength capacity of buildings or structures

The horizontal strength capacity of buildings or structures should be confirmed to have an appropriate safety margin with respect to the required horizontal loads, according to the importance. As explained above, according to the "Evaluation Guidelines," the basic consideration for the allowable limits is such that the "allowable stress limit design" is performed for the S1 earthquake motion, while the ultimate strength capacity with appropriate safety margin is performed for the S2 earthquake motion. In addition, in the third modified standardization plan, the allowable limits for maintaining the functions of structures against earthquake motion are being investigated using the allowable structural limits of the reactor buildings themselves and the allowable limits required to the buildings by the equipment and piping systems. For further details, please see "5.3.4. Investigation of the maintenance of the functions."

5.1.5 Functions of buildings and structures

(1) Structure plan

For a nuclear reactor facility, it is required to have a high enough aseismic safety level so that the loss of its safety function by earthquake must not cause a major accident. For this purpose, when a structure is designed, in addition to the structure itself, the aseismic properties of the various support structures of the equipment and piping, should also be taken into consideration in planning the structure.

For the structure, as pointed out in "5.1.2 Classification of importance of aseismic design," there are requirements on the functions as both a major equipment and an indirect support structure. As an example, for the reactor building of a BWR nuclear power plant, the building compartment of the nuclear reactor is classed as primary facility which is required to have the function of preventing leakage of radioactive substances to the outside in case of an accident accompanied by release of radioactive substances. Therefore, it belongs to class A in the importance classification. However, a portion of the building which contains a spent fuel pool also belongs to class As. In addition, it should have the function of a class As supporting structure of equipment and piping which are classified as class As.

In order to meet the various aforementioned requirements, it is important to carry out the design to satisfy the requirements of function by implementing, in addition to the layout plan, a structural plan that can avoid uplifting and eccentricity of the building.

(2) BWR buildings

BWR buildings can be generally divided into two types: MARK-I and MARK-II. In the following, for each type of building in BWR-type nuclear power plant, an explanation will be given with reference to application examples of the buildings.

a. MARK-I

Figure 5.1.5-1 shows an example of a BWR MARK-I nuclear reactor. The reactor building is a reinforced concrete building (partially made of steel frames) with a nuclear reactor compartment at its center surrounded by attached compartments. The nuclear reactor building and auxiliary building are built on a single foundation mat as an integrated structure. The plan view of this building has a nearly square shape.

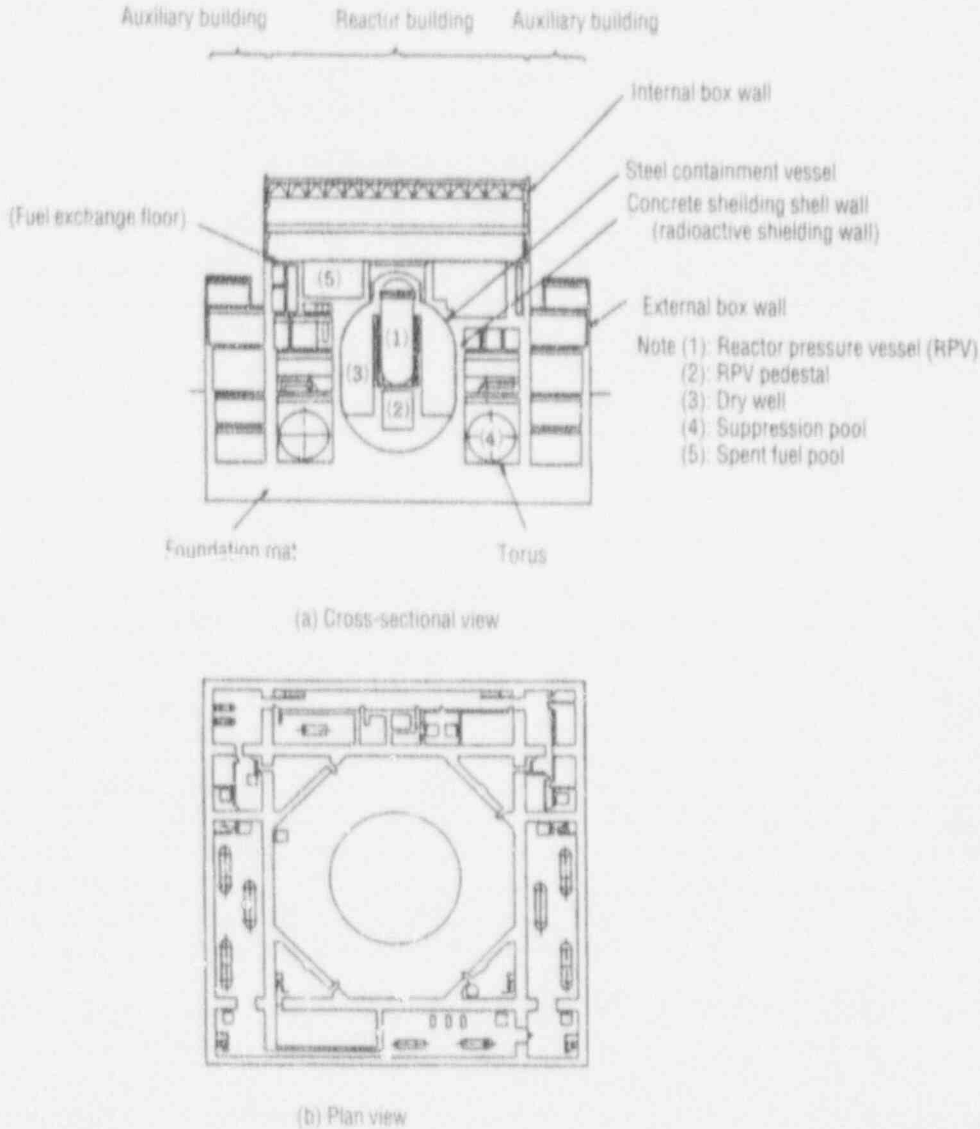


Figure 5.1.5-1. Example of 1.10 million kWe-class BWR MARK-I reactor building.

At the central portion of the reactor building, there is a steel reactor containment vessel which contains the reactor pressure containment, recirculation pump, etc. The reactor containment vessel is made of a dry well consisting of an upper hemisphere and a lower cylinder, as well as a ring-shaped suppression chamber surrounding the dry well. The periphery of the reactor containment vessel is surrounded by a reinforced-concrete radioactive shielding wall. Outside this, there are an internal box wall for dividing the reactor building and the auxiliary building as well as an external box wall that is used as the outer wall of the auxiliary building. These are the major shear walls of the reactor building. As these walls are connected by strong floor panels to form a single body, the structure is very rigid.

b. MARK-II

Figure 5.1.5-2 shows an example of a BWR MARK-II reactor building. Just as in the case of MARK-I, this reactor building is also a reinforced concrete building (partially made of steel frames) consisting of a nuclear reactor building at the central portion and peripheral auxiliary building which are set on a single foundation mat to form an integrated body. The significant difference from the MARK-I in this case is in the reactor containment vessel at the central portion of the nuclear reactor building. The containment vessel is made of an upper conical part and a lower cylindrical part. Around this reactor containment vessel, the radioactive shielding wall as well as the inner box wall and outer box wall form the major shear walls of the reactor building.

(3) PWR buildings

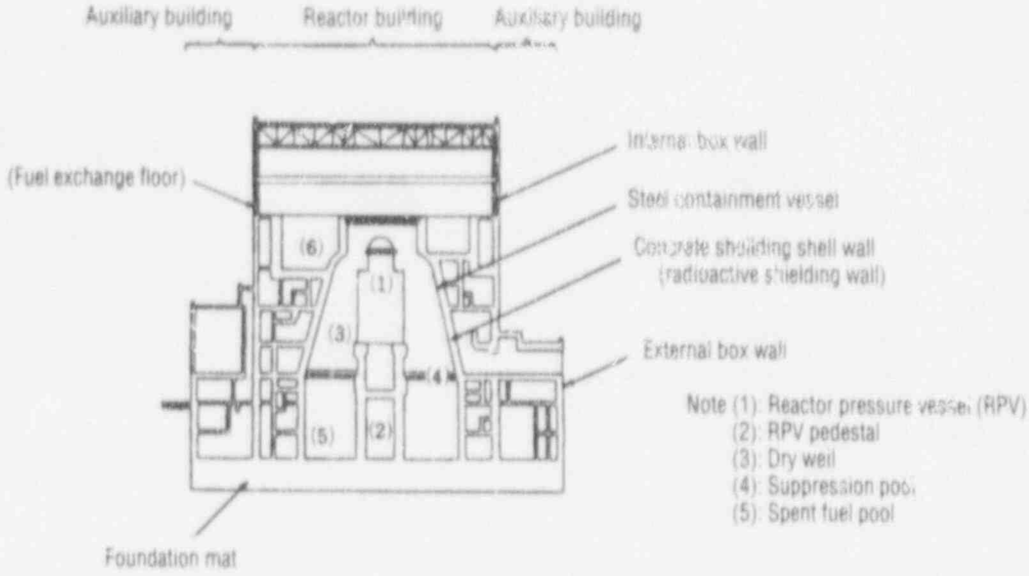
PWR buildings can be generally divided into the following three types: 2-LOOP (0.5 million kWe-class), 3-LOOP (0.8 million kWe-class), and 4-LOOP (1.10 million kWe-class). In the following, we will present several examples of the 3-LOOP and 4-LOOP buildings in some representative nuclear power plants in Japan.

a. 3-LOOP

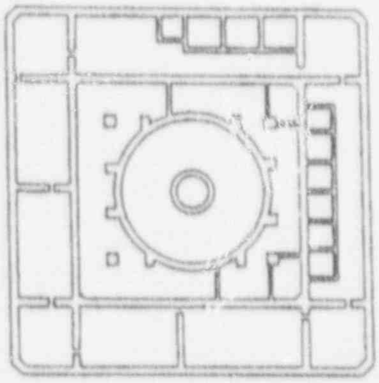
Figure 5.1.5-3 illustrates an example of PWR (3-LOOP) reactor building. The basic shape of the reactor building is designed in consideration of the requirement to improve the aseismic properties, such as building stability during an earthquake. At the center of a rectangular foundation mat, the containment vessel is installed. Around this containment vessel, peripheral buildings are arranged as part of the reactor auxiliary buildings, which are integrated with the external shielding building to form a composite plan configuration. Among the upper structural parts, a steel containment vessel and internal concrete portion are installed on the foundation mat, independently.

The steel containment vessel is a welded structure made of an upper hemispherical portion, a central cylindrical portion, and a lower dishlike portion. The internal concrete [portion] has a reinforced concrete wall structure as its primary body, which is made of primary shielding wall, steam generator chamber, pressurized chamber, and cavity wall. It contains the primary cooling equipment and other primary equipment and is placed in the center of the foundation mat.

The outer shield building is a cylindrical building made of reinforced concrete with a dome attached. A spherical dome is adopted to lower the center of weight so as to improve the aseismic properties. At a level below the operation floor level of the cylindrical portion, the peripheral building has its floor and walls integrated with each other to form a single body. The peripheral building is a 3-floor building with shear walls.

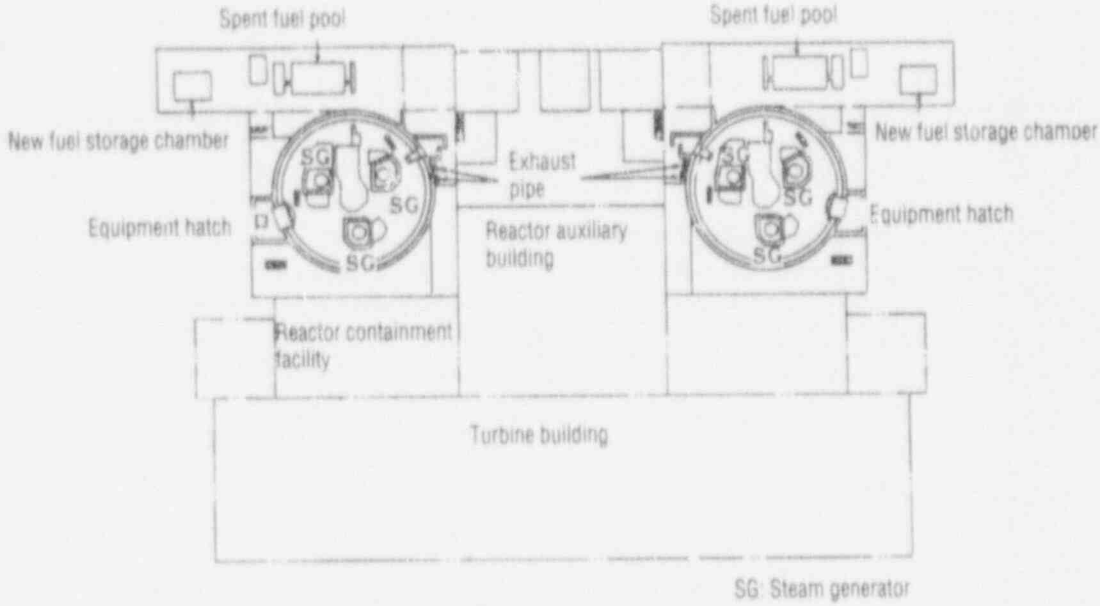


(a) Cross-sectional view

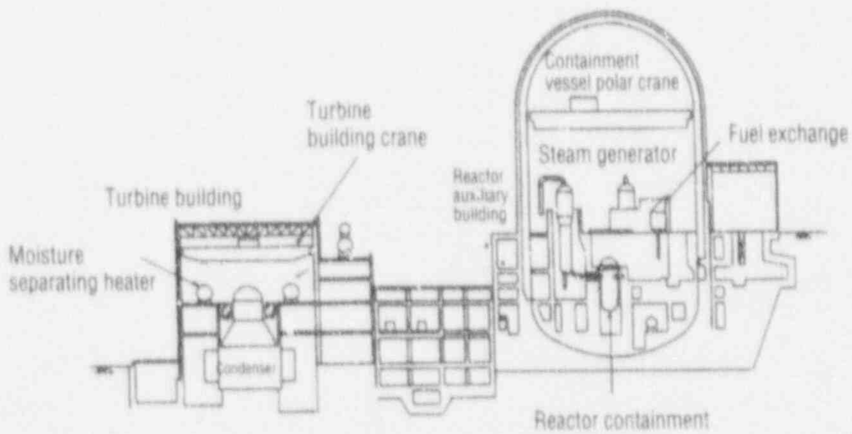


(b) Plan view

Figure 5.1.5-2. Example of 1.10 million kWe-grade BWR MARK-II reactor building.



Plan view of primary building (5-story building)



Building cross-sectional view

Figure 5.1.5-3. Example of a 0.80 million kWe-class PWR (3-LOOP) reactor building.

c. 4-LOOP

Figure 5.1.5-4 shows an example of a PWR (4-LOOP) reactor building. The reactor building is made of reinforced concrete (partially made of steel frames). It consists of a reactor compartment, a fuel handling compartment, and a peripheral auxiliary equipment compartment, which are integrated with each other to form a single body. In consideration of the building stability in an earthquake and other aseismic properties, the basic plan shape of the building is almost square. The reactor compartment is located at the center of the reactor building. It consists of a lower portion made of reinforced concrete and an octagonal upper portion made of steel frames, having the containment vessel contained in it.

The containment vessel is mounted on the same foundation mat made of reinforced concrete as the reactor building. It is made of a prestressed concrete cylindrical portion and a hemispherical dome portion, with a steel liner plate arranged on the inner surface of this pressure-proof containment to prevent leakage. It also plays the role of an external shield. The internal concrete has a reinforced concrete wall structure as its primary body which consists of a primary shielding wall, a steam generator chamber, pressurized chamber, and cavity wall. It contains the primary cooling equipment and other major equipment, and is fixed at the center of the foundation mat.

The fuel applying compartment and the peripheral auxiliary equipment compartment are arranged outside of the reactor compartment. They are mounted on the same foundation mat as the reactor compartment. The peripheral auxiliary equipment compartment is a 2-story building with shear walls arranged.

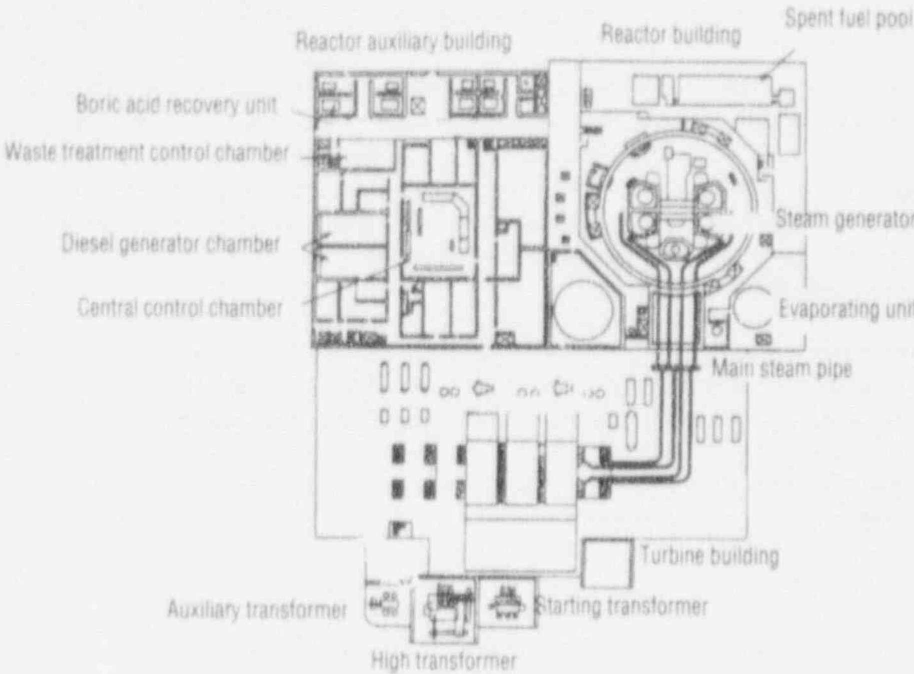
(v) Concrete containment vessel

a. General features

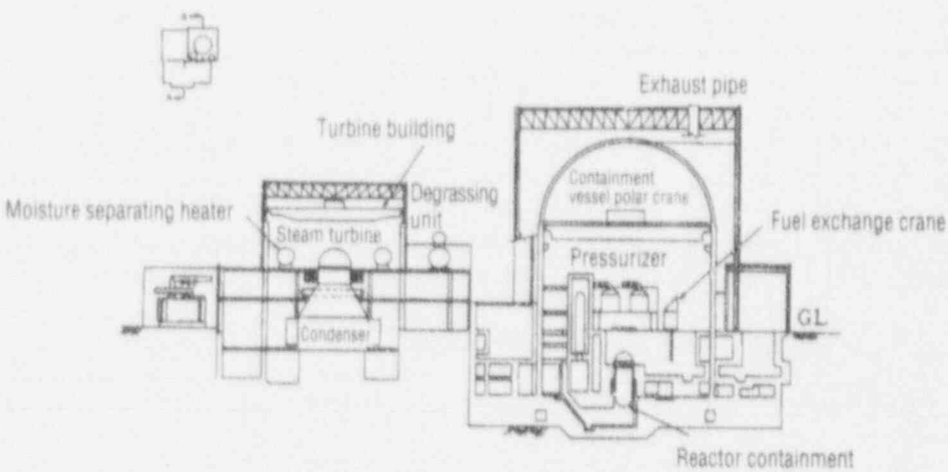
The reactor containment vessel contains the nuclear reactor as well as other equipment and piping. It plays an important role in preventing dissipation of radioactive substances to the exterior by bearing the load caused by the high pressure and temperature in case of loss of coolant accident. In the conventional schemes, in order to meet the requirements of high pressure resistance and leak-proofness, a steel containment vessel is usually adopted. However, recently, as the size of power plants increase, concrete has been used to make the containment vessel, as it is favorable for construction. Compared with steel containment vessels, concrete containment vessels have many advantages: better damping characteristics with respect to dynamic loads, a larger degree of freedom with respect to shape and wall thickness, and hence, ability for appropriate layout and design. From the viewpoint of structure, the concrete containment vessels can be divided into two types: those made of reinforced concrete and those made of prestressed concrete.

b. RCCV (containment vessels made of reinforced concrete)

In the new-type BWR plant (A-BWR) a reinforced concrete containment vessel (RCCV) is adopted as the reactor confinement vessel. For an RCCV, the pressure-proof function is played by the reinforced concrete, while the gas-tight leakage-proof function is played by the steel (liner) arranged on the inner surface of the vessel. For an RCCV, the structural design should be carried out to ensure no degradation in the pressure-proof property and leakage-proof property even under seismic load, LOCA load, hydrau-dynamic load, etc. The reactor building has a composite building form made of reinforced concrete. Usually, the RCCV and the building are structurally connected by the various floor slabs, forming an integrated structure with a common foundation mat. The RCCV has a dry well portion and a suppression portion. Water is stored in the suppression pool and can act as a heat-absorbing source in case of an accident. The dry well portion contains the main steam piping, feedwater piping, and the internal pump as the recirculating pump.



Building Plan View



Cross-sectional View of Major Buildings

Figure 5.1.5-4. Example of 0.80 million kWe-class PWR (4-LOOP) reactor building.

c. PCCV (prestressed concrete containment vessel)

In order to resist the high tensile stress caused by the internal pressure, the PCCV has its dome portion and cylindrical portion made of prestressed concrete with a prestressing force applied on the reinforced concrete beforehand, and has its base foundation mat made of reinforced concrete. A steel liner plate is applied on the inner surface of these parts. The rebars are arranged appropriately to resist the flexural stress and in-plane shear determined in the cross section. The structure is sufficiently safe against any anticipated load conditions in addition to accidents caused by loss of coolant. In this way, it can effectively prevent dissipation of radioactive substances to the exterior. Figure 5.1.5-5 shows an example of a PCCV, with the prestressed concrete portion made of a dome portion and a cylindrical portion. It has its own shielding function. In addition, in this example, leakage prevention is played by a steel liner plate arranged on the inner surface of the concrete wall. The prestress force applied on the concrete portion is obtained by pulling a tendon set made of 163 pieces of 7-mm-diameter PC steel wire to a prescribed initial force. The tendon set consists of inverse-U-shaped vertical tendons which are arranged in a good shape as projected from the upper portion of the dome, with their two ends anchored on the gallery located in the foundation mat, and horizontal tendons in 240°-hoop shape anchored on buttresses, with 3 pieces of buttresses arranged with an azimuthal angle interval of 120°.

(5) Other structures

In addition to reactor buildings and turbine buildings, there are other buildings. The building classification and the content of equipment contained in it depend on the specific plant. Tables 5.1.5-1, -2 and Figures 5.1.5-6, -7 explain the functions and structures of the other structures with reference to practical examples of BWRs and PWRs.

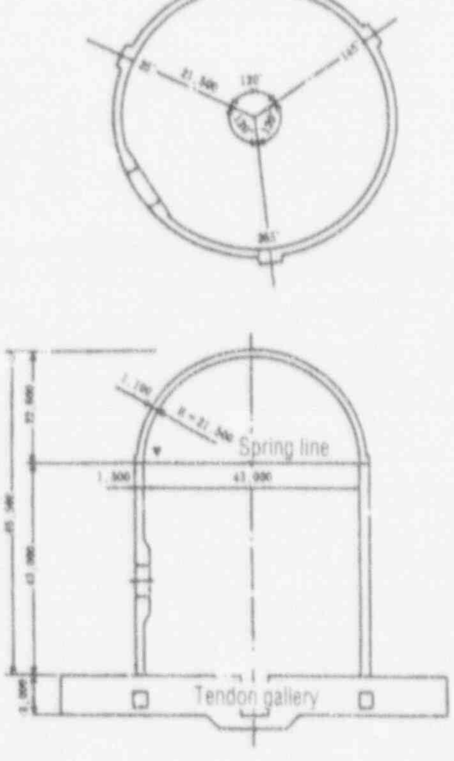


Figure 5.1.5-5. PCCV (prestressed concrete containment vessel).

Table 5.1.5-1. Example of a BWR.

Structure	Major structure	Functions
Turbine building	Reinforced concrete; steel structure and steel encased reinforced concrete structure	It is used to contain various equipment, such as the turbine generator, condenser, feedwater heater, reactor feedwater pump, etc.
Control building	Reinforced concrete; steel frame	The uppermost floor is the central control room; the lower floors are cable processing chamber and switch gear chamber
Service building	Reinforced concrete	It contains an access control facility, locker room, shower room, health care room, chemical analysis room, etc.
Waste disposal building	Reinforced concrete; steel frame	It primarily contains the facility for processing and storing liquid and solid waste from the reactor building and in the exhaust pipe of the solid waste incinerating equipment
Solid waste storage room	Reinforced concrete	For storage of solid waste packed in drum containments
Primary exhaust pipe	Steel frame; reinforced concrete (foundation)	For releasing exhaust from the air exchange and conditioning system in the reactor building, turbine building, waste processing building, as well as gases from the emergency gas processing system

Also built on the power plant site are the water processing building, water intake equipment, water releasing equipment, activated carbon rare gas holdup equipment building, seawater heat exchanger building, office building, etc.

Table 5.1.5-2. Example of a PWR.

Structure	Primary structure	Functions
Reactor auxiliary building	Reinforced concrete; steel frame	It is adjacent to the reactor containment facility and is used to contain the following equipment: chemical volume control equipment, residual heat removal equipment, waste processing equipment, fuel exchange water equipment, fuel applying equipment, fuel storage equipment, air exchange and conditioning equipment, sampling equipment, reactor auxiliary equipment cooling water equipment, emergency power source equipment, central control chamber, etc.
Turbine building	Steel frame and composite steel—R.C. structure	For containing generator, condenser, feedwater heater, feedwater pump, auxiliary equipment, etc.

In addition to the said buildings, also built on the power plant site are the solid waste storage room, water intake facility, auxiliary steam equipment, outlet facility, water releasing equipment, office building, etc.

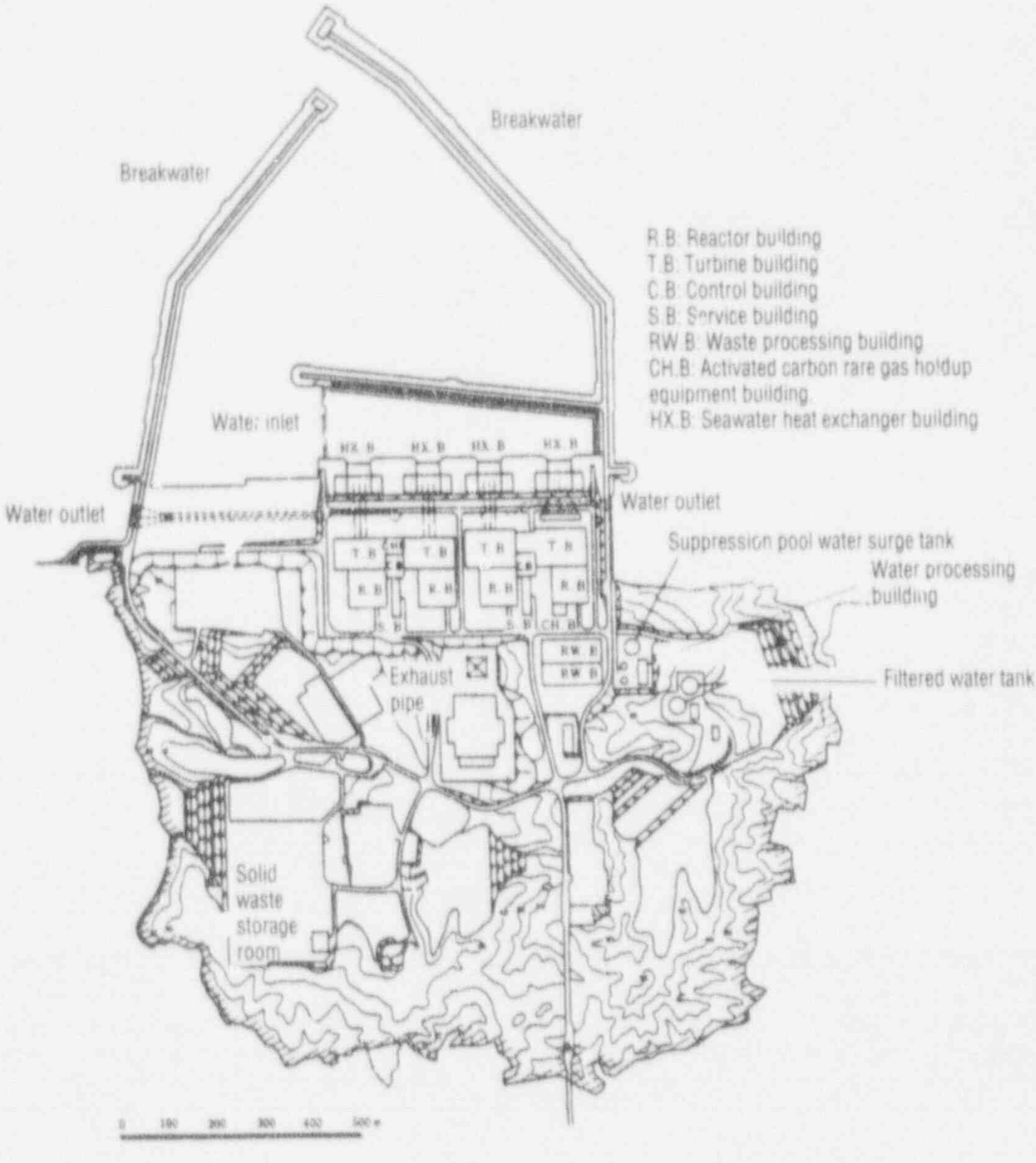


Figure 5.1.5-6. Overall layout plan of BWR power plant.

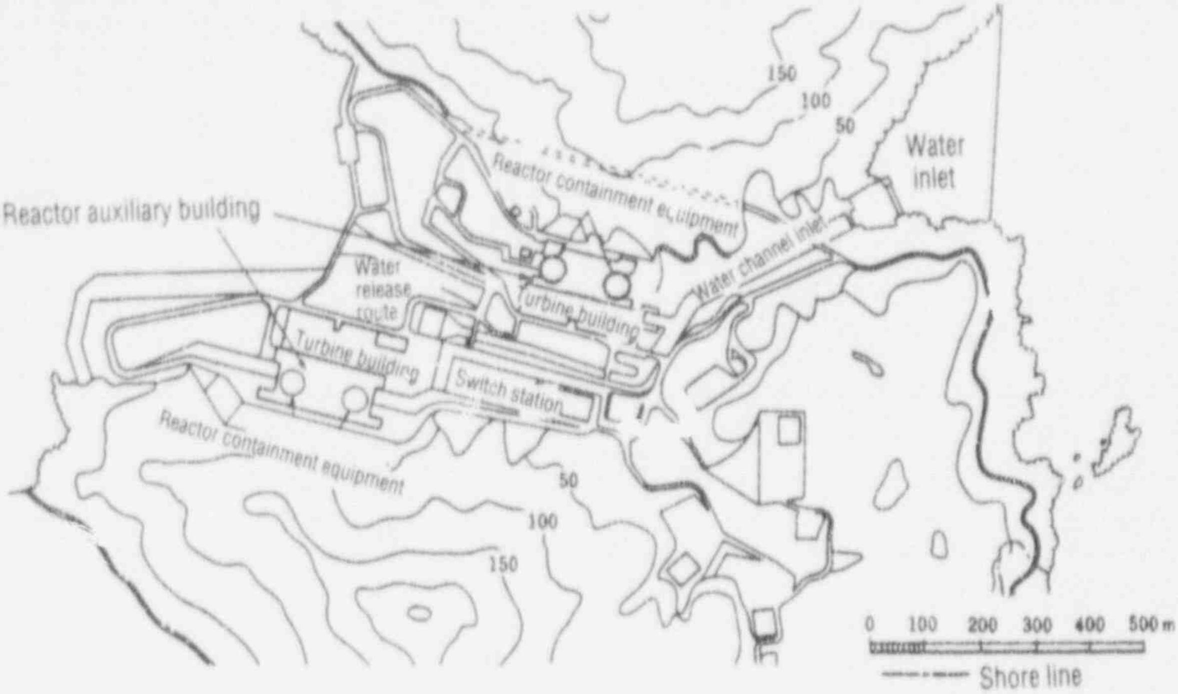


Figure 5.1.5-7. Overall layout plan of PWR power plant.

5.2 Earthquake response analysis

5.2.1 General

In this section, we will primarily discuss the earthquake response analysis of the reactor facilities. Due to the nature of operation, there is a high requirement on the safety of reactor facilities; as a result, it is required to perform earthquake response analysis carefully.

The reactor building is a typical rigid (short period) structure. Also, it is a composite structure consisting of various structural forms and materials, such as reinforced concrete, steel, steel-frame-reinforced concrete, prestressed concrete, etc. It is most important in the earthquake response analysis of this type of complicated structure to set up a dynamic model that can correctly predict the dynamic characteristics of the various structures based on the results of existing research and experimentation, and to evaluate the required building response by using appropriate analytical methods. In the following, we will describe schematically the process of earthquake response analysis from the input of an earthquake motion model to building response results.

(1) Input earthquake motion

In earthquake response analysis, the basic earthquake ground motion is the most important factor in determining the response results. In addition, the way to apply the basic earthquake ground motion at the free surface of the bedrock also affects aseismic design very importantly. When a reactor building is set on a so-called bedrock outcrop surface, and it is considered that embedment can be neglected, usually the basic earthquake ground motion is taken as the direct input earthquake motion. However, in other cases, depending on the terrain and geology of the site, embedment depth of the building, etc., the input earthquake motion to the building-soil interaction model is derived by performing a soil response analysis of the free field ground, using the methods such as a one-dimensional wave propergation theory.

(2) Soil-structure interaction

Usually, for a reactor building, as it has a rigid structure, the effect of interaction with the ground during an earthquake is significant. In particular, when the depth of embedment is great, the response of the building is affected by the embedment; hence, when a dynamic model is formed, it is important to make an appropriate model of the embedment. The ground is usually modeled by replacing the ground below the foundation mat with equivalent horizontal and rotational springs to form a sway/rocking model. In some cases, to account for the embedment, side sway springs are added to the model.

When the influences of the embedment of the building, back-fill soil, and peripheral foundation are investigated separately, or when adjacent buildings are taken into consideration, the foundation may be represented by a finite element method (FEM) model or a discrete-mass parallel model. Recently, for an analysis of the earthquake response, there is a case to use the substructuring method combining a boundary element method (BEM) and the FEM. The BEM is adopted to analize the semi-infinite soil, which is rational method for 3-dimensional problem and boundary teatment. The FEM is adopted to analize the peripheral soil of the building such as back-fill soil.

For the purpose of modeling, in many cases, the results of in-situ tests and indoor tests are used for the data on the ground properties used for evaluation of dynamic stiffness and damping. When a sway/rocking model is used to express the effects of the ground in terms of horizontal and rotational deformations, from these data, with the foundation assumed to be a homogeneous elastic body, analysis is performed by using the ground compliance and vibration admittance theory, etc. Also, for each spring, the effect of radiation damping is taken into consideration. As a result, each spring can be expressed in a complex form (complex stiffness) as a function of vibrational frequency.

When the ground is handled with the aid of a FEM model or a discrete-mass parallel model, it is possible to use different elastic constants for different ground layers. In some cases, in order to express the dissipation effect on the bottom and side boundaries of the model, a viscous damper may also be used.

(3) Superstructure model

The scheme for dividing the various parts of the reactor building into various vibration systems is related to the structural design, and it is important for forming reliable models in consideration of the overall flow of the design. In many cases, the so-called bending/shear type lumped-mass model with the mass concentrated on the floor position is used, such as the bending-shear type model in which the various structural elements stand on the foundation mat, or as the single-cantilever model with all the structural element assembled in. Also, when the stiffness evaluation is performed, in order to consider in detail the 3-dimensional effects due to orthogonal walls for various walls, openings of different sizes, etc., evaluation, in some cases, is done by FEM, etc. Further, in forming a model of the building, it is required to consider the structural elements which is important to the equipment and piping designs.

In forming models, it is important to evaluate the stiffness, damping, and other properties of the structure. The stiffness is evaluated by the various regulations of the Architectural Institute of Japan. As far as the damping is concerned, the conventionally used values (damping constants in Table 5.2.2-5) based on the existing data of vibration tests and earthquake observation are used, and the vibration equations are treated in consideration of the internal viscous damping, modal damping of each mode, strain energy proportional damping, complex stiffness, etc.

(4) Restoring force characteristics of shear walls and nonlinear uplifting characteristics of foundation mat

The major structural elements of a reactor building are box-shaped or cylindrical shear walls. For these shear walls, the restoring force characteristic curves are determined based on structural tests using many test specimens. Usually, the model of the shear wall is formed by the foregoing cantilever model with flexural and shear deformations, with the respective skeleton curve bilinearly or trilinearly idealized. To describe the hysteresis curve, for the flexural deformation, the so-called maximum point orientation type, degrading type, or other models are assumed; for the shear deformation, the origin orientation type; maximum point orientation type, or other models are assumed. In addition, when the base portion of the reactor building is subjected to a large overturning moment, a portion of the foundation mat is assumed to uplift from the ground, and geometric nonlinearity is considered for the rocking springs, which may be idealized by a trilinear model.

(5) Numerical analysis method for vibration equations

The methods for solving linear vibration equations include the spectral modal analysis method, time history modal analysis method, direct integration method, frequency response analysis method, etc. According to the spectral modal analysis method, the eigenvalues and natural vibration modes of the vibration system are derived; from the spectra of the input earthquake motion, the maximum response value of each mode is derived; then, the required number of modes is selected, and the response of the vibration system is determined using the Square Root of the Sum of the Squares method (referred to as the "SRSS" method hereinafter). According to the time history modal analysis method, for each mode, the response of the SDOF system is obtained using the Duhamel integral, etc., and the required modes are synthesized together. According to the direct integration method, the vibration equation is directly derived using the Newmark-β method, etc. According to the frequency response analysis method, the frequency response is determined in order to utilize frequency dependent stiffness and damping, followed by transformation to the time domain.

On the other hand, in nonlinear earthquake response analysis, the incremental-type equilibrium equation is an[illegible]d by successive steps with a short time increase for each step of analysis. However, as the stiffness and damping matrices change all the time, sophisticated techniques are required to treat the unbalanced forces and to calculate for convergence in the nonlinear region. The analytical methods include the Newmark-β method, Wilson's θ method, and other numerical integration schemes. Anyway, for both the linear and nonlinear analysis, it is important to study the time step interval, accuracy of solution, and numerical stability.

(6) Building response results

As explained above, by implementing earthquake response analysis, the response in acceleration, velocity, displacement, etc., of the building can be obtained. From these parameters, the shear forces and bending moments can be determined. At present, the cross sectional design of buildings is carried out primarily in terms of the shear and bending moment. However, when the functionality of structures is evaluated, as will be explained in "5.3.4(2): Consideration of allowable limits," the energy absorbing capacity of structures may also be considered as an allowable measure.

In order to study the building stability, the earth pressure on the foundation mat is determined from the vertical force and the overturning moment. In addition, investigation is performed on sliding, etc., caused by the uplifting force and shear. In order to design equipment and piping, the time history of the response acceleration of floors and other required parts is calculated. On the base of this calculation, the floor response spectrum can be calculated with the damping constants of the equipment and piping as the parameters. The aforementioned input earthquake motion, ground properties, soil spring evaluation method, building stiffness, etc., are believed to be the variation factors for the floor response spectra. For design of equipment described in Chapter 6, it is the basic requirement to use a $\pm 10\%$ broadened floor response spectrum.

5.2.2 Evaluation of properties of ground and structures

(1) Properties of ground

The dynamic analysis of a structure should be performed by using an analytical method that can reflect the status of the ground and the structural characteristics of the building. As a result, for the values of properties used in these analyses, i.e., elastic coefficient and damping constant, it is required to perform evaluation according to the specific purpose. The test methods for determining the properties of the ground include static and dynamic test methods, in situ test method, and indoor tests using samples. In the following, various geotechnical and geophysical test methods are evaluated and compared with each other in terms of the applicability to dynamic analysis, which should reflect the actual soil behavior observed in real earthquakes and simulated dynamic tests.

As the major points of this section concern the properties of the ground, with respect to the damping, the internal material damping should be primarily discussed. However, in the case of soil damping in the dynamic analysis, usually both the internal damping and the damping caused by the dynamic interaction between soil and structure need to be considered. In the following, we will present a description of the two types of dampings.

Figure 5.2.2-1 presents a schematic diagram of the content to be described here. For the elasticity parameters of the ground, it primarily shows the evaluation method of the shear wave velocity (S wave velocity). For the damping, it shows the evaluation methods for the internal damping and the dissipation damping due to dynamic interaction, separately. As far as the scheme for applying these foundation properties in the dynamic analysis is concerned, sections 5.2.3(1)b and 5.2.3(2)a will discuss the ground spring used in the sway/rocking model and the evaluation of its damping; sections 5.2.3(1)c and 5.2.3(2)b will discuss the discrete system model.

a. Elastic coefficient

(a) Test method for deriving elastic coefficient of foundation for dynamic analysis

In many cases, the results of elastic coefficients obtained for the same ground by using different test methods are different. Figure 5.2.2-2 shows the ratios of the Young's modulus of the ground obtained by using various test methods to the Young's modulus obtained from the elastic wave detection test. It can be seen from this figure that there exists significant difference in the Young's modulus for different test methods. The reasons of the difference are believed to be related to difference in strain level, difference in test conditions (in situ test vs. indoor test, static test vs. dynamic test), etc.

Figure 5.2.2-3 is a schematic diagram illustrating the difference in the measurement strain level for various test methods. It can be seen that a difference about 10^4 exists for the strain level by different test methods. Figure 5.2.2-4 compares the in situ test results (abscissa) vs. the indoor test results (ordinate). For the in situ test, it is required to check the existence of cracks in the ground and to evaluate the elastic coefficients. Figure 5.2.2-5 compares the elastic coefficient obtained from flat plate load test and that obtained from elastic wave test. It can be seen that the ratio of the static elastic coefficient by the flat plate load test to the elastic coefficient by the elastic wave test decreases as the stiffness of the rock increases.

On the other hand, for the elastic coefficient of the ground used for dynamic analysis, the elastic wave test is considered as the most appropriate evaluation of the dynamic characteristics of the ground subjected to an earthquake. Figure 5.2.2-6 shows the relationship between the equivalent shear wave velocity ($\tilde{V}s$) for the ground evaluated from the vibration test results of the block foundation and reactor building, and the shear wave velocity (Vs) obtained from the elastic wave test. Here, the elastic wave test refers to PS logging, elastic wave in adit, and elastic wave between adits. The average value of the ratio of $\tilde{V}s$ to Vs is in the range of 0.95-1.06. This indicates a good correspondence between the two velocities. Figure 5.2.2-7 shows the relation between the fundamental period (T_{OBS}) obtained from analysis on the earthquake observation records of the ground and the fundamental period (T_{COM}) obtained from analysis on the test of elastic wave. The average value of the ratio of T_{OBS} to T_{COM} is about 0.97, i.e., they are nearly the same. In addition, as shown in Figure 5.2.2-3, the strain level of the elastic wave test is below 10^{-4}; as shown in Figure 5.2.2-14, the strain level of the design input acceleration is in the order of 10^{-4}. Hence, the conclusion is that the elastic wave test, which is similar to the wave transfer phenomenon during an earthquake, is the most suitable test method for determining the elastic coefficient for dynamic analysis.

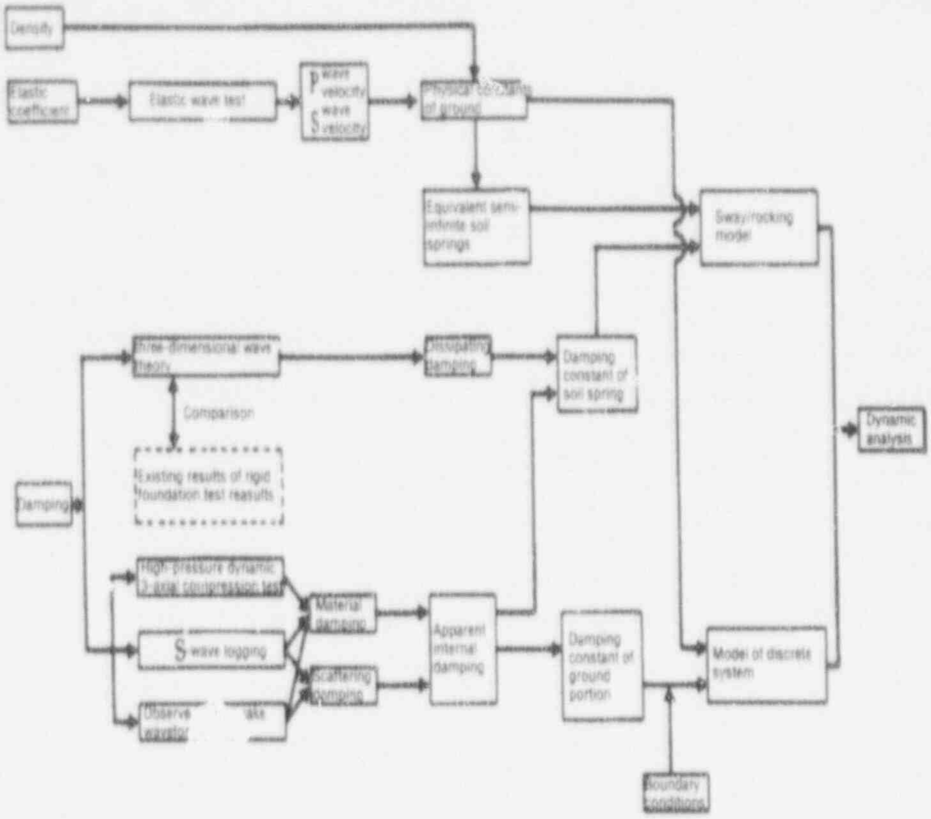


Figure 5.2.2-1. Flow chart for evaluation of properties of ground.

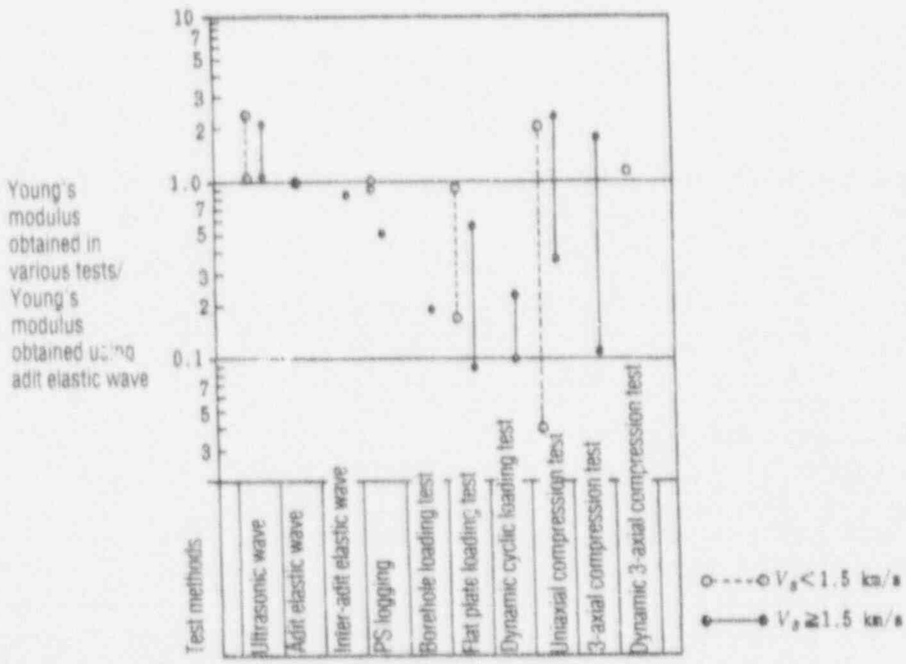


Figure 5.2.2-2. Ratio of values of Young's modulus E obtained using various test methods (Standards, Construction-4 [HYO, KEN-4]) to E obtained using adit elastic wave test.

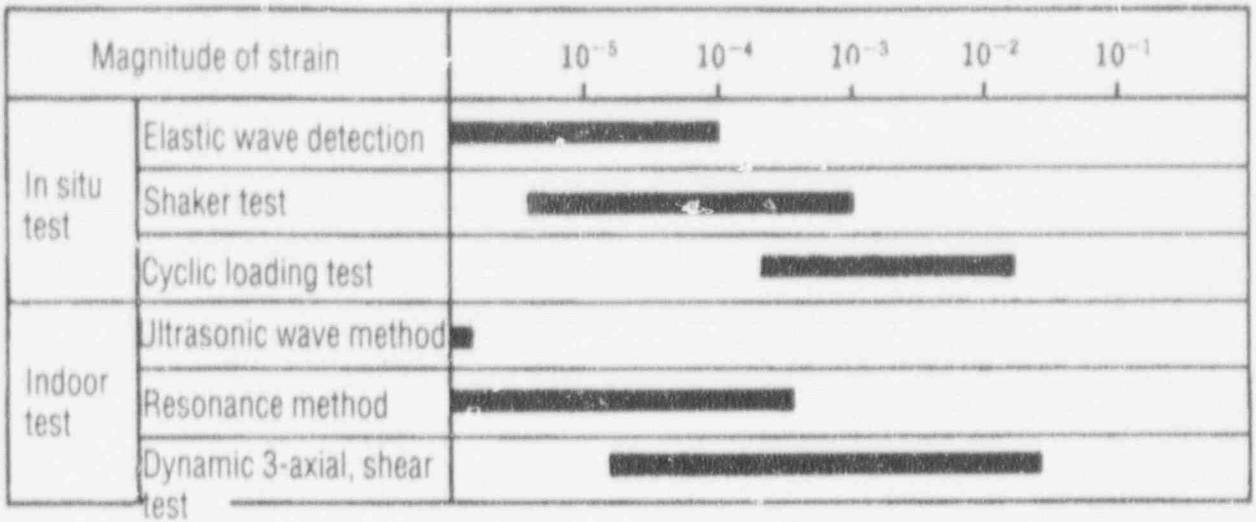


Figure 5.2.2-3. Strain levels of the various test methods (Standards, Construction-4 [HYO, KEN-4]).

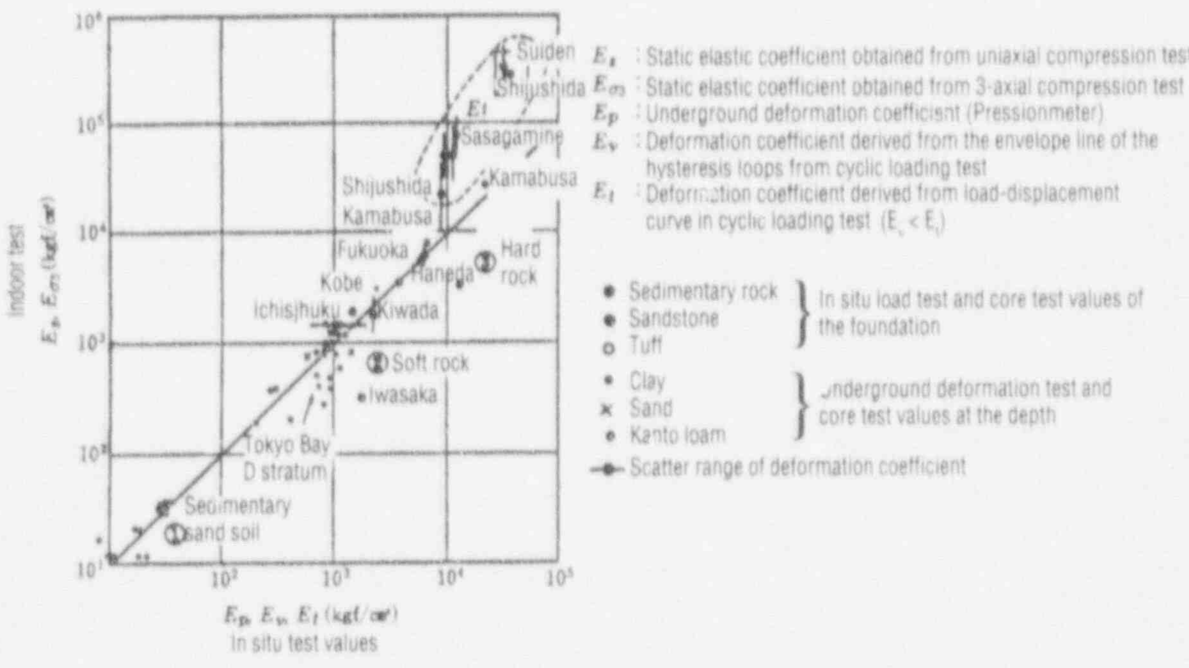


Figure 5.2.2-4. Comparison between indoor test values and in situ test values (5.2.2-2).

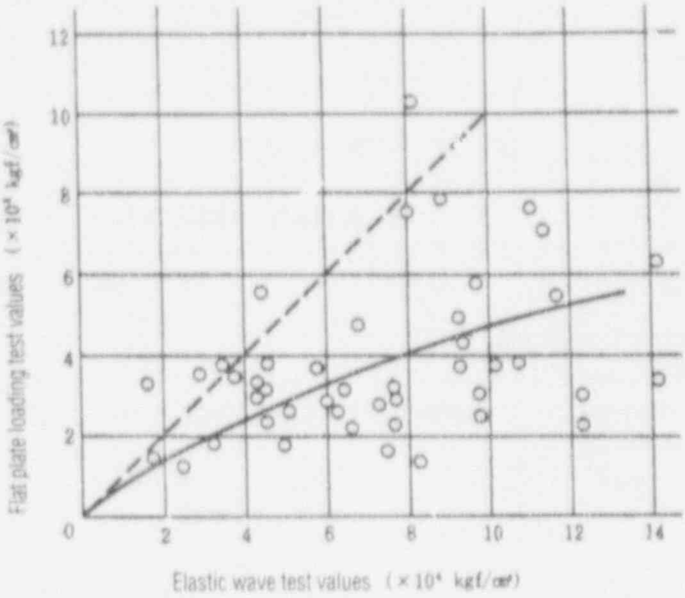


Figure 5.2.2-5. Comparison between flat plate loading test values and elastic wave test values (5.2.2-2).

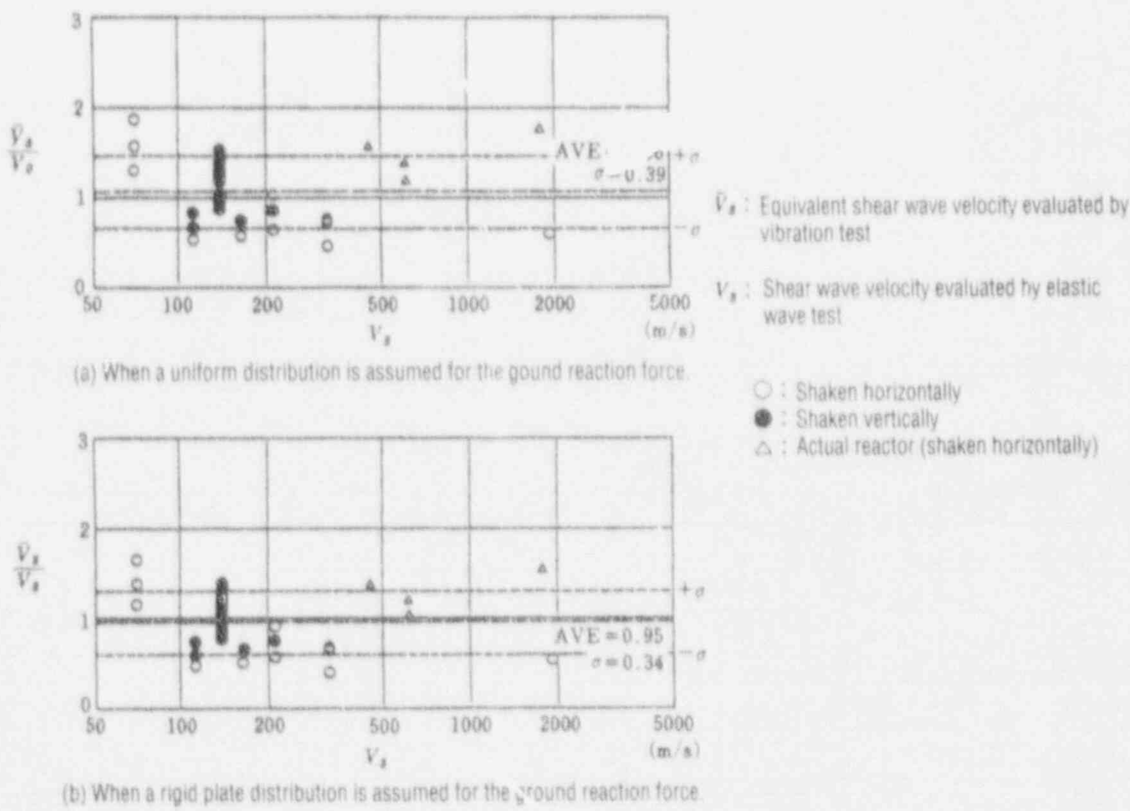


Figure 5.2.2-6. Comparison of shear wave velocities measured in vibration test and elastic wave test (Standards, Construction-4 [HYO, KEN-4]).

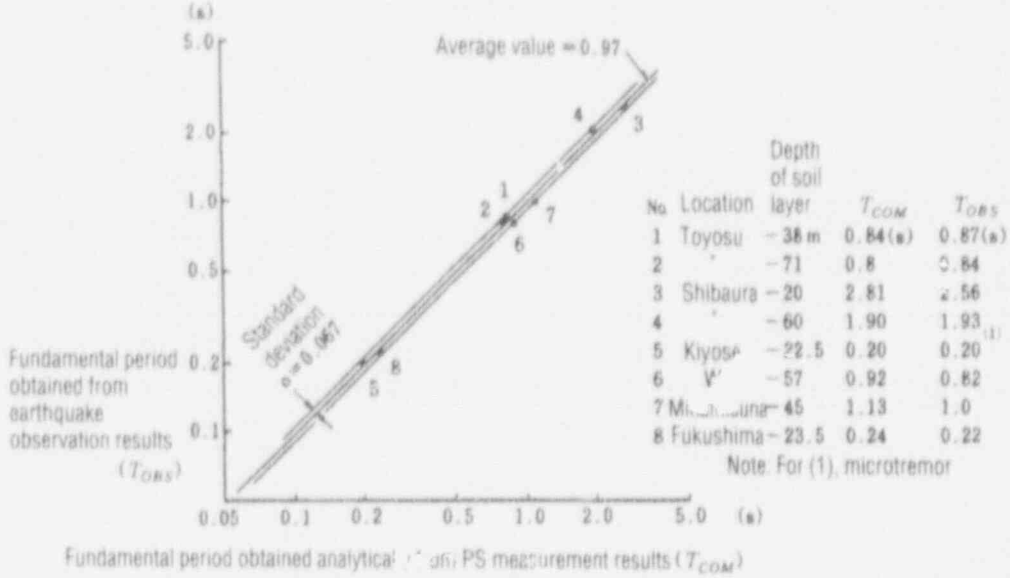


Figure 5.2.2-7. Comparison of fundamental periods of ground measured by earthquake observation and elastic wave test (Standards, Construction-4 [HYO, KEN-4]).

(b) Scatter in elastic wave test results and elastic coefficient of ground for dynamic analysis

In chapter 3, we have discussed the basic methods for evaluating the scatter in the properties of the ground, such as elastic coefficient, Poisson's ratio, specific gravity, etc. In this section, we will discuss the scatter in the elastic coefficient of the ground, which has a particularly large influence on the dynamic characteristics of the ground during earthquake, on the base of the measurement examples. Then, we will discuss the related application schemes in design.

Figure 5.2.2-8 shows the histogram of the ratio (Vs'/Vs), where Vs is the average shear wave velocity in the same ground layer in PS logging and Vs' is the velocity within a microscopic depth interval (average velocity Vs' for an interval length of 4 m). The average value and standard deviation are 1.05 and 0.19, respectively.

As shown in Figure 5.2.2-7, for the scatter in the fundamental period of the ground during earthquake, the standard deviation is as small as 0.067. This indicates that the scatter caused by the heterogeneity of the foundation can be cancelled for the overall ground layer, and the effect on the dynamic characteristics is small.

Judging from the above two examples, it can be concluded that the scatter in the elastic wave test results itself has a small effect on the evaluation of the dynamic characteristics of the ground layers as a whole, and it is acceptable to use their average value for evaluation in the practical case.

(c) Layered ground

In this section, we will discuss the situation when the layering of the ground has been confirmed by the elastic wave test (in particular, velocity layer sequence in the depth direction derived from PS measurement, etc.). In the case when dynamic analysis is performed for a layered ground, the analytical model and analytical method in consideration of the layering of the ground are used. In this case, the value of the elastic coefficient determined using the method described in the preceding sections can be used directly.

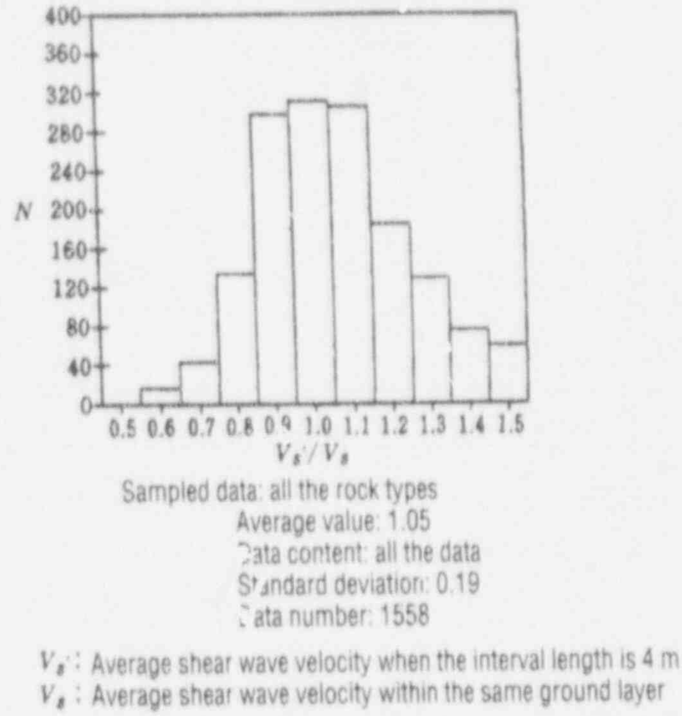


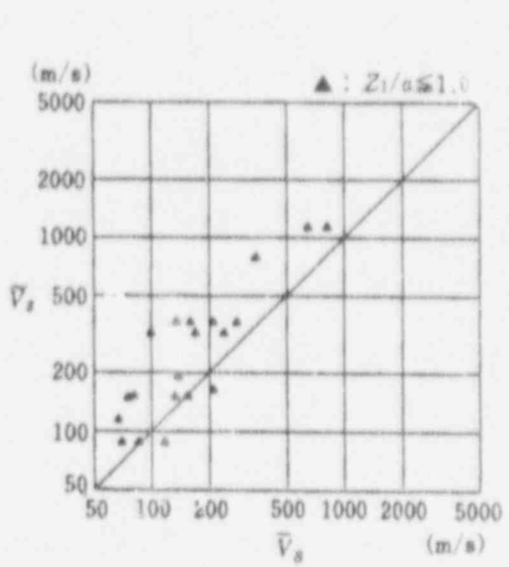
Figure 5.2.2-8. Ratio of interval velocity (Vs') to average velocity (Vs) (Standards, Construction-4 [HYO, KEN-4]).

On the other hand, it is also possible to treat it as a semi-infinite homogeneous ground. Figure 5.2.2-9 compares the equivalent shear wave velocity ($\tilde{V}s$), which was calculated to reproduce the peak vibration frequency obtained from the vibration test of the block foundation, with the shear wave velocity ($\bar{V}s$) determined using the correction method proposed by Tajimi, which accounts for the layering effect of the ground on the base of the velocity layer sequence obtained in the elastic wave test. The layer correction method proposed by Tajimi (5.2.2-1) corresponds well to the experimental results, and this indicates that the layered ground can be handled as a semi-infinite foundation.

(d) Strain dependency of stiffness

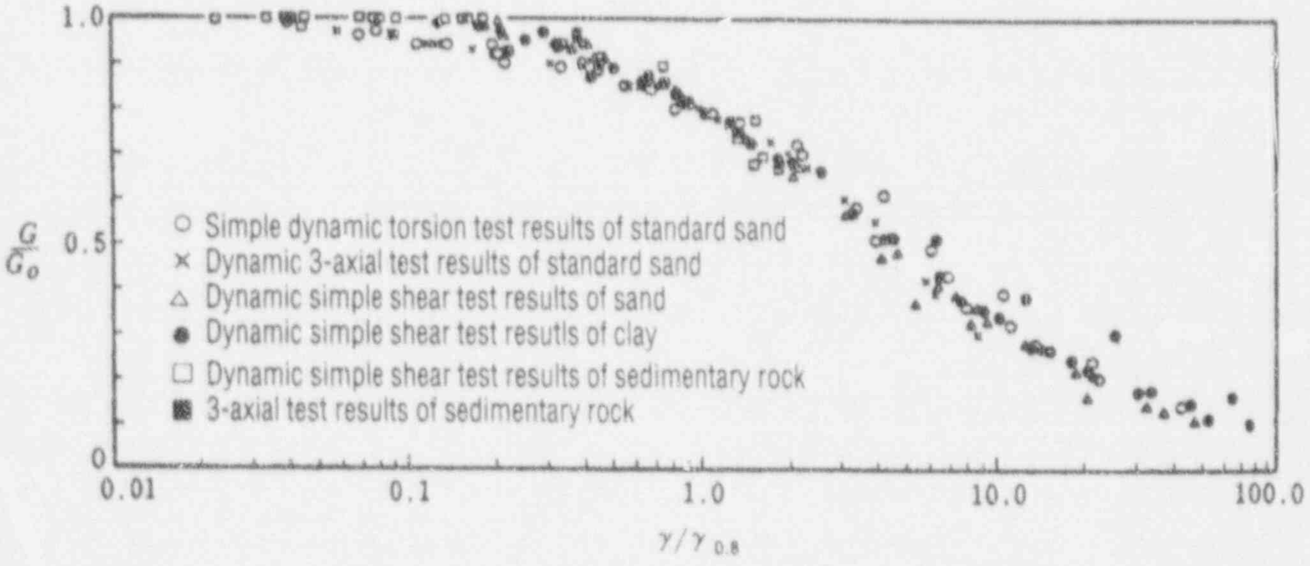
For the ground, as the strain level increases, the stiffness decreases in a nonlinear way. The nonlinearity of the ground depends on the type of soil and confinement pressure. The strain levels corresponding to decrease in stiffness are different for the sandy soil, clay sol, unsolidified ground, and rock foundation. Figure 5.2.2-10 shows the relation between the strain γ and the shear stiffness ratio G/G_0 based on the indoor test results for sand, clay, and sedimentary rock, with the relationship normalized for the strain ($\gamma/\gamma_{0.8}$; $\gamma_{0.8}$: the strain level when the value of G/G_0 becomes 0.8 of linear value). When the data are plotted using $\gamma/\gamma_{0.8}$ as shown in this figure, it is possible to represent all the soil types, from unsolidified base to sedimentary rock, by a single characteristic curve.

On the other hand, it can be seen from Figure 5.2.2-11 that the decrease pattern of the stiffness of the clay soil in the ground during earthquake corresponds well to the indoor test results. However, in this example, the data are for the unsolidified soil, and they do not include the data of the rock ground. The decrease in the stiffness in the rock ground during earthquake can be studied by using the one-dimensional wave theory as described below.



$\tilde{V}_s$: Equivalent shear velocity evaluated by the layer correction scheme by Tajimi
$\bar{V}_s$: Equivalent shear velocity evaluated by vibration test

Figure 5.2.2-9. Comparison of equivalent shear wave velocity (Standards, Construction-4 [HYO, KEN-4]).



Fi, ıre 5.2.2-10. G / G_0 vs. $\gamma / \gamma_{0.8}$ of sand, clay, and sedimentary rock (Standards, Construction-4 [HY(, KEN-4]).

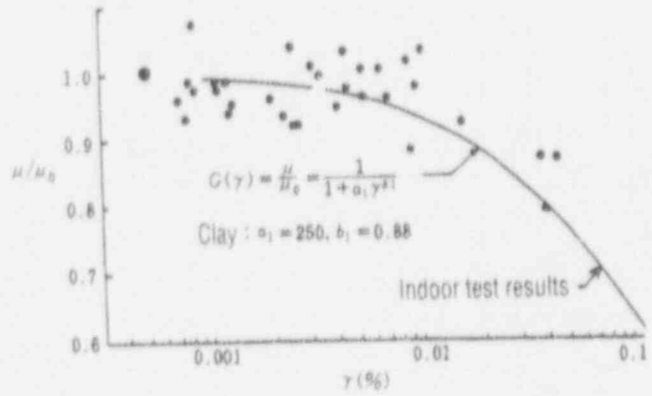


Figure 5.2.2-11. Relation of G / G_0 vs. γ during earthquakes as compared with indoor test results (5.2.2-3).

As shown in Figure 5.2.2-10, if the normalized strain is considered, it is possible to remove the various conditions, such as ground type, confinement pressure, etc. As a result, the nonlinear characteristics shown in Figure 5.2.2-12 can be assumed for performing analysis using the one-dimensional wave theory. As an example, Figure 5.2.2-14 shows the maximum strain distribution of the analytical results by using the analytical model shown in Figure 5.2.2-13. In this case, with respect to the input of the maximum acceleration of 267.4 Gal at GL. -205 m, the strain level of the sedimentary rock is in the range of 0.01-0.03%. As shown in Figure 5.2.2-12, the stiffness decreases by 1-3.5%. It is believed that even in the case of soft rock ($\bar{V}s$ = 500 m/s), the decrease in stiffness still has a small effect on the response to earthquake; for the rock ground harder than the sedimentary rock, the decrease in stiffness is not large enough to be observed in analysis

(e) Method for determining elastic coefficients of ground used in dynamic analysis

In the above, we have discussed the evaluation method based on the elastic wave test, which is believed to be the most suitable method for determining the elastic coefficients using the dynamic analysis. In order to determine the elastic coefficients, in addition to the shear wave speed, it is also required to use the compressive wave velocity (P-wave velocity) and specific gravity. For the P-wave velocity, it is possible to handle it using the evaluation method for shear wave velocity. For the specific gravity, the conventional investigation method can be used. Hence, they are not to be described further specifically here.

When the elastic coefficients of the ground is to be derived from the elastic wave velocity and the specific gravity, evaluation can be performed using the following elastic theory.

$$\left.\begin{aligned} E &= 2(1+\nu)G \\ G &= \rho V_S^2/g \\ \nu &= \{(V_P/V_S)^2-2\}/\{2(V_P/V_S)^2-2\} \end{aligned}\right\} \qquad (5.2.2\text{-}1)$$

V_P : P-wave speed of the bed(m/s)
V_S : " S-wave speed of the bed(m/s)
E : " Young's modulus of the bed(tf/m^2)
G : " Shear electric coefficient of the bed(tf/m^2)
ν : " Poisson's ratio of the bed
ρ : " Specific gravity of the bed(tf/m^3)

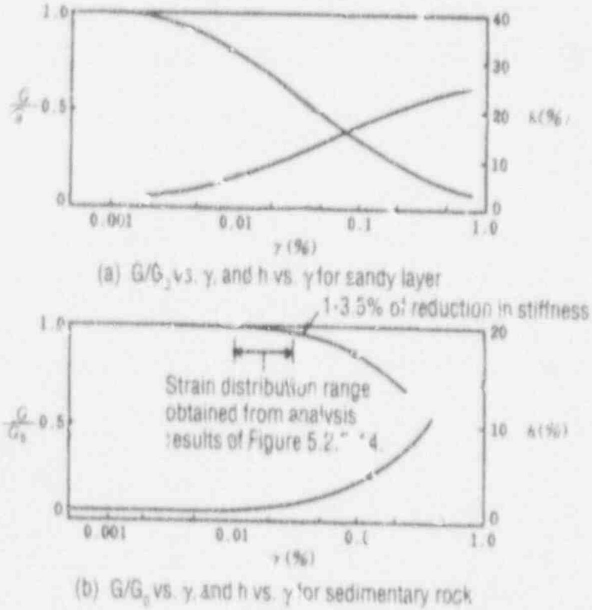


Figure 5.2.2-12. G / G_0 vs. γ, h vs. γ relationship of ground (5.2.2-4).

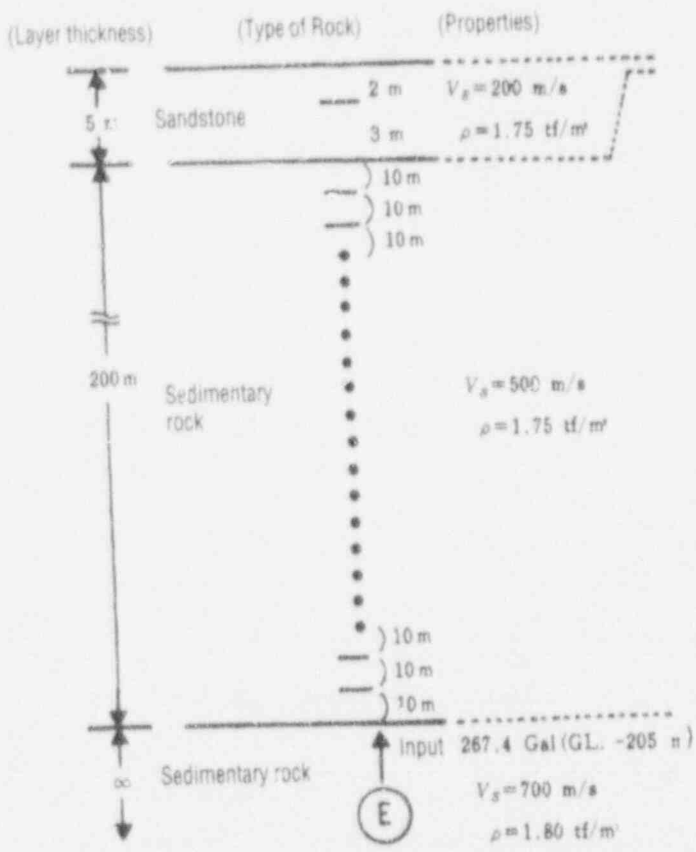


Figure 5.2.2-13. Ground model for analysis (Standards, Construction-4 [HYO, KEN-4]).

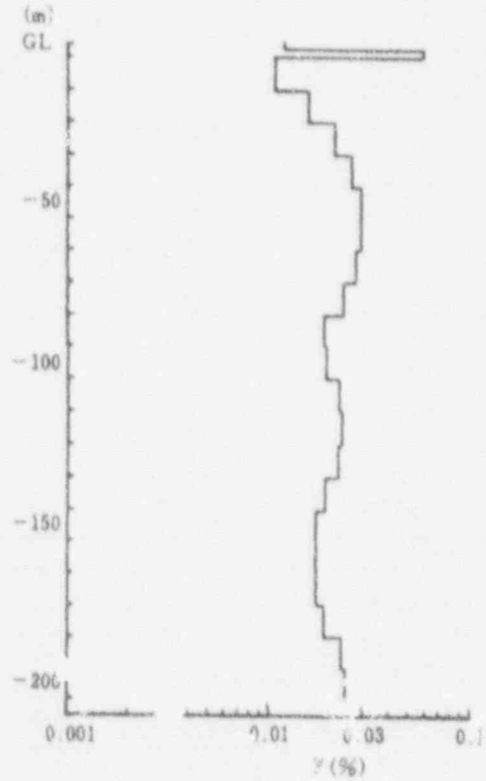


Figure 5.2.2-14. Maximum strain distribution (Standards, Construction-4 [HYO, KEN-4]).

b. Internal damping

In the process when a wave is transmitted within a ground, the major causes of damping are as follows.

- Damping caused by characteristic properties within the ground (such as friction among particles, motion of viscous fluid that fills among particles, energy consumption required for particle rearrangement, etc.), known as "material damping" here.

- Damping caused by dissipation of the wave by reflection at boundaries due to the heterogeneity and discontinuity of the ground, known as "scattering damping" here.

- Damping caused by geometric wave surface expansion (proportional to r^{-1} for the body wave, proportional to $r^{-1/2}$ for the surface wave (where r is the distance)), known as "geometric damping" here.

In the case when the analysis for earthquake response is taken into consideration, the material damping and the scattering damping become the target of the investigation. Hence, the damping determined in the interior of the ground is a combination of these damping, and it is known as equivalent internal damping. Figure 5.2.2-15 shows an example of actual measurement of the damping of the ground. In the following, we will describe the 3 types of test methods for evaluating the damping of the rock ground as well as the parameters affecting damping and damping constants.

(i) High-pressure dynamic 3-axial compression test

The purpose of this test is to understand the strain dependence of shear modulus G and damping constant h in the high-confinement pressure region (10-200 kgf/cm^2) using a high-pressure cyclic 3-axial compression tester. The confinement stress in the test is the effective overburden pressure. Figure 5.2.2-16 shows the results for GL. -74 ~ -75 m sandstone. The results of the tests performed at 3 depths of the sandstone have similar trends. The damping constant h ≈ 1-2% in the low-strain region with sheer strain $\gamma \approx 0.001\%$, and h ≈ 3% for $\gamma \approx 0.1\%$. This indicates the strain dependence of the damping. What not shown in the figure is that h ≈ 1% in the region where $\gamma \approx 0.0001$-0.02%, without strain dependency.

(ii) S-wave measurement

Among the recorded waveforms of the S-wave measured in a boring hole, the portion considered to be direct wave was selected, and the reduction (attenuation) ratio in amplitude is derived for each frequency. On the other hand, from the model of the ground the damping amount is estimated (damping due to geometric dissipation of wavefront and damping due to reflection at the boundary surface). The difference obtained by subtracting these dampings is taken as the internal damping of the ground. In this way, the damping evaluation is performed.

The vibration frequency range for the evaluation is 10-50 Hz, where a large Fourier amplitude and a stable logarithmic amplitude ratio are expected. Figure 5.2.2-17 shows the results of the sandstone portion of GL. -70 ~ -275 m. As can be seen from this figure, there exists a certain dependence on the frequency, with h = 2-4%.

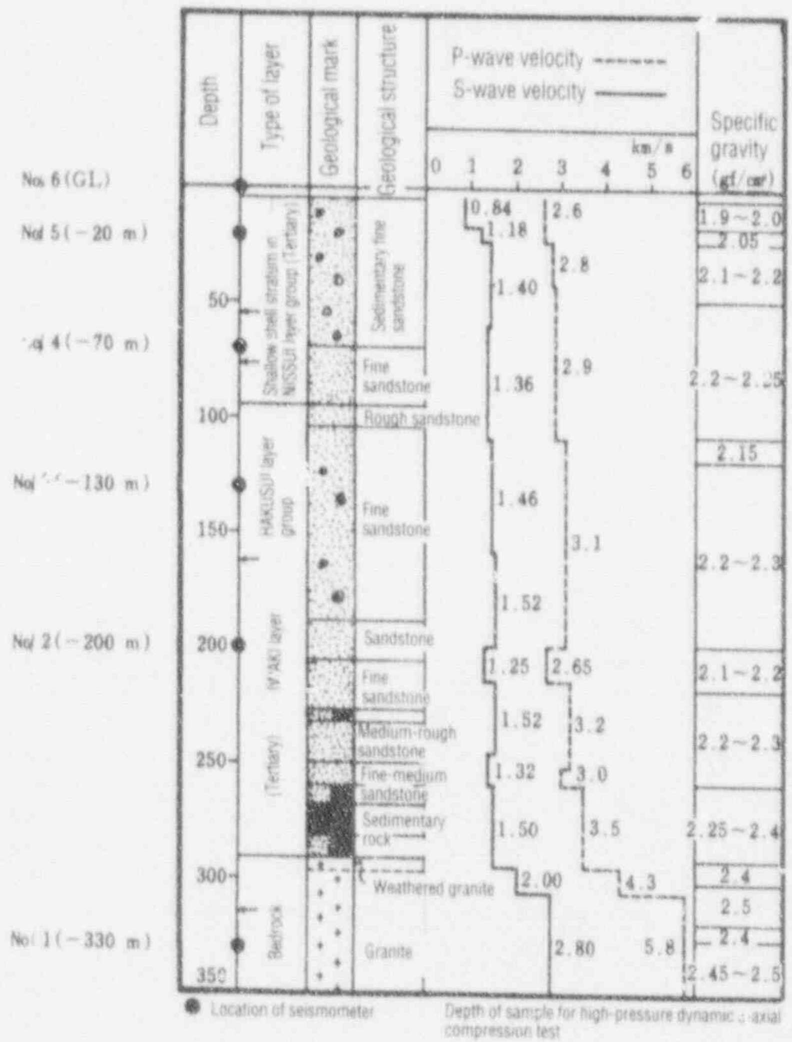


Figure 5.2.2-15. Results of in situ ground survey and locations for installation of seismometers.

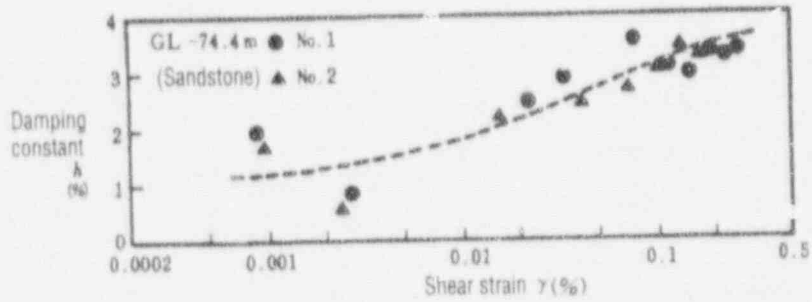


Figure 5.2.2-16. Damping constant determined in high-pressure dynamic 3-axial compression test (5.2.2-7).

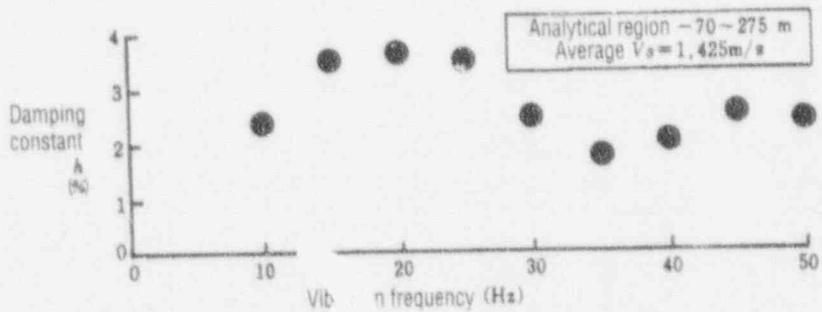


Figure 5.2.2-17. Damping c stant determined by S-wave meaurement.

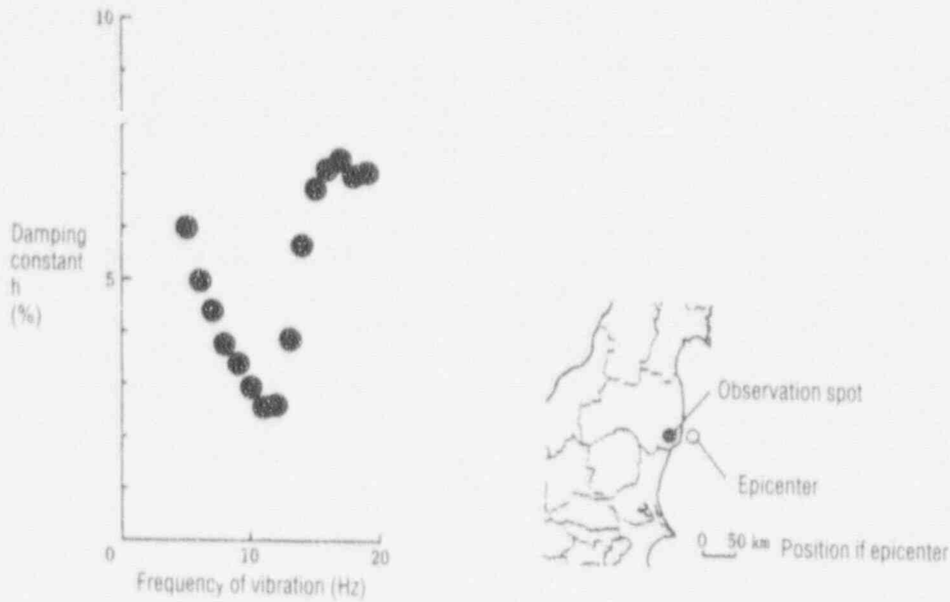


Figure 5.2.2-18. Damping constant obtained by seismic observation.

(iii) Seismic observation

The earthquake motion is RT transformed[1] and the horizontal component perpendicular to the transfer direction is considered as SH wave; the portion considered to be direct wave is extracted and its with respect to the response wave is used to represent the damping.

The records of GL. -200 m and GL. -70 m with a small variation in the ground elastic coefficients in the Tertiary layer have been used for analysis. In this case, it differs from the S-wave measurement in that only the effect at the layer boundary is removed. Table 5.2.2-18 lists the various parameters of earthquake. The analytical results are shown in Figure 5.2.2-18. It can be seen that in the frequency range of 5-20 Hz, h = 2-8%.

As can be seen the aforementioned measurement examples, for the damping constants evaluated for the same ground using high-pressure dynamic 3-axial compression test, S-wave measurement, and seismic observation, in the low-strain region, these damping constants increase in the following sequence: 0.5-2% for the high-pressure dynamic 3-axial compression test, 2-4% for the S-wave logging method, and 2-8% for the seismic observation method. The damping constant determined in the dynamic indoor test is calculated as the ratio of the energy consumed during one cycle of vibration to the maximum strain energy. Therefore, the material damping i.e., the damping due to the inelastic behavior of the ground, is obtained in this test.

Next, for the damping constant derived by elastic wave tests, such as plate tapping method, the dissipation of propagation energy due to the geometrical dispersion or reflection at a clearly recognized boundary can be separated. However, it is quite possible that separation cannot be carried out for the dissipation caused by heterogeneity or discontinuity of the ground (such as stratification, joint, existence of lenses, etc.). If this is called as scattering damping, then, the damping constant derived by the elastic wave survey is believed to include the material damping and the scattering damping.

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[1]RT transformation: The observed two horizontal components (such as the two directions of NS, EW directions) are transformed to a coordinate system having two components, with one in the direction towards the epicenter (R direction: radial) and the direction orthogonal to it (T direction: transverse).

On the other hand, although the seismic observation makes use of nearly the same analytical sequence of the elastic wave tests, it differs from the elastic wave test method in that the combination of the various wave phenomena is represented by the equivalent SH wave. Therefore, it is considered to give the equivalent damping constant.

In the dynamic indoor test, the samples used are rock elements in perfect state without cracks. On the other hand, for the elastic wave test and seismic observation, the entire layer including the effects of layering and joints is taken as the object. It is believed that the damping constant determined from the elastic wave survey and seismic observation also includes the scattering damping in addition to the material damping. Here, this portion is called temporarily as the apparent internal damping. The same behavior appears for the damping determined from seismic wave simulation analysis and spectral fitting. That is, it also represents the apparent internal damping. Recently, in the examples of evaluation of the damping of ground based on the seismic observation records, the soil damping is found to be higher-mode-decrease type, and a relatively good simulation can be obtained if the soil damping is treated as an external damping (5.5.2-5).

The various damping characteristics determined by using the above various test methods usually have the following relationship:

$$h_{\text{indoor test}} \text{ (= material damping)} < h_{\text{elastic wave test}} \text{ (= material damping + scattering damping)}$$
$$< h_{\text{seismic observation}}$$

For a ground which is relatively homogenous with no joint and with a negligible scattering damping, except for the case of an actual earthquake, the damping constants determined by indoor test and elastic wave test are nearly the same (5.5.2-6). Similar to the case of stiffness, for the strain dependency exists for the damping; as the strain level increases, the damping constant increases.

Figure 5.2.2-19 shows the dependency of the damping constants of sedimentary rock and sandstone on the normalized strain $\gamma/\gamma_{0.8}$ (where γ is the strain, $\gamma_{0.8}$ is the strain at which the stiffness becomes 0.8 that of the linear region). As can be seen from this figure, when the strain level increases, the value of the damping constant increases significantly, indicating the strain dependency of the damping constant. In addition, by using normalized strain, it is possible to express both the sedimentary rock and the sandstone by the same characteristic curve.

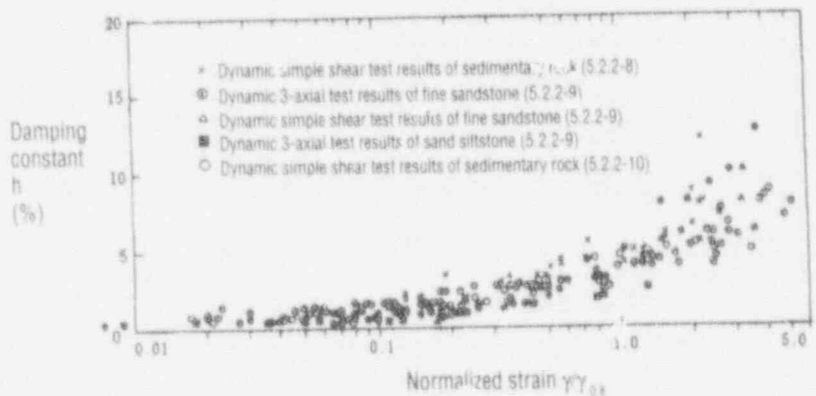


Figure 5.2.2-19. h-γ, h-$\gamma/\gamma_{0.8}$ relationship for seimentary rock and sandstone [drawn from data in references (5.2.2-8)-(5.2.2-10)].

c. Soil radiation damping

The vibration energy which has entered a structure from the foundation makes the structure vibrate, and, at the same time, is reflected from the interior of the structure and then partially dissipated into the ground. When the mechanism of soil radiation damping is to be studied experimentally, a dynamic test is performed by shaking a rigid foundation lying on the ground. In the following, we will explain the general properties of the soil radiation damping on the base of experimental data.

The vibration test data of the rigid foundation (Standards, Construction-4 [HYO, KEN-4]) are all collected from published references, including 39 papers published in Japan, and 18 papers published in other countries. When they are classified according to the types of experiments, they include 26 horizontal shaking experiments, and 31 vertical experiments. Many grounds in the tests are on unsolidified ground. However, grounds with shear wave velocity Vs higher than 400 m/s are also included. In addition, there are two sets of experimental results performed for rock grounds with Vs higher than 1,000 m/s. In most cases, the dimensions of the rigid foundations are about 2-3 m, i.e., they are small to medium models. However, there are also 9 cases in which foundations no less than 10 m are tested.

In the shaker experiment, usually, the resonance curve and phase curve are obtained, and simulation analysis is performed to evaluate the results. In order to understand the soil radiation damping properties, the rigid foundation is replaced by a sway/rocking model, and its foundation spring constants are derived from the results of each experiment, and the data are systematically analyzed. From each set of the experimental results, the sway/rocking spring constants are derived by using the horizontal displacement and rotational displacement of the foundation as well as their phase curves. Figure 5.2.2-20 shows an example of the results. It displays the complex stiffness calculated from the experimental results. The damping constants derived from the real number portion and the imaginary number portion are shown in Figure 5.2.2-21. It can be seen that the value of the soil radiation damping is not constant; as the frequency increases, the damping also increases.

As explained above, the collected data of experiments involve various ground conditions and dimensions of the rigid foundation; and the damping constants derived from the various experimental data have different magnitudes and display various trends of variation. In order to extract the characteristics of the dissipation damping caused by dynamic interaction and compare them with the theoretical solution, the results are normalized to be dimensionless with respect to the frequency. In addition, the said data are arranged by introducing the effects of the layering of the ground.

Figure 5.2.2-22 compares the experimental data with the theoretical values of the horizontal soil springs with the abscissa representing the dimensionless frequency a_0 ($a_0 = \omega \sqrt{A} / \bar{V}s$, where ω is the [illegible] frequency, A is the foundation bottom area, and $\bar{V}s$ is the equivalent shear wave velocity), and the ordinate representing the damping. The data shown in this figure are for the grounds which have a ratio of Vs in the surface layer to that in the lower layer of about 0.5, and a ratio of the thickness of the surface layer to the foundation width ($Z_1 / \sqrt{A}$) in the range of 0.5-2.0. There are significant differences among test data. However, if the data are plotted this way, the general trend of the damping constant becomes clear, i.e., they increase as the frequency increases. In this figure, the hatched portion indicates the range of the theoretical values (from the 3-dimensional wave theory) corresponding to the range of layering property. It can be seen that the experimental data (solid lines) correspond well with the theoretical values (hatched region).

In the above, we have discussed "b. Internal damping," and "c. Soil radiation damping." When the damping of the soil-structure interaction is investigated, however, it is required to make evaluation of both the soil radiation damping and the apparent internal damping of the ground for the sway/rocking model.

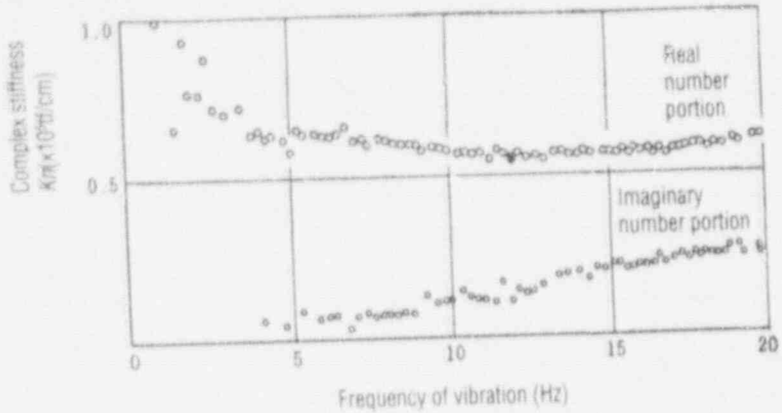


Figure 5.2.2-20. Complex stiffness [from reference (5.2.2-11)].

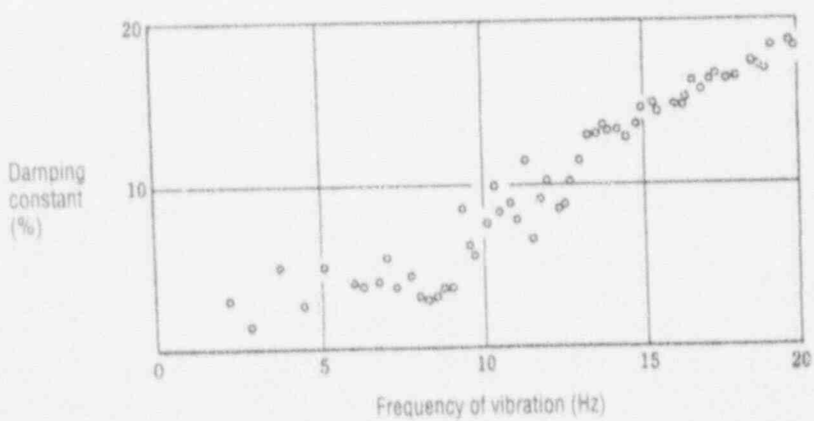


Figure 5.2.2-21. Damping constants [from reference (5.2.2-11)].

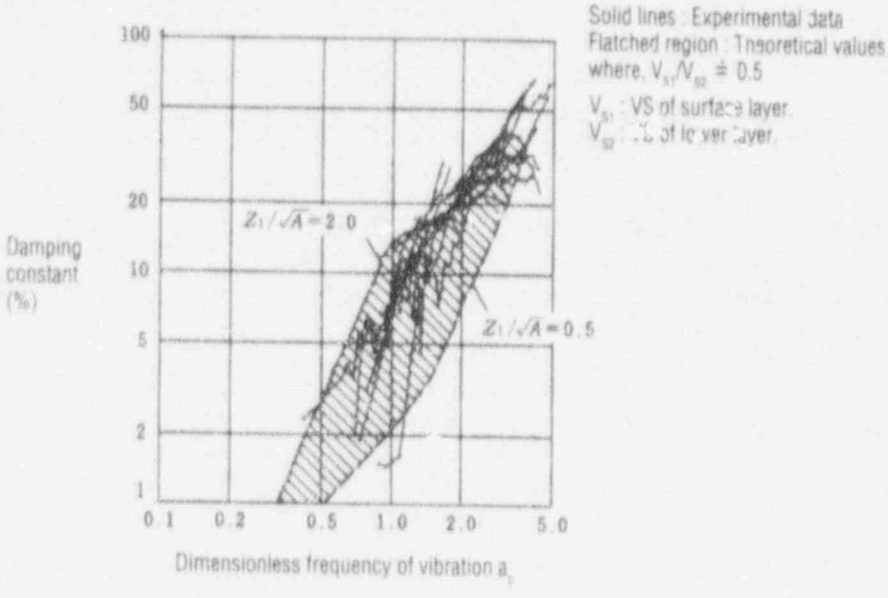


Figure 5.2.2-22. Experimental data and theoretical values of the damping constants of horizontal springs of the ground (Standards, Construction-4 [HYO, KEN-4]).

For the discrete SSI models, the basic modeling assumptions such as the boundary conditions and the way to input the earthquake motions are inter-related to the way of evaluating the soil damping. However, for the damping of a soil system, usually the internal damping is considered while the dissipation damping is accounted for at the boundaries of the model.

(2) Properties of structures

The structures in nuclear power plants are primarily made of reinforced concrete and steel frames. The properties of the materials used are defined primarily in the various standards, specifications, guidelines, etc., of the Architectural Institute of Japan. In the following, we will present a schematic description.

a. Materials used

The materials used for the major structural components are as follows.

(a) Concrete
Defined in JASS 5N "Reinforced Concrete Constructions (5.2.2-12)."

(b) Reinforcing bars
Except the case when special reinforcing bars are used, they are defined in JIS G 3112 "Steel bars used in reinforced concrete" and JIS 3117 "Reproduced steel bars for reinforced concrete."

(c) Steel
The steel used is the steel materials for nuclear power plants defined in JIS G 3101 "Rolled steel materials for conventional structures," JIS G 3106 "Rolled steel materials for welded structures," and Public Notification of the Ministry of Foreign Trade and Industry No. 501 "Technical standards for structures concerning nuclear power plants" (October 30, 1980) (referred to as "Public Notification No. 501" hereinafter).

b. Material constants

In the conventional cases, the material constants are listed in Table 5.2.2-4.

(a) Young's modulus of concrete

Figure 5.2.2-23 shows the stress-strain curve of concrete. The Young's coefficient is usually expressed by the second modulus (the slope of the straight line connecting the point on the stress-strain curve and the origin). In the conventional cases, the second modulus usually refers to the point of stress with a magnitude 1/4 or 1/3 of the concrete strength.

(b) Young's modulus of reinforcing bars and steel frames

For the steel materials used as reinforcing bars and steel frames, the Young's modulus is a constant value of 2.1×10^6 kgf/cm^2, independent of the yield point and the tensile strength. For deformed reinforcing bars, as the cross-sectional area varies in the reinforcing bar's axial direction, although a value of $1.8\text{-}2.0 \times 10^6$ kgf/cm^2 can be obtained from the stress-strain relationship for the nominal cross section, because the value of steel ratio is on a small order, the value used for design can be taken as 2.1×10^6 kgf/cm^2.

On the other hand, for the Young's modulus of deformed reinforcing bar used in experiment, measurement is performed according to the purpose. In the case when the Young's modulus of the material itself is to be determined, the ribs on the surface are cut off to form a plain bar. Then, the actual cross-sectional area is measured and the tensile test is performed. Afterwards, the Young's modulus is calculated using the actual cross-sectional area. In the case when the yield point needs to be determined in the experiment, the tensile test is performed with the surface ribs remained, and the apparent Young's modulus is calculated by using the nominal cross-sectional area. The Young's modulus of $1.8\text{-}2.0 \times 10^6$ kgf/cm^2 appearing in the test reports, etc., refers to this value.

For the deformed reinforcing bars, the names and dimensions are defined as follows in the references (5.2.2-16, 5.2.2-17).

Name: Using the rounded value of the nominal diameter.
Nominal diameter (d, mm): A value calculated from the weight of unit length.
Nominal cross-sectional area (S, cm^2): $(0.7854 \times d^2)/100$ (rounded to retain 4 significant figures)
Nominal circumference (l, cm): $0.3142 \times d$ (rounded to first digital place)
Unit weight (kgf/m): $0.785 \times S$ (rounded to retain 3 significant figures)

(c) Shear elastic coefficient

The shear elastic coefficient G is derived by using the formula listed in Table 5-2.2.4, which are well known in the conventional elastic theory.

(d) Poisson's ratio

For concrete, it is usually assumed that $\nu = 1/6 = 0.17$. Actually, however, it depends on the type, composition, age, and strength of the specific concrete used. The Poisson's ratio of steel is usually taken as $\nu = 0.3$, a value conventionally taken for steel materials.

(e) Coefficient of thermal expansion

The coefficient of thermal expansion is used, for example, in determining the thermal stress of components. The coefficient of thermal expansion of concrete at room temperature is $1.2\text{-}1.5 \times 10^{-5}/°C$ for the conventional concrete, $0.7\text{-}1.4 \times 10^{-5}/°C$ for type-1 lightweight concrete, $0.5\text{-}1.1 \times 10^{-5}/°C$ for type-2 lightweight concrete, and $0.7\text{-}1.0 \times 10^{-5}/°C$ for type-3 and type-4 lightweight concretes.

Table 5.2.2-4. Constants of materials (5.2.2-13–5.2.2-15).

Material	Young's modulus E (kgf/cm^2)	Shear elastic coefficient G (kgf/cm^2)	Poisson's ratio ν	Coefficient of thermal expansion α (1/°C)	Weight per unit volume γ (tf/m^2)
Conventional concrete[1]	$2.1 \times 10^5 \times \left(\frac{\gamma}{2.3}\right)^{1.5} \times \sqrt{\frac{Fc}{200}}$	$\frac{E}{2(1+\nu)}$	$\frac{1}{6}$	1.0×10^{-5}	2.3 [2]
Reinforcing bars	2.1×10^6	—	—	1.0×10^{-5}	7.85
Steel frames	2.1×10^6	8.1×10^5	0.3	1.2×10^{-5} [3]	7.85

[1]Lightweight concrete is excluded in this table.
[2]If not specified otherwise, the specific gravity of the reinforced concrete can be taken as 2.4 tf/m^3.
[3]1.0×10^{-5} in the case of steel frame reinforced concrete (SRC).

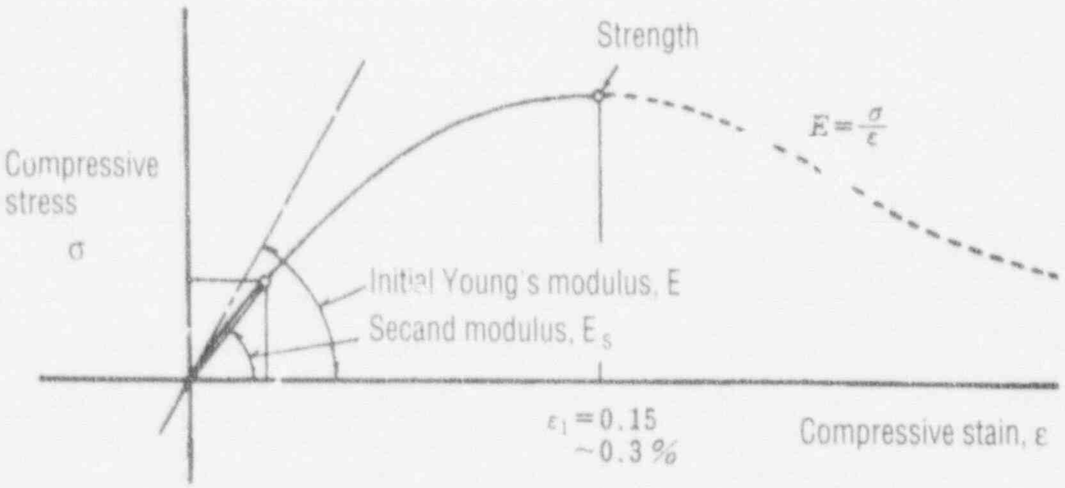


Figure 5.2.2-23. Stress-strain curve of concrete.

On the other hand, the coefficient of thermal expansion of the steel material is usually about $1.2 \times 10^{-5}/°C$. However, at room temperature, steel and concrete display nearly the same order of expansion. For RC and SRC structures, the two materials are considered to be a single body, with both coefficients of thermal expansion taken as $1.0 \times 10^{-5}/°C$.

(f) Design standard concrete strength

The design standard strength of concrete is the compressive strength of concrete used as standard in the structural calculation. The age of the material for determining the concrete strength suitable for the design standard strength depends on the characteristics of the specific structure concerned, and is specified within 91 days upon a general consideration on the design/construction conditions. However, several problems exist in this case; for example, the compressive strength of concrete depends on the test method, and the obtained values have a certain degree of scatter. Hence, for the compressive strength of concrete it is required to determine the following two points:

{1} Definition of the compressive strength in terms of test method;

{2} Judgment standard for determining how to set the design standard strength considering the scatter in the actual values of the concrete strength.

For these features, please see JASS 5N.

(g) Effects of concrete age

Figure 5.2.2-24(a) shows the effect of the concrete age on the compressive strength, with the compressive strength 28 days after casting is set to be unity. Figure 5.2.2-24(b) shows the effect of the concrete age on the elastic coefficient, with the elastic coefficient of the concrete 28 days after casting is set to be unity. Investigation is now under way on the behavior of increase in the Young's modulus and strength as the age increases.

(h) Relationship with dynamic elastic coefficient

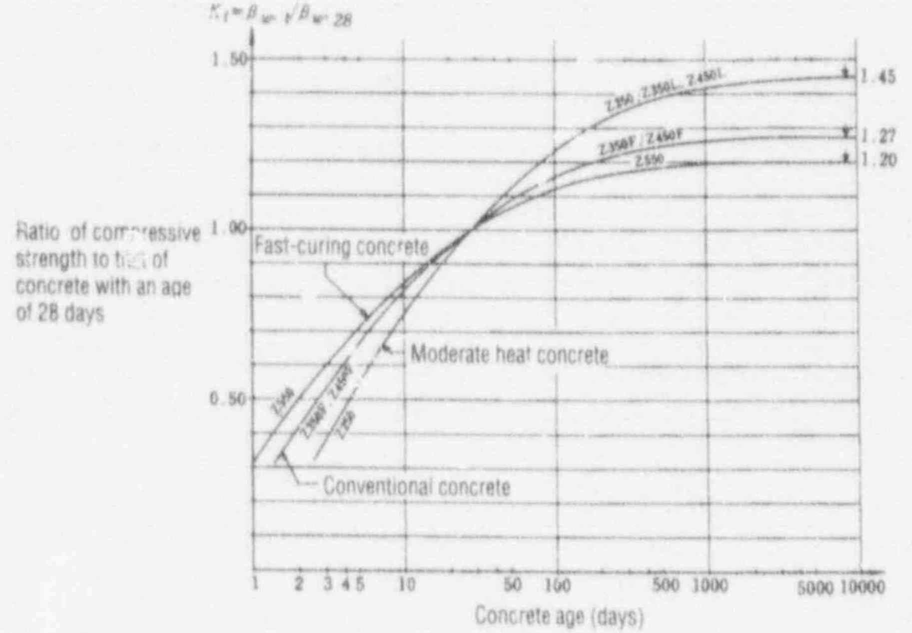
It is believed that in the range of minute deformation (low stress degree), the concrete behaves as a linear elastic material. Hence, the dynamic elastic coefficient of the concrete can be derived from the linear elastic theory. In the case of linear elastic material, the dynamic elastic coefficient is in agreement with the static elastic coefficient. However, for concrete in the low stress degree range, as it has certain nonperfect elastic properties, the value of the static elastic coefficient may not be equal to that of the dynamic elastic coefficient. For the elastic coefficient of concrete used in the dynamic analysis of design, usually, the strain level is high. For this purpose, the values given in "Table 5.2.2-4, Various constants of materials," may be used, which are recommended based on static tests at relatively high strain level.

(i) Stiffness of structural component

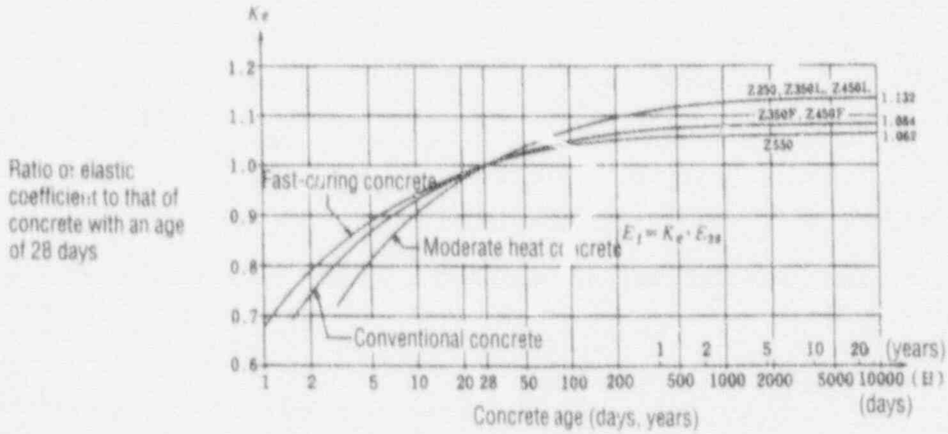
In the case when the self-constraint stress due to such as the minute deformation, minute vibration of the structures, differential settlement, temperature variation, shrinkage of concrete, etc., the elastic coefficients listed in Table 5.2.2-4 are used. In this case, the Young's modulus is determined using the values for concrete. When the effects of the steel materials are taken into consideration, the values of the cross-sectional area and the second moment of inertia are increased to appropriate equivalent values. In some cases when a detailed analysis is performed for the thermal stress, the cracks in the concrete and the effects of the steel materials are taken into account.

c. Damping constants

The values listed in Table 5.2.2-5 are currently used as the damping constants for the structures in response analyses. In the response analyses of the structure, it is as important as the stiffness evaluation to appropriately evaluate the damping properties of building and ground.



(a) Effect of age of concrete on compressive strength



(b) Effect of age of concrete on elastic coefficient

Figure 5.2.2-24. Effects of age of concrete on compressive strength and elastic coefficient (5.5.2-18).

Table 5.2.2-5. Damping constants (Standards, Construction-5 [HYO, KEN-5]).

Structural form	Damping constant h (elastic range)
Reinforced concrete structure	5%
PCCV	3%
Steel containment vessel	1%
Steel frame building structure	2%
Bolt-and-rivet joint structure	2%

From a physical standpoint, Taniguchi [5.5.2-19] classified the damping of structures as follows:

{1} Due to the radiation to the surrounding medium.
{2} Due to internal friction of the structural materials.
{3} Due to the consumption of energy through permanent deformation of the structural components.
{4} Due to friction among different parts or between the parts and other solids.
{5} Due to dispersion of elastic wave energy to the support material.

The damping of the structure is a synthesis of the above items, or several combinations of the aforementioned types. In particular, for a short-period structure such as the reactor building, the dissipation damping to the ground has a large effect. In addition, as the complicated structures such as reactor building are made of various materials and structural components, it is very important to select an appropriate evaluation method for the damping property in the seismic response analysis.

Due to the many experimental and analytical studies performed in the past, the mechanism and behavior have been gradually clarified. In the conventional analysis schemes, however, the above various damping effects are handled traditionally based on the concept, "the damping is expressed as a ratio to the critical damping for a mode (equivalent damping constant)." For example, Housner, Newmark et al. in USA have proposed various damping constants for different structures and stress levels as the damping values of nuclear power plants. In addition, the Nuclear Regulation Commi[illegible] NRC) in USA has defined the damping constants for different earthquake levels used in design (Operating Basis Earthquake (OBE), Safe Shutdown Earthquake (SSE)).

As examples of the designs of nuclear power plants in Japan, the damping constants for difference structures are listed in Table 5.2.2-5. These values roughly correspond to the data obtained in vibration tests, earthquake observation, shear wall structural tests, etc.

5.2.3 Interaction between structure/building and ground

(1) Analytical theory

a. Summary of the soil-structure interaction

A foundation supports the building, and, at the same time, acts as the medium for transferring the earthquake motion. The earthquake motion acts as an external disturbance on the building via the foundation. On the other hand, the building influences the vibration of the peripheral ground. In this way, the ground and the building affect each other's vibration patterns. Due to the dynamic interaction, the vibration of the building is transferred to the ground, causing damping of the vibration of the building. Hence, this mechanism is called soil dissipating effect or soil dissipating damping of the vibration energy (5.2.3-1) (see Figure 5.2.3-1).

The aforementioned coupled effects of building and ground can be roughly divided into the effect at the building foundation bottom portion and the effect at the underground outer side wall surface portion. Their contents are explained as follows:

(i) Effect at building foundation bottom portion

The foundation bottom portion plays the roles as the input surface of the earthquake motion transmitted from the lower layer, the primary resistant surface against the building's vibration, and the primary dissipating surface of the vibration energy.

(ii) Effect at the underground outer side wall surface portion

The underground outer side wall surface portion has the effect in increasing the input due to the soil pressure during an earthquake, as well as in increasing the horizontal resistance surface and the vibration energy dissipating surface.

In the analysis of soil-structure interaction, the effects of the resistance surfaces in above (i), (ii), and their effects as the vibration energy dissipating surfaces are evaluated using the various analytical models shown in Figure 5.2.3-2. In the following, we will present a schematic discussion on the theories of the above analytical models.

b. Semi-infinite elastic body theory

The soil springs can be determined from the relation between force and displacement, that is obtained from the elastic wave theory of the ground modeled as a semi-infinite elastic body. Several calculation methods are available for a structure lying on the ground including the ground compliance theory, vibration admittance theory, as well as other methods published in the foreign countries.

It is rather difficult to directly solve the elastic wave problem of the foundation mat-ground system. Hence, usually, the problem is solved as a stress boundary value problem under the assumption that there is a certain ground reaction distribution on the boundary between the formation mat and the ground. Usually, the ground reaction force distribution listed in Table 5-2.3-1 is adopted as the ground reaction force distribution. When the ground reaction force distribution is to be assumed, the displacement of the foundation mat is not uniform. The displacement and angle change (rotational angle) of the foundation mat listed in Table 5.2.3-1 are used for estimating the soil springs.

(a) Ground compliance theory

According to Kobori's ground compliance theory [5.2.3-2], when a dynamic force $Pe^{i\omega t}$ acting in vertical, horizontal, rotational, and torsion directions is applied on a rigid rectangular foundation mat on a semi-infinite elastic body, displacement W can be represented by the following equation:

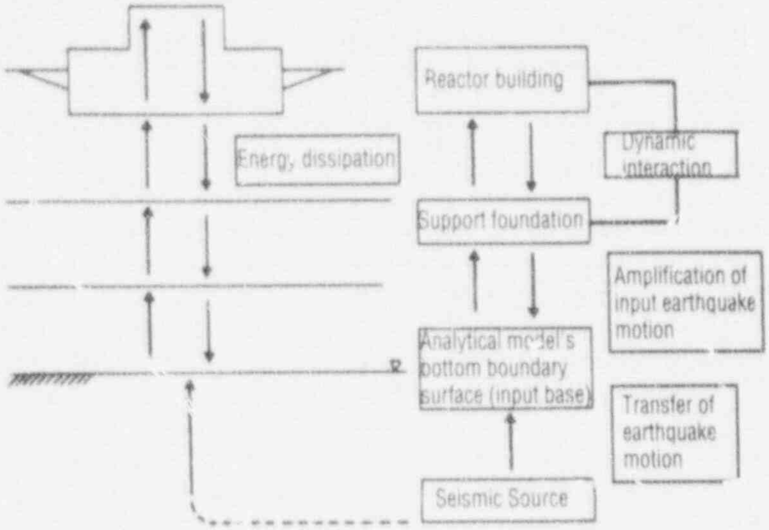


Figure 5.2.3-1. Mechanism of transfer of earthquake motion to building.

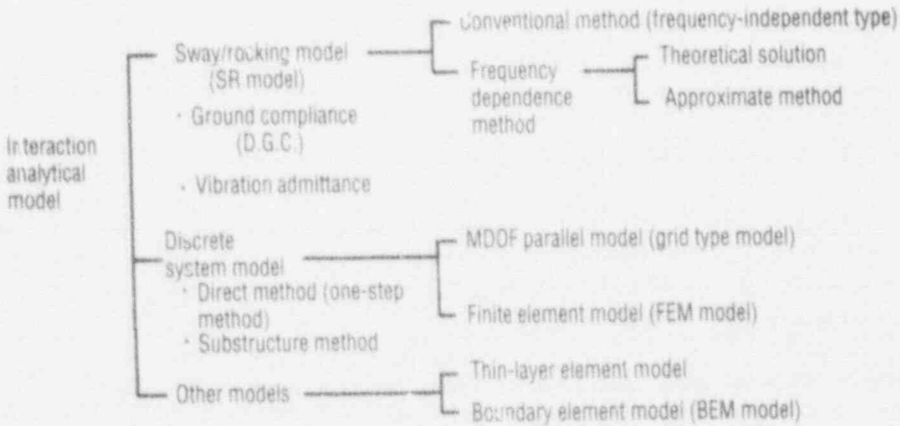


Figure 5.2.3-2. Analytical models of soil-structure interaction.

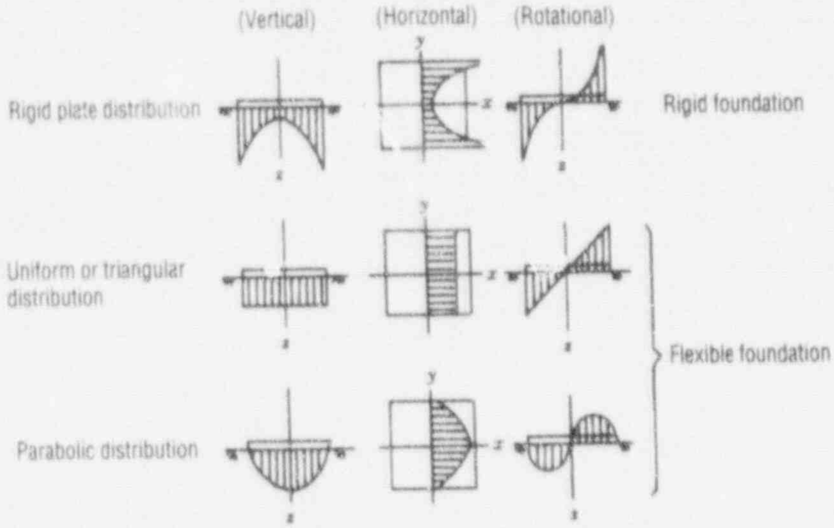
Table 5.2.3-1. Various schemes of analytical assumption, derivation method, and calculation method (Standards, Construction-5 [HYO, KEN-5]).

Item	Case			
	(1) Static spring on the base of ground compliance theory	(2) Static spring on the base of vibration admittance theory[1]	(3) Static spring on the base of vibration admittance theory[2]	(4) Static spring by Barkan et al.
Theory	Elastic theory	Elastic theory	Elastic theory	Elastic theory
Ground	Semi-infinite elastic body	Semi-infinite elastic body	Semi-infinite elastic body	Semi-infinite elastic body
Ground reaction distribution[2]	Vertical, horizontal: uniform distribution; Rotational, torsion: triangular distribution	Horizontal: uniform distribution; Rotational: triangular distribution	In each direction, any distribution patterns such as rigid plate distribution–uniform distribution (for vertical/horizontal), triangular distribution–parabolic distribution (for rotational), can be assumed	Horizontal: uniform distribution; Vertical, rotational, torsional: rigid plate distribution
Representative displacement[1]	Vertical (ω), horizontal (ω): displacement at the center of the foundation bottom; Rotational: [3]$\phi = u/b$ ($x=b$, $y=z=0$)	Horizontal: displacement at the center of the foundation bottom; Rotational: rotational angle at the center of the foundation bottom	Average displacement of various points on foundation bottom ($z=0$) Average rotational angle (arithmetic mean)	Vertical: displacement at the center of the foundation bottom; Horizontal: average displacement of the foundation bottom; Rotational: rotational angle at the center of the foundation bottom
Coefficients	$\tilde{J}_{SV}$, $\tilde{J}_{SH}$, $\tilde{J}_{SR}$, $\tilde{J}_{ST}$	α_H, α_R	α_V, $\bar{\alpha}_H$, $\bar{\alpha}_R$	β_X, β_Z, β_θ

[1]u = horizontal displacement in x-direction; v = horizontal displacement in y-direction; w = vertical displacement in z-direction.

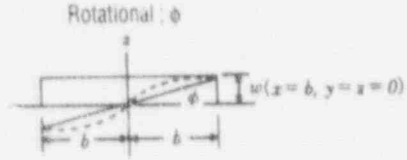
[2]Reaction force distributions.

Table 5.2.3-1 *(Cont'd)*. Various schemes of analytical assumption, derivation method, and calculation method (Standards, Construction-5 [HYO, KEN-5]).



(3) Typical displacement (ground compliance)

Rotational : φ



$$W = \frac{Pe^{i\omega t}}{dG}\{f_1(\omega) + if_2(\omega)\} \qquad (5.2.3\text{-}1)$$

where d = $\sqrt{bc}$; b, c: half the length of sides of the rectangular foundation mat shown in Figure 5.2.3-3; G: shear elastic coefficient of foundation.

In equation (5.2.3-1), $\{f_1(\omega) + if_2(\omega)\}/dG$ is the dynamical ground compliance (D.G.C). As it is the inverse of the spring constant, when D.G.C is replaced by equivalent spring constant K_{el} and dash-pot constant C_{el}, the following equation is obtained.

$$f_{1l}(\omega) + if_{2l}(\omega) = \frac{1}{K_{el} + ia_0 C_{el}} \qquad (5.2.3\text{-}2)$$

Hence,

$$\left.\begin{aligned} K_{el} &= \frac{f_{1l}}{f_{1l}^2 + f_{2l}^2} \\ C_{el} &= \frac{-f_{2l}}{a_0\left(f_{1l}^2 + f_{2l}^2\right)} \end{aligned}\right\} \qquad (5.2.3\text{-}3)$$

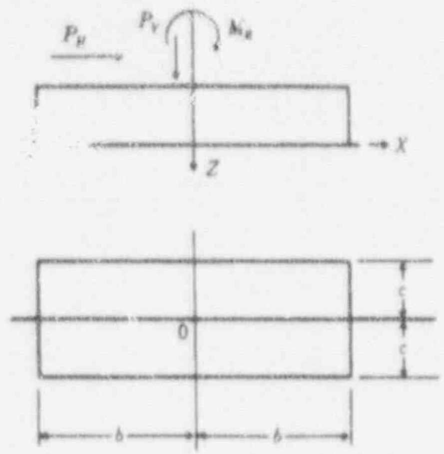


Figure 5.2.3-3. Shaking state [5.2.3-2].

where, subscript l indicates the shaking direction, such as vertical (V), horizontal (H), and rotational (R) directions.

Figure 5.2.3-4 shows $\bar{f}_1$ and $-\bar{f}_2$ in the case when the aspect ratio (c/b) is varied as the bottom area of the foundation mat is maintained constant. The abscissa represents the dimensionless frequency $\bar{a}_0 = \omega(\rho/G \cdot bc)^{1/2}$ (ρ = mass density).

The ground compliance when the frequency is 0 (zero) is called the statical ground compliance. Figures 5.2.3-5 shows the equivalent spring constant $\bar{K}_e$ and dash-pot constant $\bar{C}_e$ corresponding to $\bar{f}_1$ and $-\bar{f}_2$ shown in Figure 5.2.3-4. For the foundation spring shown in Figure 5.2.3-5, the value at frequency of 0 corresponds to a static spring represented by the following formula [5.2.3-2]:

$$\text{Vertical direction:} \quad K_V = \frac{G\sqrt{bc}}{\bar{f}_{SV}}$$

$$\text{Horizontal direction:} \quad K_H = \frac{G\sqrt{bc}}{\bar{f}_{SH}}$$

$$\text{Rotational direction:} \quad K_R = \frac{G\left(\sqrt{bc}\right)^3}{3\bar{f}_{SR}}$$

(b) Vibration admittance theory

According to Tajimi's vibration admittance theory, as a dynamic force $Pe^{i\omega t}$ in the vertical, horizontal, and rotational directions, is applied to a foundation mat surface on a semi-infinite elastic body, the corresponding displacement W can be expressed as follows [5.2.3-4].

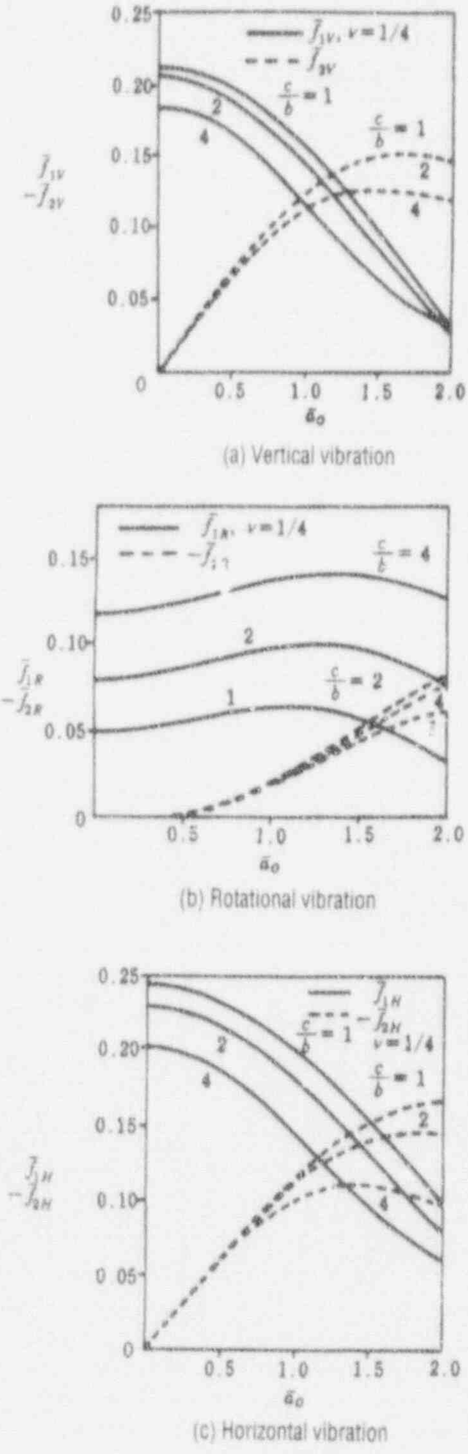
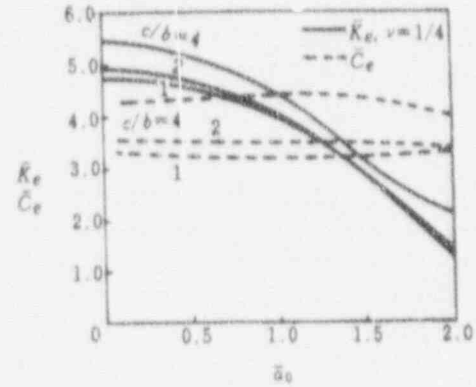
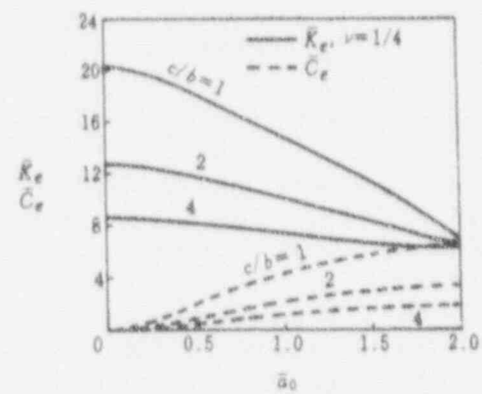


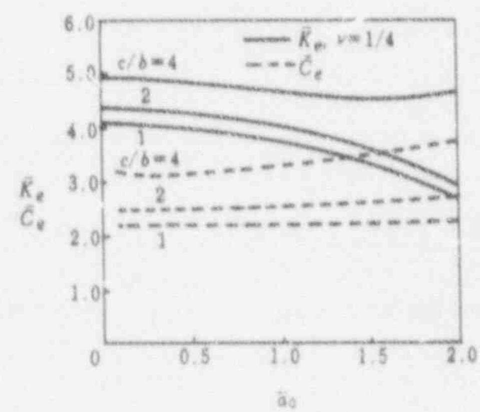
Figure 5.2.3-4. Dynamical ground compliance [5.2.3-2].



(a) Vertical vibration



(b) Rotational vibration



(c) Horizontal vibration

Figure 5.2.3-5. Equivalent spring, equivalent damping coefficients [5.2.3-2].

$$W = \frac{Pe^{i\omega t}}{K_f}\{g_1(\omega) - ig_2(\omega)\} \qquad (5.2.3\text{-}5)$$

K_f: static soil spring constant

The proportional coefficient $\{g_1(\omega) - ig_2(\omega)\}$ is called vibration admittance. Figures 5.2.3-7(a), (b) show the $g_1(\omega)$ and $g_2(\omega)$ of a rectangular foundation mat when the ground reaction force distribution are $B_V/(R^2-S^2)^{1/2}$ and $B_M S \cos\theta/(R^2-S^2)^{1/2}$, where B_V and B_M are coefficients derived from the relations with axial force P and moment M; R, S represent the plan lengths of the rectangular foundation mat shown in Figure 5.2.3-6.

As the foundation stiffness is the inverse of the vibration admittance, the following equation can be derived from equation (5.2.3-5):

$$K(\omega) = K_f \frac{1}{g_1(\omega) - ig_2(\omega)} \qquad (5.2.3\text{-}6)$$

In the vibration admittance theory, $K(\omega)$ is called the dynamic resistance coefficient.

The aforementioned analysis is performed with respect to the changes in the shape of the foundation mat and the various constants of the foundation. Therefore, it may be more practical to formulate diagrams and tables as functions of the dimensionless frequency a_0 $(= \omega\sqrt{A} / V_S)$.

Equation (5.2.3-7) shows the formulas for calculating the foundation spring with respect to vertical, horizontal, and rotational motions of a square foundation mat. Figures 5.2.3-8 (a), (b), (c) are diagrams for calculating them. These calculation formulas and diagrams are formulated from the data obtained by calculating the foundation stiffness of square foundation mats with side lengths from 30 m to 75 m with assumption of a uniform distribution of ground reaction force with respect to the vertical motion and the horizontal motion, and a triangular distribution with respect to the rotational motion. They are used for practical applications, in which the foundation stiffness is derived using the bottom area (A), second moment of area of the foundation mat, as well as the shear elastic coefficient (G) and Poisson's ratio (ν) of the ground.

$$\left.\begin{aligned}
&\text{Vertical direction:} && K_{V1} + iK_{V2} = \frac{G\sqrt{A}}{1-\nu}(\alpha_{V1} + i\alpha_{V2}) \\
&\text{Horizontal direction:} && K_{H1} + iK_{H2} = G\sqrt{A}(\alpha_{H1} + i\alpha_{H2}) \\
&\text{Rotational direction:} && K_{R1} + iK_{R2} = \frac{GZ_y}{1-\nu}(\alpha_{R1} + i\alpha_{R2})
\end{aligned}\right\} \qquad (5.2.3\text{-}7)$$

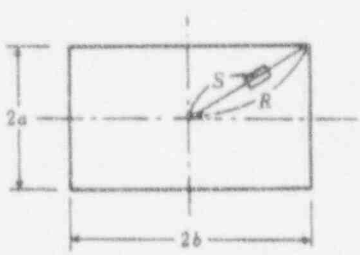


Figure 5.2.3-6. Plan of rectangular foundation mat.

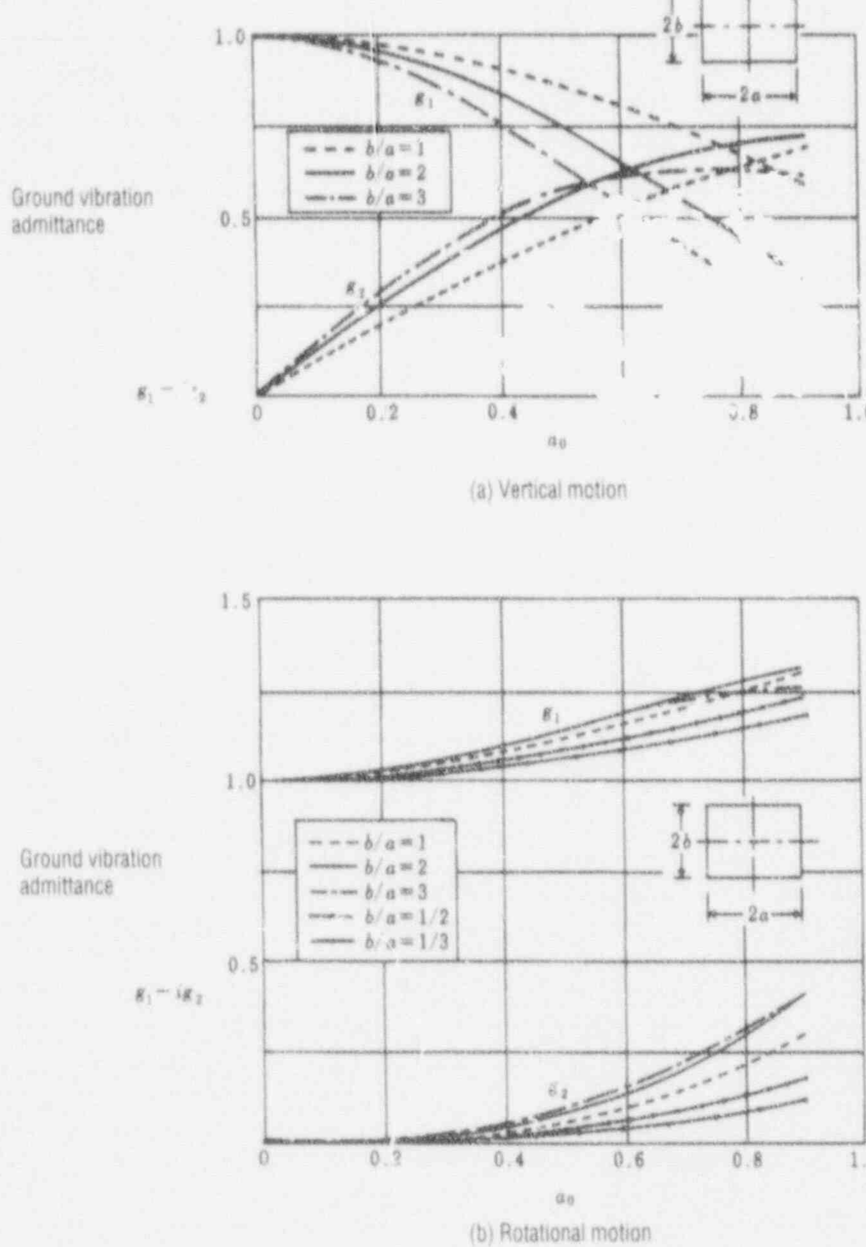
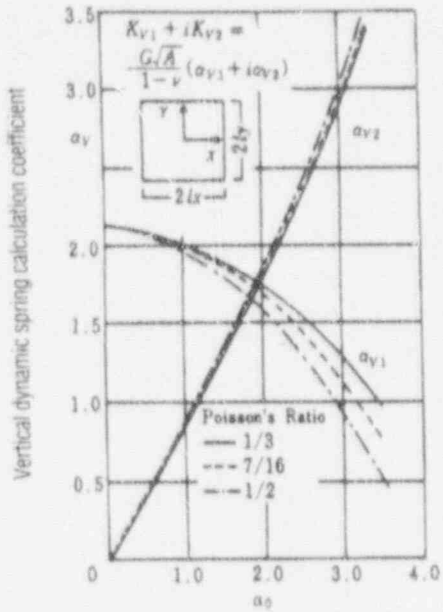
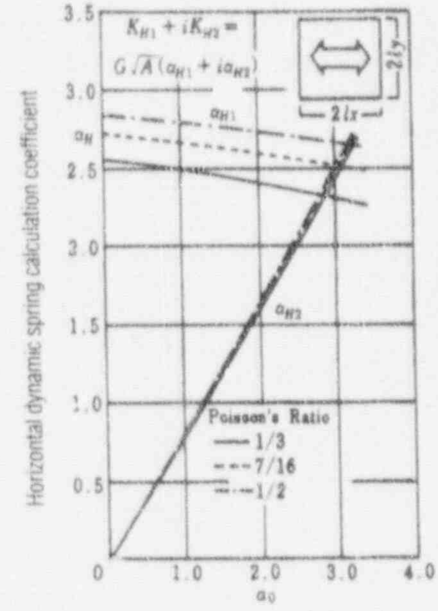


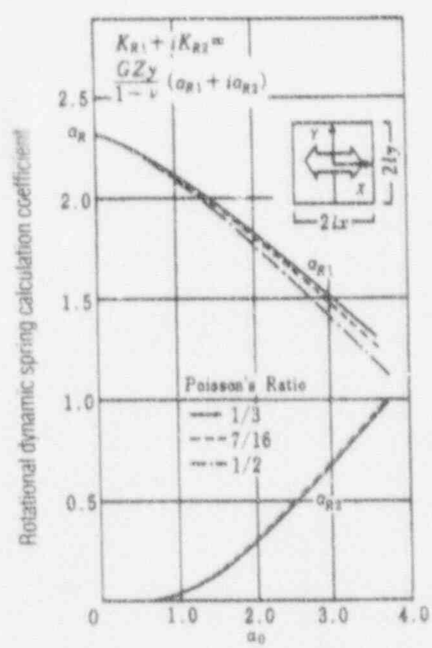
Figure 5.2.3-7. Vibration admittance of rectangular foundation [5.2.3-4].



(a) Vertical direction



(b) Horizontal direction



(c) Rotational direction

Figure 5.2.3-8. Dynamic spring calculation coefficient of square foundations.

1

IMAGE EVALUATION
TEST TARGET (MT-3)

1.0 1.1 1.25 1.4 1.6 1.8 2.0 2.2 2.5 2.8 3.2 3.6 4.0

150mm

6"

1

IMAGE EVALUATION
TEST TARGET (MT-3)

1.0 1.1 1.25 1.4 1.6 1.8 2.0 2.2 2.5 2.8

150mm

6"

1

IMAGE EVALUATION TEST TARGET (MT-3)

150mm

6"

1

IMAGE EVALUATION TEST TARGET (MT-3)

150mm

6"

In equation (5.2.3-7), when $a_0 = 0$, i.e., when the frequency is 0, the values correspond to the static foundation stiffness, which can be expressed as follows when the size of the foundation mat is $2l_x \times 2l_y$:

$$\left.\begin{aligned} &\text{Vertical direction:} \quad K_V = \frac{G\sqrt{A}}{1-\nu}\,\alpha_V \\ &\text{Horizontal direction:} \quad K_H = G\sqrt{A}\,\alpha_H \\ &\text{Rotational direction:} \quad K_R = \frac{\pi G Z_y}{1-\nu}\,\alpha_R \end{aligned}\right\} \qquad (5.2.3\text{-}8)$$

where

$$\left.\begin{aligned} \sqrt{A} &= 2\sqrt{l_x l_y} \\ Z_y &= \frac{2l_y(2l_x)^2}{6} \end{aligned}\right\} \qquad (5.2.3\text{-}9)$$

For further details concerning coefficients α_V, α_H, and α_R, please see references [5.2.3-1, 5.2.3-5, 5.2.3-6].

(c) Other theories

Barkan and Gorbunov-Passadov have derived formulas for calculating the foundation stiffness of rectangular foundation mats with respect to horizontal and rotational motion. For a foundation mat of size 2c x 2d, these calculating formulas become the following forms [5.2.3-7, 5.2.3-8]:

$$\left.\begin{aligned} &\text{Vertical direction:} \quad K_z = \frac{G}{1-\nu}\,\beta_z\sqrt{4cd} \quad \text{[Barkan]} \\ &\text{Horizontal direction:} \quad K_x = 4(1+\nu)G\beta_x\sqrt{cd} \quad \text{[Barkan]} \\ &\text{Rotational direction:} \quad K_\theta = \frac{G}{1-\nu}\,\beta_\theta 8cd^2 \quad \text{[Gorbunov-Possadov]} \end{aligned}\right\} \qquad (5.2.3\text{-}10)$$

c. Discrete system models

The ground compliance theory and vibration admittance theory shown in the above section can be applied to a uniform ground that can be assumed as a semi-infinite elastic body. However, application of these theories becomes difficult when the shape and composition of the surrounding ground becomes complex. Discrete system models, however, are effective in analyzing complex ground conditions and in evaluating the embedment effect. The discrete system models include the FEM model and MDOF parallel model. The primary features of these models are as follows.

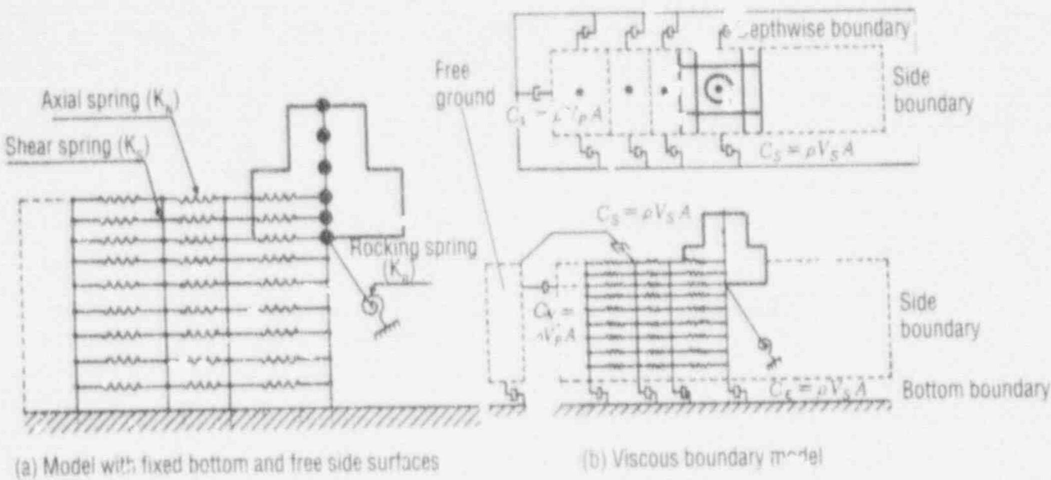


Figure 5.2.3-9. MDOF parallel ground model.

(a) Multi-degree-of-freedom (MDOF) parallel ground model

In this model, as shown in Figure 5.2.3-9(a), the foundation around the building is idealized and represented by a number of soil columns with only the horizontal degree of freedom, and the bottom of the foundation mat of the building is supported by rotational springs in this MDOF system model. Each soil column is modeled by a shear spring (K_S) and axial spring (K_N) [5.2.3-9, 5.2.3-10].

As far as the boundary conditions of the model are concerned, in Figure 5.2.3-9(a), the bottom is fixed, while the side surface is free; in Figure 5.2.3-9(b), the boundaries of the side surface and the depthwise side surface may be viscous boundaries expressed by the following equations. At the viscous boundaries, it is possible to introduce the energy dissipating effect in proportion to the relative velocity of the various nodes at the boundaries with respect to the free soil and the fixed bottom of the model.

$$\left.\begin{aligned} \text{Bottom boundary:}\quad & C_\epsilon = \rho V_S A \\ \text{Side surface boundary:}\quad & C_V = \rho V_P A \\ \text{Depth-direction boundary:}\quad & C_S = \rho V_S A \end{aligned}\right\} \qquad (5.2.3\text{-}11)$$

ρ: density of ground; A: tributary area of each damper C; V_S, V_P: S-wave velocity and P-wave velocity of ground.

(b) FEM model

The basis of the analysis theory usually is the frequency response (stationary response) of a FEM model. Here, we will only discuss some fundamental items. As shown in Figure 5.2.3-10, in this method, the model of the ground and structure is formed by using 2-dimensional or 3-dimensional finite elements. At the boundaries on the side surface and bottom, in consideration of the dissipation of the wave energy, the following viscous boundary proposed by Lysmer et al. [5.2.3-11] is adopted.

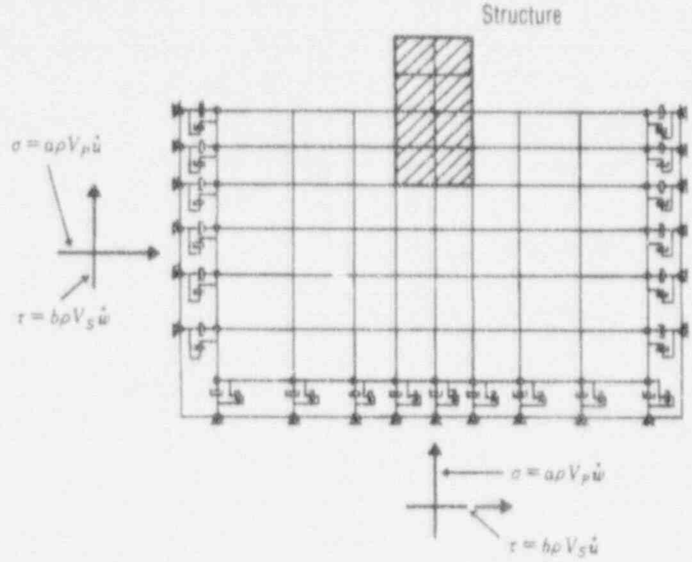


Figure 5.2.3-10. FEM model.

- Bottom boundary: Standard Viscous Boundary (referred to as S.V.B. hereinafter)

$$\left.\begin{aligned} \sigma &= a\rho V_p\dot{W} \quad (a = 1.0) \\ \tau &= b\rho V_S\dot{U} \quad (b = 1.0) \end{aligned}\right\} \tag{5.2.3-12}$$

- Side-surface boundary: S.V.B. or Rayleigh Wave Boundary (referred to as R.W.B. hereinafter)

$$\left.\begin{aligned} \sigma &= a\rho V_p\dot{U} \\ \tau &= b\rho V_S\dot{W} \end{aligned}\right\} \tag{5.2.3-13}$$

In the case of R.W.B., a, b are values depending on Poisson's ratio, depth, frequency, etc.; ρ is the density of ground; V_S, V_P are the S-wave and P-wave velocities; U, W are the horizontal and vertical displacements at the boundary, respectively. S.V.B. is a boundary absorbing the energy of the body wave; R.W.B is a boundary absorbing the energy of the surface wave (Rayleigh wave). They are mechanically equivalent to dampers.

(2) Analytical methods

a. Sway/rocking model

As shown in Figure 5.2.3-11, the ground is replaced by sway/rocking springs. This method is known as a sway/rocking model, or simply an SR model. In this model, the foundation is idealized to be a uniform semi-infinite elastic body. This method is used in calculating the foundation stiffness using theoretical formulas. It has been used in many actual examples in design and research. Recently, on the basis of the existing analytical and experimental research results, the soil spring has been investigated in an effort to find a rational yet simple method for evaluating them in the aseismic design of reactor buildings.

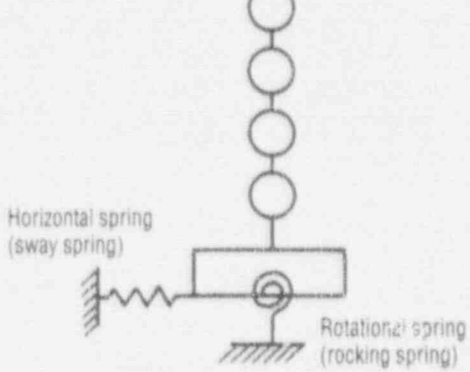


Figure 5.2.3-11. SR model.

(a) Conventional method

As shown in Figure 5.2.3-12, the foundation's dynamic resistance coefficient derived from ground compliance theory and vibration admittance theory is a function of frequency ω, and can be represented by a complex stiffness $_RK(\omega) + i_IK(\omega)$ (where $_RK$ is the real number portion, and $_IK$ is the imaginary portion). In Figure 5.2.3-12, ω_1 indicates the fundamental vibration frequency of the coupled system of the soil and building. However, if the expression $(_RK(\omega) + i_IK(\omega))$ is used directly in response analysis and eigenvalue analysis, it becomes complicated. Hence, the soil springs are usually derived using the following approximate method. This method is usually called the "conventional method."

Several methods can be used to calculate the soil springs that are independent of the frequency. In one method, as shown in Figure 5.2.3-12, the value at $\omega = 0$, i.e., $_RK(0)$, is derived; in another method, the calculation formulas (5.2.3-4 through 10) presented in the preceding section are used, and $_RK(\omega_1)$ and $_IK(\omega_1)$ are derived when $\omega = \omega_1$ in Figures 5.2.3-12. For the damping constants, in the former case, the conventionally used constant value of 5% or 10% is used for the horizontal and rotational components, irrespective of the base conditions; in another scheme, the values listed in Table 5.2.3-2 for the S-wave velocity V_S (m/s) of the ground are used. Also, when $_RK(\omega_1)$ and $_IK(\omega_1)$ are used for the complex stiffness, the damping constants can be derived using the following formulas:

$$\left.\begin{aligned} &\text{Horizontal component:} \quad h_S = {}_IK_S(\omega_1)/2\,{}_RK_S(\omega_1) \\ &\text{Rotational component:} \quad h_R = {}_IK_R(\omega_1)/2\,{}_RK_R(\omega_1) \end{aligned}\right\} \qquad (5.2.3\text{-}14)$$

(b) Frequency dependent method

This is an analytical method which considers the frequency dependence of soil spring and damping on the basis of elastic wave theory. This method includes two schemes: a theoretical solution method, in which the theoretical solution is used directly, and an approximate method. They can be selected according to the purpose (Standards, Construction-5 [Hyo, Ken-5], Research, Construction-13 [Ken, Ken-13]). According to this method, evaluation of the stiffness and damping between the ground and the building foundation's bottom, and its application in earthquake response analysis are determined using one of the following schemes:

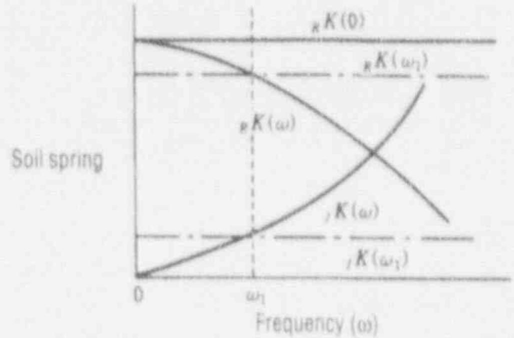


Figure 5.2.3-12. Soil spring (dynamic resistance coefficient).

Table 5.2.3-2. Damping constants of ground (Standards, Construction-5 [Hyo Ken-5]).

Component	Ground condition (V_S) (m/s)		
	500	1,000	1,500
Horizontal h_S (%)	30	20	10
Rotational h_R (%)	10	7.5	5

(i) The soil stiffness is calculated using the theoretical solution based on elastic wave theory (complex dynamic resistance coefficient depending on frequency ω). For earthquake response analysis, the theoretical solution is directly used in the frequency domain.

(ii) As an approximate method of scheme (i), the soil stiffness is calculated according to the following items {1}, {2}, {3}, and the earthquake response analysis is performed using soil springs in the frequency domain or time domain.

{1} The horizontal and rotational components of the soil springs ($\bar{K}_S$, $\bar{K}_R$), are represented by the static theoretical solutions of the elastic wave theory with frequency $\omega = 0$.

{2} The damping constants (h_{S1}, h_{R1}) of the horizontal and rotational components of the soil springs corresponding to the fundamental frequency (ω_1) of the soil/building coupled system are calculated using equation (5.2.3-15). Equation (5.2.3-15) has been obtained from a regression analysis on eigenvalue analyses in terms of frequency and damping, where the foregoing vibration admittance theory was applied to evaluate the soil springs of a typical reactor building with three soil conditions (V_S = 500, 1000, 1500 m/s).

$$\left.\begin{aligned} h_{S1} &= 17a_0 - 2 \\ h_{R1} &= 10a_0 - 10 \end{aligned}\right\} \qquad (5.2.3\text{-}15)$$

where $1.0 < a_0 < 3.0$

a_0: dimensionless frequency ($\omega_1\sqrt{A}/V_S$)

h_{S1}, h_{R1}: horizontal and rotational components of soil damping constant with respect to the fundamental frequency of the coupled system (%)

V_S: S-wave velocity of the foundation (m/s)

A: the foundation mat area (m^2)

ω: frequency (rad/s)

ω_1: nondamped fundamental frequen of soil/building coupled system (rad/s)

{3} For the damping constants (h_S, h_R) of the soil spring, equation (5.2.3-16) is used as a linear approximation.

$$\left.\begin{aligned} h_S(\omega) &= \frac{h_{S1}}{\omega_1}\omega \\ h_R(\omega) &= \frac{h_{R1}}{\omega_1}\omega \end{aligned}\right\} \qquad (5.2.3\text{-}16)$$

$h_S(\omega)$, $h_R(\omega)$: damping constants of the horizontal and rotational components of the soil springs.

According to the aforementioned method, the viscous damping coefficient and complex stiffness can be derived using equations (5.2.3-17) and (5.2.3-18), after the spring and damping constants of the ground have been calculated.

$$\left.\begin{aligned} C_s &= \frac{2h_{S1}}{\omega_1}\bar{\bar{K}}_S \\ C_R &= \frac{2h_{R1}}{\omega_1}\bar{\bar{K}}_R \end{aligned}\right\} \qquad (5.2.3\text{-}17)$$

C_S: horizontal component of viscous damping coefficient
C_R: rotational component of viscous damping coefficient

$$\left.\begin{aligned} K_S &= \bar{\bar{K}}_S\left(1 + i2h_{S1}\frac{\omega}{\omega_1}\right) \\ K_R &= \bar{\bar{K}}_R\left(1 + i2h_{R1}\frac{\omega}{\omega_1}\right) \end{aligned}\right\} \qquad (5.2.3\text{-}18)$$

b. Discrete system model

(a) Multi-degree-of-freedom (MDOF) parallel model

For the purpose of modeling, the following methods are adopted to determine the number, width, and the layer thickness of soil columns, stiffness and damping of the ground, etc. [5.2.3-10]. Figure 5.2.3-13 shows an example of the case where the foundation mat width is 81 m.

(i) Number of soil columns

The number of soil columns should be determined in consideration of the layering and geologic conditions, size of foundation mat, and the backfill status of the building's periphery. In the previous research and analytical examples, as shown in Figure 5.2.3-14, 4 to 5 soil columns are used around the central axis of the building, each representing the ground portion right below the building, as well as in the nearby, medium, and distant soil.

(ii) Soil column width

The soil column width is determined such that a soil mass with a width 4-5 times the foundation mat width B and with a depth about 1-2 times B can be divided into 4-5 soil columns. That is, with the soil column width right below the building as the foundation mat width, the width of the remaining soil columns is made as small as possible depending on the properties of the ground right below or near the building. However, as the position becomes farther away from the building, the influence of the building on the ground decreases, and the soil column width can be increased.

(iii) Layer thickness of soil columns

The layer thickness in a soil column in the vertical direction should be determined by considering the required frequency range, using the following equation as a rule of thumb. In determining the layer thickness, if viscous boundaries are used, it is possible to decrease the above (1-2 times)B by about 50%.

$$f_{max} = \frac{V_S}{(4 \sim 5)H_{min}} \qquad (5.2.3\text{-}19)$$

where f_{max}: maximum frequency (Hz); H_{min}: minimum thickness of a layer (m); V_S: S-wave velocity of the ground (m/s). For example, in the response analysis, if analysis can be performed accurately up to about 20-25 Hz, by substituting f_{max} = 25 Hz into equation (5.2.3-19), the following equation can be obtained:

$$H_{min} = \frac{V_S}{100 \sim 125} \qquad (5.2.3\text{-}20)$$

(iv) Stiffness and damping of soil

After the soil column width and the layer thickness have been determined as above, the shear stiffness (K_S) and axial stiffness (K_N) can be evaluated using the following method, where the axial spring is evaluated by assuming it as a plane strain problem.

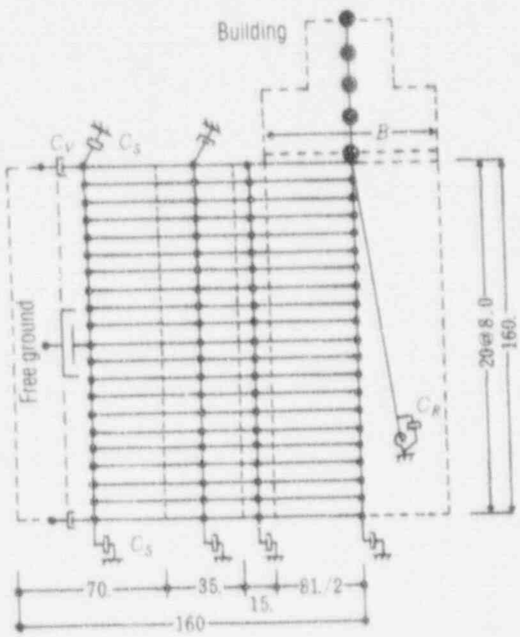


Figure 5.2.3-13. MDOF parallel model (example).

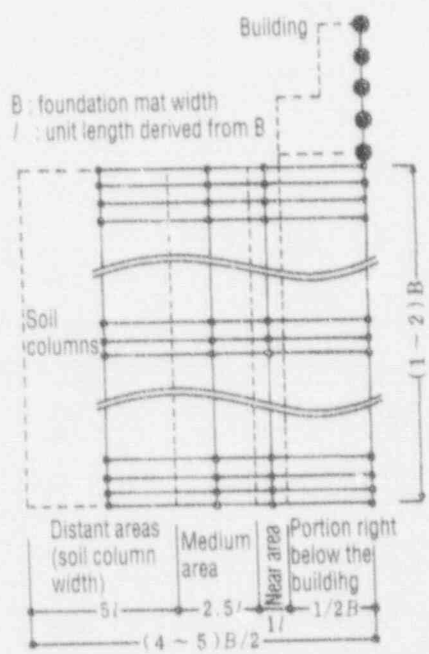


Figure 5.2.3-14. Basic factors for formation of model.

$$\left.\begin{aligned} K_S &= \frac{GDL_E}{H_E} \quad \text{(tf/cm)} \\ K_N &= \frac{EH_E D}{(1-\nu^2)L_E} \quad \text{(tf/cm)} \end{aligned}\right\} \qquad (5.2.3\text{-}21)$$

E, G, ν: Young's modulus (tf/cm^2), shear elastic coefficient (tf/cm^2), and Poisson's ratio of soil. These elastic coefficients can be evaluated using equation (5.2.2-1).

H_E, L_E, D: Layer thickness (cm), soil column width (cm), and length of building (cm), as shown in Figure 5.2.3-15.

For the damping of shear spring and axial spring, a certain constant value independent of frequency is adopted as the internal damping of the ground; the specific value of the damping constant is set in consideration of the properties of the ground and the design conditions. For the rotational spring, the method using the static analysis results of a 3-dimensional FEM model and the method using the theoretical or approximate solution of the SR model are usually used. As far as the damping is concerned, the damping constant in the SR model is being used.

(b) FEM model

When the ground is modeled by the FEM, the 3-dimensional model is more accurate and it is in better agreement with the theoretical solution. However, the 3-dimensional model requires a very long computing time. Hence, a pseudo-3 dimensional FEM model, with the foundation mat width in the depth direction (the direction perpendicular to the shaking direction) taken as a single element and the boundary in the said direction evaluated as a viscous boundary, is usually used as the analytical model for practical applications. Figure 5.2.3-16 shows an example of the 2-dimensional FEM model with a foundation mat width of 81 m. In this case, the elastic coefficients of the soil are evaluated by equation (5.2.2-1), and used for various elements.

In order to analyze the earthquake response for the soil/structure coupled system using the FEM model, the through scheme shown in Figure 5.2.3-16 in which the entire system is handled as a single body may be used. The substructuring method in which the building and base are separated at the ground boundary may also be used. According to the substructuring method, the impedance of the ground, including the effect of interaction with the bottom of the foundation mat and the embedment effect of the building, is calculated beforehand, and the results are coupled to the upper building.

Table 5.2.3-3 lists the features of the said two methods. Hence, it is required to select the method appropriately according to the purpose of the response analysis. In the following, we will discuss several factors for formation of the FEM model as shown in Figure 5.2.3-17, such as the model depth, its horizontal length, size of the elements when the ground is divided by a grid, etc.

(i) Depth from bottom of building to bottom boundary of the analytical model

The depth (H) from the building bottom to the bottom boundary of the analytical model depends on the requirement of the accuracy in calculating frequency. According to the existing references [5.2.3-18], the effect on the response is small for H of about 1/2 wavelength in the case of a fixed boundary and for H of about 1/10 wavelength in the case of a viscous boundary. However, it is appropriate to consider these values as the minimum values for the depth from the bottom of the building to the bottom boundary of the analytical model. Based on this consideration, suppose it is required to ensure accuracy in the region over 5 Hz the values of H listed in Table 5.2.3-4 can be used for the three types of ground conditions (V_S).

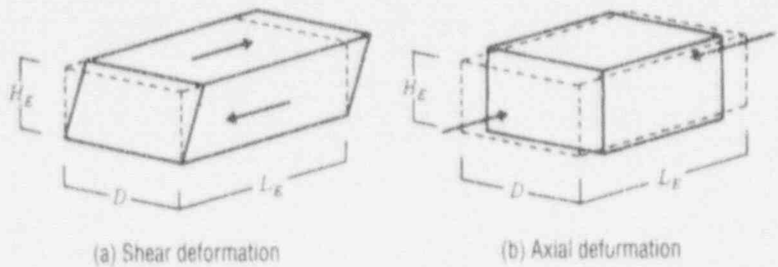


Figure 5.2.3-15. Evaluation of shear spring and axial spring.

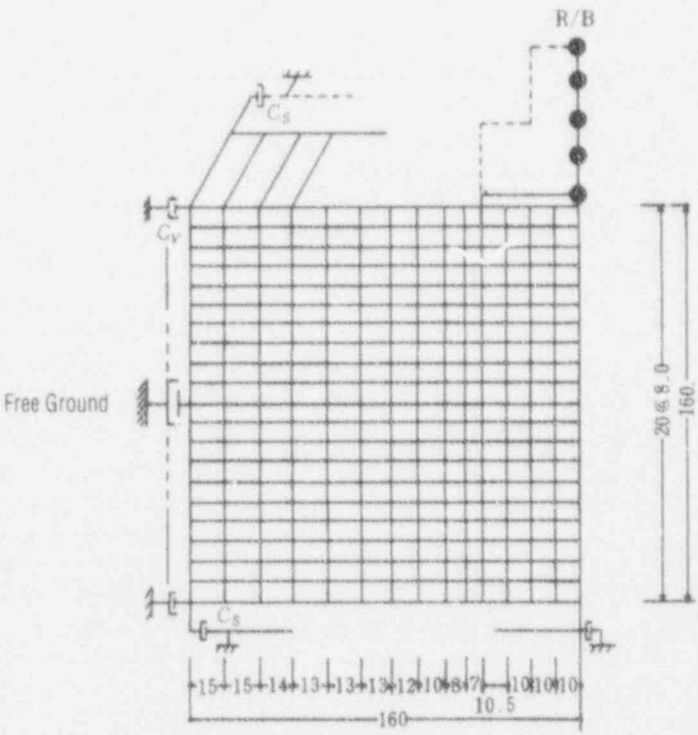


Figure 5.2.3-16. Example of FEM model.

Table 5.2.3-3. Comparison between conventional method and substructuring method [5.2.4-15].

Evaluation item	Analytical method	
	Conventional method	Substructuring method
Calculation cost	For each analytical case, repetitive calculation is performed for the entire system	As independent calculation can be performed for each step, there is no repetitive calculation
Formation of model for building	Model can be made using 2-dimensional frame elements	Precise models, such as three-dimensional model, are possible
Nonlinear analysis	Nonlinear analysis of foundation and building is possible	Nonlinear interaction analysis with the ground is impossible
3-dimensional analysis of ground	Pseudo-3-dimensional analysis is adopted	3-dimensional analysis is possible for both input and ground
Evaluation of input	Separation from the response analysis step is impossible	Input can be handled independently
Evaluation of soil stiffness	Same as above	It is possible to evaluate the soil stiffness independently
Evaluation of interaction	Same as above	It is possible to independently evaluate the interaction force
Adaptability	Combination with the other analytical method is limited	Can be combined with other analytical methods

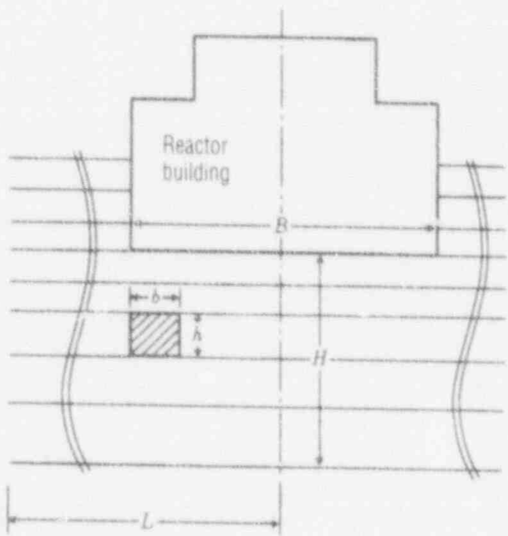


Figure 5.2.3-17. Basic factors in formation of FEM model.

Table 5.2.3-4. V_s vs. H.

Ground conditions V_s (m/s)	Wavelength with respect to f_L ($\lambda L = V_s / f_L$)	Viscous boundary		Fixed boundary	
		H(m) = λL / 10	H / B [(1)]	H(m) = λL / 2	H / B [(1)]
500	100	10	0.125	50	0.625
1,000	200	20	0.250	100	1.250
1,500	300	30	0.375	150	1.870

[(1)]Foundation mat width B = 80 m.

The results indicate that [the value of H] is up to about half the foundation mat width for a viscous boundary, and up to about twice the foundation mat width for a fixed boundary at most. Hence, when a model is chosen regardless of the ground conditions and boundary conditions, the value of H may be set at about twice the foundation mat width. Given this value of H, for the fundamental mode, it is possible to evaluate the energy dissipation effect even if fixed boundaries are assumed. For the higher-order modes, however, because reflection of energy takes place, generally speaking, adoption of a viscous boundary is appropriate.

(ii) Horizontal length of ground in model

The analytical examples up to now have indicated that in a viscous boundary model, if the horizontal length of the ground (2L) is large enough to contain one wavelength (λ) at a low frequency (f_L) in analysis, the precision of the analysis can be guaranteed for meeting the requirement of practical applications. As an example, suppose f_L = 5 Hz. Table 5.2.3-5 lists the values of 2L derived for 3 soil conditions. Judging from the above results, as a rule of thumb, the horizontal length should be in the range of 2-4 times the foundation mat width.

(iii) Dimensions of elements

When the ground and structure are divided into elements for analysis, the dimensions of the elements, for example the width (b) and height (h) in the case of rectangular elements, are related to the accuracy at a higher frequency (f_H) required for the analysis. In addition, in response analysis, the height has a larger influence on the analytical results than the width.

In consideration of the aforementioned features, Lysmer proposed that the height be about 1/3-1/12 the wavelength (λ), or about 1/5 according to the existing empirical data. Table 5.2.3-6 lists the dimensions for 3 soil conditions with a high frequency, such as 25 Hz, as a reference. In addition, as far as the width is concerned, in consideration of the balance between width and height as well as the relation with the foundation mat, it may be made smaller near the building foundation mat, and then larger farther away from the foundation mat.

c. Other ground models

Recently, a thin-story-element model and a boundary element method (BEM) have been proposed as analytical models for overcoming the shortcomings of the SR model and the discrete system model. In many cases, these analytical models are used in studying the earthquake motion incidence problem and impedance problem as well as in simulation analysis of vibration experiment results and earthquake observation results. For further details of these analytical models, please see the related references [5.2.3-14-17].

Table 5.2.3-5. V_S vs. 2L.

Soil conditions V_S (m/s)	Wavelength with respect to f_L $\lambda L = V_S / f_L$	$2L / B$ [1] $(2L / \lambda L)$
500	100	1.25
1,000	200	2.50
1,500	300	3.75

[1]Foundation mat width B = 80 m.

Table 5.2.3-6. V_S vs. h.

Soil condition V_S (m/s)	Wavelength with respect to f_H $(\lambda_H(m) = V_S / f_H)$	Height of one element (h)	
		1/8 ~ 1/12	1/5
500	20	2.5 ~ 1.7	4
1,000	40	5.0 ~ 3.3	8
1,500	60	7.5 ~ 5.0	12

d. Features of analytical models

Table 5.2.3-7 lists the features and limitations in application of the various analytical methods. In order to analyze the earthquake response, it is important to select an appropriate model for the design and analysis, with the features of the analytical models taken into consideration.

5.2.4 Linear earthquake response analysis

(1) Modeling of building/structure

a. Guidelines for modeling

The structures of reactor facilities are typical short-period structures. They are composite structures made of various structures having different structural forms and materials. Hence, the characteristics of these structures must be taken into consideration in setting up the vibration system model and for evaluating the mass, stiffness, damping, etc. The vibration system models include the single cantilever model with mass concentrated on the various floors, and the multicantilever model in which the major structures are treated independently. In addition, it is also possible to use the three-dimensional FEM model which can form a 3-D model for the roof slab, foundation mat, major structural walls, and floor panels.

Table 5.2.3-7. Features of soil-structure interaction models.

<table>
<tr><th colspan="3">Types of analysis models</th><th>Features</th><th>Limitation in application</th></tr>
<tr><td rowspan="3">Sway/rocking model</td><td colspan="2">Conventional method</td><td>The soil spring can be calculated in a relatively simple way using the static calculation formula of the elastic wave theory. The damping constant of the soil is set for each V_S independent of the frequency.</td><td>Although the response analysis results are on the safe side, the damping constants do not agree well with the theoretical solution and the actual phenomenon.</td></tr>
<tr><td rowspan="2">Frequency dependent model</td><td>Method 1 Theoretical method</td><td>When there is no embedment, 3-dimensional analysis of the foundation can be performed using the ground compliance theory and the vibration admittance theory. The theoretical values of the foundation's stiffness and damping can be calculated, with a good agreement with the actual phenomenon.</td><td>When analysis includes the embedment effect, in addition to the ground compliance theory and vibration admittance theory, it is required to evaluate the stiffness and damping of the foundation portion around the side wall by other methods.</td></tr>
<tr><td>Method 2 Approximate method</td><td>The soil spring is evaluated using the static formula of elastic wave theory; the damping is evaluated approximately from the theoretical values. This method is an approximate method for practical application on the basis of the theoretical solution.</td><td>When analysis includes the embedment effect, it is required to evaluate the stiffness and damping of the foundation portion around the side wall by other methods. The grounds suitable for this method are primarily semi-infinite uniform grounds. However, under certain limited conditions, it may also be used for a layered ground.</td></tr>
<tr><td rowspan="2">Discrete model</td><td colspan="2">MDOF parallel mode</td><td rowspan="2">The model can be formed in consideration of the embedment effect, properties of layer of the soil, irregularity of the topology, and other conditions of the soil. The 3-dimensional effect can be evaluated approximately by assuming viscous boundaries for the bottom surface, side surfaces and orthogonal side surfaces of the analytical model.</td><td>The rotational spring and damping must be calculated using the ground compliance theory and the vibration admittance theory.</td></tr>
<tr><td colspan="2">FEM model</td><td>Analysis by finite element model requires a rather long computer time compared with grid-type model. As the 3-dimensional analysis requires a very long time, usually a pseudo-3-dimensional analysis is carried out.</td></tr>
<tr><td rowspan="2">Other models</td><td colspan="2">Thin-layer model</td><td>It is possible to evaluate an infinite region by applying 3-dimensional wave theory for the periphery of the building. Compared to the FEM model, the calculation time and data gathering time are shorter.</td><td>When the model of an irregular soil is to be formed, it is required to use engineering judgment to replace it by an equivalent layered soil.</td></tr>
<tr><td colspan="2">BEM model</td><td>Because the basic solutions contain an evaluation of an infinite region, there is no need to build a model with transfer boundaries and viscous boundaries as required with FEM. In a uniform foundation, it is possible to handle the evaluation of the embedment effect of the building as a 3-dimensional problem.</td><td>As the basic solutions include evaluation of an infinite region, there exist limitations in formation of a model with respect to the variation in the type of terrain and the nonuniformity of the soil materials.</td></tr>
</table>

Evaluation of the mass and rotational inertia are usually evaluated by adopting a lumped mass system. For structures with a predominantly bending vibration, the consistent mass system is used in some cases. The stiffness evaluation is usually performed using the bending-shear beam idealization method. However, FEM is effective in performing the analysis by considering the effects of out-of-plane deformation and openings on the structural wall, deformation of floor, etc. For evaluation of damping of the structure and ground, the Voigt model and the model using complex damping are adopted for the soil dissipation and soil material dampings. In addition, when modal damping constants are needed, the strain energy proportional type [model] and the method of complex eigenvalue analysis can be used.

b. Evaluation of stiffness

(a) Bending-shear beam idealization method

For stiffness evaluation of the structures with box-shaped or cylindrical structural walls, such as a reactor buildings, the method in which the structure is substituted by bending-shear beams is usually used as its analysis is simpler than that of FEM. In this method, the web effect of the wall in the force direction and the flange effect of the wall in the direction orthogonal to the force direction are replaced by the equivalent bending-shear beams. In this way, stiffness is evaluated for the entire box structure.

The web effect and flange effect are dominant factors in determining the shear stiffness of the wall in the force direction and the flexural stiffness of the wall in the orthogonal direction; hence, calculation of the effective cross-sectional area of the web wall and flange wall is the basic step. As listed in Table 5.2.4-1, the effective cross-sectional area can be calculated from the total cross-sectional area in the case of a cylindrical wall, and from the cross-sectional areas of the web wall and the flange wall in the case of box wall, multiplied by the reduction factor caused by the shape and openings of the cross section. In particular, for the flange wall, the idea of the effective width of a slab with T-shaped beams defined in Reinforced Concrete Structure Design Standard by the Architectural Institute of Japan, published in 1982 (referred to as "RC Standard" hereinafter) is introduced.

For a flange wall, as its effective width depends on the load distribution status, cross-sectional profile, height and position, for a strict evaluation, it is required to perform FEM analysis. However, as a result of the research work carried out in the past [5.2.4-1-3], there is an approximate method for determining the effective width expressed as a function of the length and height of the web wall and flange wall of the box structure.

For stiffness evaluation of a structural wall having openings in it, the same FEM analysis for the above effective width is effective. In the design, however, it is desirable that the evaluation be carried out according to the magnitude of the opening rate [=(opening area/wall area)$^{1/2}$]. As far as the calculation method of the reduction rate of stiffness by openings is concerned, "RC Standard" (published in 1982) shows in Clause 10 the experimental results of the model test for studying the reduction in the value of D (transverse force-sharing coefficient) due to openings, and in Clause 18 the following method used by Muto in aseismic design [5.2.4-4]: The shear deformation δ_F of a structural wall with openings can be calculated using the following formula:

$$\delta_F = \frac{1}{r}\delta_S \qquad (5.2.4\text{-}1)$$

where δ_S: shear deformation of shear wall without openings.
Reduction rate = r = 1.0 - 1.25P
Opening rate: P = (opening area/wall area)$^{1/2}$

The above formula is applicable for $P \leq 0.4$. When $P > 0.4$, the solution method for a rigid-frame wall can be used. In the design, by using the said formula for calculating the reduction rate, it is also possible to set the equivalent wall thickness as (1.0-1.25P)t for a structural wall with a thickness of t.

Table 5.2.4-1. Evaluation of shear stiffness and flexural stiffness.

		Cylindrical type	Box type
Shear stiffness (GA) [1]	Cross-sectional area	Total cross section	Cross-sectional area of web wall
	Shape factor	2.0	1.0
Flexural stiffness (EI) [2]	Cross-sectional area[3]	Total cross section	The effective width of the flange wall is considered

[1]G: Elastic shear modulus of wall; A: Cross-sectional area of structural wall after the reduction by openings is taken into consideration.

[2]E: Young's modulus of wall; I: Second moment of inertia after the reduction by openings is taken into consideration.

[3]For box-shaped walls, the cross-sectional area for calculating the second moment of area with the effective width taken into consideration.

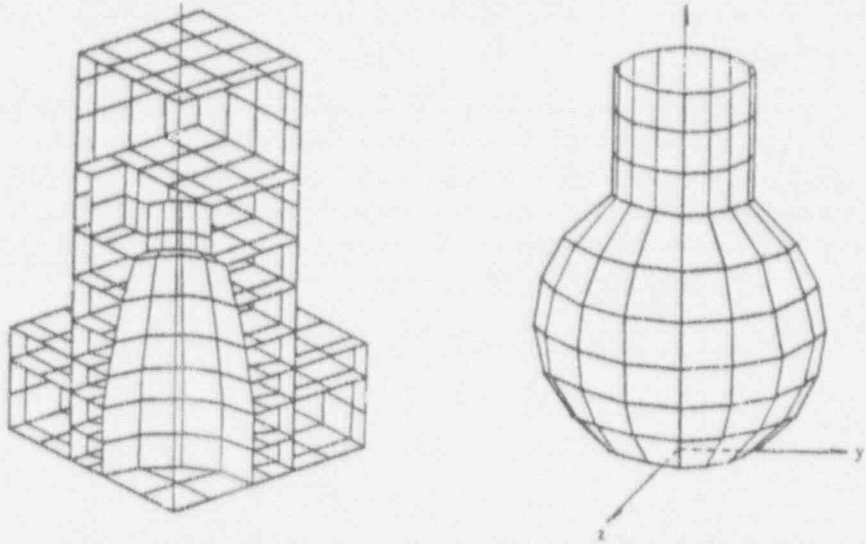
(b) FEM

FEM is an effective means for stiffness evaluation and stress analysis of buildings. It can be used to calculate the stiffness matrix used for response analysis, including the interaction effect between flat plate walls and shells having a curved surface, the flange effect of an orthogonal wall, and other 3-D effects [5.2.4-5]. In FEM analysis, the walls and the floor panel are considered to be collections of triangular and rectangular flat plate elements. three-dimensional analysis in consideration of the continuity condition of displacement with each node having 6 degrees of freedom (u, v, w, θ_x, θ_y, θ_z), or 2-dimensional analysis in consideration of only 2 degrees of freedom (u, v) may be performed.

In the analysis of a cylindrical shell, flask, or other shell shapes, the ring-shaped elements are usually used, and the problem is handled as an axisymmetric problem. However, this method cannot be applied for evaluating the characteristics of openings and asymmetric structures. In these cases, a three-dimensional analysis using flat plate trapezoidal elements with 6 degrees of freedom is performed. Figure 5.2.4-1(a) shows a three-dimensional model of reactor building. Figure 5.2.4-1(b) shows an example of a three-dimensional model of a shell with an arbitrary shape.

c. Evaluation of damping

In performing a response analysis of the structure, in addition to stiffness evaluation, it is also an important factor in determining the appropriate method to evaluate the damping performance of the ground and the structure. In particular, for earthquake response analysis of a reactor building, which is a short-period structure, the soil dissipation damping and the soil material damping have a large influence on the response characteristics of the vibration system. In this section, we will discuss the mathematical schemes for response analysis of the damping characteristics as described in Section 5.2.2 "Evaluation of properties of ground and structures" and Section 5.2.3 "Interaction between structure/building and ground."



(a) three-dimensional model of a reactor building (1/4 partial model)

(b) three-dimensional model of a shell with an arbitrary shape

Figure 5.2.4-1. Three-dimensional FEM model of structure.

(a) Viscous damping

The Voigt model is a general mechanical model of the viscoelastic body theory. In addition, its mathematical treatment can be carried out easily. Hence, this model is frequently used. For this model, in a multiple-of-degree-freedom system, between high-order frequency (ω_j) and the corresponding modal damping constant (h_j), there exists a relationship that h_j/ω_j = constant [5.2.4-6]; hence, the following method is used to form a damping matrix [C] when h_j needs to be defined to be constant or to take an arbitrary value regardless of the value of ω_j. That is, when the frequencies and damping constants of all the modes are known, the damping matrix that fits the relationship can be derived using the following formula [5.2.4-7]:

$$[C] = [M]\ [X]\ [H]\ [X]^T\ [M]^T \tag{5.2.4-2}$$

where, [M]: mass matrix; [X]: nondamped characteristic mode matrix

$$[H] : \begin{bmatrix} H_1 & & & & \\ & \ddots & & & \\ & & H_j & & \\ & & & \ddots & \\ & & & & H_m \end{bmatrix} \tag{5.2.4-3}$$

$$H_j = 2h_j\omega_j/M_j$$

ω_j: jth nondamped angular frequency,
h_j: jth damping constant
M_j: jth equivalent mass
m: maximum number

(b) Complex damping

As a method for evaluating the damping characteristics of structure, a complex spring constant model may be used [5.2.4-8]. This complex spring constant has the property that the damping constant of the vibration system can be defined as constant regardless of vibration frequency. That is, this model differs from the Voigt model in that it can define a stress, that contains an damping resistance, which is irrelevant to the strain rate and is only a function of the strain. Hence, when the damping property is considered to be uniform for the entire structure, the complex stiffness matrix is expressed by the following formula, with the imaginary portion $[K_I]$ proportional to the real portion $[K_R]$.

$$[K_R + iK_I] = [K_R](1 + i2h) \tag{5.2.4-4}$$

where h is the damping constant. The above equation is an approximate formula when h is small. Equation (5.2.4-4) shows the stiffness matrix in the region where frequency (ω) is positive. In the response analysis, formulation is carried out for ω in the positive/negative regions, with the stiffness matrix in the negative region defined as $[K_R]$ $(1 - i2h)$.

When the building/structure which is the object of response analysis is a composite structure made of RC and steel frame, if the stiffness matrix $[{}_jK]$ and damping constant ${}_jh$ are set for each structural element j, the stiffness matrix of the overall vibration system can be expressed by the following formula with respect to the displacement vector of the overall system.

$$[K_R + iK_I] = \sum_{j=1}^{m} [{}_jK_R](\ + i2{}_jh) \tag{5.2.4-5}$$

where ${}_jh$ is the damping constant of structural element j, and m is the number of structural elements.

(c) Modal damping constant

When damping constants are defined separately to different components with different structural materials or different structures, such as RC, steel frame, etc., the methods for deriving the damping constants for various orders of vibration mode, i.e., modal damping constants, include the eigenvalue analysis method and the strain energy proportional method.

According to the strain energy proportional method [5.2.4-9], first of all, the nondamped eigenvalue analysis is performed from the mass matrix [M] and stiffness matrix [K] of the vibration system, and the k-th natural mode $\{X_k\}$ is derived. Then, for each structural element, damping constant ${}_jh$ is set, and the kth damping constant h_k can be calculated from the following formula [5.2.4-10].

$$h_k = \frac{\sum {}_j h \{X_k\}^T [{}_j K] [X_k]}{\sum \{X_k\}^T [{}_j K [X_k]} \tag{5.2.4-6}$$

According to the eigenvalue analysis method, if the stiffness matrix $[K_R + iK_I]$ of the entire system is derived, by solving the eigenvalue equation (5.2.4-7), the complex eigenvalue λ_k can be derived, and frequency ω_k and damping constant h_k can be calculated using equation (5.2.4-8).

$$[\lambda^2 M + K_R + iK_I]\{X\} = 0 \tag{5.2.4-7}$$

$$\omega_k = \sqrt{\lambda_{Rk}^2 + \lambda_{Ik}^2}, \quad h_k = \lambda_{Rk} / \omega_k \tag{5.2.4-8}$$

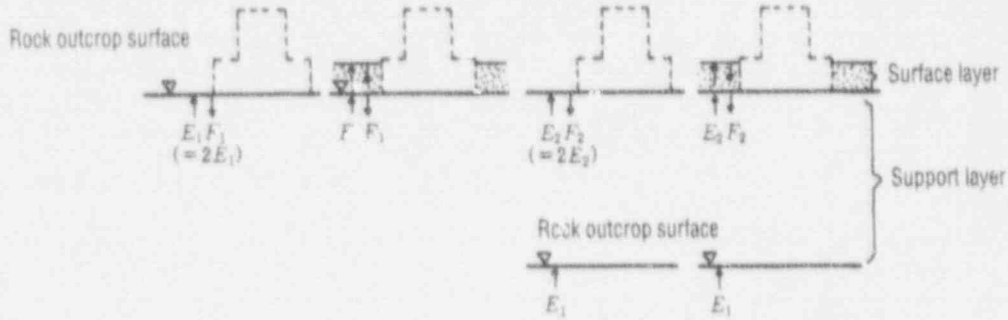
(2) Input earthquake motion for design

In the design of a reactor facility, the basic earthquake ground motion, defined as the earthquake to be considered on the surface of the rock outcrop, is used. When the building is located on the surface of the rock outcrop and the influence of the surface layer can be neglected, the basic earthquake ground motion can be used directly as the input earthquake motion. However, when the effect of the surface layer should be considered and when the surface of the rock outcrop is deeper than the bottom of the foundation mat of the building, the basic earthquake ground motion is used to calculate the earthquake motion on the bottom of the foundation mat and the bottom of the analytical model [5.2.4-11]. For the SR model and the discrete system model, the input earthquake motion can be calculated using the following methods.

a. Input earthquake motion of SR model

For the SR model, when the building is set on the surface of rock outcrop, the basic earthquake ground motion is used directly as the input earthquake motion. On the other hand, when the effect of the surface layer cannot be neglected, as shown in Figure 5.2.4-2(a), the sum of the incident wave E_1 and the reflecting wave F_1 on the bottom of the foundation mat is used as the input earthquake motion. In this analysis, one-dimensional wave analysis is performed using the incident wave E_1 as the input to the lower boundary layer of the free layer. When the effect of the surface layer is to be evaluated correctly, several methods may be used, such as the method in which a external force $(-\bar{P})$ is applied to the bottom of the foundation mat for correcting the one-dimensional wave analysis result $\{\bar{u}\}$ for the effect of a ground with a hole in it, and the method in which the ground with a hole is analyzed using a 2-dimensional discrete system model (see Figure 5.2.4-5).

The said external force $(-\bar{P})$ is a force that balances the surface force of the excavated portion. This force can be derived from the product (τA) of stress, τ, in the free ground at the bottom of the foundation mat surface and bottom surface area, A, of the standard mat [5.2.4-33]. On the other hand, when the rock outcrop surface is deeper than the foundation bottom, as shown in Figure 5.2.4-2(b), as incident wave E_1 is input, an incoming wave E_2 and an outgoing wave F_2 are calculated at the bottom of the foundation mat. When the effect of the surface layer can be ignored, $F_2 = E_2$, then $2E_2$ becomes the input earthquake motion; when the effect cannot be ignored, the input earthquake motion is given by $E_2 + F_2$.



(a) When the rock outcrop surface is identical to the foundation bottom.

(b) When the rock outcrop surface is deeper than the foundation bottom.

Figure 5.2.4-2. Calculation of input earthquake motion of SR model.

b. Input earthquake motion of discrete system model

As shown in Figure 5.2.4-3, for a discrete system model, if the position on the rock outcrop surface is set according to the site condition, as shown in Figure 5.2.4-3, incident wave E_1 is input to this position, and the incoming wave E_2 and outgoing wave F_2 at the bottom boundary of the analytical model can be derived using response analysis of the free ground; then it is possible to use $E_2 + F_2$ as the input earthquake motion.

For the discrete system model, the conventional method and substructuring method described in the preceding section can be used. According to the conventional method, when the side surface boundaries are viscous boundaries and wave boundary, as shown in Figure 5.2.4-4, the energy dissipation effect is evaluated by connecting the side surface boundary to the free ground. As far as the bottom boundary is concerned, in the case of a fixed boundary, $(E_2 + F_2)$ is taken as the input earthquake motion as shown in Figure 5.2.4-4(a); in the case of a viscous boundary, from Figure 5.2.4-4(b), $2E_2$ is taken as the input earthquake motion.

For the substructuring method, as shown in Figure 5.2.4-5(a), the building is eliminated, and the response $\{U,V\}^T$ at the boundary between the building and soil is calculated by response analysis of a foundation with a hole in it. Then, response analysis is carried out using this $\{U,V\}^T$ as the input earthquake motion on the bottom and side of the building foundation mat. Usually, however, the foundation mat is assumed to be a rigid body, the vertical response $\{V\}$ is converted to rotational motion θ, and the response analysis is performed by using horizontal motion $\{U\}$ and rotational motion θ of the building foundation mat as shown in Figure 5.2.4-5(b).

(3) Response analysis methods

a. Vibration equation

(a) SR model

In order to describe the 3-D behavior of the vibration system, it is required to have 6 degrees of freedom (u, v, w, θ_x, θ_y, θ_z) for each node of the building and the ground. Usually, the response analysis of horizontal input, as shown in Figure 5.2.4-6, is usually performed for 2 degrees of freedom for each mass, i.e., horizontal and rotational (u_i, θ_{yi}). As shown in Figure 5.2.4-6, when the earthquake motion $\ddot{u}$ is input to the support end of the model, the external force term f(t) becomes the inertial force, and is expressed by equation (5.2.4-9) according to D'Alembert principle.

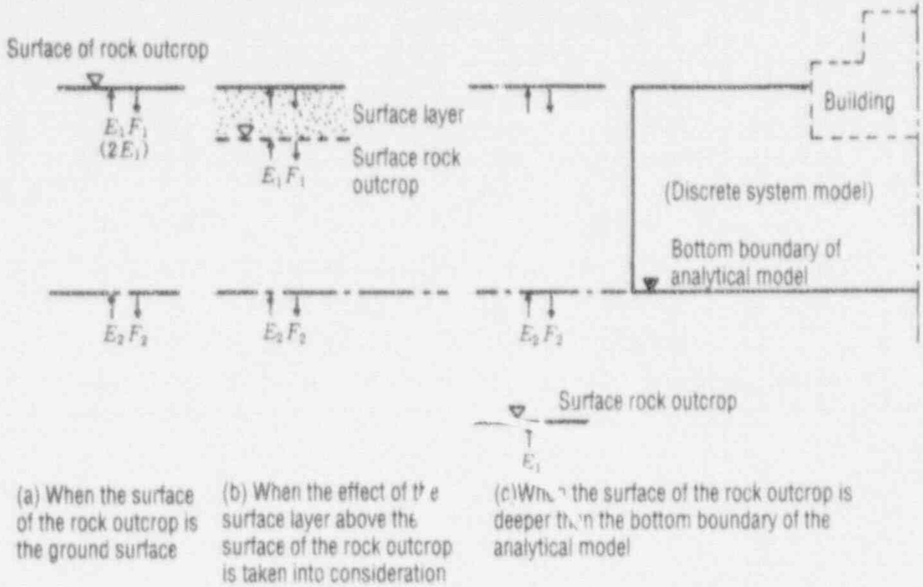


Figure 5.2.4-3. Calculation of input earthquake motion of a discrete system model.

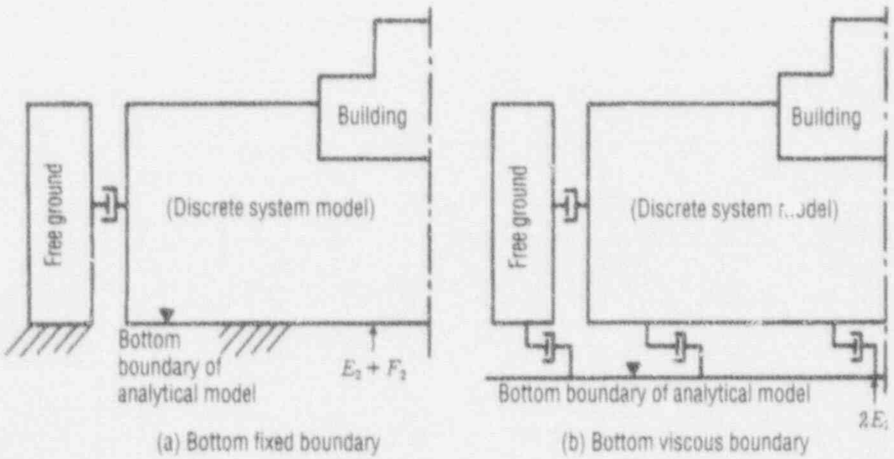


Figure 5.2.4-4. Input earthquake motion of discrete system model (through method).

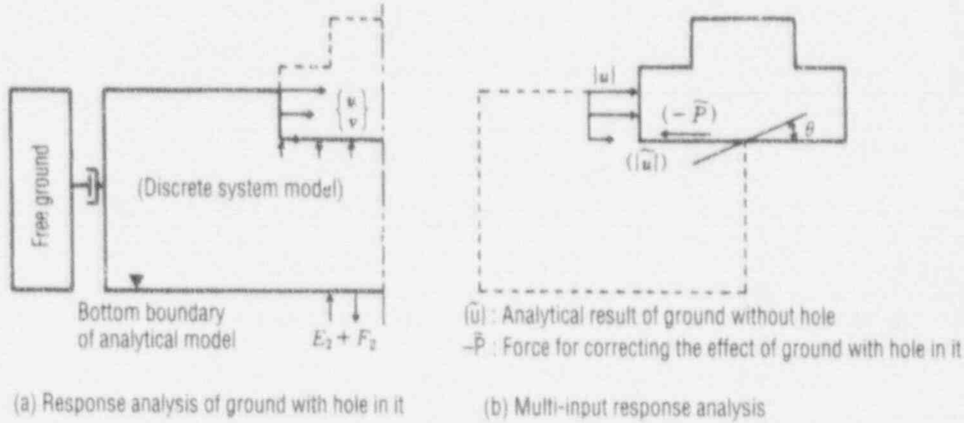


Figure 5.2.4-5. Input earthquake motion and discrete system model (substructure method).

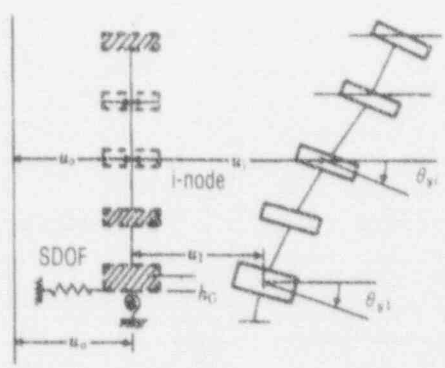


Figure 5.2.4-6. Coordinate system of SR model.

$$\{f(t)\} = -[M]\{f_0\}\ddot{u}_0 \tag{5.2.4-9}$$

$\{f_0\}$: coefficient vector for representing the load distribution.

Hence, for a multi-degree-of-freedom system with the degrees of freedom of each node represented by horizontal displacement u_i and rotational displacement θ_{vi}, the vibration equation can be expressed in the viscous damping form as follows:

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = -[M]\{f_0\}\ddot{u}_0 \tag{5.2.4-10}$$

where $[M]$: Diag $[m_n, I_n \cdots m_i, I_i, \ldots, m_1, I_1]$
m_i: mass of node 1
I_i: rotational inertia of node i

$$[K] = \begin{bmatrix} [K_F] & [K_{FS}] \\ [K_{SF}] & [\bar{K}_S + K_{SS}] \end{bmatrix} \quad [\bar{K}_S] = \begin{bmatrix} K_S & -K_S h_G \\ -K_S h_G & K_R + K_S h_G^2 \end{bmatrix}$$

$[K_F]$: stiffness matrix of building
$[K_{SF}]$, $[K_{FS}]$, $[K_{SS}]$: reaction force matrix of building
$[\bar{K}_S]$: stiffness matrix of ground
K_S;, K_R: Sway spring and rocking spring of the foundation
h_G: height of the center of gravity of the rigid foundation

$$\{u\} = \{u_n, \theta_n, \ldots, u_i, \theta_i, \ldots, u_1, \theta_1\}^T$$
$$\{f_0\} = \{1, 0, \ldots, 1, 0, \ldots, 1, 0\}^T$$

[C]: Viscous damping matrix. Similar to the above stiffness matrix, it is made of the viscous damping matrices of the building and ground.

(b) Model of discrete system

(i) MDOF parallel ground model

For this ground model, as described in section 5.2.3 "Interaction between structure/building and ground," depending on the way of treating the bottom boundary and side boundary of the foundation, the vibration equations are represented in the following different forms [5.2.4-10].

When the bottom of the foundation is a fixed boundary and its side surface is a free boundary, the stiffness matrix of the ground, $\bar{K}_S$, becomes the following formula. The vibration equation is the same as the SR model shown in the preceding item.

$$[\bar{K}_S] = \begin{bmatrix} [K_G] & [K_{Gi} h_{Gi}] \\ [K_{Gi} h_{Gi}] & K_R + \sum K_{Gi} h_{Gi}^2 \end{bmatrix} \tag{5.2.4-11}$$

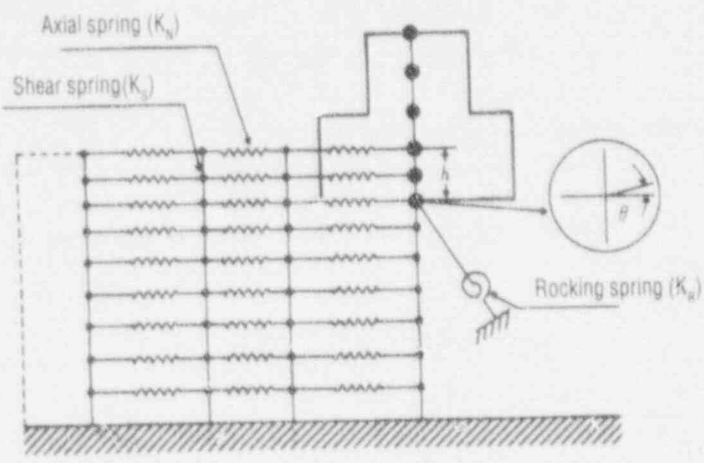


Figure 5.2.4-7. Soil spring of grid-shaped model.

$[K_G]$: stiffness matrix for the ground modeled using a concentrated mass system
$[K_{Gi}]$: part of stiffness matrix $[K_G]$ for the degrees of freedom of the foundation mat and the building as shown in Figures 5.2.4-7
K_R: rocking spring constant
h_{Gi}: height to the rotational center of K_{Gi}
$\{U_{G1} \cdots U_{Gi} \cdots U_{Gm}\}$: displacement vector of the ground.

When the boundaries on the bottom, side, and out-of-plane direction of the foundation are taken as viscous boundaries, the vibration equation becomes equation (5.2.4-12).

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = -[M]\{f_0\}\ddot{u}_0 - [C_B]\{\dot{u} - \dot{u}_f\} \qquad (5.2.4\text{-}12)$$

$[C]$: damping matrix of building and ground
$[C_B]$: damping matrix consisting of boundary dampers of the ground (diagonal matrix)
$\{\dot{u}_f\}$: velocity response of free ground.

(ii) FEM model

When the bottom, side, and out-of-plane direction are taken as viscous boundaries, the vibration equation becomes the same form as that in the above section on the MDOF parallel model. Here, we show the vibration equation for the case where the side surface is a 2-dimensional transfer boundary, the out-of-plan direction is a viscous plane and the bottom is a fixed boundary [5.2.4-13, 5.2.4-14].

$$[M]\{\ddot{u}\} + [K]\{u\} = -[M]\{f_0\}\ddot{u}_0 - \{V\} + \{F\} - \{T\} \qquad (5.2.4\text{-}13)$$

$[K]$: complex stiffness matrix of building and ground
$\{V\}$: force due to out-of-plane damper, $\{V\} = [C_0]\{\dot{u} - \dot{u}_f\}$
$\{F\}$: force due to side surface boundary, $\{F\} = [G]\{u_f\}$
$[G]$: stiffness matrix of free ground
$[C_0]$: damping matrix of out-of-plane damper (diagonal matrix)
$[T]$: force related to energy transfer on the side surface boundary
$[T] = R\{u - u_f\}$
$[R]$: frequency-dependent stiffness matrix of the side surface boundary

(c) Substructuring method

The vibration equation of the substructuring method is basically similar to the SR model, with differences in the representation of the balance between the force on the embedded portion and the force at the surfaces of surrounding soil. That is, in this case, a term {D}, the force applied from the soil to the building, or driving force as it is usually called [5.2.4-15] should be added to the right-hand side of equation (5.2.4-14). Equation (5.2.4-14) is a vibration equation representing the complex stiffness form in the overall coordinate system shown in Figure 5.2.4-8. Also, the damping can be included by representing the stiffness in equation (5.2.4-4) in the complex stiffness form.

$$[M]\{\ddot{U}\} + [K]\{U\} + \{D\} \qquad (5.2.4\text{-}14)$$

$[K] = [K_F] + [K_S(\omega)]$

$[K_F]$: complex stiffness matrix of building

$[K_S(\omega)]$: impedance matrix of the foundation

$\{D\} = [K_S(\omega)]\{U_C^*\}$

$\{U_C^*\}$: displacement of the embedded portion of a soil only, assuming a building does not exist.

As shown in Figure 5.2.4-9, without embedment, $\{U_C^*\}$ becomes displacement u_0 of the surface of the free ground, and the driving force becomes $\{D\} = K_S(\omega)u_0$.

When the absolute displacement {U} of equation (5.2.4-14) is expressed by the relative coordinate system shown in Figure 5.2.4-9, the following equation is obtained:

$$\begin{aligned} U_i &= u_0 + u_i \\ U_C &= u_0 + u_C \end{aligned} \qquad (5.2.4\text{-}15)$$

Hence, if equation (5.2.4-14) is represented by the relative coordinate system as the following equation, it comes into agreement with the vibration equation of the conventional SR model.

$$[M]\{\ddot{u}\} + [K]\{u\} = -[M]\{f_0\}\ddot{u}_0 \qquad (5.2.4\text{-}16)$$

b. Eigenvalue analysis

In response analysis, for the cases that the natural frequency and modal damping constant are necessary or the seismic response is performed by modal analysis, it is required to perform an eigenvalue analysis. However, when the stiffness matrix is a function of the frequency and is a complex matrix, or the model of the ground is formed as an MDOF system, as well as when the model is formed using a three-dimensional FEM, the matrix size of the characteristic vibration equation becomes too large. As a result, it is required to make various numerical efforts according to the matrix property and analytical purpose [5.2.4-16-19]. Table 5.2.4-2 lists the typical eigenvalue calculation methods used in response analysis. Their features are as follows:

(i) Power method (vector repetition method): This calculation method is effective in extracting some of the lower or higher order eigenvalues and characteristic modes.

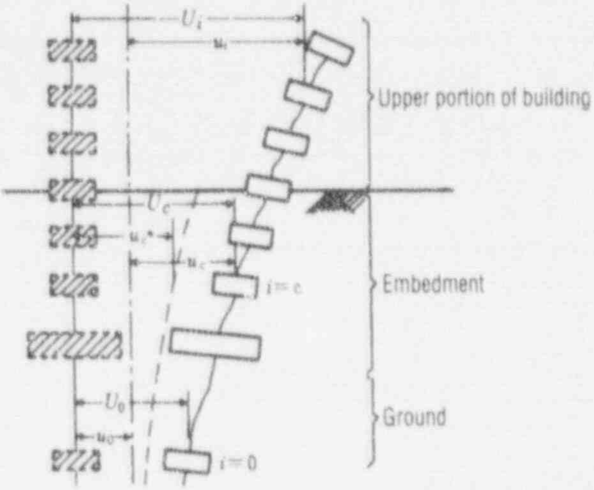


Figure 5.2.4-8. The case with embedment.

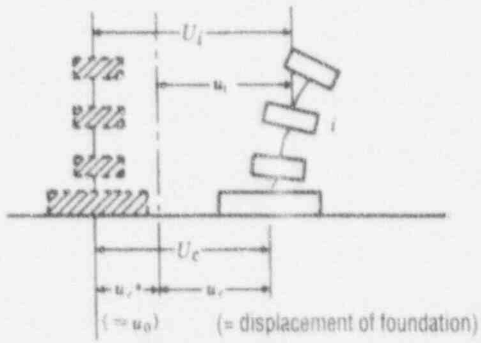


Figure 5.2.4-9. The case without embedment.

Table 5.2.4-2. Typical eigenvalue calculation methods.

Power method	Transformation method	Characteristic equation method	Others
Power method	Jacobi method	Danilevski method	Lanczos method
Inverse-power method	Q R method	Bairstow method	Determinant method Subspace method

(ii) Transformation method: This calculation method can be used for calculating all the eigenvalues and the characteristic modes. At first, only the eigenvalues are calculated. Then, in many cases, only the required eigenvectors are calculated.

(iii) Method using characteristic polynomial: In this method, the eigenvalues are calculated by exploiting the fact that the eigenvalues are the roots of the characteristic equations formed by the determinant expressed by equation (5.2.4-17).

$$\rho(\lambda) = \det|K - \lambda M| \tag{5.2.4-17}$$

This method is effective in the eigenvalue analysis of complex coefficients. For the numerical calculation algorithm and numerical analytical method of the various eigenvalue calculation methods described above, the reader is referred to reference [5.2.4-20].

Recently, software for eigenvalue analysis has been published, so that the needs of users can be met in a relatively easy way. For example, the eigenvalue analysis program package was assembled at Argonne National Laboratories in the USA on the basis of the research work of Wilkinson [5.2.4-16]. This package handles eigenvalue analysis methods for real/complex and symmetric/asymmetric matrices.

c. Seismic response analysis methods

Seismic response analysis methods can be roughly classified into the modal analysis method, the direct integration method, and the Fourier transformation method as listed in Table 5.2.4-3. The modal analysis method is for analysis of linear problems. Although it is required to perform eigenvalue analysis, by selecting appropriate lower modes, the overall time history response can be calculated at lower costs. In particular, for the spectral modal method, as the time integration is not performed, the calculation time can be further reduced. On the other hand, among the direct integration methods, the Newmark-β method, etc., are applicable for solving elastoplastic problems. When complex stiffness-form or frequency dependent-form soil springs are adopted, the Fourier transform method can be used to handle only the elastic problem.

(a) Spectral modal method

In this analytical method, the response spectral values of the various modes (jth mode, kth mode) are calculated beforehand; then, the square root of the sum of the product of the jth mode and the jth mode is calculated as the maximum response value of the system. It includes the "SRSS" method, Complete Quadratic Combination method (referred to as "CQC" method hereinafter), etc.

According to the "SRSS" method, the spectral values of the various modes (jth mode) are calculated. These values are assumed not to take place simultaneously; hence, the square root of the sum of the squares of the various modes of response are combined. That is, the maximum response value is determined using equation (5.2.4-18):

$$U_{max} = \sqrt{\sum_{j=1}^{m} |\beta_j \phi_j S_j(\omega_j, h_j)|^2} \tag{5.2.4-18}$$

where U_{max}: maximum response spectrum of various nodes

$\beta_j \phi_j$: jth participation function

$S_j(\omega_j, h_j)$: jth response spectrum; ω_j and h_j are natural frequency and damping constants, respectively.

m: maximum number of orders of the superposed mode.

Table 5.2.4-3. Classification of response analysis methods.

Response analysis method	Numerical analysis method
Spectral modal method	SRSS method, CQC method
Time history modal method (time integration method of SDOF system)	Newmark-β method, Wilson-θ method, central difference method, Runge-Kutta method, Houbolt method, external-force linearization method
Direct integration method	
Fourier transform method (frequency response analysis method)	Fast Fourier transformation (FFT)

In order to calculate the shear and bending moment on each story, it is required to calculate the shear and bending moment of each mode, and then to calculate the square root of the sum of the squares. When the characteristic frequencies of different modes are well separated from each other, the correlation between the modes can be ignored. The "SRSS" method can be applied in this case; usually, this method can be used in design. However, for a complicated structure, as the natural frequencies are close to each other, the effect of correlation is large. In this case, the "CQC" method, which enables evaluation of the correlation among modes, is an appropriate analytical method. According to this method, the maximum responses can be calculated using equation 5.2.4-19 [5.2.3-4, 5.2.4-21]:

$$U_{max} = \sqrt{\sum_{j=1}^{m}\sum_{k=1}^{m} \beta_j\phi_j S_j(\omega_j, h_j)\, \beta_k\phi_k S_k(\omega_k, h_k)\, \rho_{0jk}} \qquad (5.2.4\text{-}19)$$

where U_{max}: maximum response of each node

$\beta_j\phi_j$, $\beta_k\phi_k$: jth and kth participation factors

$S_j(\omega_j, h_j)$, $S_k(\omega_k, h_k)$: jth and kth response spectra

m: number of superposing modes

ρ_{0jk} in equation (5.2.4-19) is called the "correlation coefficient," which can be approximately expressed by the following equations:

$$\rho_{0jk} = \frac{\sqrt{8h_j h_k \omega_j \omega_k}\,(h_j\omega_j + h_k\omega_k)\,\omega_j\omega_k}{K_{jk}} \qquad (5.2.4\text{-}20)$$

$$K_{jk} = \left(\omega_j^2 - \omega_k^2\right) + 4h_j h_k \omega_j \omega_k \left(\omega_j^2 + \omega_k^2\right) + 4\left(h_j^2 + h_k^2\right)\omega_j^2\omega_k^2 \qquad (5.2.4\text{-}21)$$

When $\rho_{0jk} = 1$ for $j = k$, and $\rho_{0jk} = 0$ for $j \neq k$, the "CQC" method becomes identical to the "SRSS" method.

(b) Time history modal method

The time history modal method can be used to calculate the time history responses of the vibration equation (5.2.4-10). Suppose the jth characteristic mode of the eigenvalue analytical results is ϕ_j, the natural frequency is ω_j, the damping constant is h_j, and the participation factor is β_j, then, at each time point, the response displacement vector U(t) is given by the following equation:

$$\{U(t)\} = \sum_{j=1}^{m} \{\phi_j\} \beta_j q_{0j}(t) \qquad (5.2.4\text{-}22)$$

where m is the maximum order of the superposed modes, $q_{0j}(t)$ is the time history response displacement of a SDOF, which is the solution of the following equation with respect to the input earthquake motion $\ddot{U}_0(t)$:

$$\ddot{q}_{0j}(t) + 2h_j\omega_j\dot{q}_{0j}(t) + \omega_j^2 q_{0j}(t) = -\ddot{U}_0(t) \qquad (5.2.4\text{-}23)$$

The various methods listed in Table 5.2.4-3 can be used to perform the time integration for equation (5.2.4-23). For the modal analytical method, once the eigenvalue analysis is performed, it can be directly used for the other input earthquake motion, and very accurate responses can be obtained by superposing only the dominant modes. Hence, it is beneficial from the viewpoint of the cost of calculation. However, the modal method cannot directly account for the damping characteristics of the material. Hence, by using weighting factors for model vectors and adopting complex stiffness, the damping characteristics of the materials are reflected in determining the modal damping constant.

(c) Direct integration method

The method for directly solving the vibration equations in the time domain is called the "direct integration method," which can be used for solving both linear and nonlinear problems. The following successive integration methods in the time domain are well-used in the field of structural analysis: Newmark-β method [5.2.4-22], Wilson-θ method [5.2.4-16], and Argiris method [5.2.4-2?]. For time integration, the important factors include accuracy and stability. In order to achieve a high enough calculation accuracy, it is required to adopt a small enough calculation time interval (Δt). Usually, Δt is on the order of 1/100, 1/200, or 1/500 sec. However, it may be on the order of 1/1000 sec for a vibration system with a particularly short natural period. For details of the various numerical analysis methods, the readers are referred to the related references. In the following, we only discuss the basic features of the Newmark-β method, which is a particularly well-used time integration method.

According to the β method, parameters β and γ are used to express the displacement (U_{n+1}) and velocity ($\dot{U}_{n+1}$) of the (n + 1)'th step from the displacement (Un), velocity ($\dot{U}n$) and acceleration ($\ddot{U}_n$) of the n'th step as well as the acceleration ($\ddot{U}_{n+1}$) of the (n + 1)'th step as follows:

$$\left.\begin{aligned} U_{n+1} &= U_n + \dot{U}_n\Delta t + (\tfrac{1}{2} - \beta)\ddot{U}_n(\Delta t)^2 + \beta\ddot{U}_{n+1}(\Delta t)^2 \\ \dot{U}_{n+1} &= \dot{U}_n + (1-\gamma)\ddot{U}_n\Delta t + \gamma\ddot{U}_n\Delta t \end{aligned}\right\} \qquad (5.2.4\text{-}24)$$

According to this method, the values of β, γ in equation (5.2.4-24) correspond to the various time integration methods listed in Table 5.2.4-4. In particular, when β = 1/4 and γ = 1/2, the solution has a high stability and is widely used. For the values of γ and β, the general stable condition is expressed by formula (5.2.4-25):

Table 5.2.4-4. Combination of β and γ, and equivalent time integration method.

β	γ	Time integration method
0	1/2	Central difference method
1	3/2	Regressive difference method
1/6	1/2	Linear acceleration method
4/5	3/2	Galerkin method
1/12	1/12	Fox Goodwin method
1/4	1/2	Average acceleration method

$$\beta > (0.5 + \gamma)^2/4 \tag{5.2.4-25}$$

(d) Frequency response analysis method

The frequency response analysis method is an analytical method which allows utilization of the ground impedance depending on the complex stiffness and frequency (ω). According to this analytical method, displacement U(t), velocity $\dot{U}(t)$, acceleration $\ddot{U}(t)$, and input acceleration $\ddot{U}_0(t)$ in equation (5.2.4-10) are Fourier transformed as U(t) shown in the following equation:

$$U(\omega) = \int_{-\infty}^{\infty} U(t) e^{-i\omega t} dt \tag{5.2.4-26}$$

Hence, equation (5.2.4-10) can be transformed into the vibration equation in the frequency domain shown in the following equation:

$$[-\omega^2 M + i\omega C + K]\{U(\omega)\} = \omega^2 [M]\{f_0\} U_0(\omega) \tag{5.2.4-27}$$

When the above equation is solved for $U(\omega)$, equation (5.2.4-28) is obtained:

$$\{U(\omega)\} = [-\omega^2 M + i\omega C + K]^{-1} \omega^2 [M]\{f_0\} U_0(\omega) \tag{5.2.4-28}$$

Then, inverse Fourier transformation is performed for $U(\omega)$, giving the time history response:

$$U(t) = \frac{1}{2\pi} \int_{-\infty}^{\infty} U(\omega) e^{i\omega t} d\omega \qquad (5.2.4\text{-}29)$$

The indefinite integration of equation (5.2.4-29) is treated by Fourier series. In this case, if the component of $|\omega| > \pi/\Delta t$ can be neglected for earthquake motion $\ddot{U}_0$ sampled by Δt, the Fourier transformation can be replaced by a finite Fourier transformation. For the earthquake motion (Δt = 0.01 sec) used in conventional analysis, need to evaluate a frequency higher than 50 Hz is relatively rare; hence, the above condition can usually be satisfied.

This method has the advantage that it is able to handle the ground impedance of a complex frequency dependence type. However, it is limited to analysis in the linear region, and it is required to solve the coupled equations (5.2.4-27) for each frequency; hence, it requires a very long calculation time. This disadvantage, however, may be alleviated by reducing the calculation time by reducing the degrees of freedom for MDOF superstructures, and by using a relatively wider frequency intervals, with the response values inbetween calculated by interpolation. Also, in Fourier transformation, as the actual wave with a duration of T is treated as if it is a part of the infinitely long repeated record with a period of T, the linking effect may cause an error. This disadvantage, however, can be avoided by inserting trailing zeros at the end of the input earthquake motion.

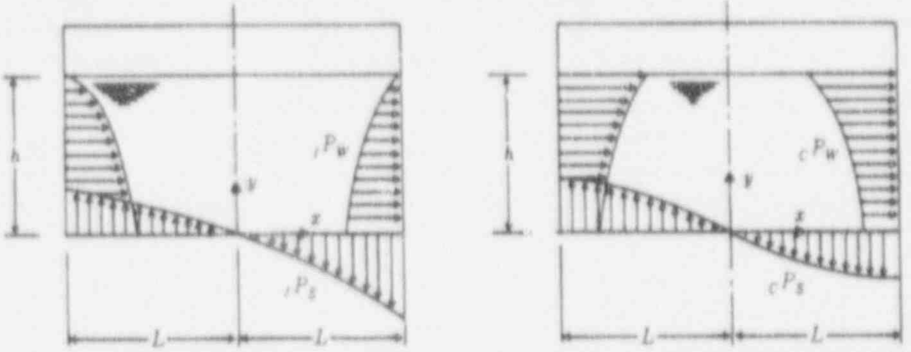
(4) Others

a. Dynamic hydraulic pressure of pool

When an earthquake motion acts on a pool containing water, the water content is shaken, and a dynamic hydraulic pressure acts on the pool wall. This phenomenon is known as "sloshing." For the spent fuel pool, tank, and containment used in the nuclear power plant, aseismic design is performed for the pool wall in consideration of the dynamic hydraulic pressure. In this chapter, we will present only the general features of the spent fuel pool. As far as the tank and containment are concerned, the readers are referred to Chapter 6.

The dynamic hydraulic pressure can be divided into the impulsive pressure caused by the inertial force of the fixed water and the convective pressure caused by the inertial force of the free water. The evaluation schemes include the finite element method [5.2.4-25], simplified calculation method [5.2.4-28] based on Housner's theory [5.2.4-26, 5.2.4-27], calculation method based on velocity potential theory [5.2.4-29, 5.2.4-30], etc. Among these 3 types of evaluation schemes, the finite element method has the highest accuracy and guarantees reasonable results. The other methods are simplified schemes for the purpose of design. They only give approximate results (Standards, Equipment-6 [Hyo, Ki-6]). In designs for practical implementation, the evaluation method of AEC TID-7024 Nuclear Reactors and Earthquakes [5.2.4-28] based on the Housner theory is used as reference. However, for the formulas shown in TID-7024, different coordinate systems are used for the impulsive pressure and convective pressure; in addition, it does not provide an explanation of how to define the input motion when the response analysis results of the building are used as input. Hence, we have rearranged the formulas conveniently for practical design. They are shown as follows.

For the impulsive pressure and convective pressure on the side wall and bottom wall of the pool, the dynamic hydraulic pressure (tf) in unit length in the depth direction in the rectangular pool shown in Figure 5.2.4-10, can be calculated using the following formulas:



(a) Impulsive pressure distribution profile (b) Convective pressure distribution profile

Figure 5.2.4-10. Dynamic hydraulic pressure distribution.

Impulsive pressure

$$P_I = \rho \ddot{X} L \frac{\sqrt{3}}{2} \frac{H}{L} \left\{1 - \left(\frac{y}{H}\right)^2\right\} \frac{\sinh\left(\sqrt{3}\,\frac{x}{H}\right)}{\cosh\left(\sqrt{3}\,\frac{L}{H}\right)}$$

Convective pressure (5.2.4-30)

$$P_C = S_A(\omega_1) \rho L \frac{5}{4} \left\{\frac{x}{L} - \frac{1}{3}\left(\frac{x}{L}\right)^3\right\} \frac{\cosh\left(\sqrt{\frac{5}{2}}\,\frac{y}{L}\right)}{\cosh\left(\sqrt{\frac{5}{2}}\,\frac{H}{L}\right)}$$

where,

$$\omega_1 = \sqrt{\sqrt{\frac{5}{2}}\,\frac{g}{L}\,\tanh\left(\sqrt{\frac{5}{2}}\,\frac{H}{L}\right)}$$: fundamental angular frequency of free water

ρ: mass of liquid per unit width ($tf \cdot s^2/m^2$)
h: depth of liquid
H: $H = h$ ($h \le 1.5L$), $H = 1.5L$ ($h > 1.5L$)
2L: width of the rectangular pool (m)
$\ddot{X}$: maximum response acceleration of the floor on which the pool is setting, or the average maximum response acceleration of the base floor and the upper-story floor (m/s^2)
$S_A(\omega_1)$: floor response spectrum for ω_1 for $\ddot{X}(t)$
g: gravitational acceleration (m/s^2)

Table 5.2.4-5. Hydrodynamic pressure equations for rectangular pools.

	Side wall	Base plate
Impulsive Pressure	${}_I P_W = \rho L \ddot{X} \frac{\sqrt{3}}{2} \frac{H}{L} \left[1 - \left(\frac{y}{H}\right)^2\right] \tanh\left(\sqrt{3}\frac{L}{H}\right)$	${}_I P_S = \rho L \ddot{X} \frac{\sqrt{3}}{2} \frac{H}{L} \frac{\sinh\left(\sqrt{3}\frac{x}{H}\right)}{\cosh\left(\sqrt{3}\frac{L}{H}\right)}$
Convective Pressure	${}_C P_W = \rho L S_A(\omega_1) \frac{10}{12} \frac{\cosh\left(\sqrt{\frac{5}{2}}\frac{y}{L}\right)}{\cosh\left(\sqrt{\frac{5}{2}}\frac{H}{L}\right)}$	${}_C P_S = \rho L S_A(\omega_1) \frac{5}{4} \left[\frac{x}{L} - \frac{1}{3}\left(\frac{x}{L}\right)^3\right] \frac{1}{\cosh\left(\sqrt{\frac{5}{2}}\frac{H}{L}\right)}$

x: horizontal coordinate with the center of the bottom pool panel as the origin

y: vertical coordinate w... the center of the bottom pool panel as the origin

Table 5.2.4-5 lists the various formulas for calculating the pressure distributions on the side wall ($x = \pm L$) and bottom ($y = 0$) (${}_I P_W$, ${}_I P_S$, ${}_C P_W$, ${}_C P_S$) according to equation (5.2.4-30). For a structure such as a spent fuel pool, because of a strong structural design to resist a major earthquake, when the dynamic hydraulic pressure is evaluated, there is almost no problem in assuming the pool as a rigid body. In addition, the stress determined in the side wall and bottom panel of the pool is usually smaller than the stress determined by the seismic shear force and thermal load.

As the aseismic design method for pools and other containment structures, the Architectural Institute of Japan has drafted "Guidelines and commentary of containment structural design [5.2.4-32]." In Chapter 4 of these guidelines (Water containments), the general aseismic design method of the pool is described; in section 4.2.3 (Loads), the dynamic hydraulic pressure is explained.

5.2.5 Nonlinear seismic response analysis

(1) Introduction

When subjected to the basic earthquake ground motion S_2, it is believed that a part of the reactor building leaves the elastic region and enters the plastic region. Therefore, for practical seismic response analysis, nonlinear analysis methods are more realistic. From the structural point of view, the reactor building is primarily made of shear walls. In order to analytically investigate its elastoplastic behavior, it is important to build an appropriate model for the restoring force characteristics of the reinforced concrete shear walls. The shear walls of the reactor building differ from the shear walls of conventional buildings in that they are primarily of box or cylindrical shapes, and contain a large amount of reinforcing bar. Recently, various institutions have performed experimental and analytical research on the restoring force characteristics of the shear walls of the reactor building. As a result, a large amount of data have been accumulated. Based on these data, it is possible to specifically determine the restoring force characteristics model used in nonlinear earthquake response analysis of buildings. For the restoring force characteristics of the shear walls, various models have been developed. Here, we will discuss only a few frequently-used models. In this section, discussion will be limited to shear walls made of reinforced concrete. When steel frames are used in the reactor building as rigid frame structures or braced structures, their hysteresis loops are of a spindle form, which is similar to a bilinear form, or a slip form, and are different from structures made of reinforced concrete. Hence, appropriate evaluation should be performed according to the "Steel structure design standards," and "Steel structure plastic design guidelines" by the AIJ.

Also, in nonlinear analysis, in addition to the elastoplastic behavior of the building, the nonlinear effects in the soil-structure interaction should also be taken into consideration. As a method conventionally used at present, analysis is performed by using soil springs; the uplifting due to excessive overturning moment of the foundation can be handled by using a nonlinear rocking spring. In addition, when the behavior of embedment soil, etc., is considered, it is required to handle the problem by considering the nonlinearity of the soil material. In this section, we will present the present methods used for nonlinear seismic response analysis, and will also show some items believed to be required for aseismic design of reactor buildings in the future.

(2) Restoring force characteristics of structure

As pointed out above, the shear walls of reactor buildings have the shapes of boxes, cylinders, cones, etc., in which the inner walls and outer walls orthogonal to the seismic force play the role of flanges. In addition, compared with conventional shear walls, the wall is much thicker and the amount of shear reinforcing bars is larger; hence, even after cracking, the total (concret + rebars) strength of the wall is expected to increase. Investigations of the characteristics of the restoring force of the shear walls have been performed by model experiments and numerical analysis by the finite element method. In this way, useful information has been accumulated, and various proposals have been made.

In the elastoplastic response analysis of the reactor building, the model of the building is formed by bending-shear beams. Corresponding to this scheme, as shown in Figure 5.2.5-1, flexural deformation and the shear deformation are separated from each other, with hysteresis loops determined for each of them independently; as these deformations are added, the total deformation of the component is obtained. In the following, the restoring force characteristics of the shear deformation and flexural deformation will be discussed with respect to the skeleton curve and the hysteresis loop.

a. Skeleton curve of reinforced concrete shear wall

In the elastoplastic earthquake response analysis of the reactor building, the trilinear approximation shown in Figure 5.2.5-2 on the basis of the M-ϕ relation and the τ-γ relation of the major aseismic elements, i.e., box wall and cylindrical wall, is used. The methods in determining the first turning point, the second turning point, and the ultimate point include the conventionally used method and the scheme proposed in the Electric Power Joint Research (referred to as "EPJR" hereinafter) program (Research, Construction-2 [Ken, Ken-2]).

(a) Shear deformation (τ-γ relation)

In the region between the origin and the first turning point, the elastic stiffness K_e is

$$K_e = {}_CG \qquad (5.2.5\text{-}1)$$

where ${}_CG$ is the shear modus of elasticity of the concrete. In the conventional method, for both the box wall and the cylindrical wall, the shear stress τ_1 at the first turning point is determined by the following formula:

$$\tau_1 = 0.1F_C \qquad (5.2.5\text{-}2)$$

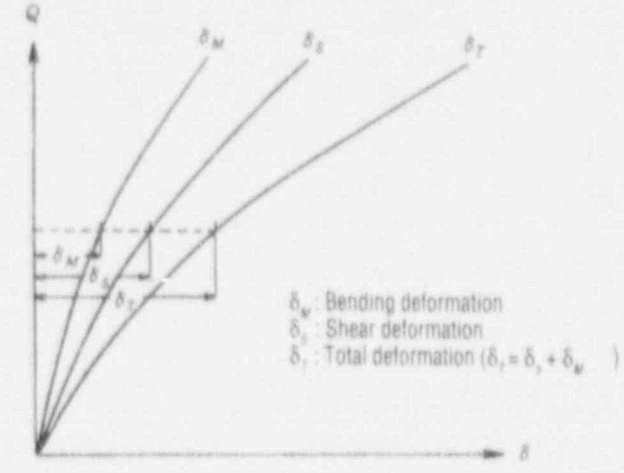


Figure 5.2.5-1. Bending deformation and shear deformation.

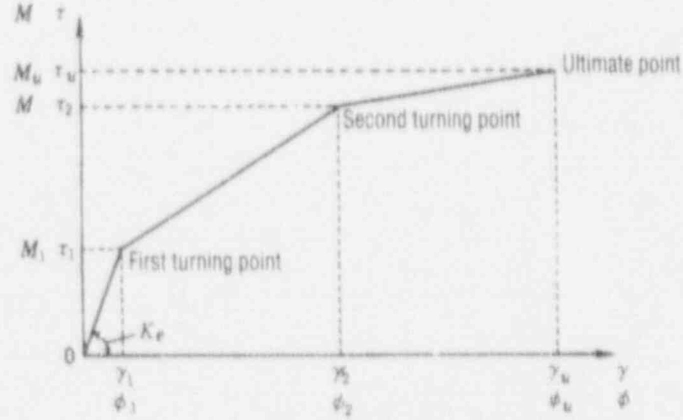


Figure 5.2.5-2. Trilinear skeleton curve.

The corresponding shear strain γ_1 is determined by the following formula:

$$\gamma_1 = \tau_1 / {}_cG \qquad (5.2.5\text{-}3)$$

On the other hand, according to the EPJR scheme, the shear stress at the first turning point is determined by the following formula:

$$\tau_1 = \sqrt{\sqrt{F_C}\left(\sqrt{F_C} + \sigma_v\right)} \qquad (5.2.5\text{-}4)$$

The corresponding shear strain is determined by using equation (5.2.5-3). According to the conventional method, for both the box wall and the cylindrical wall, the shear stress τ_2 at the second turning point is

$$\tau_2 = 1.5\tau_1 \qquad (5.2.5\text{-}5)$$

The corresponding shear strain γ_2 is determined by using Kokusho's formula [5.2.5-5] as follows:

$$\gamma_2 = \left(\frac{\tau_2}{\tau_1}\right)^{2.72} \times \gamma_1 \qquad (5.2.5\text{-}6)$$

On the other hand, according to the EPJR scheme, the shear stress and the corresponding shear strain at the second turning point are determined as follows by using the values at the first turning point:

$$\tau_2 = 1.35\tau_1 \qquad (5.2.5\text{-}7)$$

$$\gamma_2 = 3\gamma_1 \qquad (5.2.5\text{-}8)$$

At the ultimate point, according to the conventional method, the shear stress τ_u is defined using Hirozawa's formula [5.2.5-4] as follows:

$$\tau_u = \frac{0.0679 P_v^{0.23}\left(F_C + 180\right)}{\sqrt{M/QD + 0.12}} + 2.7\sqrt{P_h \, {}_w\sigma_y} + 0.1\sigma_v \qquad (5.2.5\text{-}9)$$

where, P_v: longitudinal reinforcing bar ratio (%), P_h: transverse reinforcing bar ratio, σ_v: axial compression stress in longitudinal direction, M/Q: shear span, D: wall length in the force-acting direction in the case of a box wall, or outer diameter in the case of a cylindrical wall. In particular, the following formula is used for PCCV:

$$\tau_u = 5.0\sqrt{F_C} \qquad (5.2.5\text{-}10)$$

The deformation γ_u at the terminating point can be derived by using Kokusho's formula, with τ_2 in equation (5.2.5-6) replaced by τ_u. On the other hand, according to the EPJR scheme, the shear stress at the terminating point is determined by using Yoshizaki's formula [5.2.5-6] as follows:

$$\left.\begin{aligned} \text{when} \quad \tau_S < 4.5\sqrt{F_C} \quad \tau_u &= \left(1 - \frac{\tau_S}{4.5\sqrt{F_C}}\right)\tau_0 + \tau_S \\ \tau_0 &= \left(3 - 1.8\frac{M}{QD}\right)\sqrt{F_C} \\ \tau_S &= \frac{P_v + P_H}{2}\,{}_S\sigma_y + \frac{\sigma_v + \sigma_H}{2} \\ \text{when} \quad \tau_S \geq 4.5\sqrt{F_C} \quad \tau_u &= 4.5\sqrt{F_C} \end{aligned}\right\} \quad (5.2.5\text{-}11)$$

where n: Young's modulus ratio, σ_H: axial compression stress in the transverse direction. The corresponding shear strain is determined as follows:

$$\gamma_u = 4.0 \times 10^{-3} \quad (5.2.5\text{-}12)$$

Table 5.2.5-1 summarizes the results of determination of turning points and ultimate point in the τ-γ relation.

(b) Bending deformation (M-ϕ relation)

In the region from the origin to the first turning point, the elastic stiffness K_e is determined by the following formula:

$$K_e = {}_CE \cdot I_e \quad (5.2.5\text{-}13)$$

where, ${}_CE$ is the Young's modulus of concrete, I_e is the effective second moment of inertia. In the calculation, the value of I_e is evaluated by accounting for the effective cross-sectional area with the flange effect of the wall orthogonal to the external force taken into consideration in the case of a box wall, or it is evaluated by using the total cross-sectional area as the effective area in the case of a cylindrical wall.

According to both the conventional method and the EPJR scheme, moment M_1 and curvature ϕ_1 at the first point are defined as follows using the formulas for columns for both the box wall and the cylindrical wall:

$$M_1 = \left(f_t + \frac{N}{A_e}\right) Z_e \quad (5.2.5\text{-}14)$$

$$\phi_1 = \frac{M_1}{K_e} \quad (5.2.5\text{-}15)$$

where $f_t = 1.2\sqrt{F_C}$, F_C: concrete compression strength, N: axial force, A_e: effective cross-sectional area with reinforcing bars taken into consideration, Z_e: effective section modulus.

Moment M_2 at the second turning point is determined using conventional methods for the box wall and cylindrical wall. For the box wall, the following approximate formula for column [5.2.5-1] is used:

Table 5.2.5-1. τ–γ relation.

	Conventional method	EPJR scheme
τ_1	$\tau_1 = 0.1F_C$	$\tau_1 = \sqrt{\sqrt{F_C}\left(\sqrt{F_C}+\sigma_V\right)}$
γ_1	$\gamma_1 = \tau_1 / {}_CG$	Same as left
τ_2	$\tau_2 = 1.5\tau_1$	$\tau_2 = 1.35\tau_1$
γ_2	Kokusho's formula, equation (5.2.5-6)	$\gamma_2 = 3\gamma_1$
τ_u	Box wall, cylindrical wall: Hirosawa's τ_u formula, equation (5.2.5-9); for PCCV, $\tau_u = 5.0\sqrt{F_C}$	Yoshizaki's formula, equation (5.2.5-11)
γ_u	Kokusho's formula, equation (5.2.5-6)	$\gamma_u = 4.0 \times 10^{-3}$

$$M_2 = 0.8a_t \cdot {}_s\sigma_y \cdot d + 0.5N \cdot d\left(1 - N/\left(A_g F_C\right)\right) \tag{5.2.5-16}$$

For the cylindrical wall (including PCCV), the formula of chimney [5.2.5-2] is used:

$$\left.\begin{aligned} M_2 &= 2tr^2 \sin\theta_0 \left(2{}_s\sigma_y P_v + 0.85F_C\right) \\ \theta_C &= \frac{1}{2{}_s\sigma_y \cdot P_v + 0.85F_C}\left(\frac{N}{2tr} + \pi{}_s\sigma_y \cdot P_v\right) \end{aligned}\right\} \tag{5.2.5-17}$$

where a_t: reinforcing bar cross-sectional area in a flange wall, A_g: total cross-sectional area, ${}_s\sigma_y$: reinforcing bar yield stress, d: distance between centers of tensile/compression flanges (d = 2r + t in the case of the cylindrical wall), r: wall center radius, t: wall thickness, P_v: longitudinal reinforcing bar ratio (%). For both the box wall and cylindrical wall, the curvature ϕ_2 at the second turning point is determined using the following formula from their respective values of M_2:

$$\Phi_2 = \frac{M_2}{\alpha_y \cdot K_e} \tag{5.2.5-18}$$

where, α_y is the reduction rate in stiffness:

$$\alpha_y = 0.15 + 0.3P_t \tag{5.2.5-19}$$

where $P_t = P_g/2$ (P_g: reinforcing bar ratio). On the other hand, according to the EPJR scheme, M_2 and ϕ_2 are taken as the flexural moment and curvature when the tensile reinforcing bar yields.

For the moment M_u at the ultimate point of cylindrical walls, in the case of the conventional analysis, the chimney formula (5.2.5-17) is used for evaluating the cylindrical wall in the same way as in the case of M_2 at the second turning point. For the box wall, the following Hirozawa's formula for M_u [5.2.5-4] is used:

$$M_u = 0.9a_t \cdot {}_s\sigma_y \cdot d + 0.4a_w \cdot {}_s\sigma_y \cdot d + 0.5N \cdot d(1 - N/(A_g F_C)) \tag{5.2.5-20}$$

where a_w is the web steel bar area. The curvature ϕ_u corresponding to this M_u value can be calculated as the curvature when the strain reaches 3000 μ for the extreme fiber of the concrete, under the following assumptions [5.2.5-3]:

{1} The concrete on the compression side is within the elastic range.
{2} The concrete does not bear tensile stress.
{3} The strain is assumed to remain in-plane.
{4} Although actually two or more rows of reinforcing bars are arranged, they are assumed to be replaced by an equivalent row of reinforcing bars located at the center of the thickness of the wall.

On the other hand, according to the EPJR scheme, calculation is made by using the total plastic formula, and the curvature corresponding to M_u is calculated as follows:

$$\left.\begin{array}{lll} \phi_u < 20\phi_2 & \text{then} & \phi_u = 0.004/X_{nu} \\ \phi_u \ge 20\phi_2 & \text{then} & \phi_u = 20\phi_2 \end{array}\right\} \tag{5.2.5-21}$$

where, X_{nu} is the distance from the extreme compression fiber to the centroid of the full-plastic cross-section. Table 5.2.5-2 summarizes the definitions of turning points and ultimate point for the M-ϕ curve.

b. Hysteresis loop of reinforced concrete shear wall

In order to perform elastoplastic seismic response analysis, after determining the skeleton curves, it is required to determine the hysteresis rule for repeated unloading and loading process. In the following, we will discuss the rules for determining the hysteresis rules in the elastoplastic seismic response analysis of the reactor building, with respect to the τ-γ relation and M-ϕ relation, respectively.

(a) Shear deformation (τ-γ relation)

Compared with the bending deformation, for the restoring force characteristics of the shear deformation, the hysteretic energy consumption is smaller. Figure 5.2.5-3 shows an origin-oriented model conventionally used as the hysteretic model of the shear deformation of the reactor building. Within the range where the absolute value of the shear strain γ does not exceed the previous maximum response value, the straight line connecting this maximum response point and the origin represents the restoring force characteristics. That is, during unloading from the skeleton curve, [the point] moves on the straight line toward the origin; for re-loading, it stays on the same straight line. Then, as the response point reaches the skeleton curve, the response point moves along the skeleton curve until unloading takes place. This model has the shear behavior feature that the hysteresis energy is not consumed at all if the previous maximum point is not exceeded; while the stiffness decreases together with the damage. However, as the response point returns to the origin during unloading, the residual strain due to damage accumulation cannot be taken into account.

Table 5.2.5-2. M-ϕ relation.

	Conventional method	EPJR scheme
M_1	$M_1 = (f_t + N/A_e) \cdot Z_e$ $f_t = 1.2\sqrt{F_C}$	Same as left
ϕ_1	$\phi_1 = M_1/({}_C E \cdot I_e)$	Same as left
M_2	Box wall: column approximate formula, equation (5.2.5-16) Cylindrical wall[1]: chimney formula, equation (5.2.5-17)	M_y [2]
ϕ_2	$\phi_2 = M_2/(\sigma_y \cdot K_e)$ $\sigma_y = 0.15 + 0.3P_t$	ϕ_y [2]
M_u	Box wall: Hiroxawa's M_u formula, equation (5.2.5-20) Cylindrical wall[1]: chimney formula, equation (5.2.5-17)	Full-plasticity formula
ϕ_u	Umemura, Takana's method	If $\psi_u < 20\phi_2$, $\phi_u = 0.004 / X_{nu}$ If $\phi_u \geq 20\phi_2$, $\phi_u = 20\phi_2$

[1]The cylindrical wall includes PCCV.
[2]M_y and ϕ_y are the flexural moment and curvature when the reinforcing bar on the tensile side yields.

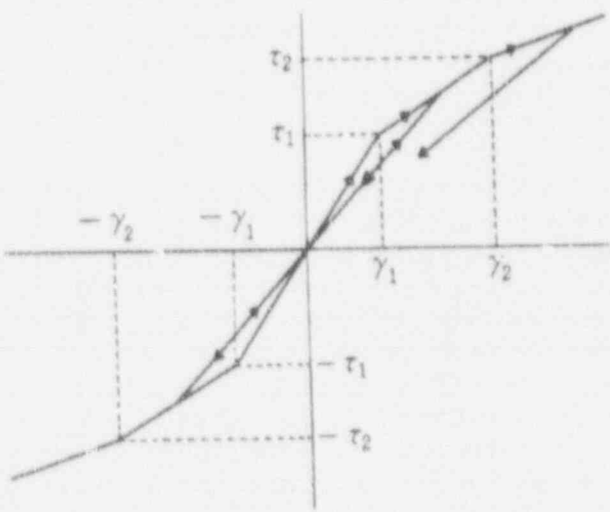


Figure 5.2.5-3. Origin-oriented model.

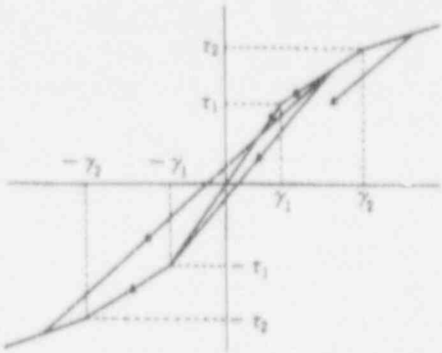


Figure 5.2.5-4. Maximum point-oriented model.

Different from this origin-oriented model, the EPJR scheme (Research, Construction-2 [Ken, Ken-2]) uses the maximum point-oriented hysteresis loop shown in Figure 5.2.5-4. In the range where the absolute value of the shear strain does not exceed the previous maximum response value, the restoring force characteristics are represented by the straight line connecting the maximum response point and the maximum response point on the opposite side (or the first turning point on the opposite side when the maximum response value on the opposite side has not exceeded the first turning point). That is, during unloading from the skeleton curve, the response point moves on the straight line toward the maximum response point on the opposite side. During re-loading, the response point stays on the same straight line. Then, as the response point reaches the skeleton curve, the response point moves on the skeleton curve until unloading takes place. Just as in the case of the origin-oriented model, in this model, too, no hysteresis energy is consumed in the range within the maximum response point. However, the decrease in stiffness in accompany with damage is taken into consideration. In addition, the effect of the residual strain caused by the damage which cannot be evaluated in the origin-oriented model is considered.

In the slip type model [5.2.5-8], the so-called slip phenomenon is taken into consideration in describing the restoring force characteristics of the shear deformation, i.e., in repeated deformation cycles with a medium or small amplitude after a large deformation, the stiffness is low; as the deformation increases, the stiffness increases, and the strength is also increased. It is believed that the slip phenomenon is caused by cracks in the concrete and failure of the bonding between the reinforcing bars and concrete. According to the slip type model, the loop area and the slope near the point with zero deformation (slip amount) can be evaluated more appropriately than in the origin-oriented model. This is an advantage. However, as the hysteresis rules are complicated, it is not yet actually used.

(b) Bending deformation (M-ϕ relation)

The restoring force characteristics of the bending deformation have the following features: with the same displacement amplitude, the hysteresis curve is more stable and the hysteresis energy area is also large; under a lower load, the stiffness is high, while as the load increases, the stiffness decreases, forming a spindle-shaped hysteresis loop. Usually, the maximum point-oriented model is used as the hysteresis model of the bending deformation. As the hysteresis rules and characteristics of the maximum point-oriented model have been discussed with respect to the shear deformation (τ-γ relation), they are not repeated here. However, when this model is used for the bending deformation (M-ϕ relation), there is a tendency to underestimate the hysteresis energy than in the actual phenomenon. Hence, the following degrading trilinear model has been proposed.

The model which is based on the trilinear skeleton curve and uses the Masing type hysteresis rule to represent the restoring force characteristics is called normal trilinear model. However, according to this model, as the decrease in stiffness does not take place during the unloading process, it is impossible to incorporate the maximum point-orientation property, the feature of the bending hysteresis of reinforced concrete; in addition, there is a tendency that the hysteresis energy consumption is overestimated compared with experimental results. In consideration of these problems, Fukada [5.2.5-27] proposed a degrading trilinear model by introducing a decrease in stiffness during unloading process into the normal trilinear model to enable change in the consumption of the hysteresis energy by adjusting the stiffness reduction rate. According to this model, for an amplitude smaller than the second turning point, it follows the same hysteresis rule as the conventional bilinear model. On the other hand, as unloading takes place from a point on the skeleton curve over the second turning point, the stiffness decreases, and [the point] is oriented to the original maximum response point on the opposite side according to the hysteresis rule of the bilinear model. In addition, in the stable loop after the second turning point, the hysteresis-related equivalent viscous damping becomes a certain constant value depending only on the ratio of yield strength to stiffness at the first and second turning points. This is another feature of this model.

Other degrading trilinear models for the elastoplastic analysis of the reactor building include the model proposed by Muto et al. [5.2.5-7] and the model proposed by EPJR (Research, Construction-2 [Ken, Ken-2]). The model proposed by Muto et al. is a combination of the two types: origin-oriented type and stiffness-degradation type. The hysteresis rules are as follows: in the region between the first and second turning points, it is an origin-oriented type; after the second turning point, it reaches the ϕ-axis with a stiffness which is determined by connecting the second turning point and the origin; after the ϕ-axis, it goes toward the original maximum point on the opposite side.

As shown in Figure 5.2.5-5, the degrading trilinear model of EPJR has the following features: Before the maximum response value exceeds the second turning point, the hysteresis rule is the same as that of the maximum point-oriented type model which is conventionally used. Hence, the stable loop has no area. As the maximum value exceeds the second turning point, a parallelogram-shaped stable loop pointing to the maximum response point on the opposite side (or the second turning point on the opposite side in the case when the maximum response value on the opposite side has not exceeded the second turning point) is defined, and the hysteresis energy is consumed. In this case, the shape of the parallelogram is determined by the fact that the equivalent viscous damping is given according to the maximum curvature. The turning point of the parallelogram is the point obtained by subtracting $2M_1$ from the maximum response value. The unloading stiffness of the stable loop is used as the stiffness for repetition within a stable loop.

(3) Restoring force characteristics of ground

a. Foundation uplifting nonlinearity

When the seismic input to the reactor building becomes large, uplifting of the foundation may take place. It is thus required to find the effects of this phenomenon on the foundation and the superstructure. Figure 5.2.5-6 shows the state of a uplifting foundation. In this case, the foundation is a rectangular rigid foundation mat with length L and width B. Acting on this foundation mat are a vertical force N and an overturning moment M. As moment M increases with respect to vertical force N, the foundation changes from a complete grounding state to a uplifting state, with the foundation rotated by θ with respect to the horizontal plane and partially peeled off from the foundation, causing a decrease in the grounding length to D. In this case, the earth pressure determined on the outermost edge of the grounding side of the foundation is called edge stress (P_t), and the moment under which the uplifting phenomenon starts taking place is called uplifting threshold overturning moment. In the case when the skeleton curve of the uplifting rocking spring is induced, the following assumptions are made:

{1} The linear distribution of the ground reaction force.
{2} No tensile force acts between the foundation and ground.
{3} The vertical force N is always constant, its applying point does not shift.

Of course, if assumption {1} is not made, the ground reaction distribution can be handled in a more realistic way [5.2.5-11, 5.2.5-12].

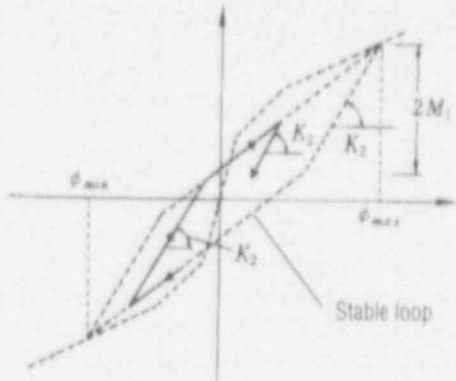


Figure 5.2.5-5. Degrading model.

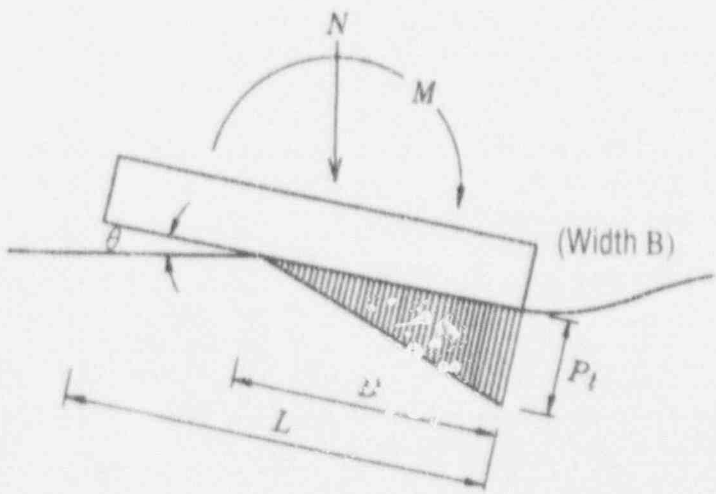


Figure 5.2.5-6. Foundation uplifting model.

Under the aforementioned assumptions {1}-{3}, the relation between moment M and rotational angle θ after uplifting is as follows:

$$\frac{M}{M_0} = 3 - 2\sqrt{\frac{\theta_0}{\theta}} \qquad (5.2.5\text{-}22)$$

where, M_0 is uplifting threshold overturning moment, and θ_0 is uplifting threshold rotational angle. When equation (5.2.5-22) is used, application of the vertical seismic force with respect to vertical force N becomes a problem. In this case, nonlinear analysis is performed for the uplifting phenomenon subjected to both the horizontal and vertical ground motions simultaneously. It has been found that the results obtained by using the M-θ relation with consideration of the input vertical motion are nearly the same as the results obtained by using the M-θ relation without considering the vertical motion. Hence, usually it is acceptable just to use the M-θ relation without taking the vertical motion into consideration (Research, Construction-13 [Ken, Ken-13]). Figure 5.2.5-7 shows the skeleton curve of equation (5.2.5-22). In the practical analysis, the multi-linear line indicated by the dash-dot line in the figure can be used as an approximation.

It is believed that due to uplifting of the foundation, the stiffness of the horizontal spring also changes depending on the contact rate just as in the case for the rotational spring. However, its influence on the actual response analysis is very small and can be ignored. As a result, the value before uplifting can be used directly regardless of the contact rate. In addition, as far as the change in the dissipation damping due to uplifting is concerned, usually, the values of the damping of the rotational spring and horizontal spring before uplifting can be used directly, with their dependence on the contact rate ignored in the analysis. On the other hand, EPJR (Research, Construction-13 [Ken, Ken-13] has proposed a scheme in which the damping coefficient of the rotational spring is changed in the same way as the reduction rate of the stiffness, while the value of the damping coefficient of the horizontal spring before uplifting is used directly.

b. Evaluation of contact rate

For seismic response analysis, the contact rate of the foundation mat can be evaluated by using the following formula from the static equilibrium between the maximum overturning moment and the ground reaction moment as derived from the seismic response analysis by ignoring the influence of the vertical earthquake motion and assuming a triangular distribution of the ground reaction force.

$$\eta = \frac{1}{2}\left(3 - \frac{M}{M_0}\right) \qquad (5.2.5\text{-}23)$$

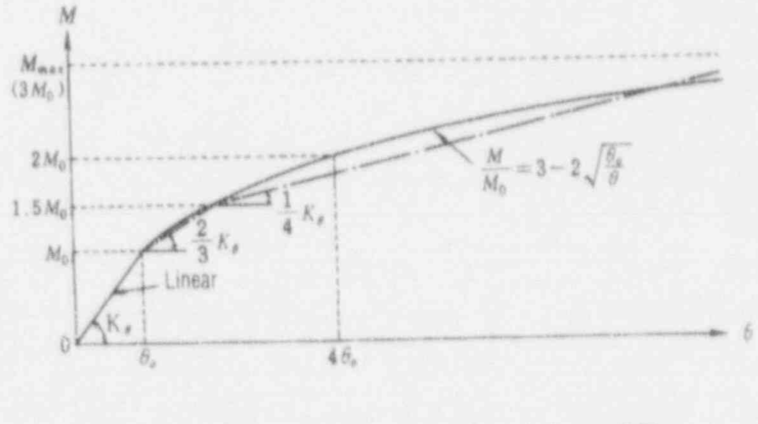


Figure 5.2.5-7. M-θ curve for foundation uplifting.

where, η is contact rate, M is the maximum overturning moment, and M_0 is uplifting threshold moment. In addition, also proposed is an energy balance scheme which can be used to derive the contact rate in a simpler way without resorting to the equilibrium of forces.

c. Effects of elastoplastic characteristics of ground materials

In addition to the elastoplastic characteristics of the building and the nonlinearity of the foundation uplifting, the nonlinearity of the embedment soil and other ground materials in the periphery of the building also affects the response behavior of the building. To account for the nonlinearity of the stress-strain relation of the soil, the seismic response analytical methods can be divided into the equivalent linearization method and the time-history integration method. According to the equivalent linearization method [5.2.5-13], the cyclic stress vs. strain relation of soil at a certain strain amplitude is represented by a linear viscoelastic model having an equivalent shear elastic coefficient and hysteretic damping constant, and the vibration analysis is performed in the frequency domain in which the shear elastic coefficient vs. strain amplitude relation and the hysteresis damping constant vs. strain amplitude relation are given. As this method usually has a shorter calculation time, it is effective for design purposes. However, because it does not account for the changes in time of the equivalent shear elastic coefficient and the hysteresis damping constant, it cannot be used in the case when the material characteristics of the soil change significantly in time, such as in the case of liquifaction. On the other hand, the time-history integration method is a method in which the time histories of the soil internal stress and strain are traced successively. It requires a hysteresis model for the stress-strain relation. Typical hysteresis models include the Ramberg-Osgood model [5.2.5-14], the Hardin-Drnevich model [5.2.5-15], the Martin-Davidenkov model [5.2.5-16], etc. In the case when the strain level is high and the soil material displays prominent nonlinearity, this method can provide analytical results more reliable than those obtained by using the equivalent linearization method. For the elastoplastic restoring force characteristics of the actual ground, in consideration of the fact that near the ground surface, the vertically incident SH wave component is prominent, only the relation between the shear strain and shear stress in the horizontal plane is taken into consideration in most of the current analyses.

(4) Nonlinear response analytical method

a. Nonlinear vibration equation

Analysis of the nonlinear vibration equations may be performed by using the incremental method, the iterative method, or a mixture of these two methods. For the static nonlinear analysis, various numerical analytical schemes have been proposed. On the other hand, for the dynamic nonlinear analysis, the effective analytical methods are limited because the time region is divided into short time intervals for numerical analysis. Usually, the factors considered in selecting the nonlinear analysis method are as follows:

{1} Loading condition on the system
{2} Restoring force characteristics of the system
{3} Required analytical accuracy
{4} Calculation size and time

In the practical analysis, it is required to select the most suitable nonlinear analytical method on the basis of the above factors.

The incremental equations of motion used in the nonlinear response analysis is as follows [5.2.5-17].

$$M_t\Delta\ddot{U}_t + C_t\Delta\dot{U}_t + K_t\Delta U_t = R'_{t+\Delta t} \tag{5.2.5-21}$$

$$R'_{t+\Delta t} = R_{t+\Delta t} - F_i - F_d - F_e \tag{5.2.5-25}$$

where M_t, C_t, K_t are the mass, damping and stiffness matrices at time t, respectively; $\Delta\ddot{U}_t$, $\Delta\dot{U}_t$, ΔU_t are increments of acceleration, velocity, and displacement, respectively; F_i, F_d, F_e, R_t are the inertial force, damping force, internal resistant force and external force vectors, respectively.

At present, there is yet no established theory for evaluation of the viscous damping effect after a portion of the building goes outside the linear region and enters the plastic region. Evaluation may be performed with the viscous damping assumed to be constant without change from that in the linear region, or, it may also be performed by using a variable damping assumed to be proportional to the tangential stiffness. Equation (5.2.5-24) is for the case with a variable damping. In the case of constant damping, it becomes the following form:

$$M\Delta\ddot{U}_t + C\Delta\dot{U}_t + K\Delta U_t = R_{t+\Delta t} - M\ddot{U}_t - C\dot{U}_t - F_e \tag{5.2.5-26}$$

b. Nonlinear numerical calculation method

The typical methods that can be used to perform successive integration of increment-type vibration equations (5.2.5-24) or (5.2.5-26) for each calculation time interval include the Newma[illegible] ethod and the Wilson-θ method [5.2.5-17]. Both of those two methods are considered to be a modified or i[illegible] d version of the linear acceleration method, which assumed a linear change of acceleration over the time p[illegible] om t to t + Δt.

For the incremental method using the above successive integration scheme, in the minute calculation time interval Δt from a certain time point to the next time point, a varying external force increment is applied on the system; during the period, the system is represented by a linear vibration equation with the restoring force characteristics of the system assumed to be unchanged. Although these methods are usually effective in analyzing the nonlinear vibration behavior, it is nevertheless required to solve for the inverse matrix of the linear combination of mass, damping, and stiffness matrices during the calculation process. This calculation must be carried out corresponding to the stiffness matrix and damping matrix which vary at each time point of the calculation process. Therefore, as the degrees of freedom are increased, a longer calculation time is needed.

On the other hand, for the iterative method, although the iteration number is large, there is nevertheless no need to calculate the inverse matrix; hence, the calculation time can be significantly shortened as compared to the incremental method for solving certain problems. A commonly well-used effective iteration method in nonlinear vibration analysis is the load correction method [5.2.5-18]. In this method, the matrix is determined by using the initial stiffness defined in the linear region. With this matrix taken as unchanged, the external forces are adjusted iteratively to meet the nonlinearity of the vibration equations.

c. Points for attention in the numerical calculation

(a) Treatment of turning points due to variation in the stiffness

The turning points, which occur when the stiffness in the next time point differs from the stiffness in the preceding time point, include the following two types:

{1} The direction of component deformation changes.
{2} Although the direction of the component deformation is the same as in the preceding step, a turning point is overpassed.

The following 4 schemes may be used to perform numerical integration for a time interval which includes a turning point.

{1} A method in which the unbalanced force of the component overpassing the turning point is calculated and is transformed to the unbalanced nodal forces of the vibration equations; in the next step, the unbalanced force is subtracted from the external nodal forces.
{2} A method in which when the turning point is overpassed at the (i + 1)th step, the boundary time is calculated to estimate the weighting factors in terms of the time interval. Using this weighting factors and linear interpolation, the stiffness of i-th and (i + 1)-th steps are combined to correct the stiffness matrix and damping matrix; the recalculated values are taken as the values of the (i + 1)th step; then, in the following (i + 2)th step, correction is made again for the correct stiffness in the interval. In this way, the calculation is moved forward.
{3} A method in which the boundary point is calculated by interpolating the response values of the two points sandwiching the boundary point.
{4} A method in which the calculation time interval is redivided to be finer in the vicinity of the turning point, so that the unbalanced force due to variation in the stiffness can be minimized.

Among the aforementioned methods, with methods {1} and {2} can be performed with a constant time interval; methods {3} and {4} can be made by varying the time interval.

(b) Treatment of moment within element

For the rotational angle vs. moment relation and shear deformation vs. shear force relation, the values are taken as constant for each element. On the other hand, for the curvature vs. moment relation, the values of the curvature are different at the two ends of the beam component. Hence, the following adjusting methods are used.

{1} The end moments of an element are averaged to calculate curvature and to judge yielding.
{2} The larger moment is used to calculate the curvature and to judge yielding.
{3} Each end is evaluated separately, and the stiffness reduction rates are averaged.
{4} The beam model using the transfer matrix method is applied.

In addition, the following models may also be used: plastic hinge method, dividing beam model, parabolic model, condensation method, etc.

(c) Time interval and accuracy of solution of numerical integration

In the nonlinear seismic response analysis, due to the decrease in stiffness, high-order frequency components appear significantly on the response acceleration waveform, and the waveform becomes disturbed. As a result, the maximum value of the response acceleration is sensitive to the change in the calculation time interval. Hence, in order to ensure accuracy in the nonlinear seismic response analysis of the same order as that in the linear analysis, it is required to make the calculation time interval much shorter than that used in the case of linear analysis. While in the linear analysis, the results obtained with Δt = 0.002 sec are almost the same as those obtained with Δt = 0.008 sec, in the nonlinear analysis, significant differences are developed between the case with Δt = 0.004 sec and the case with Δt = 0.001 sec: about 6% for the response acceleration, about 5% for the shear force, and about 9% for the overturning moment; as the results obtained with Δt = 0.002 sec are compared with those obtained with Δt = 0.001 sec, the difference is about 5% for the response acceleration and about 3% for the shear force and overturning moment. As a result, some authors indicated that Δt = 0.002 sec might be a reasonable value for performing the nonlinear analysis [5.2.5-24].

(5) Nonlinear response characteristics

a. Nonlinear response behavior of building

The nonlinear analysis in the aseismic design of the reactor building is performed to find the response behavior of the building subjected to the basic earthquake ground motion S_2. Usually, as the plasticity increases, when the response of an elastoplastic system is compared with the response of an elastic system, the displacement increases while the restoring force decreases. Hence, as the results of the electroplastic analysis are compared with those of the elastic analysis, in the case when the ductility factor is not very large, although the response displacement is increased a little, the response acceleration is reduced; even in the case when the elastic analytical results reach the yield strength of the design, it is still possible that there exists a certain margin in the yield strength.

Up to now, the aseismic design of the reactor building is primarily performed by using elastic analysis. However, in order to provide sufficient safety margin in maintaining the functions of the building as well as the equipment/piping system, it is also required to investigate the plastic behavior for the sake of economical design of buildings. In order to perform rationale aseismic design with elastoplastic behavior taken into consideration, the correct response state of the building should be found. For this purpose, it is very important to perform elastoplastic analysis by using the skeleton curve and hysteresis loop which are able to fully represent the restoring force characteristics of the plastic region beyond the first turning point. In addition, the results of the nonlinear response analysis are also used in the study of the function maintenance and the safety margin as discussed in section 5.3.4 "Investigation of the maintenance of the function" and in section 5.3.5 "Safety margin."

b. Effect of nonlinear analysis on the floor response spectrum

Newmark has proposed a method in which the ductility factor is used to derive the nonelastic design spectrum from the elastic design spectrum [5.2.5-21, 5.2.5-22]. The nonlinear responses of the equipment system were studied using this method; as the mass ratio of the equipment to the building is increased, in addition to the independent damping mechanisms of the building and equipment, the damping mechanism due to the interaction between these two portions also plays an important role. In this case, it is required to perform analysis of the building-equipment coupled system. However, in the case of a small mass ratio, the dynamic behavior of the equipment is usually evaluated by using the floor response spectrum. When the nonlinear seismic response analysis is performed, the important data in the equipment design include the changes in the shape of the floor response spectrum and the difference compared with the results of linear analysis. The factors that affect the floor response spectrum by the nonlinear analysis include the characteristics of the input earthquake motion, characteristics of the restoring force of the building, uplifting characteristics, etc. These factors are combined to display very complicated properties. Attention should also be paid to the accuracy of the numerical integration method used for the analysis.

In the nonlinear analysis, usually the resonant frequency of the system is decreased. However, as the peak of the spectrum shifts towards the side of longer period, at the same time, responses tend to increase at the short-period portion. It has been found that this increase on the short-period portion tends to become more significant for a nonlinear elastic system that is represented by a uplifting nonlinearity, and it tends to become less significant for an elastoplastic system. However, further investigation is required in order to find the general trend with respect to the relation with the input seismic wave characteristics and selection of numerical integration. Kawakatsu et al. [5.2.5-23] have performed spectral analysis for the nonlinear floor response of buildings having origin-oriented type, degrading trilinear type, or slip type recovery force characteristics, and have investigated the effects of the input earthquake motion and the restoring force characteristics of the building. It has also been reported that if the foundation uplifting nonlinearity is analyzed directly as a continuous function without using the trilinear model, the response increase in the short-period portion becomes less significant. In addition to analytical works, foundation uplifting tests are also being performed to determine whether the response increase in the short-period portion during uplifting is caused by second-order or higher-order periods of the building, or caused by the second and third high-frequency components of the ground motion itself.

5.2.6 Investigation of the building stability

As explained above, from the seismic response analysis, the response acceleration, velocity, displacement, etc., of the building can be obtained. In addition, the overturning moment of the overall building and the shear force at the building base can be obtained. The ground should be stable against these forces. In order to confirm the stability of the building, evaluation is made of two factors: contact pressure of the foundation bottom and sliding of the building. For the foundation uplifting phenomenon, it may be analyzed using an appropriate analytical scheme, such as that described in Section 5.2.5 (3) "Restoring force characteristics of ground." The seismic forces required for the stability evaluation include both the dynamic seismic force and the static seismic force.

(1) Evaluation of contact pressure

Evaluation of the contact pressure is made to see whether the contact pressure, which is evaluated by taking the vertical seismic force and underground water buoyancy into consideration together with the maximum overturning moment calculated by a static analysis or the dynamic seismic response analysis, is within the margin limits. Calculation of the contact force is done according to Clause 19: "Regulations on independent footing foundation" [5.2.6-3] in "Rules and explanation of construction base structure design," by the Architectural Institute of Japan (1974).

According to the "Evaluation guidelines," the allowable limits are defined as follows: "The allowable stress determined to be appropriate with respect to the basic earthquake ground motion S_1, etc., is taken as the allowable limits. In addition, with respect to basic earthquake ground motion S_2, it should have an appropriate safety margin over the ultimate strength." This is explained as follows: "The allowable limits for the contact pressure is defined as the short-term allowable support force with respect to the basic earthquake ground motion S_1 (2/3 the limit bearing capacity [5.2.6-3]), and it should have an appropriate safety margin to the limit bearing capacity (strength of the foundation) with respect to the basic earthquake ground motion S_2."

With respect to the basic earthquake ground motion S_1, the design is usually selected to ensure that the response of the building is not too far away from the elastic behavior. However, if the contact pressure is below the short-term allowable stress, it is also possible to carry out the design by evaluating the influence of uplifting of the foundation. In order to find the safety margin with respect to the limit bearing capacity for the basic earthquake ground motion S_2, the fact that a safety factor of 1.5 is used for S_1 can be regareded as a basis for engineering judgment.

(2) Evaluation of sliding

Just as in the case of the contact pressure, evaluation of sliding is also performed by taking the vertical seismic force and water buoyancy into consideration together with the shear force transferred from the building to the foundation as determined by the static analysis or the dynamic seismic response analysis. Evaluation of sliding include the following two items:

{1} Evaluation of sliding only at the foundation bottom
{2} Evaluation of sliding with embedment taken into consideration

Selection of the above two items is made with respect to the response analysis model. The rule of selection is as follows: for the soil-structure interaction, if only the foundation bottom is taken into consideration, {1} is applied; if embedment is taken into consideration, {2} is applied.

For evaluation of sliding of the foundation bottom, a conventionally used scheme is to divide the shear resistance [5.2.6-1] by the safety factor (see the above description on the evaluation of the contact pressure), and use it as the allowable limit.

$$H_u = CA + V \tan\phi \qquad (5.2.6\text{-}1)$$

where H_u: shear resistance acting between the bottom and ground (tf)
V: vertical force acting on the bottom (tf)
A: effective load-bearing area of bottom (m^2)
C adhesive force between bottom and base (tf/m^2)
ϕ frictional angle between bottom and ground (degrees)

When embedment is taken into consideration, the appropriate evaluation method should be selected with respect to the dynamic analytical method. For example, when a dynamic analysis using 2-dimensional FEM model is adopted, the obtained response is transformed to the equivalent static load; this load is applied on a sliding plane in the FEM model, and the shear resistance and safety factor are calculated for this plane, and the results are compared with the margin limit [5.2.6-2]. In this case, as sliding of the ground is included in the investigation, it is desirable that the safety factor be set according to Chapter 4 "Safety evaluation of ground and aseismic design of underground structures."

5.3 Stress analysis and structural design

5.3.1 Introduction

Stress analysis and evaluation of the various parts of structure are done by using the stress and deformation determined using the static seismic force calculation method and the linear or nonlinear response analysis shown in the section of seismic response analysis. The stress analysis method, including modeling of the various components of the structure, should be determined by taking the shape of the structure and load conditions into consideration. The structure and form of a reactor building are complex, and the wall thickness and slab thickness of the structure are much larger than those of the conventional building. Hence, the FEM analysis is mainly used as stress analysis. In this section, we will present in the form of lists the analytical methods of the various parts of BWR MARK-I, BWR MARK-II, PWR 2 LOOP 2, 3 LOOP, and 4 LOOP. In addition, the following major items in stress analysis will be discussed.

{1} Input method and modeling of composite structure
{2} Formation of analytical model for thick concrete structures of the containment facilities such as foundation mat
{3} Evaluation of springs used in stress analysis of foundation mat
{4} Method of consideration of earth pressure in stress analysis
{5} Treatment of thermal stress in combination with S_1 seismic stress
{6} Accuracy of FEM analysis

Proportioning of the cross sections of the various parts of the structure is performed in principle by using the various rules just as for the conventional building. As pointed out above, however, the walls of the reactor building are thicker than those of conventional buildings and the reactor building is partially of a complex shape. Therefore, certain special considerations may be required. The following features will be explained in particular for reference.

{1} Evaluation method of combined stress
{2} Evaluation method of the cross section of foundation mat or other thicker concrete components.
{3} Design method of anchor bolts
{4} Evaluation method of flat slab structure
{5} Evaluation method of shear wall with openings
{6} Composite structure
{7} Splicing method for large-diameter reinforcing bars

With respect to the stress in S_1 and S_2 earthquakes, and the stress under required load combination, it is required to investigate the function of preventing leakage, function of preventing successive accident, and supporting function. As far as the limits for maintaining said functions of the various portions of the building, there is no quantitative standard at present. At present, as a rule of thumb, the criteria of S_1 for the allowable stress level design, and that of S_2 and S_1 + LOCA for the ultimate strength design are commonly used. In the following, we will discuss the treatment method of the allowable limit values for the various functions required.

In the section of safety margins, an evaluation is shown with respect to the static seismic force and the dynamic seismic force. For the safety margin with respect to static seismic force, although the quantitative standards are not yet determined, those for conventional buildings may be taken as a reference for comparison. For the safety margin with respect to the dynamic seismic force, although the value of the margin index is not yet explicitly shown, in order to cover the safety requirement, a sufficient margin is made in the actual design. Researches are now on-going for the margin index and its quantitative standards. It will be possible to make a judgment from the results of this research work in the future.

5.3.2 Stress analysis

(1) Outlines of building shape and structural form

In order to determine the method for stress analysis and the method of modeling, it is required to make a good consideration of the type of the structure (reinforced concrete structure, steel frame/reinforced concrete structure, steel frame structure, etc.) and the shape of the structure (cylindrical, square, etc.). Both the BWR and PWR reactor buildings have their primary structures made of reinforced concrete. In many cases, however, the columns of the uppermost story of BWR are made of steel frame/reinforced concrete structures. In addition, in some cases, the roof and floor are made of steel frame structure. The fuel applying building of PWR also has a steel frame structure. In addition, in some cases, the pedestal of the BWR reactor pressure containment and the internal concrete of PWR have steel frame/reinforced concrete structure. The shell wall of BWR MARK-II, the external shielding building of PWR 2 LOOP and 3 LOOP, and PCCV of 4 LOOP have a cylindrical shell structure made of reinforced concrete. The inner concrete structure of the reactor containment vessels of PWR 2 LOOP, 3 LOOP, and 4 LOOP is a structure made of polygonal wall corresponding to the LOOP number. The spent fuel pool of BWR and the spent fuel pool of PWR are made of thick concrete walls and basemat. They are box-shaped structures supported on a building or foundation. For the peripheral buildings of BWR, PWR, and box-shaped wall and the orthogonal walls are the major aseismic elements. In addition, the foundation mat shared by BWR and PWR has a thick concrete structure. For both BWR and PWR, these different structures of the reactor building are set on a single foundation mat. Tables 5.3.2-1 to 5.3.2-4 list the structural forms of the various parts of BWR MARK-I, MARK-II, PWR 2 LOOP, 3 LOOP, and 4 LOOP. Young's modulus of the concrete used in the stress analysis is calculated according to "RC Standards."

(2) Loading conditions

Another important factor in determining the stress analysis method and modeling is related to the loading conditions which are applied to the model or obtained from the analysis. For example, when the analytical method and analytical model are to be determined for the shear wall, foundation slab, etc., it is necessary to determine whether the load conditions of the analytical object are in-plane type, out-of-plane type, or their combination. In addition, when the seismic stress is to be calculated for each component, it is required to use a stress analytical model, where shear force, flexural moment, acceleration, and displacement obtained from the static and response analyses, are applied in an appropriate and conservative faction.

Table 5.3.2-1. Present status of stress analysis and evaluation methods (BWR MARK-I reactor building).

Component			External/internal walls	Shell wall	Foundation of reactor	Spent fuel pit	Frame	Foundation mat
Structural form			Wall structure	Wall structure Upper portion: spherical segment shape Lower portion: cylindrical shape	Wall structure	Wall structure	Wall structure	Plate structure
Stress analysis	Vertical load		Frame analysis	FEM analysis	FEM analysis	FEM analysis	Frame analysis	FEM analysis
	Horizontal load	S_1 earthquake[3]	Frame analysis	FEM analysis	FEM analysis	FEM analysis	Frame analysis	FEM analysis
		S_2 earthquake	—[4]	—[4]	FEM analysis	FEM analysis	—	FEM analysis
	Load of LOCA (except thermal load)		—	—	FEM analysis	—	—	—
	Thermal load		—	FEM analysis	FEM analysis	FEM analysis	—	—

[3]The dynamic earthquake force based on basic earthquake ground motion S_1, and the static earthquake force are represented.
[4]The safety margin is investigated using the seismic response analysis results.

Table 5.3.2-1 *(Cont'd)*. Present status of stress analysis and evaluation methods (BWR MARK-I reactor building).

Component		External/internal walls	Shell wall	Foundation of reactor	Spent fuel pit	Frame	Foundation mat
Structural form		Wall structure	Wall structure Upper portion: spherical segment shape Lower portion: cylindrical shape	Wall structure	Wall structure	Wall structure	Plate structure
Evaluation methods	Normal operation	Long-term allowable stress design	Long-term allowable stress design	Long-term allowable stress design	Long-term allowable stress design	Long-term allowable stress design	Long-term allowable stress design
	S_1 earthquake[3]	Short-term allowable stress design	Short-term allowable stress design	Short-term allowable stress design	Short-term allowable stress design	Short-term allowable stress design	Short-term allowable stress design
	S_2 earthquake	Study on safety margin	Study on safety margin	Ultimate strength design	Ultimate strength design	—	Ultimate strength design
	LOCA[2]	—	Short-term allowable stress design	Short-term allowable stress design	Short-term allowable stress design	—	—
	S_1 + LOCA[2]	—	—	Ultimate strength design	Ultimate strength design	—	—
	In storm or heavy snow	—	—	—	—	Short-term allowable stress design[1]	—

[1]Generally, short-term allowable stress design is performed, but in specific region, long-term allowable stress design is performed.
[2]For the pressure and temperature of LOCA, the time lag is taken into consideration.
[3]The dynamic earthquake force based on basic earthquake ground motion S_1, and the static earthquake force are represented.

Table 5.3.2-2. Present status of stress analysis and evaluation methods (BWR MARK-II reactor building).

Component			External/internal walls	Shell wall	Foundation of reactor	Floor diaphragm	Spent fuel pit	Frame	Foundation mat
Structural form			Wall structure	Wall structure Upper portion: spherical segment shape Lower portion: cylindrical shape	Wall structure	Plate structure	Wall structure	Wall structure	Plate structure
Stress analysis	Vertical load		Frame analysis	FEM analysis	FEM analysis	FEM analysis	FEM analysis	Frame analysis	FEM analysis
	Horizontal load	S_1 earthquake[3]	Frame analysis	FEM analysis	FEM analysis	FEM analysis	FEM analysis	Frame analysis	FEM analysis
		S_2 earthquake	—[4]	—[4]	FEM analysis	—	FEM analysis	—	FEM analysis
	Accident load (except thermal load)		—	—	FEM analysis	FEM analysis	—	—	FEM analysis
	Thermal load		—	FEM analysis	FEM analysis	FEM analysis	FEM analysis	—	FEM analysis

[3]The dynamic earthquake force based on basic earthquake ground motion S_1, and the static earthquake force are represented.

[4]The safety margin is investigated using the seismic response analysis results.

Table 5 3.2-2 *(Cont'd)*. Present status of stress analysis and evaluation methods (BWR MARK-II reactor building).

Component		External/internal walls	Shell wall	Foundation of reactor	Floor diaphragm	Spent fuel pit	Frame	Foundation mat
Structural form		Wall structure	Wall structure Upper portion: spherical segment shape Lower portion: cylindrical shape	Wall structure	Plate structure	Wall structure	Wall structure	Plate structure
Evaluation methods	Normal operation	Long-term allowable stress design	Long-term allowable stress design	Long-term allowable stress design	Long-term allowable stress design	Long-term allowable stress design	Long-term allowable stress design	Long-term allowable stress design
	S_1 earthquake[3]	Short-term allowable stress design	Short-term allowable stress design	Short-term allowable stress design	Short-term allowable stress design	Short-term allowable stress design	Short-term allowable stress design	Short-term allowable stress design
	S_2 earthquake	Study on safety margin	Study on safety margin	Ultimate strength design	—	Ultimate strength design	—	Ultimate strength design
	LOCA[2]	—	Short-term allowable stress design	Short-term allowable stress design	Short-term allowable stress design	Short-term allowable stress design	—	Short-term allowable stress design
	S_1 + LOCA[2]	—	—	Ultimate strength design	Ultimate strength design	Ultimate strength design	—	Ultimate strength design
	In storm or heavy snow	—	—	—	—	—	Short-term allowable stress design[1]	—

[1]With snow deposits, the long-term allowable stress design should be performed according to the specific region.
[2]For the pressure and temperature of LOCA, the time lag is taken into consideration.
[3]The dynamic earthquake force based on basic earthquake ground motion S_1, and the static earthquake force are represented.

Table 5.3.2-3. Present status of stress analysis and evaluation methods (PWR 2 LOOP, 3 LOOP reactor building).

Component			Outer shield building	Inner concrete structure	Reactor external building: Spent fuel pit	Reactor external building: Fuel handling bldg.	Reactor external building: Other buildings	Foundation mat
Structural form			Cylindrical shell structure	Made of shear walls with irregular closed cross section and floor panel having multiple openings	Walls, floor panels	Steel frame and brace structure	Wall/flat slab structure; having multiopening shear walls	Plate structure
Stress analysis	Vertical load		Axisymmetric shell or FEM analysis	Frame analysis or FEM analysis	Frame analysis or FEM analysis	Frame analysis	Frame analysis or FEM analysis	—
Stress analysis	Horizontal load	S_1 earthquake[3]	Axisymmetric shell or FEM analysis	Frame analysis or FEM analysis	Frame analysis or FEM analysis	Frame analysis	Frame analysis or FEM analysis	FEM analysis
Stress analysis	Horizontal load	S_2 earthquake	—[4]	—[4]	Frame analysis or FEM analysis	—	—	FEM analysis
Stress analysis	Accident load (except thermal load)		—	Frame analysis or FEM analysis	—	—	—	—
Stress analysis	Temperature load		Axisymmetric shell or FEM analysis	FEM analysis	FEM analysis	—	—	FEM analysis (including outer shield bldg., internal concrete, and peripheral bldg.)

[3]The dynamic earthquake force based on basic earthquake ground motion S_1, and the static earthquake force are represented.
[4]The safety margin is investigated using the seismic response analysis results.

Table 5.3.2-3 *(Cont'd)*. Present status of stress analysis and evaluation methods (PWR 2 LOOP, 3 LOOP reactor building).

Component		Outer shield building	Inner concrete structure	Reactor external building			Foundation mat
				Spent fuel pit	Fuel handling bldg.	Other buildings	
Structural form		Cylindrical shell structure	Made of shear walls with irregular closed cross section and floor panel having multiple openings	Walls, floor panels	Steel frame and brace structure	Wall/flat slab structure; having multiopening shear walls	Plate structure
Evaluation methods	Normal operation	Long-term allowable stress design	Long-term allowable stress design	Long-term allowable stress design	Long-term allowable stress design	Long-term allowable stress design	Long-term allowable stress design
	S_1 earthquake[3]	Short-term allowable stress design	Short-term allowable stress design	Short-term allowable stress design	Short-term allowable stress design	Short-term allowable stress design	Short-term allowable stress design
	S_2 earthquake	Investigation of safety margin	Investigation of safety margin	Ultimate strength design	Investigation of safety margin	Investigation of safety margin	Ultimate strength design
	LOCA[2]	Ultimate strength design	Ultimate strength design	—	—	—	Ultimate strength design
	S_1 + LOCA[2]	Ultimate strength design	Ultimate strength design	—	—	—	Ultimate strength design
	In storm or heavy snow	—	—	—	Short-term allowable stress design[1]	—	—

[1]With snow deposits, the long-term allowable stress design should be performed according to the specific region.
[2]For the pressure and temperature of LOCA, the time lag is taken into consideration.
[3]The dynamic earthquake force based on basic earthquake ground motion S_1, and the static earthquake force are represented.

Table 5.3.2-4. Present status of stress analysis and evaluation methods (PWR 4 LOOP reactor building).

Component			Containment	Inner concrete structure	Reactor external building: Spent fuel pit	Reactor external building: Fuel handling bldg.	Reactor external building: Other buildings	Foundation mat
Structural form			Cylindrical shell structure	Made of shear walls with irregular closed cross section and floor panel having multiple openings	Walls, floor panels	Steel frame and brace structure	Wall structure, having shear walls with multiple openings	Plate structure
Stress analysis	Vertical load		FEM analysis	FEM analysis	Frame analysis or FEM analysis	Frame analysis	Frame analysis or FEM analysis	
Stress analysis	Horizontal load	S_1 earthquake[3]	FEM analysis	FEM analysis	Frame analysis or FEM analysis	Frame analysis	Frame analysis or FEM analysis	FEM analysis
Stress analysis	Horizontal load	S_2 earthquake	FEM analysis (confirmation of bldg.'s durability, maintenance of functions)	—[4]	Frame analysis or FEM analysis	—[4]	—[4]	FEM analysis
Stress analysis	Accident load (except thermal load)		FEM analysis	FEM analysis	—	—	—	FEM analysis
Stress analysis	Temperature load		FEM analysis	FEM analysis	FEM analysis	—	—	FEM analysis

[3]The dynamic earthquake force based on basic earthquake ground motion S_1, and the static earthquake force are represented.
[4]The safety margin is investigated using the seismic response analysis results.

Table 5.3.2-4 *(Cont'd)*. Present status of stress analysis and evaluation methods (PWR 4 LOOP reactor building).

Component		Containment	Inner concrete structure	Reactor external building			Foundation mat
				Spent fuel pit	Fuel handling bldg.	Other buildings	
Structural form		Cylindrical shell structure	Made of shear walls with irregular closed cross section and floor panel having multiple openings	Walls, floor panels	Steel frame and brace structure	Wall structure, having shear walls with multiple openings	Plate structure
Evaluation methods	Normal operation	Based on the scheme shown in "5.4. Concrete containment."	Long-term allowable stress design	Long-term allowable stress design	Long-term allowable stress design	Long-term allowable stress design	Based on the scheme shown in "5.4. Concrete containment."
	S_1 earthquake[3]		Short-term allowable stress degree design	Short-term allowable stress degree design	Short-term allowable stress degree design	Short-term allowable stress degree design	
	S_2 earthquake		Safety margin investigation	Ultimate strength design	Safety margin investigation	Safety margin investigation	
	LOCA[2]		Ultimate strength design	—	—	—	
	S_1 + LOCA[2]		Ultimate strength design	—	—	—	
	In storm or heavy snow		—	—	Short-term allowable stress degree design[1]	—	

[1]With snow deposits, the long-term allowable stress design should be performed according to the specific region.
[2]For the pressure and temperature of LOCA, the time lag is taken into consideration.
[3]The dynamic earthquake force based on basic earthquake ground motion S_1, and the static earthquake force are represented.

(3) Analytical methods

a. Analytical models and modeling

As pointed out above, the reactor building has a complicated structure with structural walls and slabs thicker than those of conventional buildings. Hence, in many cases, the entire building and its various portions are analyzed using FEM method. Tables 5.3.2-1 to 5.3.2-4 list the analytical methods of the various portions (buildings) of BWR MARK-I, MARK-II, PWR 2 LOOP, 3 LOOP, and 4 LOOP. Figure 5.3.2-1 shows an example of the analytical model of PWR 3 LOOP.

As far as the stress analysis is concerned, the elastic analysis is usually done with respect to the short-term allowable stress design and the ultimate strength design. In the recent years, for the ultimate strength design, partial elastoplastic analysis is also carried out.

b. Points for attention with respect to stress analysis

For the structural analysis of conventional buildings, a reasonable simplified analytical scheme is adopted. However, for reactor buildings, because the structure and form are complicated as pointed out above, FEM analysis is primarily used for conducting the stress analysis. For the case which is difficult to use the existing standards, calculations of the cross section and evaluation of the safety are done by taking the experimental results, etc., as reference. For stress analysis, the primary points for attention are as follows:

(a) Input method and model formation scheme for composite structures

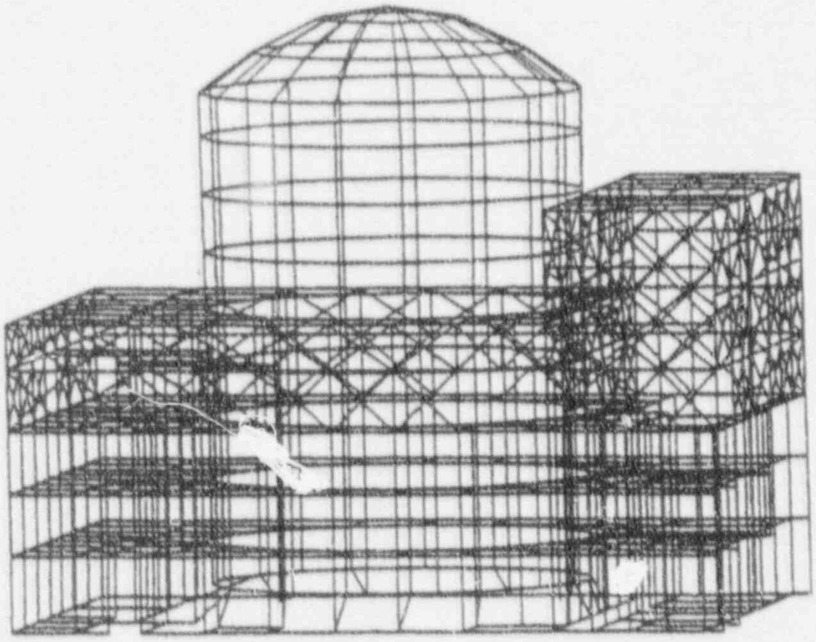
First, let's look at the input method of the seismic response analysis results to the stress analysis model. For BWR as well as PWR 2 LOOP, 3 LOOP reactor buildings, they are usually composite structures made of different types of structures. For these composite structures, it is required to consider the input method for deriving the appropriate stress for the cross-sectional design of the various portions.

At present, the seismic response analysis is usually carried out by using the lumped mass model shown in Section 5.2 "Seismic response analysis." For stress analysis of the upper building, the maximum value of the acceleration or shear force output in the seismic response analysis is adopted, and nodal load or body force is acted on the FEM model. As a result, the derived stress of each part becomes more or less the evaluation on the side of safety. However, if the same detailed model is used for both the response and stress analysis, the computer requires an extremely long computing time. This problem is to be solved in the future.

As far as the analysis of foundation is concerned, the basic scheme used is that the stress for the maximum overturning moment is given as the external force according to the upper structure's reaction force distribution, to derive the stress; then, a partial engineering check-up is carried out for the stress during operation or in case of accident. For the method of distribution of static seismic force in a composite structure, various methods can be used for calculating the story shear force coefficient in a parallel structure with different characteristics. As far as the distribution of shear force is concerned, it is believed to be more reasonable to consider dynamic vibration properties (such as the tendency of participation factors) as much as possible.

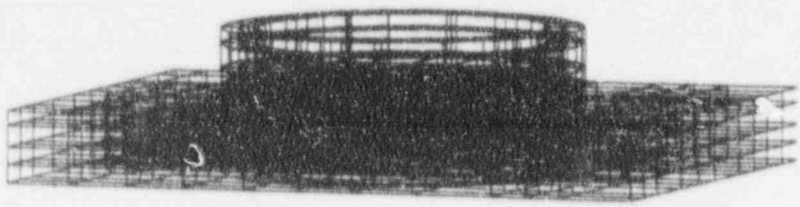
(b) Formation of analytical models for containment facility's foundation mat and other thick concrete structures

When models for FEM analysis are to be formed for the containment facility's foundation mat and other thick concrete structures, the appropriate element type for analysis model should be selected in consideration of the geometric shape of the foundation and matching of the upper structure and the foundation. Usually, in the three-dimensional FEM analysis of a thick mat part, such as the foundation mat of a reactor building, solid elements are adopted. In addition, plate elements with out-of-plane shear taken into consideration may also be adopted.

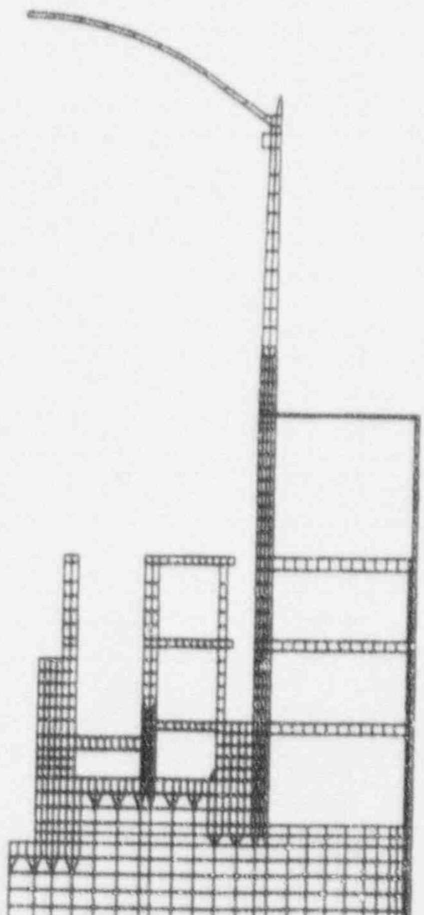


External shielding building, peripheral building

Figure 5.3.2-1(a). Stress analysis model of PWR 3 LOOP reactor building.



Foundation mat



Thermal stress analytical model

Figure 5.3.2-1(b). Stress analysis model of PWR 3 LOOP reactor building.

(c) Evaluation of spring in stress analysis of foundation mat

When the stress analysis of the foundation mat is performed, in addition to the FEM model of the soil underneath and surrounding the foundation, the discrete spring method may also be used as the calculation method. In this case, the value of spring at each node can be calculated by multiplying the foundation reaction force coefficient of the vertical spring, which is used in the stress calculation in the conventional operation when the vertical load is dominant, with the bearing area of each node. When the moment is dominant in the earthquake, the stress can be calculated from the foundation reaction force coefficient of the rotational spring. The formula for calculating the reaction force coefficient should be selected appropriately in consideration of the stiffness of the building and soil, shape of the foundation, etc., [5.3.2-12, 5.3.2-13].

(d) Treatment of earth pressure in stress analysis

The earth pressures to be considered in the stress analysis of building include the static earth pressure, which always applies, and the dynamic earth pressure, which applies during the earthquake. For the static earth pressure, evaluation can be performed according to the "Construction foundation structural design standards" by the Architectural Institute of Japan. The earth pressure during the earthquake is considered to have an active earth pressure state and a passive earth pressure state. Evaluation of the earth pressure load is usually done by adopting the method proposed by Mononobe and Okabe. In addition, research being carried out on the appropriate evaluation method of dynamic earth pressure. In general, for the earth pressure load applied to a building, an out-of-plane force is applied on the outer peripheral wall of the underground portion. In addition, if there exists a difference in the peripheral earth pressure between the two sides, a horizontal force is applied for the underground portion of the entire building.

(e) Treatment of thermal stress

Combination of the S_1 seismic stress and the thermal stress is done according to the flow chart shown in Figure 5.3.2-2. The stress analysis is done on the base of elastic stiffness by using the temperature load derived in the thermal conduction analysis. In this case, as the thermal stress decreases as the stiffness of the part decreases due to cracks, etc., of the concrete part, the thermal stress is decreased. Table 5.3.2-5 lists the decrease in thermal stress. For details, please see the design method [5.3.2-1], related experiments [5.3.2-2 to 7] and the related standards [5.3.2-8]. In addition, for the stress due to the combination with the thermal stress, the crack cross-sectional method [5.3.2-9 to 11] can be cured to calculate the stress level of the reinforcing bars, etc., for checking.

(f) Accuracy of FEM analysis

Regarding the model discretization and the precision associated with the selected elements, an appropriate selection should be made corresponding to the analytical purpose. For the element division and its precision, the calculation method described in Zienkiewicz, O.C. "Matrix finite element method" can be used as a reference. In addition, at present, some foreign institutions are conducting research on the element division and precision of FEM analysis, such as "NAFEMS (Normal Agency for Finite Elements Methods and Standards)" (UK). This work will also be available for reference.

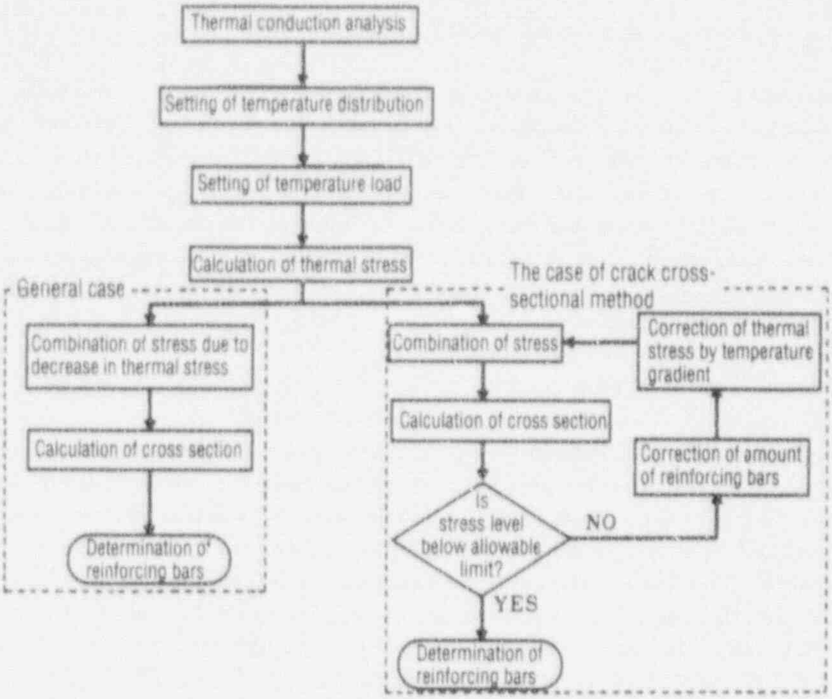


Figure 5.3.2-2. Flow chart of thermal stress design.

Table 5.3.2-5. Load combinations and thermal stress.

Allowable stress state	Load combinations	Thermal stress
Long-term	1 $(D + L) + O + T_1$	Reduced to 1/2
Short-term	2 $(D + L) + O + T_1 + K_1$ 3 $(D + L) + LO + T_2$	Reduced to 1/3
Ultimate	4 $(D + L) + O + K_2$ 5 $(D + L) + LO + K_1$	The thermal stress is not considered

Symbols -- D + L: dead load, live load, etc.; O: operation load; LO: load in L-accident; T_1: temperature load in operation; T_2: temperature load in L-accident; K_1: seismic force due to S_1 earthquake; K_2: seismic force due to S_2 earthquake.

5.3.3 Cross-sectional design

(1) Combination of stresses

The combination of the seismic stress and other stresses is done according to the method of stress combination described in Section 5.1.4 "Load combination and allowable limits." Tables 5.3.2-1 to 5.3.2-4 list the present status of stress analysis and evaluation methods for the containment vessel interior and foundation mat of BWR and PWR reactor buildings, PCCV of PWR, etc. It is required to carry out the cross-sectional design by combining the stresses for an S_1 earthquake and coolant loss accident (referred to as "LOCA" or "accident" hereinafter). Here, the accident stress to be combined refers to temperature load, etc., acting for a long time after the accident (about 1 month after the accident takes place).

The seismic stress includes stress caused by the horizontal earthquake motion and stress caused by the vertical earthquake motion. In this case, the stress, which is caused by the vertical earthquake motion and is to be combined with the stress caused by the horizontal earthquake motion, is the stress calculated assuming the vertical seismic intensity corresponding to 1/2 of the maximum acceleration of basic earthquake ground motion S_1 or S_2 for both the S_1 and S_2 earthquakes.

(2) Cross-sectional calculation method

a. Introduction

Just as in the case of conventional buildings, calculation of the cross sections of reactor building is also carried out by using the various standards for an appropriate evaluation. For example, in the case of allowable stress level design for a reinforced concrete structure against stress in S_1 earthquake, for the shear walls for which the in-plane shear is the dominant factor, and for the floor slab, column, beam, etc., for which the out-of-plane load is the dominant factor, evaluation is primarily carried out according to the "RC Standards." For the ultimate strength design, the data described in the "Yield strength and deformation properties of aseismic design of buildings" by the Architectural Institute of Japan can be taken as a reference.

In the case of reactor buildings, although the primary structure is the reinforced concrete structure, some of them also use partial steel frame structure, and steel frame/reinforced concrete structure. The cross-sectional design for the beam, column, brace, etc., of the steel frame structure is performed according to the "Steel structure design standards" by the Architectural Institute of Japan (1973) (referred to as "S Standards" hereinafter). On the other hand, for the steel frame/reinforced concrete structure, the design is carried out according to the "Steel frame/reinforced concrete structure calculation standards" by the Architectural Institute of Japan (1975) (referred to as "SRC Standards" hereinafter). For the ultimate strength design of the steel frame structure, the "Steel structure plasticity design guidelines" by the Architectural Institute of Japan may be used as a reference.

In principle, the aforementioned various standards are used for designing the cross sections of the various parts. However, for the reactor buildings, as the walls are thicker than those of conventional buildings, and they are partially of complicated shapes, certain special rules should be observed in this case. Some examples are shown below.

b. Features for special consideration in cross-sectional design

(a) Evaluating method of combined stress

The cross-sectional evaluation of combined stress may be performed as follows: for each axis, the amounts of reinforcing bars are derived according to the methods for column, beam and shear wall in "RC Standards" with respect to bending, tension (or compression), and in-plane shear force, followed by adding up the required reinforcing bar amounts. In addition, "RC Standards" may also be used for the out-of-plane shear force of the plate.

For the cross-sectional design method of a reinforced concrete column acted by both axial force and biaxial bending force at the same time, the method of the old ACI standard 318-63 described in commentary Section 15 of "RC Standards" can be used. In addition, several researchers have conducted research on the composite stress. In the following, we will present several references related to the design methods.

Umemura, Aoyama, et al. [5.3.3-1] have proposed a method for calculating the in-plane ultimate strength of RC shear walls acted on by out-of-plane bending force based on a series of experimental results. In this scheme, according to the out-of-plane bending stress, the wall is divided to compressed zone and tension zones. For each zone, the minimum of Q1 and Q2 are evaluated to represent the shear strength; Q1 is determined from the shear force due to in-plane bending moment; and Q2 is the calculated shear strength. Then, they are added and the sum is taken as the in-plane ultimate strength of the overall wall.

Kobata, Takeda, et al. [5.3.3-2] have proposed a method for calculating the in-plane and out-of-plane ultimate strength of a wall acted on by in-plane and out-of-plane bending and shear on the base of experimental results. In this scheme, for each of the two axes (in-plane and out-of-plane), the correlation curve between bending and shear strength is calculated, and their envelope strength lines are taken as the ultimate strength correlation curve of the overall wall.

Aoyama and Yoshimura [5.3.3-3] have carried out experimental research on the ultimate strength of a wall acted on by in-plane shear force and out-of-plane force. In this case, as the correlation between the in-plane force and out-of-plane force are normalized and represented by arc, [the theory] becomes in good agreement with the experimental results. As this is expressed by a diagram and table, Equation (5.3.3-1) and Figure 5.3.3-1 can be used for representing this correlation. This equation can be effectively used to evaluate shear walls and slabs.

$$\left(\frac{Q_u}{{}_cQ_u}\right)^2+\left(\frac{M_u}{{}_cM_u}\right)^2 = 1$$

where Q_u: in-plane shear strength

${}_cQ_u$: in-plane shear strength when there is no out-of-plane bending, as determined from the various empirical formulas

M_u: out-of-plane flexural strength

${}_cM_u$: out-of-plane flexural strength when there is no in-plane shear, as determined by using the e-function method and approximate calculation method.

(b) Cross-sectional evaluation method of thick concrete structures such as foundation mat, etc.

The foundation mat of a reactor building is a plate with large plane dimensions and thickness. Usually, evaluation is made by using the cross-sectional calculation method of column and beam of "RC Standards" for the stress derived by using FEM analysis (solid elements, plate elements, etc.). When stress analysis is done using FEM analysis (solid elements) for cross section designed by "RC Standards," the stress shown in Figure 5.3.3-2 is evaluated with respect to the flexural moment, axial force, and shear force according to following Equations (5.3.3-2) to (5.3.3-4), so that the amount of reinforcing bars is calculated for both the upper end and the lower end.

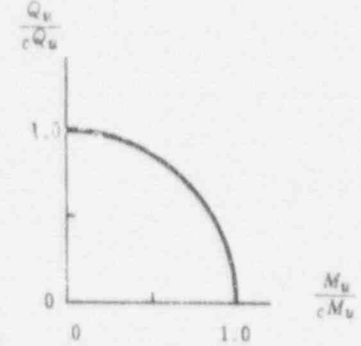


Figure 5.3.3-1. Correlation between in-plane shear and out-of-plane bending for plate.

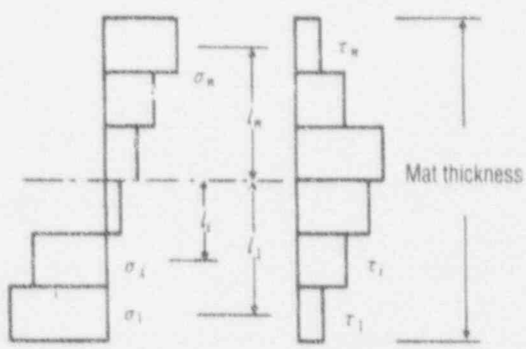


Figure 5.3.3-2. Calculation method of M, N, Q from FEM analytical stress.

$$M = \sum_i \sigma_i A_i l_i \qquad (5.3.3\text{-}2)$$

$$N = \sum_i \sigma_i A_i \qquad (5.3.3\text{-}3)$$

$$Q = \sum_i \tau_i A_i \qquad (5.3.3\text{-}4)$$

where M: flexural moment of the part
N: axial force of the part
Q: shear force of the part
σ_i: tensile or compressive stress level of element (i)
τ_i: shear stress level of element (i)
A_i: area of element (i)
l_i: distance to the center of element (i)

In addition, the design rules for thick members in ACI Standard 349-80 [5.3.3-4] can be used as a reference for designing thick concrete structures. Thermal cracks may be developed due to the heat of hydration. In this case, one measure in the design is to increase the amount of reinforcing bars used.

(c) Design method of anchor bolts

The design of building's anchor bolts is described in the "Guidelines and commentary of designs of various composite structures" by the Architectural Institute of Japan [5.3.3-5]. In addition, the design method of the support structures for equipment and piping is described in "JEAG 4601, Supplement-1984." In addition, at present, research is being carried out on anchor bolts. The results of this work will be available for future designs.

When the shear force and tensile force are too large to be handled by anchor bolts, it is possible to arrange a concrete surrounding the foot of the steel frame, or to embed the steel frame [foot] in the lower concrete. In this case, the yield strength evaluation can be performed according to the amended version of the "SRC Standards" [5.3.3-6]. In addition, Wakabayashi, et al. [5.3.3-7] may also be used as a reference. The design method of the anchor bolts of equipment and piping is described in Chapter 6 "Aseismic design of equipment and piping."

(d) Evaluation method of flat slab structure

For a reactor building, the walls and slabs usually are relatively thick for the shielding function and aseismic function. In many cases, the floor is made of flat slabs, forming a wall-slab structure or a column-slab structure. The stress distribution in the slab usually can be evaluated according to the description of flat slabs in Section 1 of "RC Standards." In addition, in the case of the wall-slab structure, it is also appropriate to derive the stresses in the wall and slab by FEM analysis, and to evaluate the slab according to the cross-sectional calculation method of column or beam defined in "RC Standards." In addition, for the flat slabs, Kano and Yoshizaki [5.3.3-8] have studied the fracture of the column-slab joint portion; Kikuchi [5.3.3-9] has studied the moment transfer between column and slab. Their results are described in "RC Standards."

(e) Evaluation of a shear wall with openings

In principle, the yield strength evaluation of the shear walls with openings, and the determination of opening reinforcement are carried out according to the portion concerning shear walls in Section 18 of "RC Standards." However, detailed standards are not yet available for the design method of shear walls having various complicated openings. The following are research data for reference.

Aoyama, et al. [5.3.3-10 and 5.3.3-11] have compared the features of various reinforcement methods for a single-opening wall. Yoshizaki, et al. [5.3.3-12] have proposed a scheme for reinforcing a shear wall having a number of small openings in it. The Construction Research Data (No. 6) by the Building Research Institute of the Ministry of Construction [5.3.3-13] proposed a calculation method of reinforcing bars using the edge stress calculation formula of a number of openings. In the following, we will present the methods proposed by Aoyama, et al. and by Yoshizaki, et al.

(i) Evaluation method of single-opening wall [5.3.3-10 and 5.3.3-11]

The minimum value of the ultimate strengths calculated by using the following formulas assuming various failure lines (such as those shown in Figure 5.3.3-3) is taken as the ultimate strength of the wall. This method is applicable for both circular and square openings.

$$Q_u = Q_s + Q_c \tag{5.3.3-5}$$

$$Q_s = Q_{sh} + Q_{sv} + Q_{sD} \tag{5.3.3-6}$$

$$Q_{sh} = a_{sh} \cdot \sigma_y \tag{5.3.3-7}$$

$$Q_{sv} = 0.425 (d/l_0) a_{sv} \cdot \sigma_y \tag{5.3.3-8}$$

$$Q_{sD} = a_{sD} \cdot \sigma_y \cdot \cos\theta \tag{5.3.3-9}$$

$$Q_c = \tau_c \cdot t \cdot l \tag{5.3.3-10}$$

$$\tau_c = (1.9 - 0.7h/l) \sqrt{F_c} \tag{5.3.3-11}$$

where Q_u: ultimate shear strength of the wall
Q_c: shear force carried by the concrete
Q_s: shear force carried by reinforcing bars
t: wall thickness
Q_{sh}: shear force carried by transverse reinforcing bars
l_0: bending span of the reinforcing bars
Q_{sv}: shear force carried by the longitudinal reinforcing bars due to the dowel action
h/l: shear span ratio
d: reinforcing bar diameter
Q_{SD}: shear force carried by diagonal reinforcing bars (reinforcement of opening)
F_c: design standard strength of concrete
σ_y: yield strength of reinforcing bars
a_{sh}: total cross-sectional area of transverse reinforcing bars
a_{sv}: total cross-sectional area of longitudinal reinforcing bars
a_{SD}: total cross-sectional area of diagonal reinforcing bars
θ: angle of diagonal reinforcing bars.

(ii) Evaluation method of multiple openings [5.3.3-12]

With various failure lines assumed (such as those shown in Figure 5.3.3-4), the sum of the ultimate strength is derived using the following formulas for each case. Then the minimum value among these sums is taken as the ultimate strength of the wall. This method is applicable for both the circular and square-shaped openings.

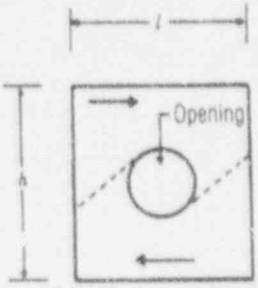


Figure 5.3.3-3. Example of failure lines assumed (in the case of a single opening).

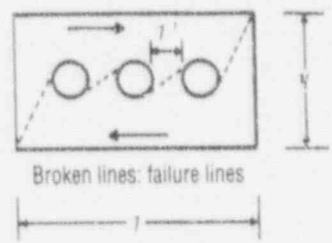


Figure 5.3.3-4. Examples of failure lines assumed (in the case of multiple openings).

$$Q_u = \Sigma Q_w + Q_x \tag{5.3.3-12}$$

$$\Sigma Q_w = \Sigma_i \tau_w \cdot {}_iA \tag{5.3.3-13}$$

$$Q_x = \sqrt{2}\, a_x \cdot \sigma_y \tag{5.3.3-14}$$

where Q_u: ultimate shear strength of wall
${}_i\tau_w$: ultimate shear stress level of portion (i), calculated using various strength formulas
${}_iA$: cross-sectional area of portion (i) (il·t)
${}_il$: wall length of portion (i)
t: wall thickness
Q_x: shear force borne by the diagonal reinforcing bars of the opening portion
a_x: total cross-sectional area of the diagonal reinforcing bars
σ_y: yield strength of the reinforcing bars

(f) Composite structures

For the composite structures, "Guidelines and commentary of designs of various composite structures" published by the Architectural Institute of Japan [5.3.3-5], describes the design guidelines for the composite beams, composite slabs of deck plates and concrete, composite structures of steel-frame and reinforced-concrete structural walls, as well as various anchor bolts for usual configuration. Among the composite structures made of steel plates and concrete, for the concrete-covered steel pipe and concrete-filled steel pipe, the cross-sectional calculation method and ultimate strength evaluation method are described in the amended version of "SRC Standards" [5.3.3-6]. In addition, Yamada et al. [5.3.3-14] and Wakabayashi et al. [5.3.3-15] have performed researches on the ultimate strength capacity of the steel pipe components; Kato, et al. [5.3.3-16] have performed researches on the shear strength of the concrete-filled steel plate walls.

(g) Method of splicing large-sized reinforcing bars (D29-D51)

For the reactor buildings, large-sized deformed reinforcing bars are often used, with mechanical joints often used as the splices for the reinforcing bars. At present, lap splices are used. Although they are beneficial from an economic point of view and have the ability to shorten the construction time, their applications are nevertheless limited by the standards. For the reactor building, due to its special structural form, it is believed that the lap splice may be well used for the large-sized reinforcing bars. From this point of view, experimental research is being performed on the structural characteristics of the joints of large-sized reinforcing bars.

In this research work, on the basis of the multiple tests on the structural characteristics in the shear stress and tensile stress, the conclusion is that in the case of up to D38 reinforcing bars are used, 40 d (d; diameter of the rebar) is enough length for lap length when covering thickness is 1.5 d. Therefore, 40 d is applicable even if several splices are located in the same cross-section. For the lap splices of D41 and D51, a further investigation is yet to be made. At present, in principle, mechanical joints are used for those.

5.3.4 Investigation of the maintenance of the functions

(1) Required functions and components

For the stresses in S_1 and S_2 earthquakes, as well as stresses under the required load combinations, it is required to investigate the function-maintenance ability of the various parts. The topics for the investigation include leak-proof functions, function to prevent secondary accidents, the supporting function, etc.

The leak-proof functions include airtightness and watertightness. Airtightness can prevent the release of radioactive substances to the outside of the building in an S_1 earthquake, and are required for the reactor building (MARK-I, II), outer shield building (2, 3 LOOP), and reactor peripheral auxiliary equipment building (4 LOOP). The watertightness refers to the good performance of the liner on the portion used for the storage of radioactive liquids. It is required for the spent-fuel pool (MARK-I, II), spent-fuel pit (2, 3, 4 LOOP), bottom foundation mat of the containment vessel (MARK-II), etc.

The function of preventing secondary accidents refers to the ability to prevent hazards in the safety function of the higher grade equipment caused by damage, falling, or an overturning of the lower, aseismic-grade structure. It is widely required for the various portions of the building against the earthquakes S_1 and S_2.

The supporting function indicates that the motion from the original supporting position due to overturning or shifting of the equipment is within the allowable limit, or that the relative movement of equipment and piping does not cause any damage. This should be confirmed with respect to both earthquake S_1 and earthquake S_2. Almost all part of the reactor building should be considered for the evaluation of the supporting function as pipes end equipment are located throughout the building.

For components and their functions, an investigation has been conducted in the Third Amendment Standardization Survey (referred to as "Third Amended Standard" hereinafter) (Standard, Construction-7 [HYO, KEN-7] and Standard, Construction-8 [HYO, KEN-8]). The results are shown in Table 5.3.4-1 and Figures 5.3.4-1 to 2.

Table 5.3.4-1. Allowable limits in the function maintenance of the various parts of reactor buildings as required by the equipment system.

	Part		Earthquake	Criteria of equipment system	Characteristics for maintaining the functions	Margins, etc.				
Leak-proof function	Reactor primary building Outer shield Reactor peripheral auxiliary equipment building	(MARK-I,II) (3 LOOP) (4 LOOP)	S_1	Leakage of radioactive substances to the outside of the building in S_1 earthquake can be prevented	A negative pressure is maintained					
	Spent fuel pool Spent fuel pit	(MARK-I,II) (3,4 LOOP)	S_1 S_2	The liner part in the storage portion of the liquid is flawless	No cracks are developed in the liner portion	Allowable membrane strain of concrete containment-vessel liner is used				
	Containment-vessel bottom foundation mat Foundation mat	(MARK-II) (4 LOOP)	S_1 S_2	Prevent excessive deformation in the foundation mat that the liner of the concrete containment vessel cannot follow		MARK-II Membrane strain Compression Tensile	 S_1 0.004 0.002	 S_2 0.005 0.003	4 LOOP Membrane strain Compression Tensile	 S_1 S_2 0.005 0.003
Function in preventing secondary accidents	Reactor primary body foundation Shell wall Reactor primary building Reactor auxiliary building Diaphragm floor Internal concrete Fuel-treatment building Reactor peripheral auxiliary equipment building Outer shield	(MARK-I,II) (MARK-I,II) (MARK-I,II) (MARK-I,II) (MARK-II) (3,4 LOOP) (3,4 LOOP) (3,4 LOOP) (3 LOOP)	S_1 S_2	Prevent the accidents that the Class-As or Class-A equipment safety functions are lost due to damage, fall or overturns of the lower aseismic grade structures	Structure or its portion is not damaged	Allowable limit of structural body				

Table 5.3.4-1 *(Cont'd)*. Allowable limits in the function maintenance of the various parts of reactor buildings as required by the equipment system.

	Part		Earth-quake	Criteria of equipment system	Characteristics for maintaining the functions	Margins, etc.
Supporting function	Containment-vessel bottom foundation mat	(MARK-I,II)	S_1 S_2	Equipment and piping are restrained in the allowable space from the original supporting positions - No shift, overturn, or falling of the equipment - No relative move in the supporting point - No disconnection in the supporting point	Parts I - No excessively large deformation - Anchor portion, etc., are flawless Parts II - They can support parts I - They do not cause excessively large deformation in parts I Parts III - No collapse	Parts I - (within the structural allowable deformation limit, within the allowable movement of equipment) - No yield mechanism is formed in the out-of-plane direction Parts II - (within the structural allowable deformation limit, within the allowable motion amount of Parts I) - No yield mechanism is formed in the vertical direction Parts III - Allowable limit of component
	Containment-vessel bottom external foundation mat	(MARK-I,II)				
	Reactor primary body foundation	(MARK-I,II)				
	Shell wall	(MARK-I,II)				
	Spent fuel pool	(MARK-I,II)				
	Diaphragm floor	(MARK-II)				
	Reactor primary building	(MARK-I,II)				
	Reactor auxiliary building	(MARK-I,II)				
	Foundation mat	(3,4 LOOP)				
	Internal concrete	(3,4 LOOP)				
	Spent fuel pool	(3,4 LOOP)				
	Fuel-treatment building	(3,4 LOOP)				
	Reactor peripheral auxiliary equipment building	(3,4 LOOP)				
	Outer shield	(3 LOOP)				

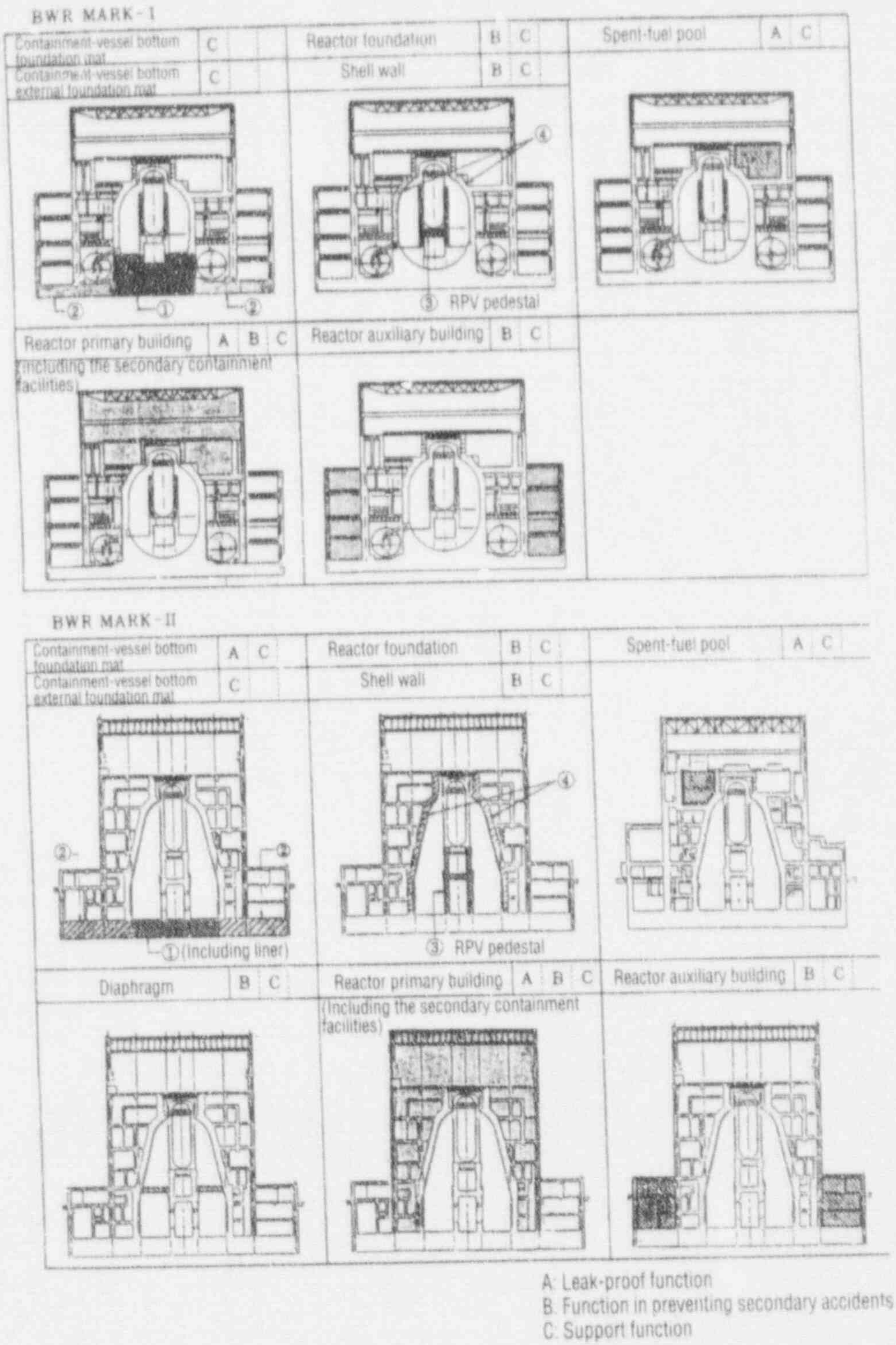


Figure 5.3.4-1. Various parts of reactor building (BWR) with function-maintenance ability required.

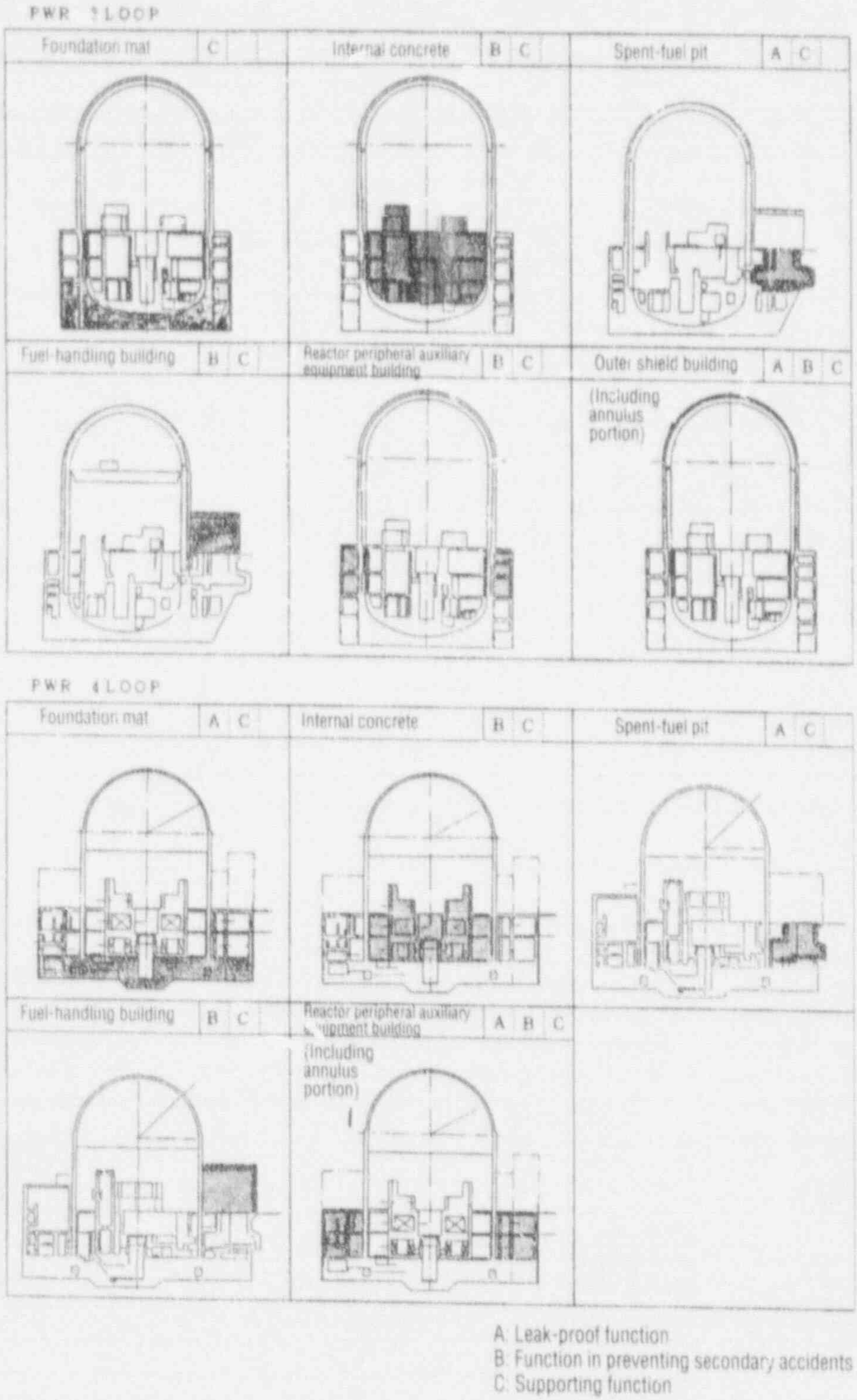


Figure 5.3.4-2. Various parts of reactor building (PWR) with function-maintenance ability required.

(2) Consideration of the allowable limits

At present, there is yet no standard/rule on the limits of the various parts of the building to maintain the functions as described in above section (1). Hence, at present, the criteria for maintaining the functions are taken as follows: for S_1, the allowable stress design is performed; for S_2 and S_1 + LOCA, the ultimate strength design is performed. The design for the ultimate strength level is performed with a sufficiently large safety margin, e.g., the design for out-of-plane shear reinforcement is designed based on allowable stress concept. Tables 5.3.2-1 to 4 list the presently used design criteria. In this section, we will schematically present the treatment method of the allowable limits on the basis of the results of an investigation in the Third Amended Standard (Standard, Construction-7 [HYO, KEN-7] and Standard, Construction-8 [HYO, KEN-8]) (see Table 5.3.4-1). In addition, EPJR is also performing research in the allowable limits, which will be explained in Section 5.3.5 "Safety margin."

a. Leak-proof function (airtightness)

In order to prevent the leakage of radioactive gas from the building to the outside, the interior of the building is designed to maintain a certain degree of negative pressure with respect to the exterior. For this purpose, the design of building is currently performed based on allowable stress design. However, once the airtightness of walls with cracks is quantitively established, the structural limit can be defined in terms of shear angle as related to the degree of cracking in shear walls.

b. Leak-proof function (watertightness)

In the present design, the allowable value for the strain of the liner [5.3.4-1] is applied at the bottom-foundation mat portion of the confinement vessel. It is also being used for the fuel pool and pit. However, the strain may be limited too low considering the properties of steel in preventing the leakage of the liquid. As long as there are no cracks in the liner (including the welded portion), the localized buckling, etc., may be allowed.

c. Function in preventing secondary accidents

Examples of secondary accidents include the falling of part of the concrete structure of the ceiling or wall, or the falling of a crane, causing damage to the equipment that are important for safety. In the latter case, it will be allowable if the supporting function for the crane can be satisfied. In the former case, it is believed that it will be acceptable if the structure or its portion does not collapse. That is, the margin can be determined the same as that for the [entire] structure. This value may be defined within a certain safety margin with respect to the maximum yield point of the structure. According to the Third Amended Standard, the safety factor is recommended to be 3 based on energy concept, i.e., the allowable level is defined such that the area defined by the skeleton curve becomes 1/3 of the area corresponding to the maximum strength.

d. Supporting function

Depending on the scheme for supporting the equipment and piping, the supporting function is considered for 3 ranks of parts (with a different meaning for the titles of the various parts). Parts I include the floor, walls, ceiling, etc., which directly support the equipment and piping. Parts II include the walls, columns, beams, etc., that support parts I. Parts III include the walls, etc., that maintain the stability of parts II.

According to the definition of the supporting function, for parts I, the requirement is that there should be no excessively large deflection or deformation of the overall part. As far as the local portions are concerned, the anchor bolts, studs, and other anchor portions and steel-frame mounting portions should be flawless. Parts II must be able to support parts I, and they should not cause excessively large deformation in parts I. For parts III the requirement is that they are acceptable so long as they do not collapse.

It is rather difficult to make a quantitative formulation of above qualitative guidelines. Some examples are shown in Table 5.3.4-1. In addition, as the equipment reaction force, etc., act on the walls and floor in out-of-plane directions, it is required to evaluate the behaviors of these parts acted upon by both the in-plane force and out-of-plane force. However, the experimental data for the yield strength, etc., are not yet sufficient. At present, the design is usually carried out as the stress level is held within the allowable stress. On the other hand, for the local portions, it is required to confirm the strength of the anchor portion. However, for the strength of anchors embedded in a concrete, which is under plastic condition, further study is to be made in the future.

(3) Ultimate strength design

In the investigation of the function maintenance, a determination is made with reference to the criteria of the function of the equipment system. Hence, if it can be confirmed that the criteria for maintaining the function are satisfied, from the viewpoint of preventing secondary accidents, structural design may be performed with a certain degree of margin with respect to the ultimate strength or deformation. Table 5.3.5-1 lists the major ultimate strength formulas considered at present to investigate the ultimate-strength design of the building system. For example, as pointed out above, for the parts which are reinforced for out-of-plane shear and are acted upon by in-plane and out-of-plane forces, a sufficient margin can be obtained by holding the stress level well within the allowable limits.

5.3.5 Safety margin

(1) Evaluation against static seismic force

a. Safety margin with respect to required horizontal strength

The "Evaluation Guidelines" [5.3.5-1] pointed out that an appropriate safety margin is needed for the horizontal strength of the structure with respect to the required strength according to the degree of importance. If this is directly expressed it says that the value of the ratio of the "horizontal strength/required strength" is the index of the static safety margin.

The horizontal strength can be evaluated on the basis of the shear strength and flexural strength of the component. The typical formulas for the strength are described in the references listed in Table 5.3.5-1. In reactor building, the shear wall is almost never determined by the flexural yield capacity; instead, the horizontal strength is usually evaluated by the sum of the shear strength of the shear wall. The strength formulas should be compared with the restoring force characteristics used in the nonlinear response analysis (see Section 5.2.5(2) "Restoring-force characteristics of structure"). According to the Construction Standards, the required horizontal strength can be evaluated using the following equation:

$$Q_{un} = D_S \cdot F_{eS} \cdot Q_{ud} \qquad (5.3.5\text{-}1$$

where Q_{un}: required horizontal strength of each story (tf)
D_S: structural characteristic coefficient of each story
F_{eS}: shape characteristic coefficient of each story
Q_{ud}: horizontal force in each story due to seismic force (tf)

b. Regarding the value of D_S

The value of D_S needed to evaluate the required horizontal strength is determined by appropriately evaluating the vibration damping characteristics and the ductility. For the reinforced concrete structure portion, the present design was 0.5 or a larger value. This value is believed to be rather conservative in consideration of the fact that, compared with a conventional building, the box cross section of the reactor building has a greater

Table 5.3.5-1. Typical component strength evaluating formulas.[1]

Component	Shear strength	Flexural yield strength
Reinforced concrete shear wall	[5.3.5-2] [5.3.5-3] Third Amended Standard[2] (Standard, Construction-6 [HYO, KEN-6])	[5.3.5-2] [5.3.5-4] [5.3.5-5] [5.3.5-7]
Reinforced concrete beam, column	[5.3.5-4] [5.3.5-6]	[5.3.5-7] [5.3.5-8]
Steel bracse	[5.3.5-9]	—
Steel beam, column	—	[5.3.5-9]
SRC beam, column	[5.3.5-10]	[5.3.5-10]

[1]The numbers in the table refer to the numbers of references which describe the evaluating formulas.
[2]$5.0\sqrt{F_C}$.

restraining effect and the fact that the amount of reinforcing bars is large. The ductility can be evaluated based on the experimental data for the reactor building shear walls accumulated recently. The results of the investigation indicate a D_S value of 0.4-0.45 even when the scale effect due to the difference between the test specimens and the actual building is taken into consideration (Standards, Construction-3 [HYO, KEN-3]). Hence, in the future evaluation of the D_S value, these data will be used.

c. Lower limit of the safety margin

As far as the quantitative standard of the safety margin is concerned, an investigation is now underway. For the conventional buildings, the value is taken to be 1.0. Therefore, the safety margin can be estimated quantitatively by comparing with conventional buildings. In some cases (Standard, Construction-3 [HYO, KEN-3]), in the first-phase design, the safety margin is approximately evaluated by estimating the strength of a building designed with an allowable stress level against 3 times the specified seismic force. In another scheme (Standard, Construction-3 [HYO, KEN-3]), in the second-phase design, the 3-times margin in the first-phase design is considered as the margin in terms of energy. When this is converted to the yield strength, the margin becomes about $\sqrt{3}$.

In addition, recently, based on the experimental data of the reactor building's shear walls, an investigation was performed on the allowable limit for the aseismic design (Research, Construction-2 [KEN, KEN-2]). It proposed the following reference value for the structure made of reinforced concrete.

$$Q_A = Q_u/1.5 \tag{5.3.5-2}$$

where Q_a: reference value of the allowable limit for the shear
Q_u: ultimate shear strength in each story ($= \tau_u \cdot A_s$)
τ_u: according to equation (5.2.5-11)
A_S: effective shear cross-sectional area

This reference value is based on engineering judgment by performing a quantitative evaluation of the scatter in the ultimate strength capacity of the shear walls, and also by accounting for the supporting function.

(2) Evaluation of dynamic seismic f...

a. Safety margin with respect to S_2 earthquake

As pointed out in "Evaluation Guidelines" [5.3.5-1], under the action of basic earthquake ground motion S_2, the overall structure should have a safety margin in the deformation ability; and also it should have an appropriate safety margin with respect to the ultimate strength capacity. In this way, the reactor facility is made with a higher structural safety than that of the conventional structure. If the criteria for the function maintenance of the various parts are stricter than those for the ultimate state, the margin is believed to be guaranteed by actually confirming the function maintenance. There is still no detailed standard regarding the criteria for maintaining the functions of the various parts as explained in Section 5.3.4 "Investigation of the maintenance of the functions." Hence, in the practical design work, a sufficient margin is usually required. In addition, the evaluation of the safety margin is still being investigated by various organizations.

Recently, the following reference value has been proposed for the deformation corresponding to the margin limit of the reinforced concrete shear walls (Research, Construction-2 [KEN, KEN-2]).

$$\gamma_a = \gamma_u/2.0 \tag{5.3.5-3}$$

γ_a: reference value of the allowable limit concerning the shear strain level
γ_u: ultimate shear strain level of each story, 4.0×10^{-3}, see equation (5.2.5-12)

This value is determined by a quantitative evaluation of the scatter in the ultimate deformation of the shear wall by experiments, with an additional margin to account for the uncertainties in the design. It corresponds to the reference value of the allowable limit of the strength described in "(1)c. Lower limit of safety margin" in terms of the combined safety margin (i.e., $1.5 \times 2.0 = 3.0$).

5.4 Concrete containment vessel

5.4.1 General features

(1) Introduction

The containment vessel contains the reactor and important equipment and piping. It should be able to prevent the dissipation of radioacti e substances if the radioactive substances leak from the equipment and piping. When the containment vessel is made of concrete, the structure is primarily made of reinforced concrete or presiress concrete. The containment vessel differs from a pressure vessel in that it is not always acted upon by an internal pressure, and its function is to form a barrier for the leakage of radioactive substances in case of an accident. However, since the internal pressure is also increased in an accident, the containment vessel is also designed as a pressure-resistant containment. For a conventional concrete containment vessel, a thin steel plate is arranged on the inner side as a liner which can prevent leakage. The reinforced concrete or prestressed concrete structure is designed to have a sufficiently high yield strength against the internal pressure. For PWR, when a concrete containment vessel is used, the design internal pressure is relatively high, and the containment vessel's internal volume is also large; hence, PCCV wing prestressed concrete is used to make the semispherical dome and the cylindrical shell body for the vessel in PWR 4 LOOP plant. Also, for BWR, research and development are being carried out for the practical application of a concrete containment vessel. In this case, the containment vessel has a simple shape made of a cylindrical shell and a top slab. Since the internal pressure is lower than that of PWR, and the volume is smaller, the containment vessel may be made of reinforced concrete.

(2) Outline of technical standards

a. "Technical standards of concrete containment vessels for nuclear power plants (draft)" [5.4.1-1]

As the concrete containment vessels for nuclear power plants were introduced in Japan, in order to form suitable standards for them, in August, 1975, the "Seminar on Technical Standards of Concrete Containments for Nuclear Power Plants" was convened. After many technical seminars and verification tests, "Technical Standards of Concrete Containment Vessels for Nuclear Power Plants (drafts)" was drafted in November, 1979. Afterwards, in June, 1981, the "Technical Seminar on Concrete Containment Vessels" was convened by the Nuclear Power Plant Technical Advisory Committee for reinvestigating the content of said technical standards (draft), which was then amended on the basis of the results obtained in the later tests. In November, 1981, the amended version of the standard draft was formulated. The standard (draft) consists of the following 6 chapters: Chapter 1, Introduction; Chapter 2, Design; Chapter 3, Materials; Chapter 4, Implementation; Chapter 5, Liner; and Chapter 6, Test/Inspection.

In Chapter 1, the standard (draft) treats the concrete containment vessel as two types: reinforced concrete structure and prestressed concrete structure; it defines the rules suitable for the concrete portion and the liner portion of the steel plate. Chapter 2 defines the design of the concrete portion which bears the structural strength of the concrete containment vessel. It contains the basic contents and detailed design methods of the types and combinations of design loads, design margin for each material, calculation of the stress, and the structural analytical method. Chapter 3 and 4 define the materials used in the concrete portion and it's construction. Chapter 5 defines the design, manufacture, construction, and inspection of the liner. Chapter 6 defines the tests and inspection methods for confirming the sound performance (pressure resistance, leak-proof property) of the completed concrete containment vessel.

b. "Guidelines and commentary of the design of concrete containment vessels for nuclear power plants" [5.4.1-2]

These guidelines were drafted in August, 1978 by the Subcommittee on Nuclear Power Plant Concrete Structures of the Structure Standards Committee of the Architectural Institute of Japan. The design of a nuclear power plant concerns the techniques in various respects. In determining the design conditions for the concrete structures, attention should be paid to the following facts: each plant has its own design specification and the containment vessel made of a concrete structure requires a license from the Agency on Science and Technology and the Ministry of International Trade and Industry.

This draft consists of the following chapters: Chapter 1, Introduction; Chapter 2, Materials; Chapter 3, Loads and their combination; Chapter 4, Design and analytical method; Chapter 5, Design margin; Chapter 6, Detailed design scheme; Chapter 7, Design of liner; Chapter 8, Design of opening portion and through-holes; Chapter 9, Inspection and tests.

(3) Types of loads for design

For the design of a concrete containment vessel, the following loads should be taken into consideration.

a. Normal loads

Loads that always act irrespective of the state of the power plant:

{1} Dead load (D)
{2} Live load (L)
{3} Prestress load (F)

Loads in operation

Loads that act in the normal operation or during an abnormal transition period in the operation:

{1} Operation pressure load (P_1)
{2} Operation piping load (R_1)
{3} Operation thermal load (T_1)

c. Accident loads

The accident loads include the loads in L-accidents and the loads in J-accidents. The load in LOCA refers to the internal pressure, thermal load and piping loads caused by thermal expansion of the piping. The J-accident load refers to the force acting by the high-temperature, high-pressure jet flow and the reaction force caused by an e... ..ng jet in the case of a J-accident, which converge over a short period of time.

{1} Pressure load in L-accident (P_2)
{2} Piping load in L-accident (R_2)
{3} Thermal load in L-accident (T_2)
{4} Load in J-accident (R_3)

d. Loads related to natural phenomena

{1} Seismic load

The following loads are based on the "Evaluation Guidelines."

S_1 seismic load (K_1): The seismic force caused by basic earthquake ground motion S_1 or the static seismic force, whichever is larger.

S_2 seismic load (K_2): The seismic force due to basic earthquake ground motion S_2.

The importance class of the containment vessel is class A_S, for which the safety function should be maintained against an S_2 earthquake.

{2} Weather loads

Wind load (W): According to Implementation Clause 87 of the Construction Standards.
Snow load (S): According to Implementation Clause 86 of the Construction Standards.

e. Load during test

This is the load with the test state taken into consideration. The internal pressure P_0 in the test (the pressure load with 1.15 times the design internal pressure) is taken into consideration.

f. Other loads

Depending on the site of the plant, the fall of a certain flying object, etc., should be taken into consideration. These loads depend on the specific site of the plant.

(4) Load combinations for design and load state

When the containment vessel is designed, the various loads with different properties as described in said item on the types of loads for design must be taken into consideration. These loads are different from each other with respect to the probability of occurrence, simultaneous nature of the loads, etc. Hence, it is required to consider the appropriate combination of the loads for the design for each load state with consideration given to the status of the various loads. The load states of the combinations of the various loads for design are as follows:

Load state I: The state in the conventional operation.
Load state II: The state with the weather loads taken into consideration in the conventional operation, the state of the abnormal transient period of the operation, and the state in the test.
Load State III: The state in an S_1 earthquake or L-accident.
Load State IV: The state assumed for a safety evaluation of the containment vessel.

Table 5.4.1-1 lists the load combinations and load coefficients.

(5) Design margins

a. Concrete

The design standard strength of concrete is over 210 kgf/cm^2 in the case of reinforced concrete, and over 300 kgf/cm^2 in the case of prestressed concrete. Among the allowable stresses for concrete, stress state 2 indicates the stress state when the load combination including thermal load is considered.

(a) Allowable stress for the compression of concrete
See Table 5.4.1-2.

(b) Out-of-plane shear stress bearable by concrete in the bottom portion
When it is required to investigate the punching shear due to the jet force, jet reaction force, etc., the section of punching shear of the formation defined in item 19 of "RC Standards" can be used. In this case, the allowable shear stress of the concrete has the values listed in Table 5.4.1-3, with a load state IV identical to load state III.

(c) Allowable bearing stress of prestressed concrete anchor portion
Lower than $0.45\ F_c \times (A_c/A_1)^{1/2}$ and $0.9\ F_c$, where A_c is the anchor area and A_1 is the anchor plate area.

(d) Temperature limit of concrete
When the temperature limit listed in Table 5.4.1-4 is satisfied for the concrete temperature, the allowable for the concrete is determined using the values given in this section. In addition, it can be assumed that there is no variation in the properties of the concrete. On the other hand, even when the temperature is over the limit, as long as it can be confirmed by experiment that the temperature does not seriously affect, the value defined in this section can be considered as reference. All condition except normal operation and long-term condition in a steady state, refers to "short-term". In this case, transient state refers to "short-term".

b. Reinforcing bars

(a) Allowable stress for reinforcing bars
See Table 5.4.1-5.

(b) Allowable stress for out-of-plane shear reinforcing bars of the bottom portion
See Table 5.4.1-6.

(c) Allowable bond stress of reinforcing bars with respect to concrete
See Table 5.4.1-7.

c. PC bars

The allowable stress for PC bars is the smaller one in the values listed in Table 5.4.1-8. When the simple tensile material is made of a collection of PC steel bars, the allowable stress value of PC bars can be used. What are listed in Table 5.4.1-8 are values when a prestress is introduced. For the PC bar after completion of the anchoring, $0.7\ F_u$ or $0.8\ F_y$ whichever is smaller, is taken as the allowable stress.

Table 5.4.1-1. Load combinations and load factors.

Load state	No.	Load time	Load factors							
			D	L	F	P_1	R_1	T_1	S	W
		Name	Dead load	Live load	Prestressed load	Pressure in operation	Operation piping load	Operation thermal load	Snow load	Wind load
I	1	In normal operation	1.0			1.0	1.0	1.0		
II	2	In test	1.0							
	3	In a storm	1.0			1.0	1.0	1.0		1.0
	4	In snow	1.0			1.0	1.0	1.0	1.0	
III	5	In S_1 earthquake	1.0			1.0	1.0	1.0		
	6	In L-accident	1.0							
	7	In (L-accident + S_1 earthquake)	1.0							
IV	8	In S_2 earthquake	1.0			1.0	1.0			
	9	In L-accident	1.0							
	10	In J-accident	1.0							
	11	In (L-accident + S_1 earthquake)	1.0							
	12	In (L-accident + hurricane)	1.0							1.25
	13	In (L-accident + hurricane)	1.0						1.25	

For the load in parentheses, there is no need to combine the pressure determined right after the coolant-loss accident and the maximum piping load with the seismic load, etc.

A combination of the loads should be appropriately performed by investigating the state of occurrence of the load and the timing of the generation of stress.

When it is clearly determined that the evaluation will not become more strict when a combination is considered with the other loads, the evaluation of such a load combinations may be omitted.

Table 5.4.1-1 *(Cont'd).* Load combinations and load factors.

Load state	No.	Load time / Name	Load factors P_0 Internal pressure in test	P_2 Pressure in L-accident	R_2 Piping load in L-accident	T_2 Thermal load in L-accident	R_3 Jet force and jet reaction force	K_1 S_1 seismic load	K_2 S_2 seismic load
I	1	In normal operation							
II	2	In test	1.0						
	3	In a storm							
	4	In snow							
III	5	In S_1 earthquake						1.0	
	6	In L-accident		1.0	1.0	1.0			
	7	In (L-accident + S_1 earthquake)		(1.0)	(1.0)	1.0		1.0	
IV	8	In S_2 earthquake							1.0
	9	In L-accident		1.5	1.0				
	10	In J-accident					1.0		
	11	In (L-accident + S_1 earthquake)		1.0	1.0			1.0	
	12	In (L-accident + hurricane)		1.25	1.0				
	13	In (L-accident + hurricane)		1.25	1.0				

For the load in parentheses, there is no need to combine the pressure determined right after the coolant-loss accident and the maximum piping load with the seismic load, etc.

A combination of the loads should be appropriately performed by investigating the state of occurrence of the load and the timing of the generation of stress.

When it is clearly determined that the evaluation will not become more strict when a combination is considered with the other loads, the evaluation of such a load combinations may be omitted.

Table 5.4.1-2. Allowable compression stress of concrete.

Load state	Stress state I	Stress state II
I, II	0.33 F_C	0.45 F_C
III	0.66 F_C	0.75 F_C

Table 5.4.1-3. Allowable shear stress for bottom concrete.

Load state	Out-of-plane shear stress (kgf/cm^2)
I, II	Lower than F_C / 30 and (5 + F_C / 100)
III	1.5 times the above

Table 5.4.1-4. Temperature limit of concrete (°C).

Acting state of temperature load	Temperature of concrete	
	General portion	Local portion
Long-term	65	90[(1)]
Short-term	175	350[(2)]

(1) Penetration portion of piping, etc.

(2) The portion which receives a high-temperature jet flow due to rupture of the piping.

Table 5.4.1-5. Allowable stress for reinforcing bars (kgf/cm^2).

Load state	Tensile and compressive [strength][1],[2]					
	SR24	SR30	SD30	SD35	SD40	Welded metal mesh
I, II	1600	2000	2000	2000	2000	2000
III	2400	3000	3000	3500	4000	—

[1]For reinforcing bars SD35, SD40 with diameter smaller than 25 mm, the tensile/compression [strength] allowable for load states I and II is set at 2200 kgf/cm^2.

[2]For reinforcing bars SD35, SD40 with diameters of 51 mm, the tensile/compression [strength] allowable for load states I and II is reduced to 1800 kgf/cm^2.

Table 5.4.1-6. Allowable stress for out-of-plane shear reinforcing bars (kgf/cm^2).

Load state	Tensile and compression					
	SR24	SR30	SD30	SD35	SD40	Welded metal mesh
I, II	1600	2000	2000	2000	2000	2000
III	2400	3000	3000	3000	3000	3000

Table 5.4.1-7. Allowable bond stress between reinforcing bars and concrete (kgf/cm^2).

Load state	Plain steel bars	Deformed steel bars
I, II	Lower than 6/100 F_C and 13.5	Lower than 1/10 F_C and (13.5 + 1/25 F_C)
III	1.5 × (same as above)	1.5 × (same as above)

Table 5.4.1-8. Allowable tensile stress of PC bars.

Load state	Tensile
I, II, III	0.75 F_u or 0.85 F_y

F_u: Standard tensile strength of PC bars
F_y: Standard yield-point strength of PC bars

Table 5.4.1-9. Limit values for load state IV.

Material	Stress	Limit strain
Concrete	Membrane 0.65 F_C Membrane + bending 0.85 F_C (Rectangular, parabolic distribution)	0.003
Reinforcing bars	Standard yield strength	0.005

d. Limit values with respect to load state IV
See Table 5.4.1-9.

e. Application of allowable values

For load states I-III, elastic allowable stress design is performed. For load state IV, it is out of the elastic range for the load combinations assumed for a safety evaluation of the containment vessel. However, since the significant plastic state is not yet reached in this region, the strength design is still performed.

For load states I-II, since hurricanes and snow are included for the various load contents, it may not necessarily be a long-term load. However, in consideration of the correspondence to load state III, the importance of the containment vessel is taken into consideration on the basis of Japanese design system, and the allowable for a long-term load is used.

For load state III, the allowable stress level in a short-term load is used for the design. For load states I-III, the thermal load is taken as the design load. When the thermal stress is combined with the stress caused by other external forces, stress state 2 is applicable. In this case, considering the self-limiting property of the thermal stress, the allowable stress for the concrete is increased from that in stress state 1.

For load state IV, the safety is investigated from the strength design based on the ultimate strength capacity of the cross section. In this case, the limits for the reinforcing bars and concrete listed in Table 5.4.1-9 are used. In addition, for state IV, the thermal load is not added to the load combinations.

5.4.2 PCCV

(1) Introduction

The general shape of PCCV (prestressed concrete containment vessel) has a cylindrical shell portion attached with a dome and has a bottom made of a flat plate. The shell portion has a prestressed concrete structure to reduce the membrane tensile stress. The bottom has a reinforced concrete structure which can guarantee the structural strength. The airtightness is maintained by a liner applied on the inner surface of the concrete structure. Figure 5.4.2-1 shows the structural concept of PCCV.

The liner applied on the inner surface is used as the form for the concrete structure. Hence, it is reinforced by a stiffener, which is also used as the fixing material of the concrete. On the inner side and outer side of the concrete cross section, reinforcing bars are applied in two directions, i.e., in both the longitudinal and transverse directions. At the central portion of the cross section, a longitudinal tendon with a prestress applied to it is set; at the outer portion, a tendon is set in the circumferential direction. In the scheme illustrated in this figure, the circumferential tendon is of 3-buttress form with 3 places of anchorages separated by 120° from each other. In addition, a 2-buttress PCCV scheme has also been proposed.

Figure 5.4.2-2 illustrates an example of the arrangement of the tendons. In this example, for the circumferential tendons in the conventional portion, they are fixed in a buttress manner every 240° with a distance of 30 cm or 60 cm from each other. In the vertical direction, 90 pieces in the inverted U shape are set up in two directions. There are 180 locations for anchorage on the tendon gallery. Generally speaking, the design of the tendons is taken as a means for reducing the membrane tensile stress determined by a high internal pressure. The tendon force is designed on the basis of an evaluation of the effective prestress force in consideration of the loss in the tensile force after 40 years of operation.

(2) Structural analysis

a. General features

As far as the structural analysis is concerned, for the load combination described in Section 5.4.1(4) "Load combination for design and load state," in principle, the stress is calculated by performing an elastic analysis on the basis of the elastic stiffness of the components. However, for the discontinuous portion, opening portion, peripheral portion of the penetration portion, and other portions with concentrated stress, the stress can be calculated in consideration of the effect of the plastic deformation of the part.

b. Analytical schemes

In consideration of the various loads acting on the containment vessel and their combinations, as well as the loads acting locally and the effects of the opening portion, it is rather difficult to represent all of these factors by a single analytical model. Usually, the stress analysis of the containment vessel may be carried out using the following methods: a method in which the containment vessel is represented by an axisymmetric model, with the opening portion neglected; or a method in which FEM is used for the analysis of a 180° model obtained by vertically dividing the model of the containment vessel containing an opening portion, etc., or a 360° model, i.e., the model of the entire body.

The results of the various stresses calculated using these methods are combined in application. Figure 5.4.2-3 shows an example of the stress analytical model of the shell portion of a containment vessel. For the vicinity of an opening portion or other local portions which require a more detailed analysis of the stress, the portion can be extracted for the more detailed analysis.

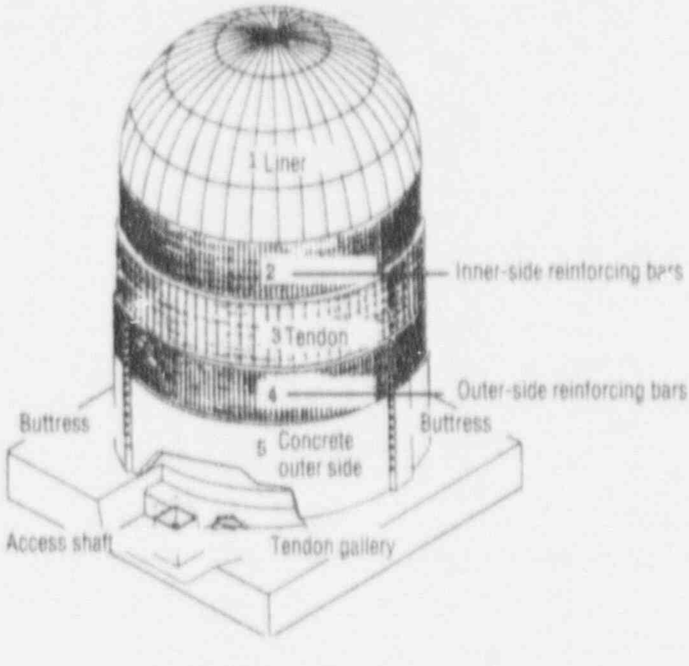


Figure 5.4.2-1. Structural concept of PCCV.

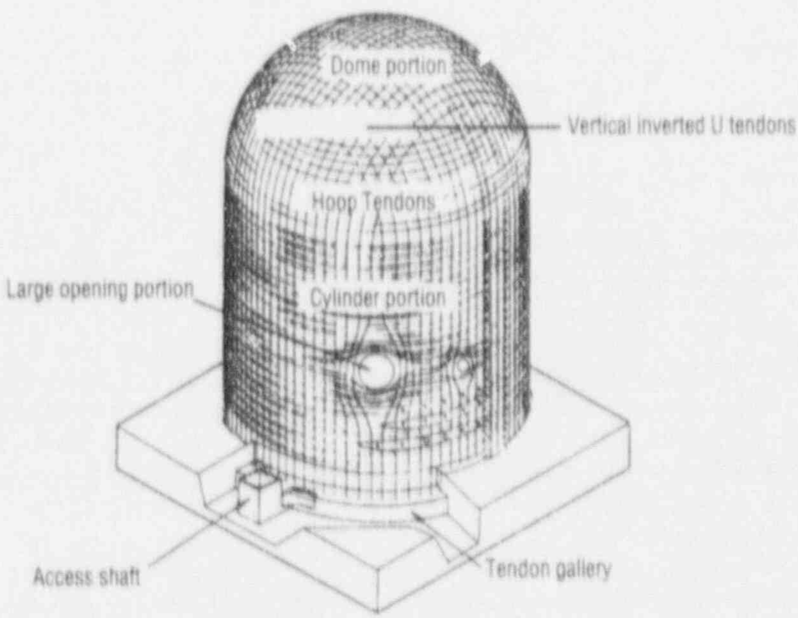


Figure 5.4.2-2. Layout example of tendons.

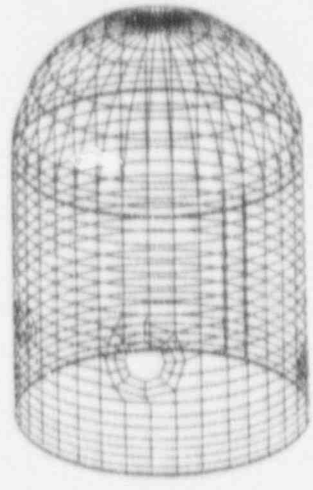


Figure 5.4.2-3. Stress analysis model example.

(3) Cross-sectional design

The stresses derived in the structural analysis are combined for each load state. The allowable values suitable for the stresses are used to perform the cross-sectional design.

a. Design for membrane and flexural stresses

For design related to the membrane and flexural stresses, usually, the schemes described in "RC Standards" are used for performing the cross-sectional design. For the shell portion, the membrane stress is combined with the in-plane shear stress to calculate the equivalent membrane stress for evaluation [5.4.1-1].

b. Design for shear stress

For the shell portion, the ultimate strength capacity is determined using the empirical formula proposed for the in-plane shear stress of cylindrical shear walls, and the actual shear stress should not become higher than this ultimate strength capacity. For the out-of-plane shear stress, the ultimate strength capacity is calculated for the cylindrical foot portion, which is dominant in the design, using the empirical formula (5.4.1-1) proposed for the out-of-plane shear stress in test condition and in LOCA condition. In the design for the shear stress, for both the in-plane and out-of-plane directions, it is taken as 1/2 the ultimate strength capacity for load states I and II; it is taken as 3/4 the ultimate strength capacity for load state III. For the bottom portion or a portion other than the shell portion, the schemes described in "RC Standards" are used.

c. Design for thermal stress

For a concrete containment vessel, it is required to investigate the effect of the thermal stress. The thermal stress is a type of self-limiting stress, and it decreases as the plasticity increases. In consideration of this property of the thermal stress, the thermal stress cannot be treated as an external stress. Instead, the increase in the stress due to an increase in the thermal load is calculated by accounting for the existing stress and the amount of plasticity.

In the evaluation of the ultimate strength capacity, it has been found from the experimental results that the structural strength subjected to thermal loads is the same as the structural strength without a thermal load; hence, in principle, the thermal stress may be excluded from the evaluation of the ultimate strength capacity.

Since elastic design is performed for up to load state III, in one method, the s. ess is calculated in consideration of the plastic stiffness of components under the stress caused by the other load. In another method, the cross-sectional stiffness is replaced by the equivalent elastic stiffness which has been reduced by the apparent coefficient, which is combined with the stress caused by external forces. In this case, the reduction coefficient (stress residual rate) is up to 1/3 for [load] state III, and up to 1/2 for load states I and II.

d. Analytical checkup for designed portion

When the cross-sectional design is carried out, the locations required for the design should be appropriately selected. For example, the shell portion contains a conventional portion and discontinuous portion; the bottom portion contains a conventional portion and a boundary portion with the upper structure; etc. The cross-sectional design should be performed in consideration of the local peak stress caused by the discontinuity.

For the layout of reinforcing bars determined after performing the cross-sectional design of PCCV, the following items are checked by analysis:

{1} In each load state, the possibility and amount of cracking, and the stress levels of the concrete and reinforcing bar
{2} Deformation amount
{3} Reduction rate for thermal load

(4) Experimental checkup

As far as the experiments concerning PCCV are concerned, some preliminary experiments have been performed since the mid '60s. Since PCCV was actually adopted for the reactor containment vessel, various experiments for the verification were carried out. The major design loads for PCCV include seismic loads S_1 and S_2, pressure load and thermal load in an accident, etc. The experiments are carried out primarily for {1} in-plane shear, {2} out-of-plane shear, {3} thermal stress, {4} liner, etc. In this section, we will present large-scale experiments with an earthquake as the primary load among the numerous experiments performed for the various loads and their combinations.

These experiments (5.4.2-1) are a large-scale model test for PCCV against the seismic load, with the dimensions of the models being 1/8 and 1/30 the dimensions of the actual size, respectively. Its purpose is to confirm the soundness of the structure and the reliability of the design. For the 1/8 model, a static horizontal test is performed. Since the model is large, the test has the characteristics of an actual-proving test. For the 1/30 model, the plan is to arrange both a static horizontal test and a dynamic vibration test for the same model. Since the static test and dynamic test are performed at the same time, the dynamic behavior of the structure can be clarified. This is quite attractive. Figure 5.4.2-4 shows the flow sheet of the experimental research work.

The results of the experiment in both the horizontal static test and the vibration test can be summarized with respect to the following aspects: {1} failure made and ultimate strength capacity, {2} decrease in stiffness, {3} damping constant. From the design values and experimental data, it can be seen that good results are obtained in all of the following aspects: {1} behavior when a prestress is introduced, {2} behavior when an internal pressure is applied, {3} behavior under the designed seismic load, {4} margin of resistance against the design load, {5} design reliability.

The major experiments related to PCCV, including the above experiments, are implemented as a part of the tests for confirmation of the technical standards. These experimental results are reflected in the amended draft of the Technical Standards of Concrete Containment Vessels. As far as the confirmation tests of the technical standards are concerned, the investigation is primarily made on the in-plane shear, foot-portion out-of-plane shear, and thermal stress of the concrete containment vessel. For further details, please see the articles of Aoyagi (5.4.2-2) and Akino and Watanabe (5.4.1-1).

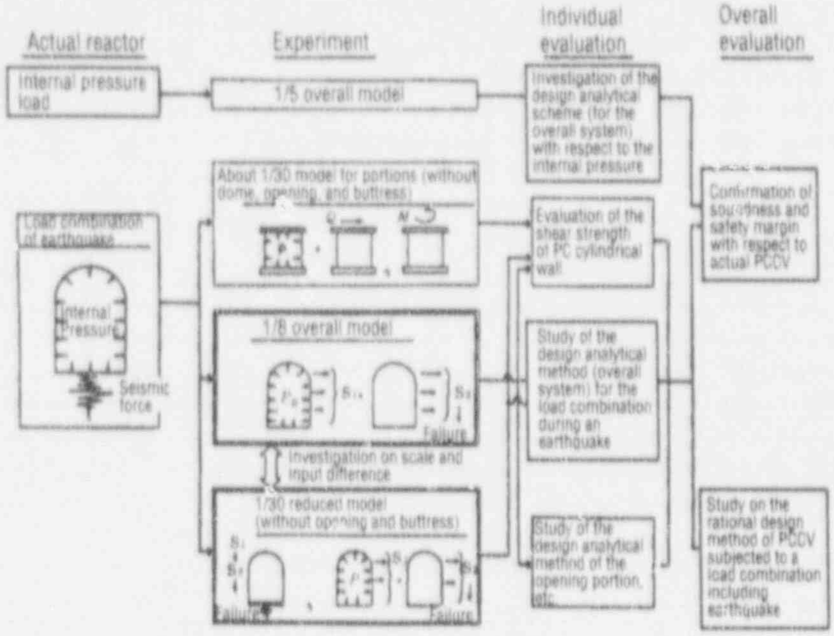


Figure 5.4.2-4. Flow chart of experimental research.

5.4.3 RCCV

(1) Introduction

At present, RCCV (reinforced concrete containment vessel) is being developed in Japan as the containment vessel of BWR. Here, we will discuss the design concepts and methods of the various types of RCCV. Figure 5.4.3-1 shows the structure of RCCV, which is a reinforced concrete structure made of a cylindrical shell, a circular top slab, and a foundation mat. The cylindrical shell portion and the top slab portion are integrated with the reactor building's structure via the floor slab and the pool girder, respectively. A steel liner is applied on the inner surface of RCCV to maintain the airtightness. Using the longitudinal and transverse stiffeners welded onto it, it is fixed on the RCCV.

(2) Structural analysis

a. General features

The general features of the design load and load combination are similar to those in the case of PCCV. However, in the case of RCCV, the SRV load characteristics of BWR (the dynamic pressure load released to the suppression pool to control the pressure variation during the operational process) and the chugging load (the dynamic pressure load applied on the suppression pool through the vent tube in the case of an accident due to rupture of the main steam pipe) should also be taken into consideration as the design loads. The structural analytical method and the cross-sectional calculation method are the same as those in the case of PCCV. In addition, for RCCV, an evaluation of the concrete crack width and evaluation of the soundness of the structure of the containment vessel are also carried out.

b. Analytical schemes

In principle, the stress analysis for the design stress calculation is an elastic analysis. However, for the portion in which a decrease in the thermal stress and stress redistribution due to concrete cracks take place, an

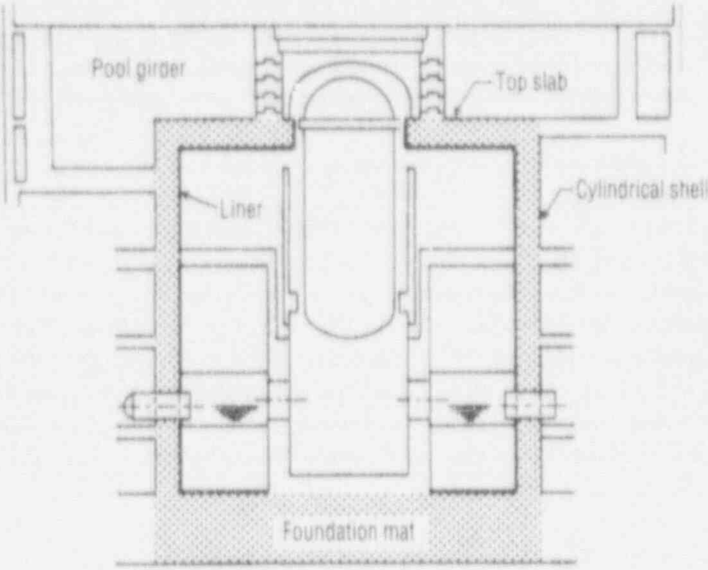


Figure 5.4.3-1. Profile of RCCV.

equivalent elastic analysis and elastoplastic analysis are carried out, with the results taken into consideration in the cross-sectional design. Since both the structural shape and loads are complicated, the elastic analysis makes use of FEM. On the other hand, for the temperature distribution analysis (with a model finely divided in the heat transfer direction), stress analysis of the general portion (with an axisymmetric model, or three-dimensional model), local stress analysis (three-dimensional model), etc., the model used is selected according to the specific content of the analysis.

The design stresses are calculated by multiplying the basic stresses of the elastic-stress analytical result subjected to the basic load, by the load coefficient listed in Table 5.4.1-1; then the stresses are added. As the thermal stress is decreased due to the cracks in the concrete, said basic stress is multiplied with a thermal stress reduction coefficient, depending on the load state, and the result is then used as the basic stress.

(3) Cross-sectional design

In the case of cross-sectional design, for the design stress with respect to the various design loads and their combinations, the allowable stress design and the ultimate strength design are carried out according to the load state described in Section 5.4.1(4) "Load combinations for loads and load state."

(4) Experimental verification

The shape of the RCCV shown in Figure 5.4.3-1 is a prototype, which was the first construction of this type in Japan. Hence, proving tests were performed for the following purposes.

(i) Confirmation of the soundness of RCCV against the external design forces

Although the design is carried out as an elastic design, at the design load level, the cross sections are subjected to biaxial tensile stresses (tension-tension), which exceed the tensile strength of the concrete. Hence, the amount of cracks and the stress redistribution caused by the cracks are determined by experiments, and the soundness of RCCV is confirmed.

(ii) Confirmation of the safety margin of RCCV with respect to the primary design load

By comparing the primary design loads (pressure load, thermal load, seismic force, and their combinations) with the ultimate capacity obtained from experimental results, the safety margin is confirmed.

(iii) Confirmation of the applicability of design standards and establishment of an evaluation of method of items not defined in the standards

"Technical Standards for Concrete Containment Vessels Used in Nuclear Plants (draft)" primarily concerns PCCV. It defines the design methods on the basis of various experimental data related to the containment vessel structure made of a cylindrical portion, dome portion, and a foundation mat portion. However, as RCCV shown in Figure 5.4.3-1 is compared with PCCV, the wall thickness and shape are somehow different; hence, experiments should be performed to confirm the applicability of the technical standards (draft). On the other hand, the top slab of RCCV is a donut-shaped circular plate. When a pressure load is applied, an in-plane tensile stress and an out-of-plane shear stress are generated at the same time. At present, there are as yet no research data on the shear yield strength with respect to this combination of stresses. Hence, basic experiments should be carried out to establish the evaluation method.

(iv) Evaluation of the appropriateness of the design analysis method

The analytical method and design method used in the design are applied for the experimental simulation to see if these methods are appropriate. In this case, the emphasis of the evaluation is placed on evaluating whether it is appropriate to incorporate the actual phenomena of stress redistribution and thermal stress reduction caused by cracks in the concrete in a safe and suitable way into the design method.

5.5 Analysis examples

5.5.1 BWR (MARK-II)

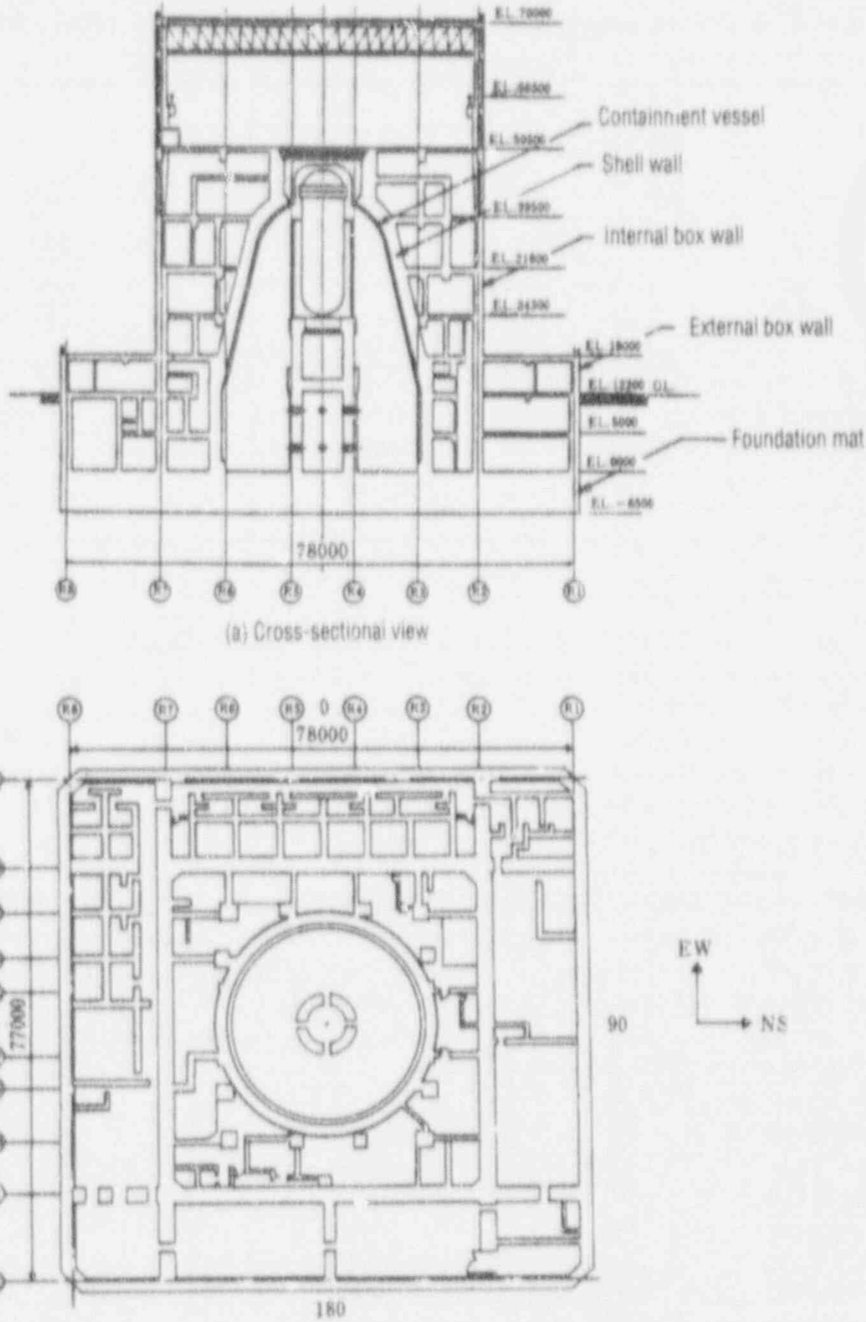
As an analytical example of the BWR reactor building, we will discuss MARK-II (1.10 million kWe class) and will present the results including the seismic response analysis and the cross-sectional design of the primary portion of the building.

(1) General features of the building

The building is primarily made of a reinforced concrete structure, with its roof made of a steel frame (see Figure 5.5.1-1). At the middle part of the building, there is an 8-story reactor compartment, with a 4-story auxiliary compartment at its periphery. The building has a height of about 77 m as measured from the bottom of the foundation. The reactor compartment and the auxiliary compartment are located on the same foundation mat with an integrated structure. The lower portion of the building has a plan shape measuring 80.5 m × 81.5 m. The foundation mat is directly set on a support rock ground. The total weight of the building is about 0.33 million ton. The building's structure has a shell wall, internal box wall, and external box wall arranged in sequence from the center surrounding the containment vessel. These shear walls are interconnected through the floor slabs. The overall building is a structure with a very high stiffness.

(2) Analysis conditions

The reactor building belongs to class A or class A_S. The reactor compartment, as a secondary containment facility, belongs to design class A. As far as the seismic force acting on the reactor building is concerned, the dynamic seismic force calculation from the S_1 seismic response analysis or the static seismic force derived by some other calculation, whichever is larger, is taken as the horizontal seismic force, which is then combined with the vertical seismic force derived from the vertical seismic coefficient to form the design seismic force.



(b) Plane view

Figure 5.5.1-1. BWR MARK-II (1.10 million kWe class) reactor building (Standard, Construction-5 [HYO, KEN-5]).

Table 5.5.1-1. Basic earthquake ground motions (Standard, Construction-1 [HYO, KEN-1]).

			Magnitude (M)	Distance to epicenter Δ (km)	Peak acceleration (Gal)	Peak velocity (kine)	Phase characteristics
S_1	S_1-1	Long distance	8.4	90	286.15	16.09	Taft 1952 EW
	S_1-2	Short distance	7.0	20	267.4	17.15	El Centro 1940 NS
S_2	S_2-1	Long distance	8.5	68	407.1	26.26	Hachinohe 1968 EW
	S_2-2	Short distance	6.5	7.2	340.13	13.24	Cholame Shandon 1966 N 40 W

The cross section of the building is designed on the basis of the allowable stress determined according to the Construction Standards, etc., from the stress calculated as a combination of the seismic load, the load that always acts, and the load that acts during the operation. In addition, since a portion of the reactor building belongs to the class A_S facility and it contains the equipment and piping of the A_S class, the analysis should be performed by considering the nonlinearity of the various portions of the building and the foundation with respect to an S_2 earthquake. The basic earthquake ground motions used in the analysis include two waves of an S_1 earthquake and two waves of an S_2 earthquake listed in Table 5.5.1-1. Figure 5.5.1-2 (a)-(d) show their standard spectra and the response spectra of the simulated seismic waves. Figure 5.5.1-3 (a)-(d) show their respective acceleration time histories. Figure 5.5.1-4 shows the profile of the ground. For the ground supporting the building, the velocity of S-wave V_S = 500 m/s. The rock outcrop surface is a rock surface which is 160 m below the building position and has V_S = 700 m/s. In addition, for the input earthquake motion for the seismic response analysis of the reactor building, the seismic response analysis of the ground is performed using the 1-dimensional wave theory on the basis of the basic earthquake ground motions applied at the rock outcrop surface; obtained response motions at the building's foundation are used as the input seismic motions.

(3) Seismic response analysis

a. Analysis model

As shown in Figure 5.5.1-1, a seismic force is acting on the EW direction in this example. In this example, for the building's seismic response analysis model, the embedment effect is relatively small. Hence, as shown in Figure 5.5.1-5, the sway/rocking model (multi-cantilever model of the bending shear-type building) is adopted. In this figure, the various structural elements of the building are shown. The damping constant of the building is assumed to be 5%.

The properties of the ground are listed in Table 5.5.1-2, with the spring constant derived using the static theoretical solution based on the elastic wave theory (Novak's theoretical solution is used to derive the side-surface spring constant). The calculation results are listed in Table 5.5.1-3. The damping of the ground is treated using the approximate method in which the frequency dependence of the soil damping is determined approximately (see Section 5.2.3(2) "Analytical methods").

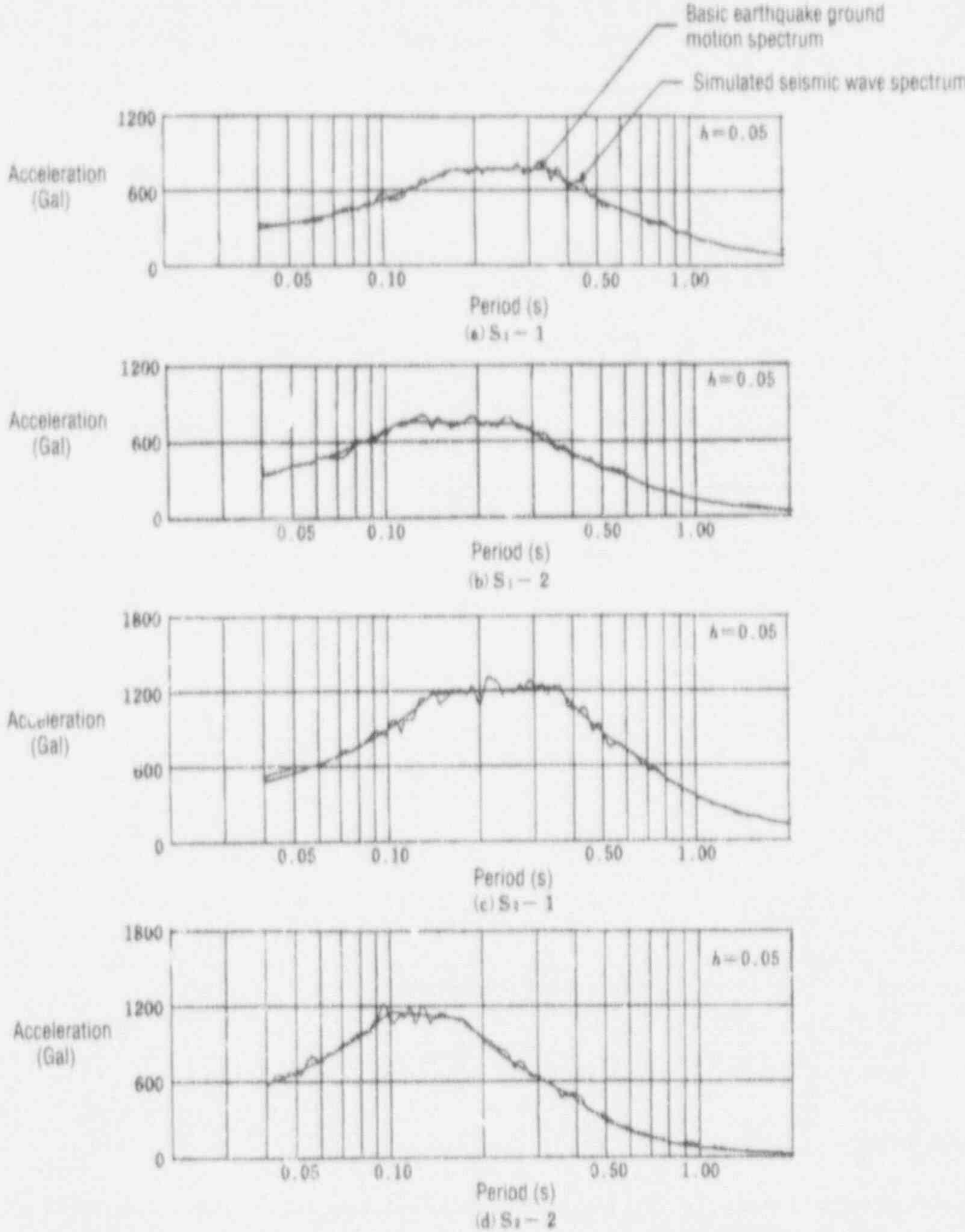


Figure 5.5.1-2. Acceleration response spectra of basic earthquake ground motion and simulated seismic wave (Standard, Construction-1 [HYO, KEN-1]).

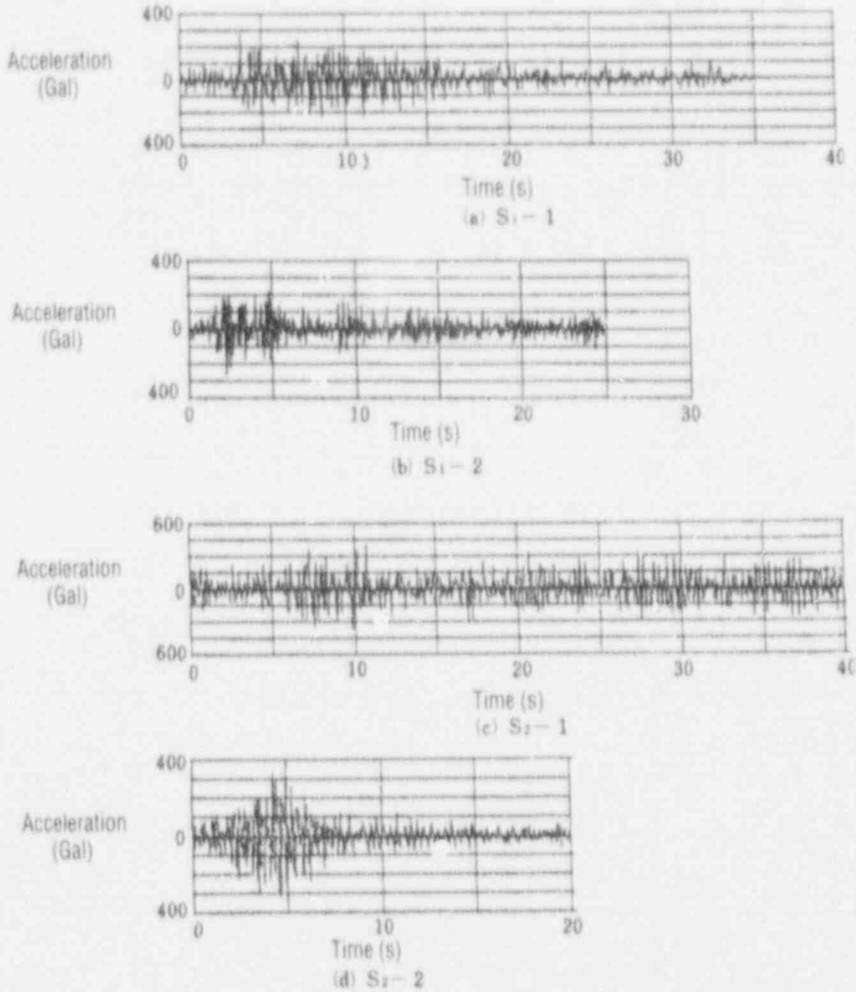


Figure 5.5.1-3. Acceleration time history of simulated seismic wave (Standard, Construction-1 [HYO, KEN-1]).

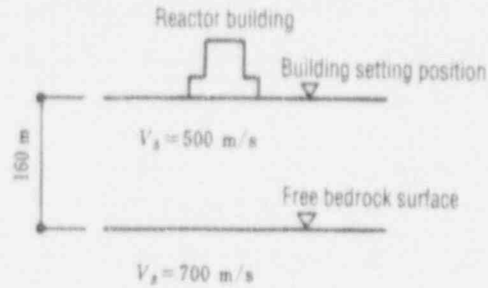


Figure 5.5.1-4. Profile of ground.

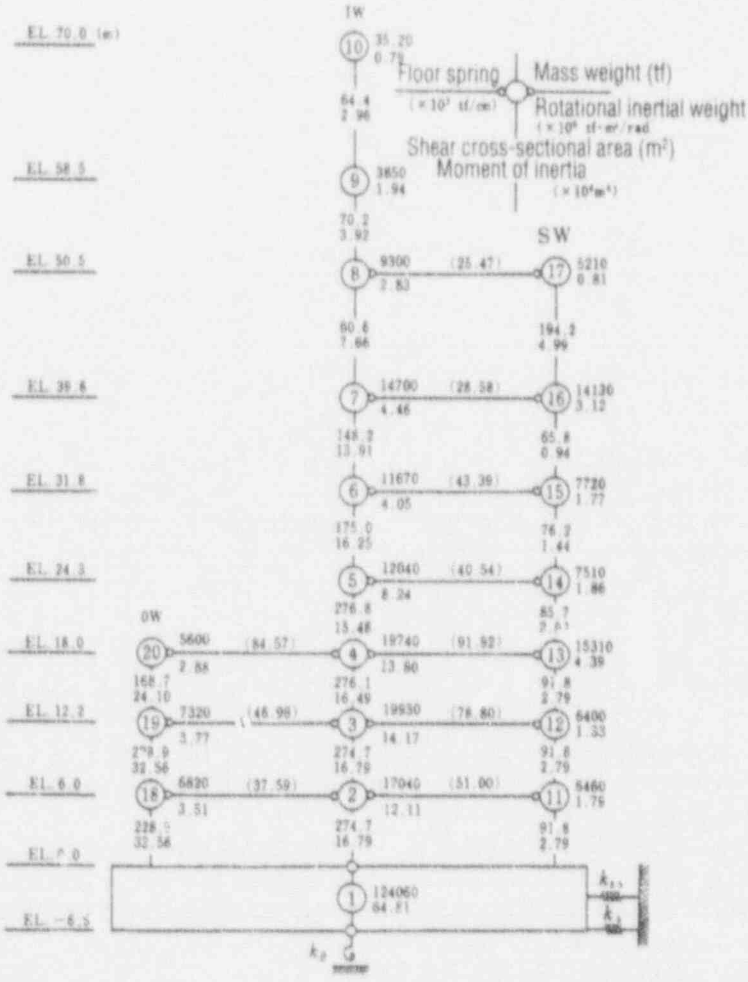


Figure 5.5.1-5. Seismic response analysis model and parameters.

Table 5.5.1-2. Properties of ground.

Shear wave velocity	(V_S)	m/s	500	700
Ground density	(ρ_t)	tf/m^3	1.7	1.8
Poisson's ratio	(ν)		0.42	0.41
Shear elastic coefficient	(G)	tf/m^2	4.34×10^4	9.00×10^4
Young's modulus	(E)	tf/m^2	1.23×10^5	2.54×10^5

Table 5.5.1-3. Spring constants of ground.

Bottom horizontal spring	(K_S)	tf/m	9.22×10^6
Bottom rotational spring	(K_ϕ)	tf·m/rad	1.63×10^{10}
Side horizontal spring	(K_{SS})	tf/m	5.96×10^5

b. Eigenvalue analysis results

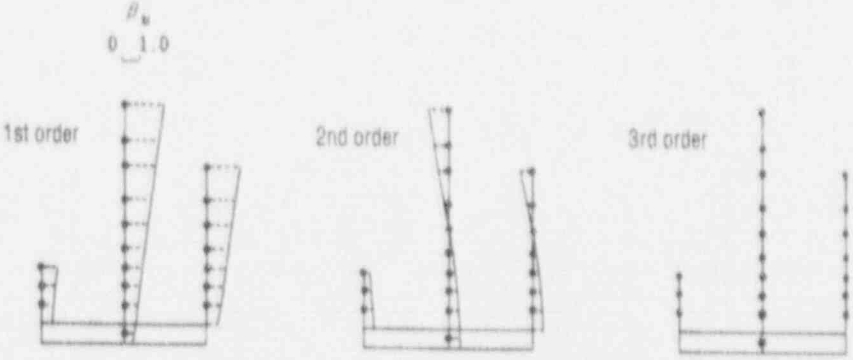
Figure 5.5.1-6 shows the eigenvalue analysis results. The first-order natural period of the building as the soil-structure interaction system is about 0.478 sec.

c. S. seismic response analysis

Figures 5.5.1-7 (a)-(d) show the maximum response values as the results from the seismic response analysis. For a long-distance earthquake, the maximum acceleration distribution has a value of about 270 Gal at the foundation position and about 800 Gal at the roof position (the acceleration amplification factor at the roof is about 3 times that of the foundation). For a short-distance earthquake, it becomes about 230 Gal at the foundation position and about 770 Gal at the roof position (the factor is also about 3.3 times). At the first floor of the building, the maximum story shear is about 69,900 t (story shear coefficient: 0.36) for a long-distance earthquake and about 53,400 t (story shear coefficient: 0.27) for a short-distance earthquake. The average shear stress of the shear wall (the value obtained by dividing the response shear stress by the effective shear cross-sectional area) is up to 15.3 kgf/cm^2 (long-distance earthquake, internal box wall, 5th floor). In addition, this figure also shows the shear stress and bending moment caused by the static seismic force.

Mode number	Period (sec)	Participation factor
1	0.478	1.83
2	0.247	0.97
3	0.131	0.06
4	0.114	0.00
5	0.105	0.17
6	0.076	0.06
7	0.069	0.01
8	0.068	0.01
9	0.066	0.02
10	0.053	0.01

(a) Natural period and participation factor



(b) Natural vibration mode (participation factor)

Figure 5.5.1-6. Eigenvalue analysis results.

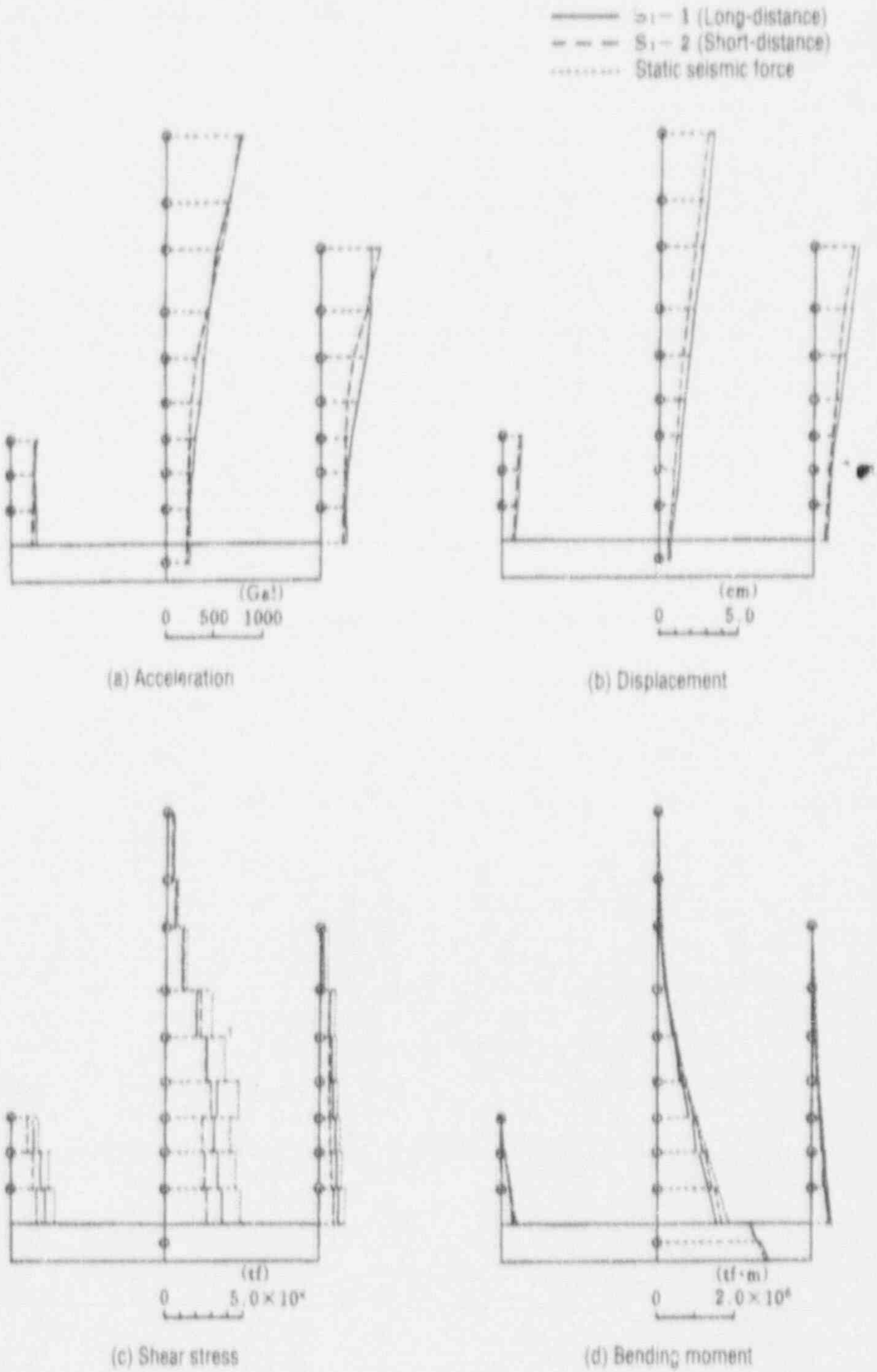


Figure 5.5.1-7. Maximum response value (S_1 earthquake).

d. S_2 seismic response analysis

(a) Determination of restoring force characteristics for various parts

(i) Restoring force characteristics of building

For the restoring force characteristics of the bending-shear beam elements, the skelton curves and hysteresis loops are evaluated separately for flexure and shear deformation springs according to the EPJR scheme. (see Section 5.2.5 (2) "Restoring force characteristics of structure".) Table 5.5.1-4 lists the various parameters of the skeleton curve.

(ii) Rotational spring of ground

For the rotational spring of the ground, the geometrical nonlinearity due to the foundation uplifting is taken into consideration, and the M - θ relation of the trilinear-type foundation uplifting shown in Figure 5.5.1-9 exists between the moment and the rotational angle. (see Section 5.2.5 (3) "Restoring force characteristics of foundation.")

(b) S_2 seismic response analytical results

Figures 5.5.1-10 (a)-(d) show the maximum response values of the building due to S_2 earthquakes. Among the two seismic waves, for the long-distance earthquake that can provide a large-response to the building, the maximum acceleration is about 440 Gal at the foundation position and about 1,200 Gal at the roof position (with an acceleration magnification factor of about 2.7 times). The maximum shear force of the story is about 97,500 t at the first floor position of the building (with a story shear coefficient of 0.50) and the average shear stress of the shear wall is up to 21.9 kgf/cm^2 (at the 5th floor position of the internal box wall). In this case, the first turning point of the τ - γ skeleton curve is slightly overpassed, and the other parts of the building are almost within the elastic range. Figure 5.5.1-11 shows the plotted results on the skeleton curve for the response values of the lower portion of the building.

e. Contact rate of the foundation

The contact rate can be calculated using the formula derived from the static balance (see equation 5.2.5-23) shown in Section 5.2.5(3)b "Evaluation of contact rate," with results shown in Table 5.5.1-5. In this case, for both the S_1 and S_2 earthquakes, the contact rate is 100%, i.e., no uplifting occurs.

(4) Stress analysis and cross-sectional design of major structural parts.

a. Shear wall

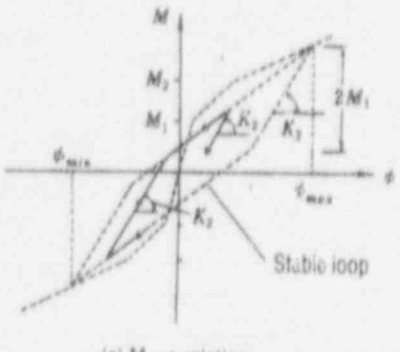
(a) Shear stress in the shear wall

Figure 5.5.1-12 shows the flow chart for the design of an shear wall. The shear stress distribution in components is determined for designing the various parts of the shear wall (shear stress and flexural moment) and to calculate the interstory displacement used in the design of a frame structure. The horizontal load during an earthquake is assumed to be totally resisted by the three shear walls, i.e., shell wall, internal box wall, and external box wall, and the horizontal displacements of these walls on each floor are assumed to be identical.

An analysis is performed using the design external forces which envelope the results of the response analysis of the S_1 earthquake and the static seismic force derived using other methods. The loads are applied on the model shown in Figure 5.5.1-13. In this case, a correction due to torsion is made for the shear stresses.

Table 5.5.1-4. Skeleton curve parameters concerning bending and shear.

	Level (m)	$\tau - \gamma$ relation						$M - \phi$ relation					
		τ_1 (kgf/cm^2)	γ_1 ($\times 10^{-4}$)	τ_2 (kgf/cm^2)	γ_2 ($\times 10^{-4}$)	τ_3 (kgf/cm^2)	γ_3 ($\times 10^{-3}$)	M_1 ($\times 10^5$ tf·m)	ϕ_1 ($\times 10^{-6}$ rad/m)	M_2 ($\times 10^5$ tf/m)	ϕ_2 ($\times 10^{-5}$ rad·m)	M_3 ($\times 10^5$ tf·m)	ϕ_3 ($\times 10^{-4}$ rad/m)
IW	58.5	18.2	2.03	24.6	6.08	55.7	4.00	2.8	4.5	4.6	4.2	10.5	4.8
	50.5	19.6	2.18	26.5	6.54	55.2	4.00	4.2	5.1	7.3	4.4	15.6	5.5
	39.8	20.1	2.23	27.1	6.69	54.9	4.00	8.6	5.4	15.2	4.5	32.2	6.0
	31.8	20.1	2.23	27.1	6.69	56.6	4.00	15.6	5.4	29.0	4.5	64.8	7.4
	24.3	20.7	2.30	27.9	6.90	54.7	4.00	19.2	5.6	35.1	4.6	73.4	7.3
	18.0	19.8	2.20	26.7	6.60	53.5	4.00	17.0	5.2	50.4	4.4	68.4	6.5
	12.2	20.5	2.28	27.7	6.84	51.7	4.00	19.2	5.5	59.1	4.6	72.4	7.1
	6.0	21.2	2.36	28.6	7.08	55.9	4.00	20.8	5.9	72.3	4.9	89.5	10.8
	0.0	22.1	2.46	29.8	7.38	56.[illegible]	4.00	22.4	6.4	79.5	5.1	94.3	13.3
SW	39.8	16.8	1.87	22.7	5.60	58.1	4.00	0.6	13.2	2.1	15.2	3.1	21.0
	31.8	21.7	2.41	29.3	7.2[illegible]	56.3	4.00	2.6	13.3	5.7	9.8	7.8	15.5
	24.3	22.7	2.52	30.6	7.56	56.4	4.00	3.8	12.7	8.1	8.7	11.0	13.2
	18.0	23.5	2.61	31.7	7.83	56.9	4.00	5.2	12.1	10.7	7.9	14.4	11.5
	12.2	25.7	2.86	34.7	8.58	62.2	4.00	7.5	12.7	15.2	7.3	20.0	8.7
	6.0	26.8	2.98	36.2	8.94	63.3	4.00	8.0	13.7	16.0	7.4	20.8	8.1
	0.0	27.8	3.09	37.5	9.27	64.6	4.00	8.6	14.7	16.8	7.5	21.6	7.5
OW	12.2	16.6	1.84	22.3	5.52	57.4	4.00	12.7	2.5	29.5	2.6	59.6	5.1
	6.0	17.3	1.92	23.4	5.76	58.2	4.00	18.6	2.7	46.5	2.6	90.8	3.2
	0.0	18.2	2.02	24.6	6.06	57.9	4.00	20.4	3.0	51.0	2.7	94.8	5.2



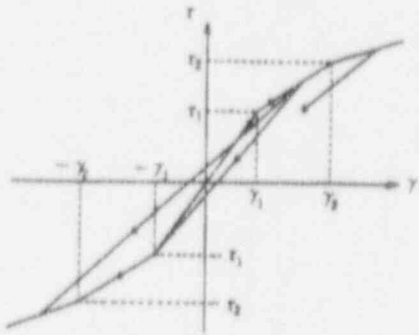


Figure 5.5.1-8. Restoring force characteristics of building (Research, Construction-2 [KEN, KEN-1]).

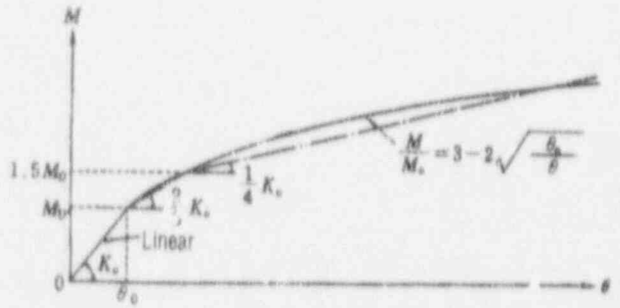


Figure 5.5.1-9. Foundation uplifting M–θ relation.

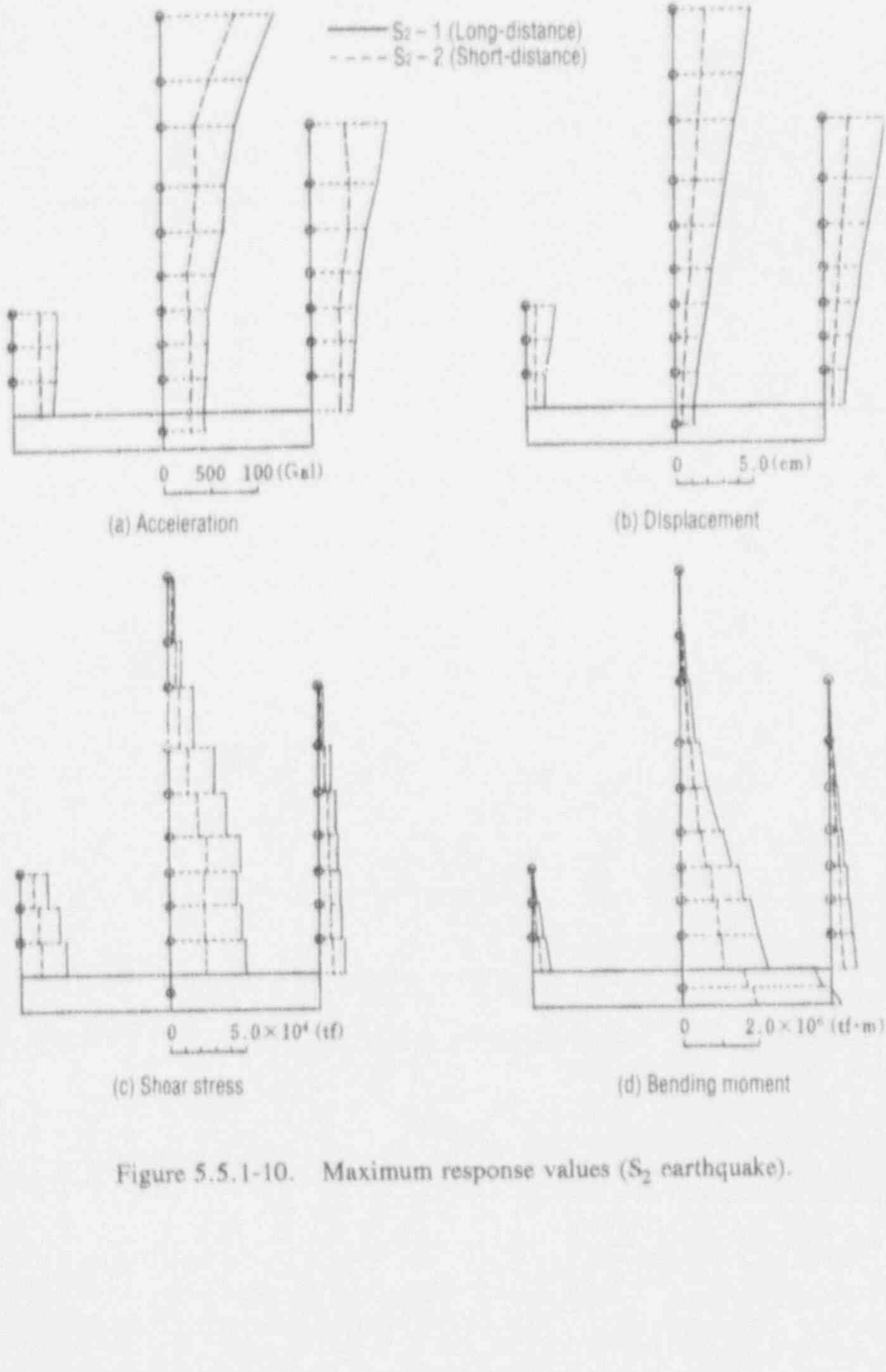


Figure 5.5.1-10. Maximum response values (S_2 earthquake).

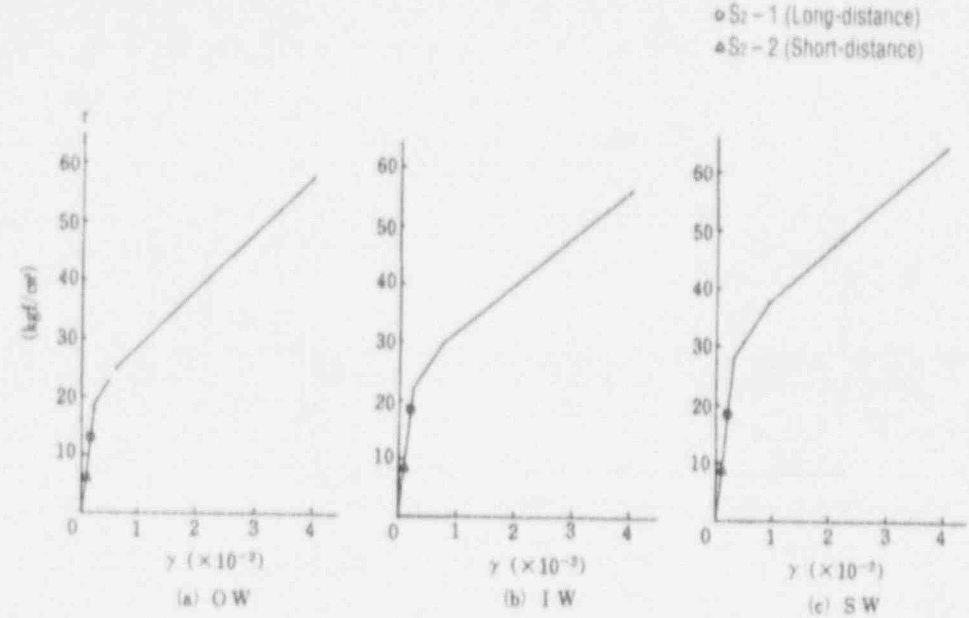


Figure 5.5.1-11. Restoring force characteristics and response values of lower portion of building.

Table 5.5.1-5. Contact rate of foundation.

Direction	Item	Earthquake			
		S_1		S_2	
		Long-distance	Short-distance	Long-distance	Short-distance
EW	Overturning moment ($\times 10^6$ tf·m)	2.87	2.73	4.21	1.91
	Eccentric distance e (m)	9.0	8.6	13.2	6.0
	Eccentricity ratio (e/L)	0.11	0.11	0.16	0.07
	Contact rate of foundation (%)	100	100	100	100

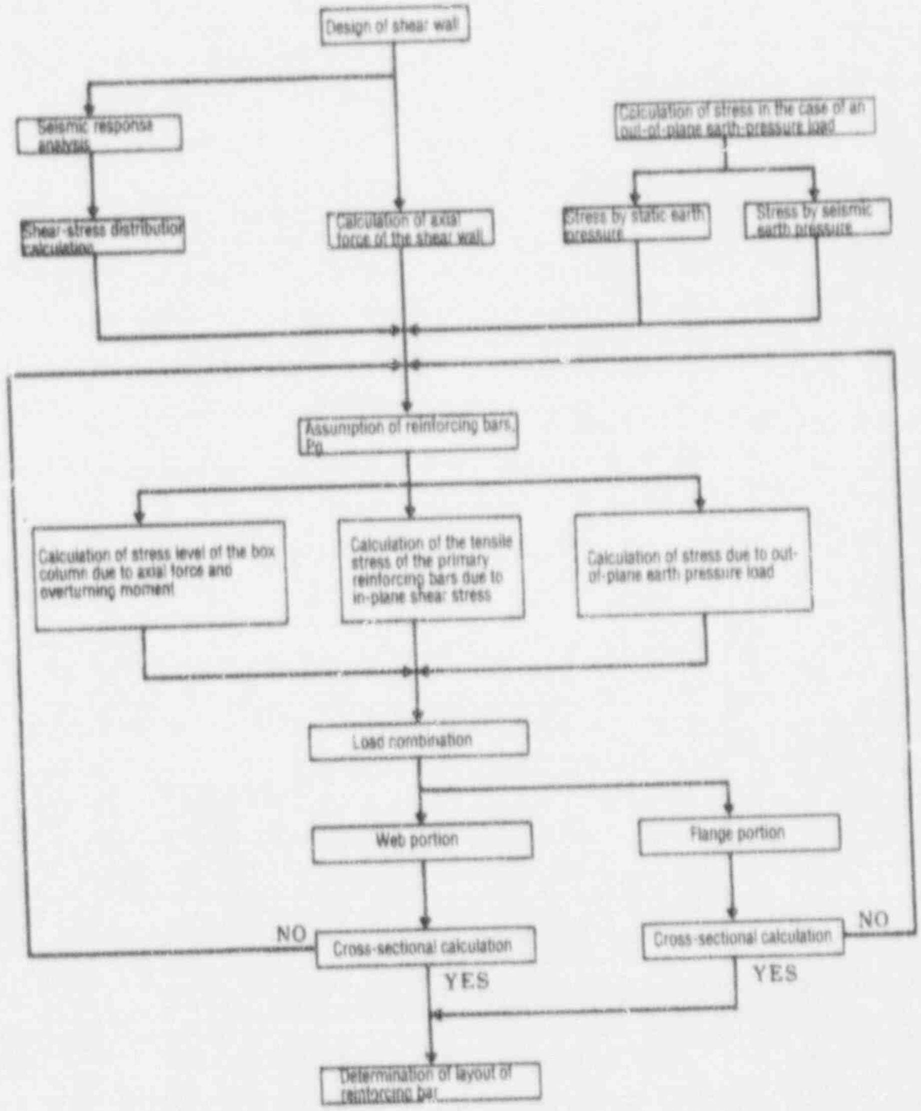


Figure 5.5.1-12. Design flow chart of shear wall.

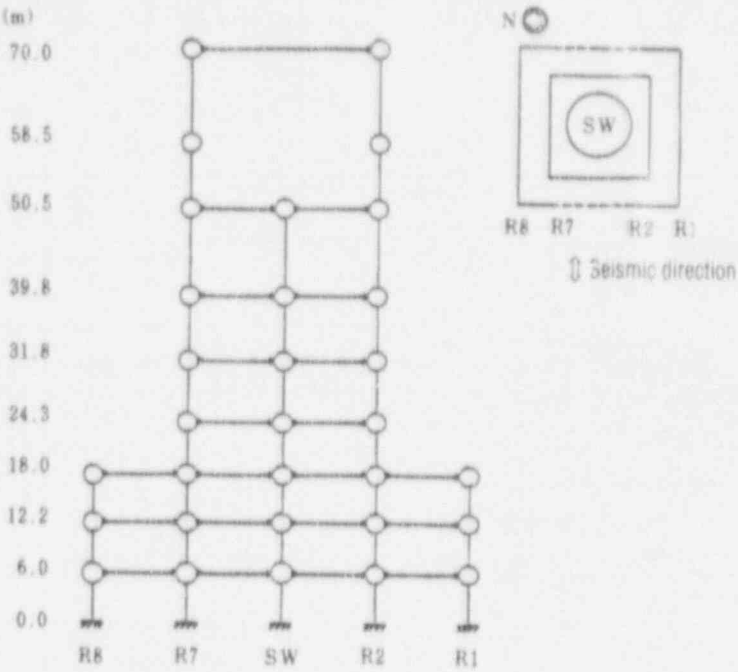


Figure 5.5.1-13. Model for analysis of shear stress distribution (EW direction).

(b) Cross-sectional design of the box walls

The box walls are designed to withstand the shear stress and bending moment determined in the above section on "Shear stress in shear walls." In addition, for the underground portion of the external box wall, in addition to said stress, safety should be maintained with respect to the earth pressure.

In the cross-sectional calculation, the shear stress is totally carried by the reinforcing bars. With respect to the bending moment, each box wall is treated as a column in the design. In addition, the stress caused by the earth's pressure is also taken into consideration. Figure 5.5.1-14 shows an example of the cross-sectional design example of the box wall.

b. Foundation mat

(a) General features of analysis

Figure 5.5.1-15 shows the design flow chart of the foundation mat. For the foundation mat, an analysis is performed by FEM as a plate supported on an elastic foundation. In addition, since the foundation mat is almost symmetric with respect to the NS axis and EW axis passing through the center of the reactor, an analysis can be performed for the half portion as divided by the EW axis.

Figure 5.5.1-16 shows the analytical model and its coordinate system. The model is divided into elements of quadrangles and triangles. Each element is taken as a plate element made of homogeneous isotropic material. For each element, the bending of the plate and the membrane stress are taken into consideration at the same time. For the bending of the plate, the influence of the out-of-plane shear deformation is also taken into consideration.

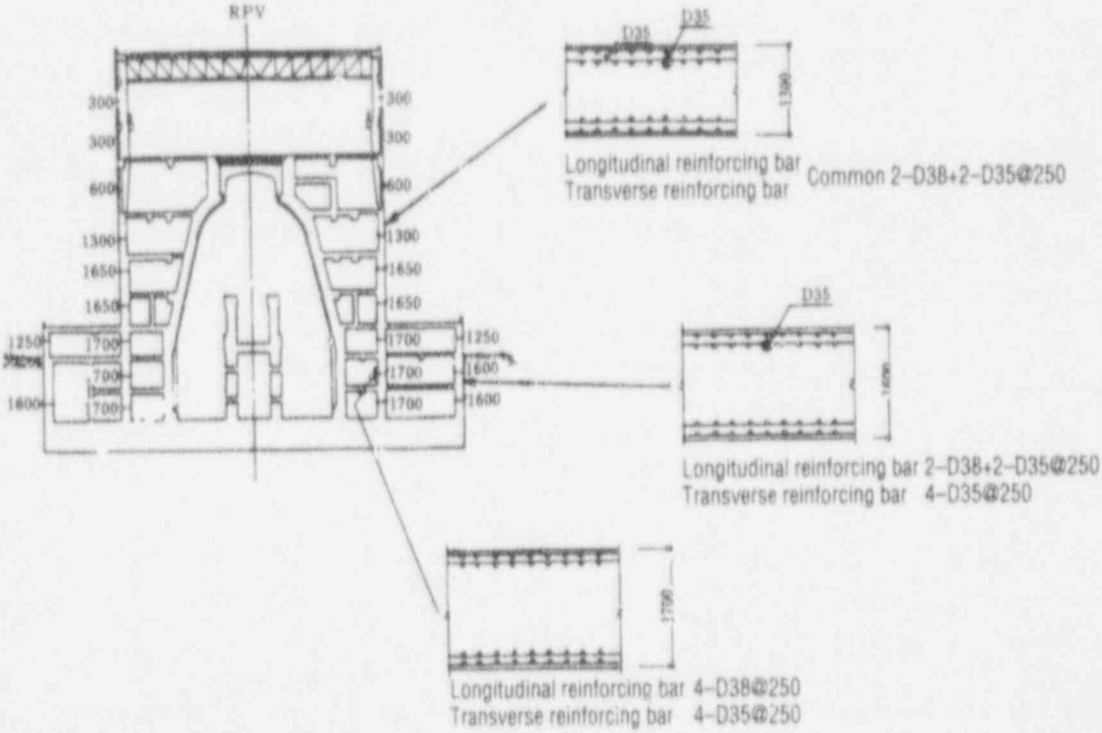


Figure 5.5.1-14. Example of cross-sectional design of box wall.

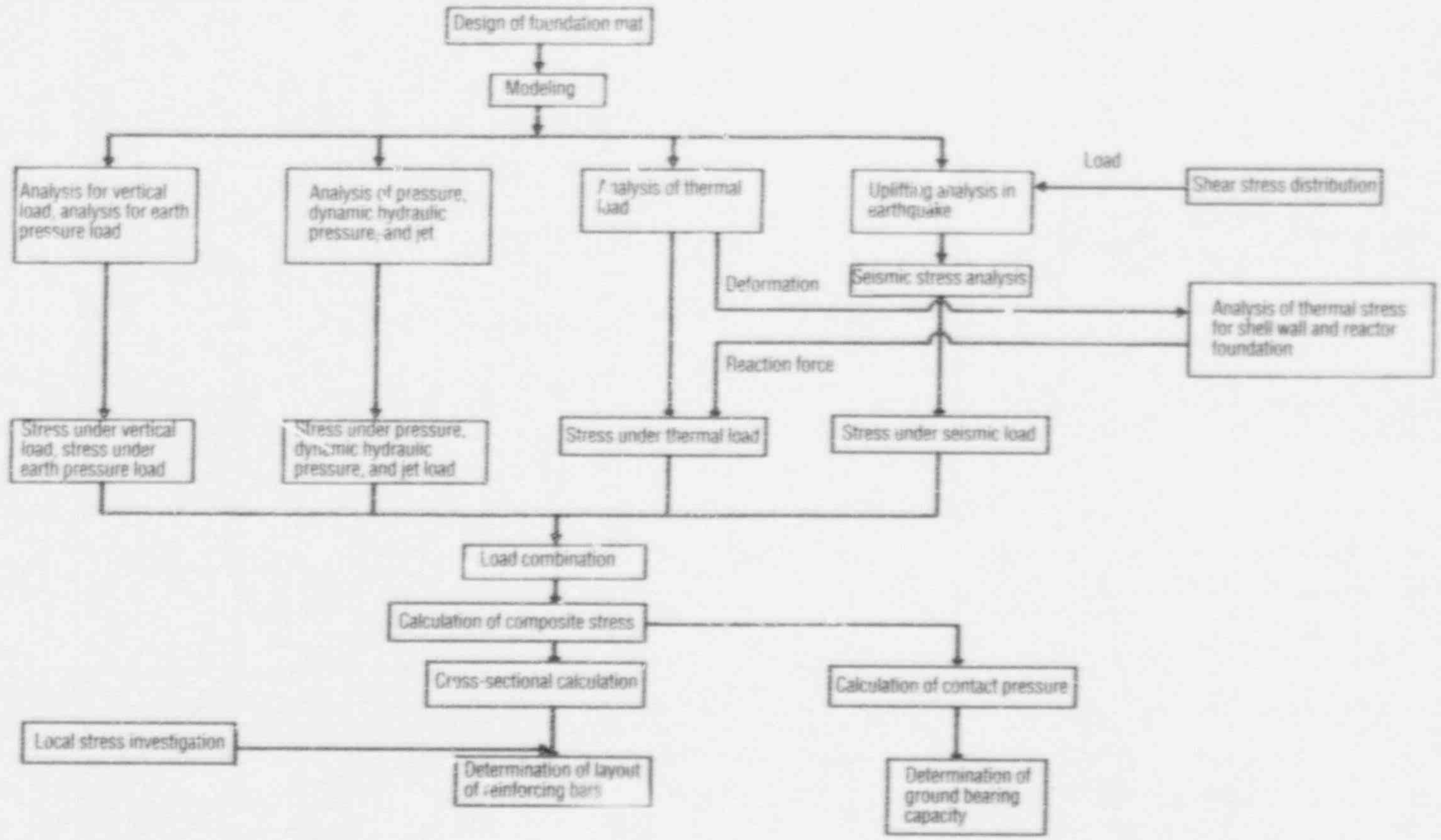


Figure 5.5.1-15. Design flow chart of the foundation mat.

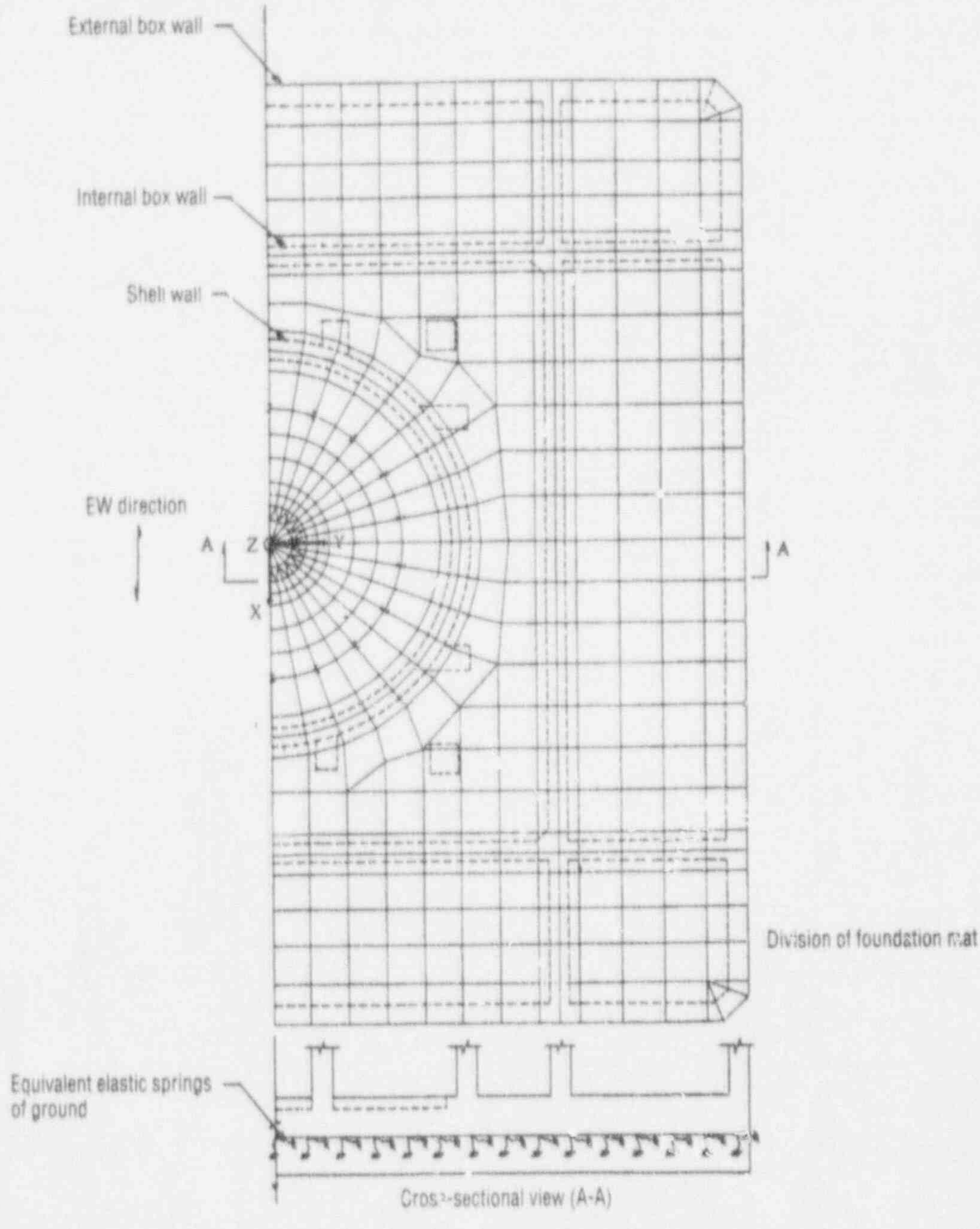


Figure 5.5.1-16. Analytical model and coordinate system of the foundation mat.

By accounting for the stiffening effects of the shear walls rising from the foundation mat (external box wall, internal box wall, and shell wall), the "flat-plane assumptionn" is made for this portion. The supporting ground is represented by the equivalent elastic spring in the model. However, it is presumed that no tensile force acts on the spring. In addition, the thermal stress used in the cross-sectional design is the reduced value to account for the flexural cracks of the concrete.

(b) Cross-sectional design of the foundation mat

Based on the axial stress (membrane stress), flexural moment, and out-of-plane shear stress of the foundation mat determined by the analysis, for the various portions of the foundation mat, the amount of reinforcing bars is determined by regarding the cross-sections in X and Y-directions as columns, and based on the "RC Standards." In addition, the in-plane shear stress is considered in calculating the required number of reinforcing bars. For the out-of-plane shear stress, the increase in the allowable stress due to the shear span ratio (M/Qd, where M is the bending moment, Q is the shear stress, and d is the effective diameter) is also considered.

For the various portions, the cross-sectional calculation is performed for all of the load cases. Among these, the layout of the reinforcing bars is determined for the largest required number of reinforcing bars. As the function-maintenance ability of the load combination is investigated, the cross-sectional calculation for the cases listed as (5) and (6) in Table 5.5.1-6 is performed based on the ultimate strength design. Figure 5.5.1-17 shows the general features of the layout of reinforcing bars of the foundation mat obtained from the results of the cross-sectional calculation.

(c) Contact pressure determined on the ground

Table 5.5.1-7 lists the highest contact pressure obtained from the FEM analysis of the foundation mat. As can be seen from this table, the contact pressure on the ground has a sufficient margin with respect to the allowable bearing capacity.

c. Shell wall

(a) General features of analysis

The shell wall is the primary shielding wall of the reactor. It is set on the peripheral side of the reactor containment vessel. It is a reinforced concrete structure with a truncated cone-shaped upper portion and a cylindrical lower portion. As a structure, it acts as an shear wall in bearing the seismic force together with the outer wall of the building. In addition, it also bears the load coming from the upper portion, the stress transmitted from the frame structural portion, and the thermal stress due to the temperature gradient in the wall's thickness direction caused due to the increase in temperature within the reactor containment vessel. Figure 5.5.1-18 shows the design flow chart of the shell wall.

In the structural analysis, the model of the shell wall is made by taking it as a collection of plate elements, and an elastic analysis is performed by FEM. The divided elements have a quadrangle shape with each element made of homogeneous isotropic material. For each element, the bending and membrane stress of the plate are taken into consideration at the same time. For the bending behavior of the plate, the effect of the out-of-plane shear deform. [illegible] taken into consideration. The analytical models are shown in Figures 5.5.1-19 and 5.5.1-20. For the sh[illegible], it is assumed that the upper end of the foundation mat is fixed. However, the deformation caused by the temperature of the foundation mat in the thermal load is also taken into consideration. In addition, the restraining effect of the slab is considered by replacing it with a bar elements. Since the structure of the shell wall is almost symmetric, and analysis can be made for the half portion as divided by the EW axis. In addition, the thermal stress used in the load combination is the reduced value in consideration of the flexural cracks of the concrete.

Table 5.5.1-6. Load comb ations (foundation mat).

Load combination		Allowable stress
(1)	D + O	Long-term
(2)	D + O + L*	
(3)	D + O + L	Short-term
(4)	D + O + S_1*	
(5)	D + O + S_2	Investigation of function maintenance
(6)	D + O + L + S_1*	

The combination of (5) and (6) is taken into consideration for the design of the reinforced concrete mat of the bottom portion of the reactor containment vessel.

D: Deal load (self weight and equip nent support load, suppression pool water weight, etc.)

O: Loads in conventional operation (live load applied on the equipment, load caused by air bubble pressure when relief safty valve is activated, etc.)

L*: Internal load in an accident (maximum pressure load when coolant is lost in accident)

L: Lo d in accident (load caused by pressure, temperature, and steam blown down in an accident involving the ss of coolant)

S_1*: Seismic load caused by basic earthquake ground motion S_1 or static seismic force

S_2: Seismic load due to basic earthquak ground motion S_2

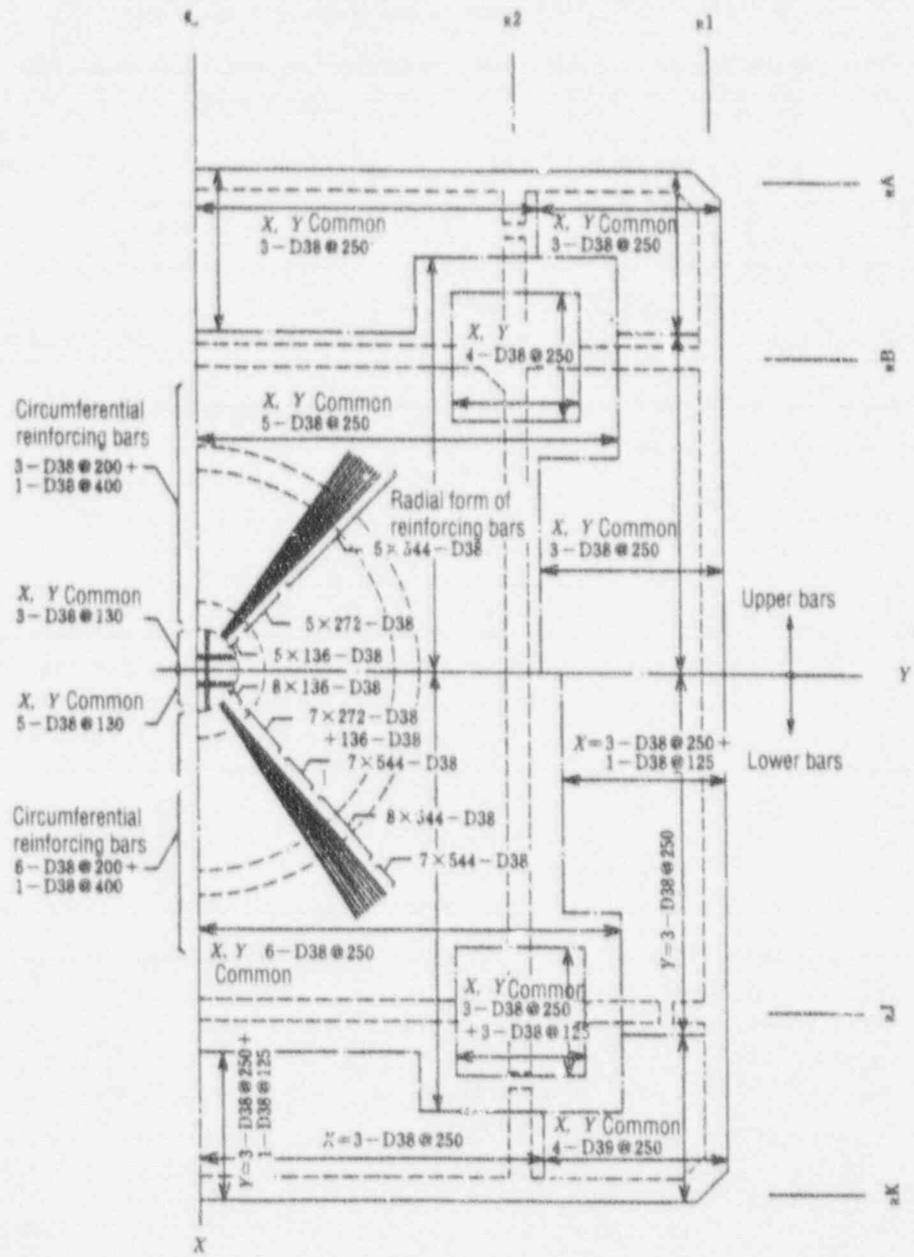


Figure 5.5.1-17. Schematic diagram of layout of reinforcing bars of foundation mat.

Table 5.5.1-7. Maximum contact pressure (units: tf/m^2).

	Load combination (see Table 5.5.1-6)		Maximum contact pressure	Allowable bearing capacity
Long-term	(1)		67	165
Short-term	(4)	Vertical seismic, upward	97	330
		Vertical seismic, downward	107	330

Table 5.5.1-8. Load combination (shell wall).

Load combination		Allowable stress level
(1)	$V + T_0$	Long-term
(2)	$V + T_L$	Short-term
(3)	$V + T_0 + S_1^*$	

V: Stress applied from frame structural portion on various portions of shell wall; a combination of shell wall weight, piping load, equipment load, live load, etc.

T_0: Thermal load in conventional operation

T_L: Thermal load in accident

S_1^*: Seismic load caused by basic earthquake ground motion S_1 or static seismic forces

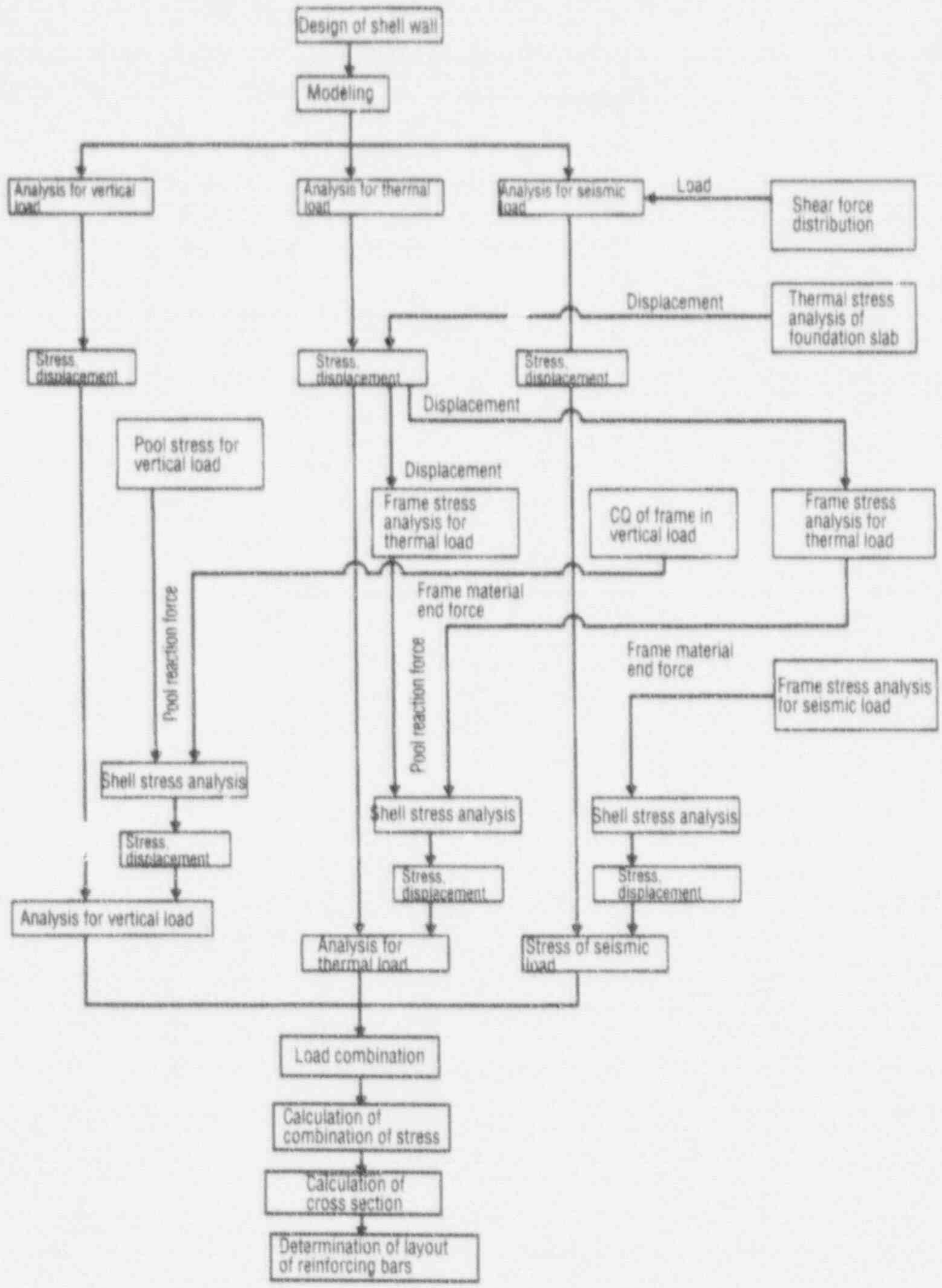


Figure 5.5.1-18. Design flow chart of shell wall.

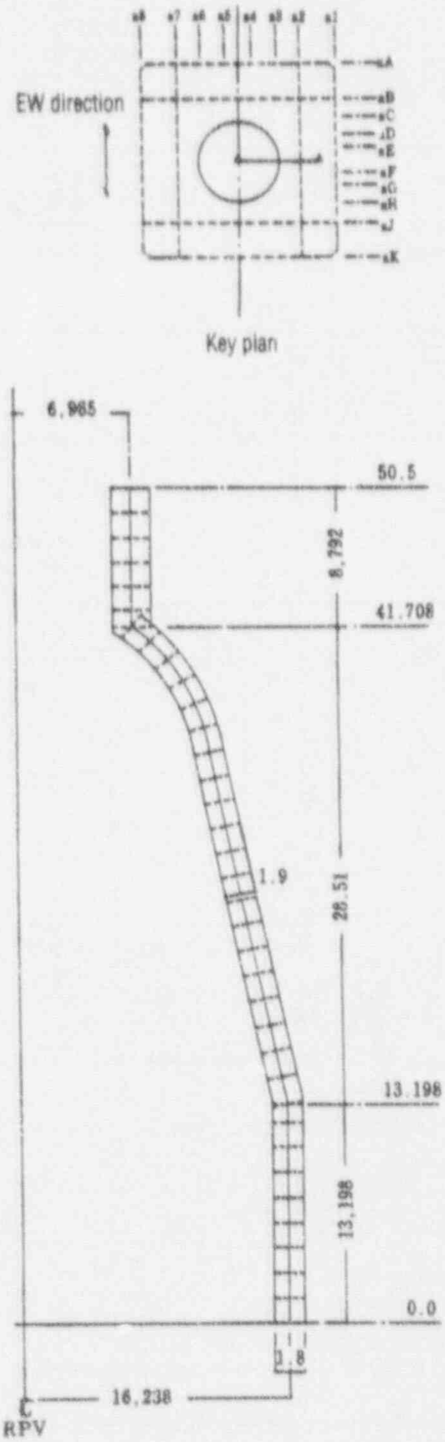


Figure 5.5.1-19. Element division cross-sectional view (units: m).

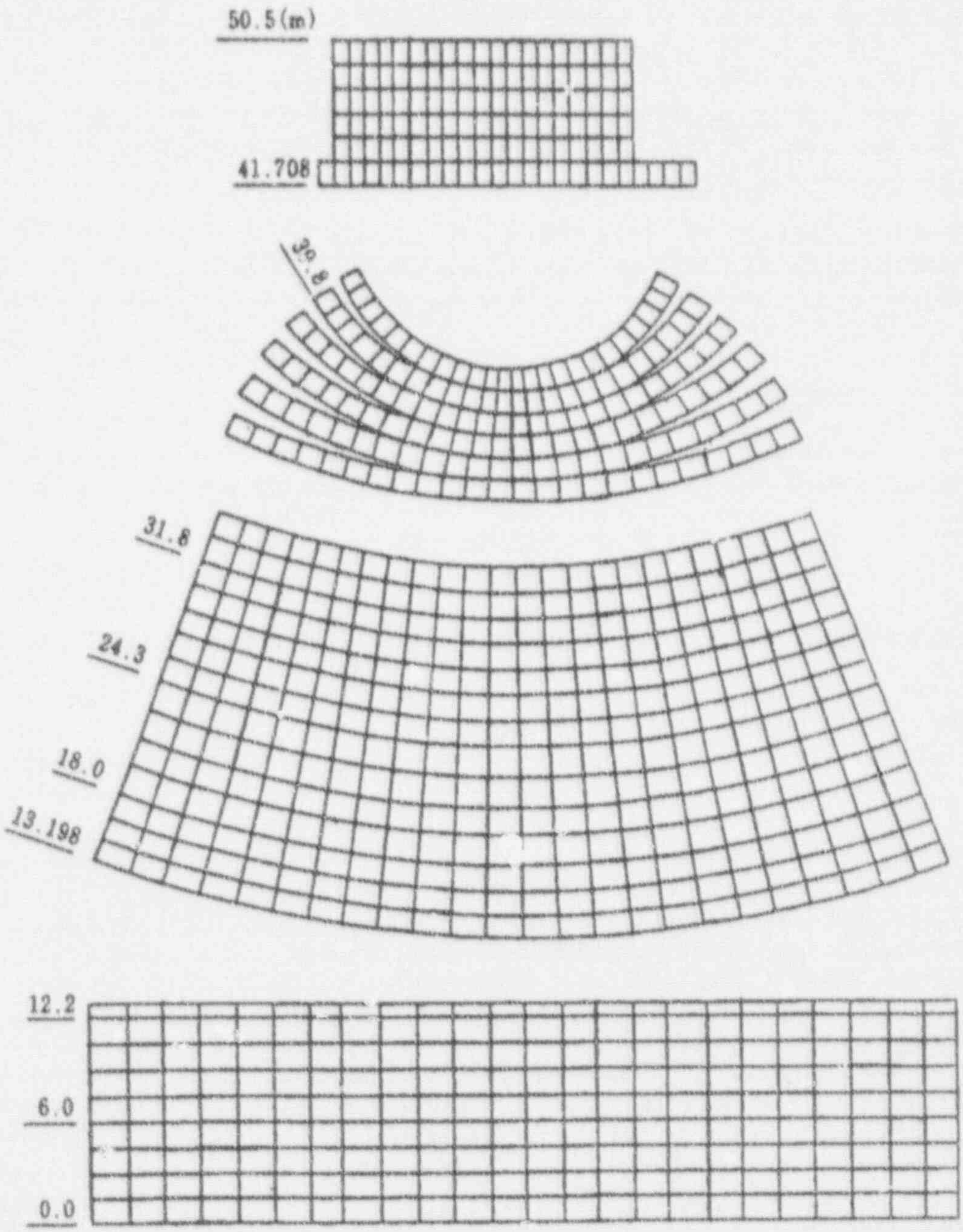


Figure 5.5.1-20. Shell wall element division diagram (units: m).

(b) Design of cross section of shell wall

Based on the axial force (membrane stress), bending moment, and out-of-plane shear stress of the shell wall determined in the analysis, and with the various portions of the shell treated as virtual components in the X and Y directions, the number of reinforcing bars is calculated using the formula for the columns in "RC Standards." In addition, the required number of the reinforcing bars is determined by adding the number of reinforcing bars needed for the in-plane shear stress. In addition, for the out-of-plane shear, the allowable shear stress is increased in consideration of the shear span ratio.

The calculation of the cross section is performed for each portion with respect to all the load cases. The largest amount of reinforcing bars in this calculation determines the layout of the reinforcing bars. Figure 5.5.1-21 shows the schematic of the layout of the reinforcing bars for the shell wall as obtained in the calculation of the cross section.

d. Study on horizontal strength capacity of building

In order to ensure an appropriate safety margin of the horizontal strength capacity (Q_u) with respect to the required horizontal strength capacity (Q_{un}), the horizontal strength capacity is calculated under the following assumptions:

(a) For each story, the horizontal strength capacity is the sum of the horizontal strength capacity of the various shear walls.

(b) The horizontal strength capacity of the shear wall is calculated using the following formula:

$$Q_u = \left(\frac{0.0679 P_{te}^{0.23} (F_c + 180)}{\sqrt{M/(QD) + 0.12}} + 2.7\sqrt{\sigma_{wh} \cdot P_{wh}} + 0.1\sigma_0 \right) \times A_e$$

where P_{te}: equivalent tensile reinforcing bar ratio

σ_{wh}: strength of material of the shear reinforcing bar

P_{wh}: shear reinforcing bar ratio ($P_{wh} \leq 1.2\%$)

σ_0: average axial stress with respect to the total cross-sectional area

A_e: shear effective cross-sectional area of structural wall.

The calculation of the required horizontal strength capacity for each story of the building is performed using the method described in Section 5.3.5(1) "Evaluation of static seismic force." Here, the structural characteristic coefficient (D_s) is taken as 0.5. Table 5.5.1-9 lists the calculated strength capacity of various stories as compared to the required horizontal strength. The horizontal strength capacity has a rather large margin with respect to the required horizontal strength capacity.

5.5.2 PWR (4 LOOP)

As an example of the analysis of the PWR reactor building, we will present the general features of a PWR 4 LOOP plant, from the seismic response analysis to cross-sectional design of the primary portion of the building.

(1) General features of the building

In a reactor building, the superstructures, i.e., reactor containment vessel (PCCV), internal concrete, and containment-vessel peripheral building, are independently installed on a single foundation mat (see Figure 5.5.2-1). In the following, we will present the general futures of these structures:

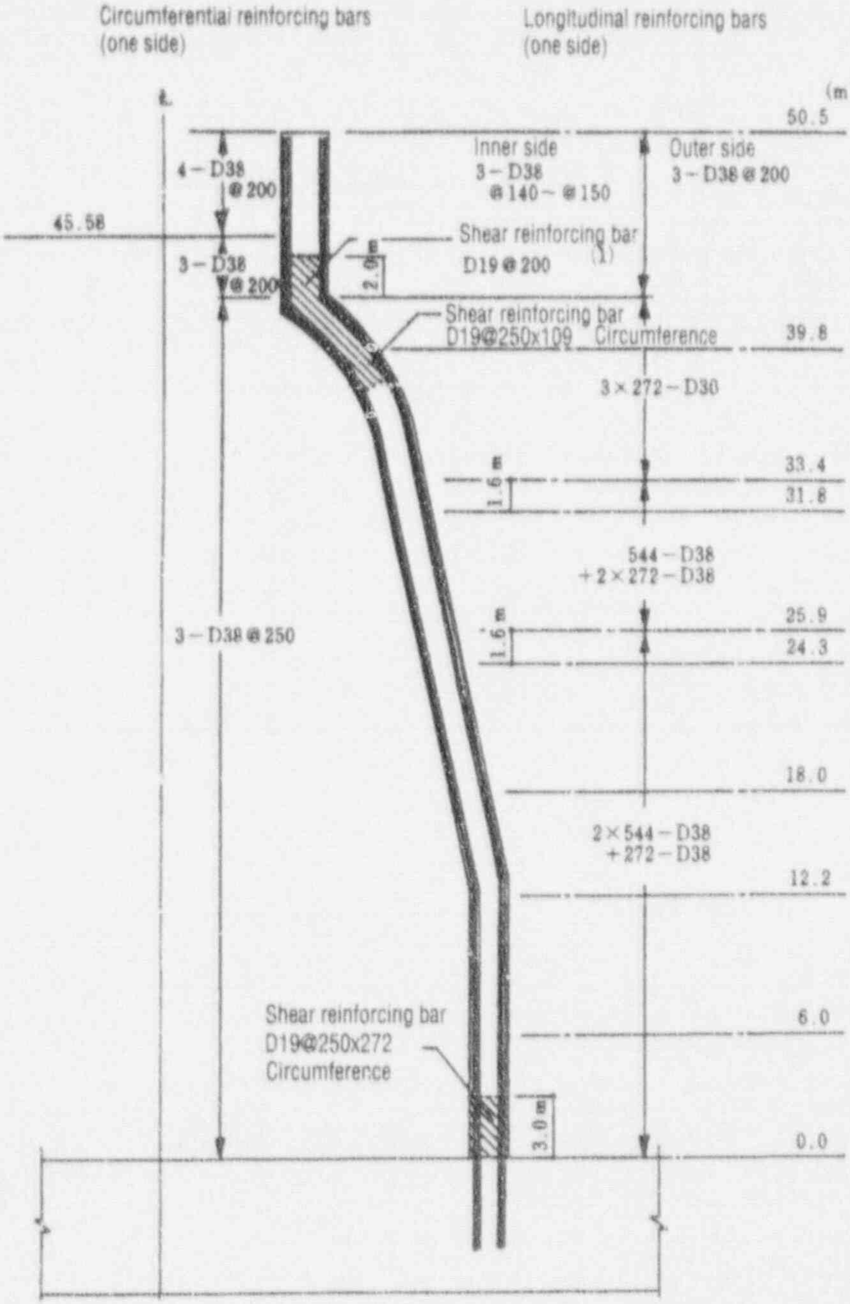


Figure 5.5.1-21. Schematic diagram of layout of reinforcing bars in shell wall.

Table 5.5.1-9. Comparison of horizontal strength capacity.

Floor	Horizontal strength capacity Q_u ($\times 10^3$ tf)	Required horizontal strength capacity Q_{un} ($\times 10^3$ tf)
Crane floor	26.11	3.07
6th floor	28.69	5.94
5th floor	106.05	15.57
4th floor	87.62	31.13
3rd floor	98.74	39.83
2nd floor	134.37	47.14
1st floor	206.83	60.27
Underground 1st floor	237.88	69.99
Underground 2nd floor	237.18	77.72

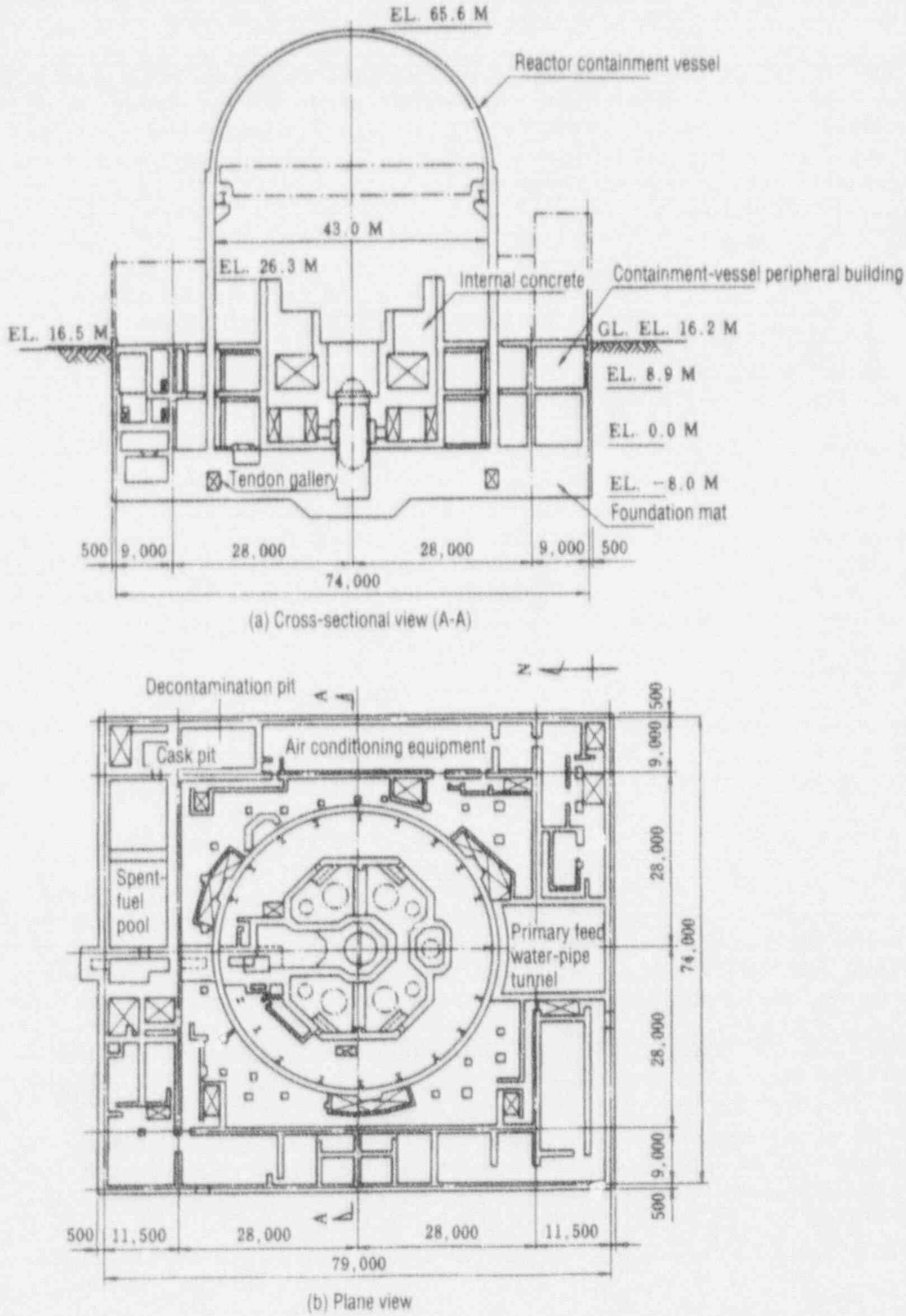


Figure 5.5.2-1. Reactor building of PWR 4 LOOP (1.10 million kW class).

a. PCCV

PCCV is a reactor containment vessel made of prestressed concrete with an inner diameter of 43 m, an internal height of about 64 m, and attached with a semispherical dome. The wall thickness is 1.3 m at the cylindrical portion and 1.1 m at the dome portion. For PCCV, using a prestressing system, the concrete portion is always kept in a membrane-compression stressed state. In addition, in order to maintain the sealing property of the containment vessel, a 6.4-mm-thick steel liner is applied.

b. Internal concrete

The internal concrete is installed within PCCV. It consists primarily of a primary shielding wall around the reactor at the center of the reactor building and a secondary shielding wall that forms the steam-generating chamber and pressurizer chamber; it is made of a reinforced concrete wall and contains the major equipment of the primary cooling facility. On the periphery of the secondary shielding wall, reinforced concrete floor plates (EL 8.9 m and El 16.5 m) are installed and are supported by a concrete wall and steel frame arranged on its periphery.

c. Containment-vessel peripheral building

The peripheral building is a 2-story building which has nearly the same shape as that of the foundation mat, i.e., nearly square, and it made of a reinforced concrete structure (partially made of a steel frame). Since PCCV is arranged at the central portion, for the floor plates EL 8.9 m and EL 16.5 m, a circular gap is formed on the periphery of PCCV, so that PCCV is structurally isolated. In the building's general portion, shear walls with a thickness of 1.0-1.5 m are almost symmetrically arranged. A portion of the wall forms the spent-fuel pool wall.

d. Foundation mat

The foundation mat is a reinforced concrete structure measuring 80 m × 75 m, with a thickness of 8.0 m. It supports PCCV, internal concrete, and the containment-vessel peripheral building; in addition it forms the bottom portion of the containment vessel.

(2) Analytical conditions

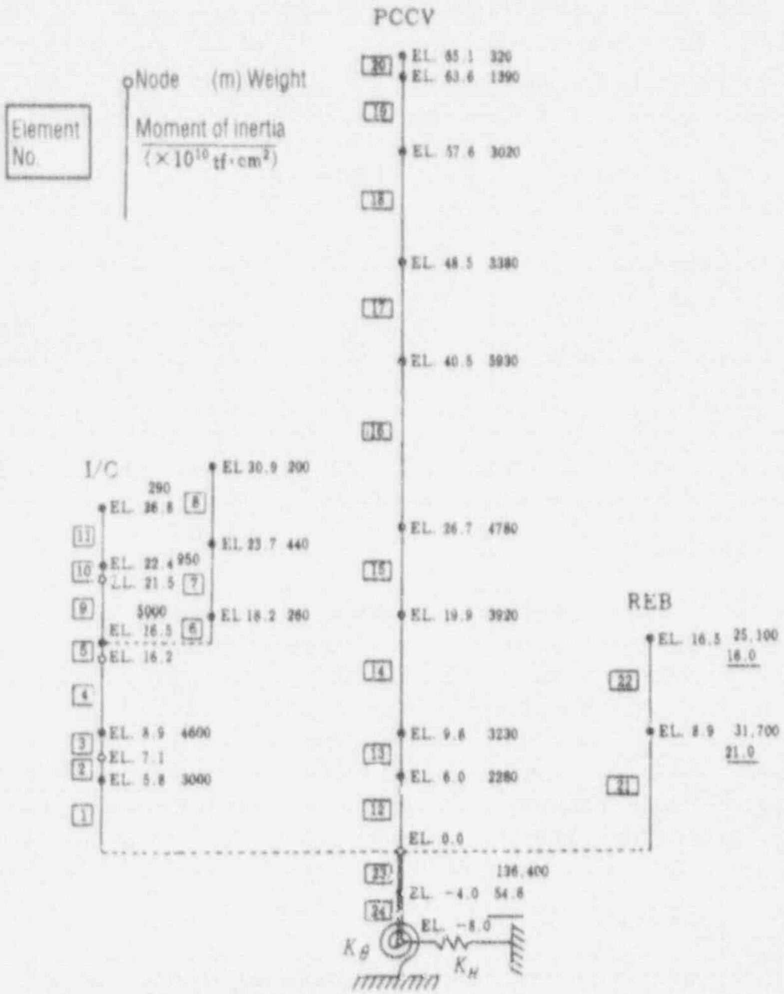
Since the reactor building contains the facilities of class A or class A_S, a dynamic analysis should be conducted against earthquake S_1 and earthquake S_2 in consideration of the site conditions. The seismic force for the reaction building is either the dynamic seismic force obtained in an S_1 earthquake response analysis or the static seismic force calculated otherwise, whichever is larger. It is then combined with the vertical seismic force determined by applying the vertical seismic coefficient.

The cross sections of the building are then designed on the basis of the allowable stress defined in the Construction Standards, etc., with respect to the stress determined from a combination of seismic load, the long-term load, and the accident load. The aseismic safety of the building is evaluated with respect to the S_2 earthquake by performing the seismic response analysis in consideration of the nonlinearity of the various portions of the building and ground. Here, the shear wave velocity (V_S) of the ground supporting the building is 1000 m/s. In addition, the basic earthquake ground motion listed in Table 5.5.1-1 is used as the input earthquake motion.

(3) Seismic response analysis

a. Analysis model

As shown in Figure 5.5.2-2, the sway/rocking model is usually used as the model for the seismic response analysis of the building. In this model, for the superstructure, the reactor containment vessel (PCCV), internal concrete (I/C), and containment-vessel peripheral building (REB) are replaced by independent discrete mass system models, respectively, forming a bending-shear type of a multicantilever model integrated at the base portion. Figure 5.5.2-1 shows an analytical example when the seismic force acts in the EW direction.



Component	E (tf/cm^2)	G (tf/cm^2)	F_C (kgf/cm^2)
I/C	230	98	240
PCCV	304	130	420
REB	230	98	240
Base Mat	230	98	240

Figure 5.5.2-2. Model for response analysis of building.

As far as the stiffness is concerned, the shape factor of the shear cross-sectional area of a cylindrical wall with respect to the total cross section is taken as 2.0; the shape factor of the shear cross-sectional area of the box wall with respect to the web wall area is taken as 1.0. The second moment of inertia of the cylindrical wall is calculated as effective over the entire cross section; the calculation for the box wall is performed with the effective width of the flange portion taken into consideration. In addition, for the stiffness, the evaluation method using a continuous body with FEM is adopted for the design. Table 5.5.2-4 lists the various stiffness parameters of the building.

The properties of the ground are listed in Table 5.5.2-1. The spring constant is calculated using Barkan's formula under the assumption that the ground is a semi-infinite elastic body. (see Table 5.5.2-2.) As far as damping is concerned, the various data determined for the various building structural types as listed in Table 5.5.2-3 are used, with the data determined from the shear wave velocity (V_S = 1000 m/s) of the ground.

b. Eigenvalue analysis results

The results of the eigenvalue analysis are shown in Table 5.5.2-5 and Figure 5.5.2-3. The fundamental period of the building of the system integrated with the ground is 0.235 sec.

c. S_1 seismic response analysis

Figures 5.5.2-4 (a)-(d) show the maximum response values obtained as a result of seismic response analysis. The maximum response acceleration of the foundation position and the PCCV top portion are about 340 Gal and about 1700 Gal (the acceleration magnification factor of the PCCV top portion to the foundation is about 5) for a long-distance earthquake; they are about 400 Gal and about 2000 Gal (the acceleration magnification factor is also about 5) for a short-distance earthquake. The maximum response shear force at the PCCV foot portion is about 28000 t (story shear coefficient: 0.99) for a long-distance earthquake and about 26500 t (story shear coefficient: 0.94) for a short-distance earthquake. The average shear stress level is about 31 kg/cm^2 for the long-distance earthquake. In addition, the figure also shows the shear stress and bending moment due to the static seismic force.

d. S_2 seismic response analysis

(a) Determination of restoring force characteristics for various portions

(i) The bending moment and shear stress of the flexural shear part are determined from the skeleton curve and hysteresis loops described in the EPJR scheme as pointed out in Section 5.2.5(2) "Restoring force characteristics of structure." Table 5.5.2-6 lists the various parameters of the skeleton curve for the various portions of the building.

(ii) Rotational spring of ground

For the rotational spring of the ground, the geometric nonlinearity due to foundation uplifting is taken into consideration for the formation of the moment vs. rotational angle (M - θ) relation (see Figure 5.2.5-7) of the trilinear-type foundation uplifting phenomenon as described in Section 5.2.5(3) "Restoring force characteristics of ground."

(b) S_2 seismic response analysis results

Figures 5.5.2-5 (a)-(d) show the maximum response values of the building due to an S_2 earthquake. Among the two seismic waves, for the long-distance earthquake that gives larger responses, the maximum response acceleration is about 630 Gal at the foundation position and about 2800 Gal at the PCCV top position (the acceleration magnification factor is about 4.4). At the PCCV foot portion, the maximum response shear force is about 41000 t (story shear coefficient: 1.45), and the average shear stress level is about 46 kgf/cm^2. This is slightly higher than the first turning point on the $\tau - \gamma$ skeleton curve. Figure 5.5.2-6 shows the results of the response analysis for the lower portion of each building as plotted on the skeleton curve. For all of the other portions of the building, except the PCCV foot portion, it is within the elastic range.

Table 5.5.2-1. Properties of foundation.

Shear wave velocity	(V_S)	1000 m/s
Soil density	(ρ)	2.0 tf/m^3
Poisson's ratio	(ν)	0.40
Shear modulus of elasticity	(G)	2.04×10^5 tf/m^2
Young's modulus	(E)	5.71×10^5 tf/m^2

Table 5.5.2-2. Spring constant of foundation.

Sway spring K_H (tf/cm)	Rocking spring K_θ (tf·cm/rad)
4.17×10^5	4.88×10^{12}

Table 5.5.2-3. Damping constants of various portions.

Building	Reinforced concrete portion	5%
	Prestressed concrete portion	3%
Foundation	Sway spring	20%
	Rocking spring	7.5%

Table 5.5.2-4. Stiffness parameters of building.

Element No.	Shear cross-sectional area (m^2)	Second moment of inertia (m^4)	Element No.	Shear cross-sectional area (m^2)	Second moment of inertia (m^4)
1	147	17,400	13	90	44,420
2	130	15,100	14	90	44,420
3	134	15,200	15	90	44,420
4	102	11,500	16	90	44,420
5	102	11,500	17	83	39,780
6	13	180	18	76	29,240
7	13	180	19	76	13,480
8	13	180	20	44	610
9	28	1,000	21	610	265,600
10	13	36	22	460	230,400
11	13	36	23	5,100	2,810,000
12	90	44,420	24	5,100	2,810,000

Table 5.5.2-5. Natural period and modal damping.

	Natural period (sec)	Modal damping (%)
1st order	0.235	6.05
2nd order	0.148	13.71
3rd order	0.094	9.35
4th order	0.086	5.02
5th order	0.067	3.16
6th order	0.059	5.00
7th order	0.050	5.58
8th order	0.042	5.05
9th order	0.038	3.51
10th order	0.037	4.97

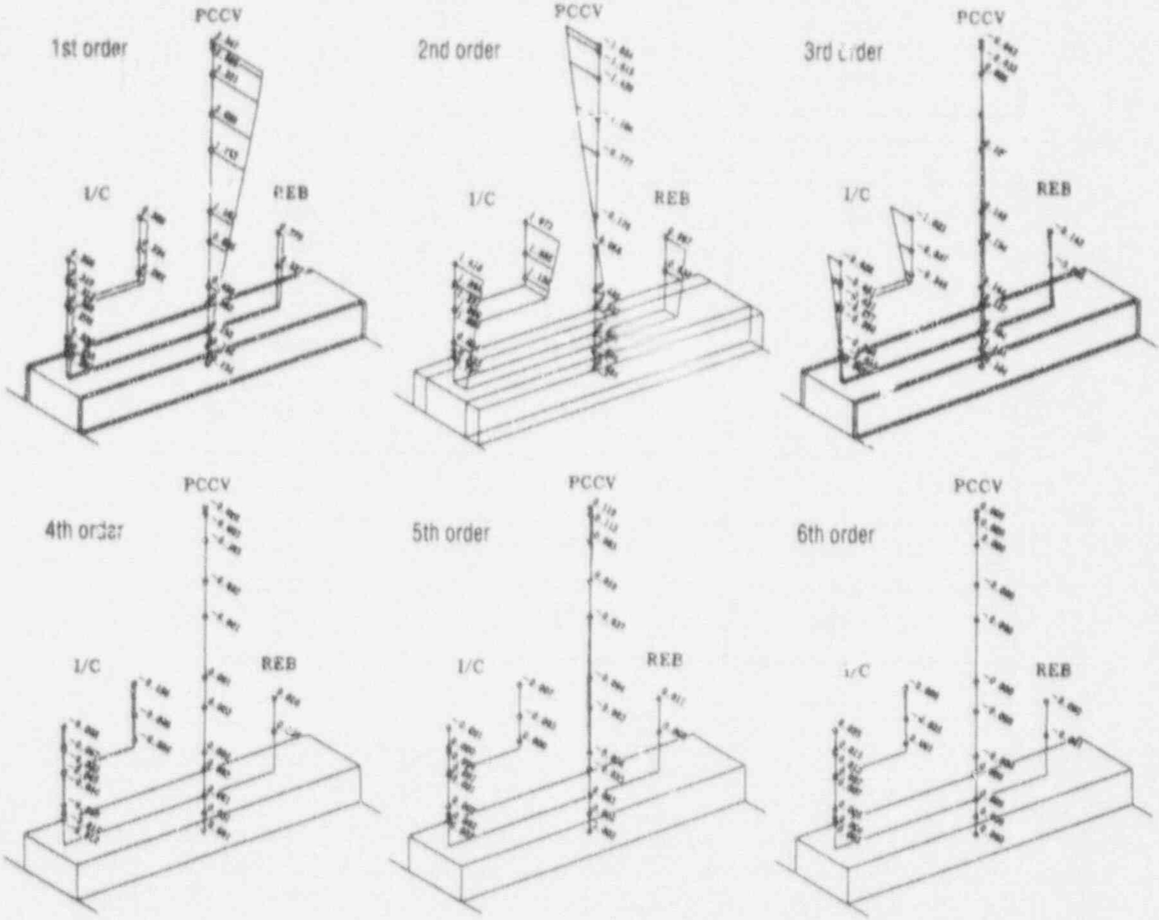


Figure 5.5.2-3. Diagrams of participation factor.

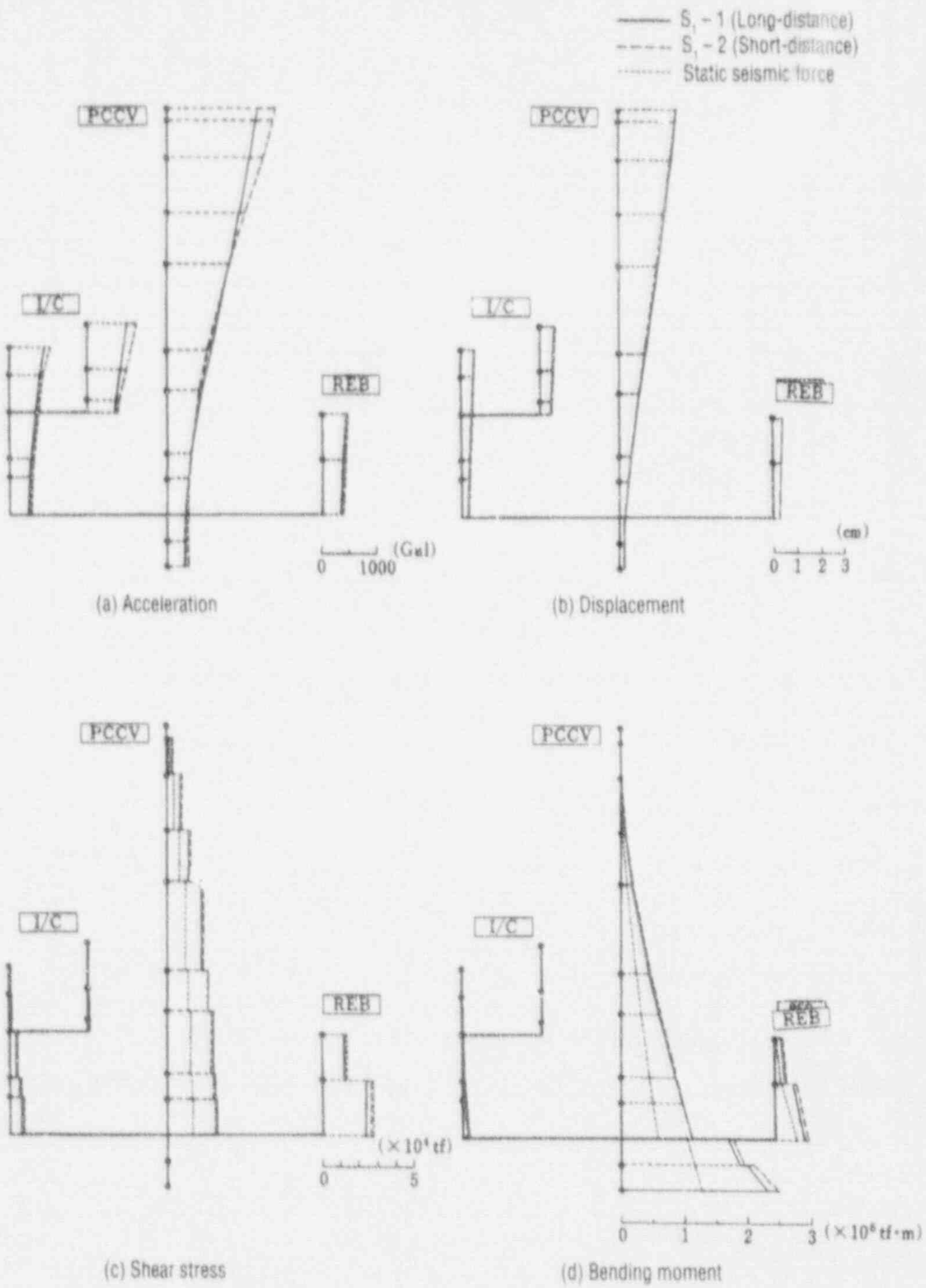


Figure 5.5.2-4. Maximum response value (S_1 earthquake, EW direction).

Table 5.5.2-6. Skeleton curve parameters related to bending and shear.

	Element No.	$\tau - \gamma$ relation						$M - \phi$ relation					
		τ_1 (kgf/cm^2)	γ_1 ($\times 10^{-4}$)	τ_2 (kgf/cm^2)	γ_2 ($\times 10^{-4}$)	τ_3 (kgf/cm^2)	γ_3 ($\times 10^{-3}$)	M_1 ($\times 10^4$ tf·m)	ϕ_1 ($\times 10^{-5}$ rad/m)	M_2 ($\times 10^4$ tf·m)	ϕ_2 ($\times 10^{-5}$ rad/m)	M_3 ($\times 10^4$ tf·m)	ϕ_3 ($\times 10^{-5}$ rad/m)
I/C	1	17.9	1.81	24.2	5.43	66.7	4.0	35.4	0.770	143.	9.42	202.	81.4
	2	17.7	1.79	23.9	5.37	69.7	4.0	29.7	0.759	129.	9.10	178.	160.
	3	17.6	1.78	23.8	5.34	69.7	4.0	29.7	0.754	131.	9.11	182.	160.
	4	17.2	1.74	23.2	5.22	60.7	4.0	20.7	0.712	75.6	8.93	111.	143.
	5	17.2	1.74	23.2	5.22	62.8	4.0	20.7	0.712	75.6	8.93	111.	143.
	6	17.2	1.74	23.2	5.22	46.5	4.0	1.35	2.95	3.31	35.5	4.39	577.
	7	16.7	1.69	22.6	5.07	46.1	4.0	1.29	2.83	3.24	35.3	4.31	590.
	8	15.9	1.61	21.5	4.83	43.7	4.0	1.14	2.61	1.81	33.1	2.44	662.
	9	16.2	1.64	21.9	4.92	41.4	4.0	5.62	2.15	13.5	26.7	18.1	534.
	10	16.7	1.69	22.6	5.07	46.6	4.0	0.446	5.33	0.688	67.7	1.49	1160.
	11	15.8	1.60	21.3	4.80	39.0	4.0	0.422	4.83	0.561	66.6	1.31	1332.
PCCV	12	42.0	3.23	56.7	9.69	92.2	4.0	208.	1.34	454.	6.73	573.	41.6
	13	41.6	3.20	56.2	9.60	92.2	4.0	193.	1.31	360.	6.54	444.	52.6
	14	41.3	3.18	55.8	9.54	92.2	4.0	187.	1.30	326.	6.46	397.	59.1
	15	41.0	3.16	55.4	9.48	92.2	4.0	185.	1.28	323.	6.44	394.	60.1
	16	41.0	3.16	55.4	9.48	92.2	4.0	183.	1.28	307.	6.41	371.	63.8
	17	Taken as elastic (dome portion)											
	18												
	19												
	20												
REB	21	17.9	1.80	24.1	5.40	57.6	4.0	183.	0.278	643.	3.27	960.	45.7
	22	16.8	1.70	22.7	5.10	55.1	4.0	143.	0.252	425.	3.15	666.	55.1

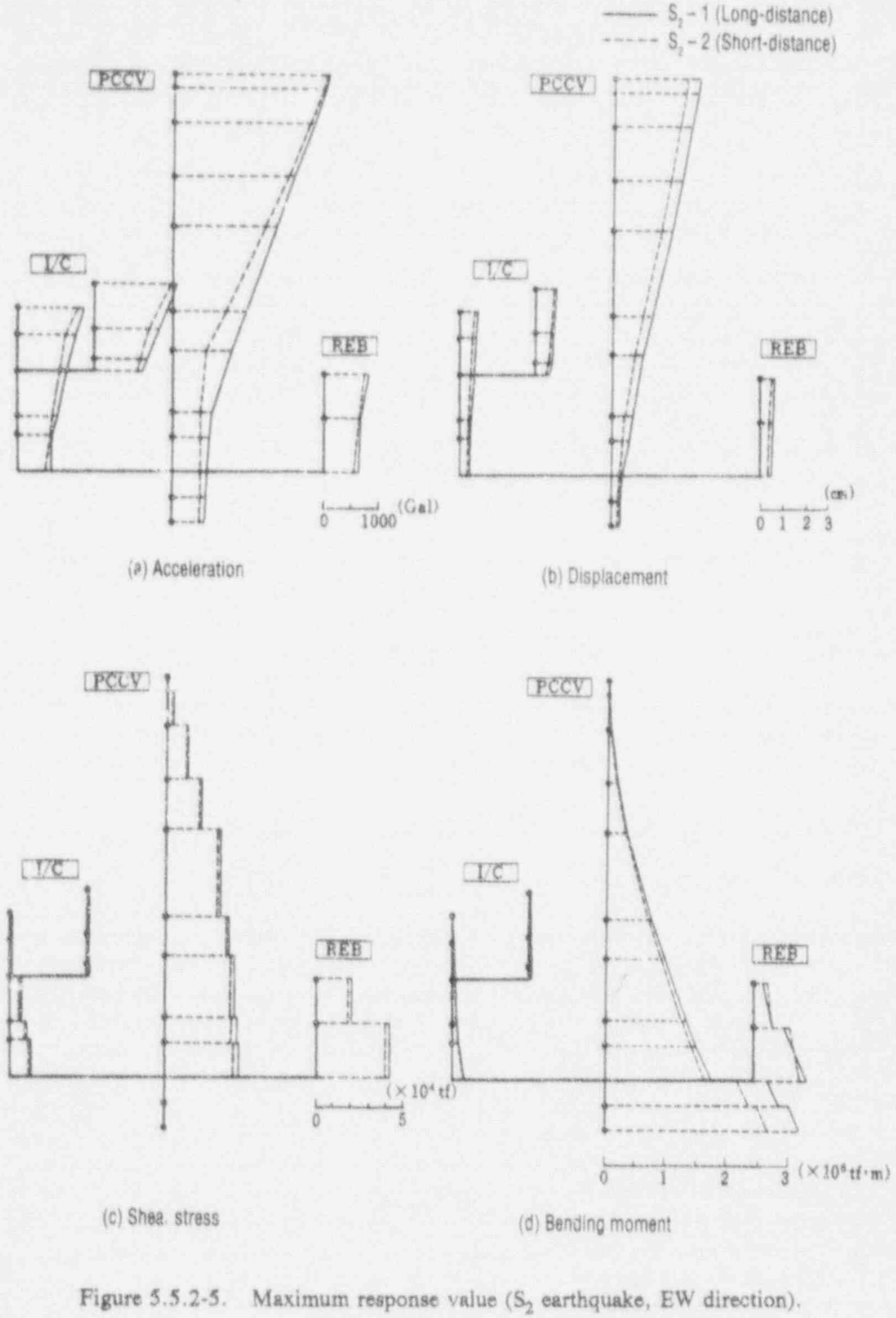


Figure 5.5.2-5. Maximum response value (S_2 earthquake, EW direction).

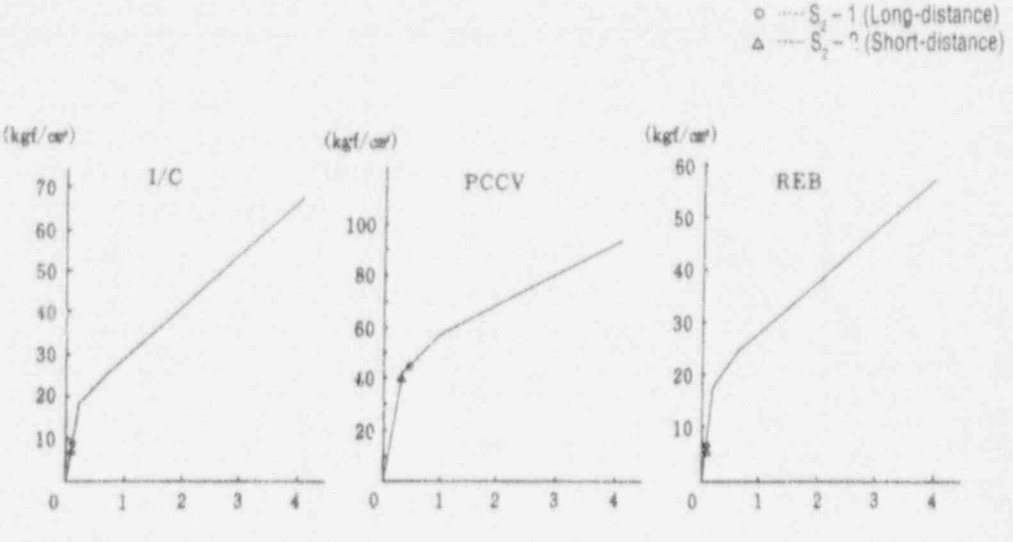


Figure 5.5.2-6. Restoring force characteristics and response values of lower portions of various buildings.

e. Contact rate of foundation

The contact rate of the foundation mat bottom is calculated according to Section 5.2.5(3)b "Evaluation of contact rate," with the results listed in Table 5.5.2-7. In this analysis, the contact rate is 100% in the S_1 earthquake, and no uplifting occurs. In addition, the contact rate is 96% in an S_2 earthquake (long-distance).

(4) Stress analysis and cross-sectional design of primary structural portions

a. Containment-vessel peripheral building

As shown in Figure 5.5.2-1, the containment-vessel peripheral building is built in the periphery of the reactor containment vessel (PCCV). It is a 2-story building made of reinforced concrete (partially made of a steel frame) with a plan size measuring 75 m × 80 m. In the general portion of the building, shear walls with a thickness of 1.0 m to 1.5 m are arranged almost symmetrically, with a portion forming the used-fuel pool. The design flow chart is shown in Figure 5.5.2-7.

Figure 5.5.2-8 shows the finite element model for the stress analysis of the primary walls and primary slabs (EL 16.5 m, El 8.9 m) subjected to the horizontal seismic load. For the analytical model, EL 0.0 m is taken as fixed and the model of the slab and shear wall is formed using the in-plane stress flat-plate elements. The horizontal load is given as the nodal load in proportion to the concrete volume. Table 5.5.2-8 lists the combinations of the loads taken into consideration for the design.

With respect to the stresses in various parts obtained as the result of stress analysis, the cross-sectional calculation of the shear wall is performed using the following methods.

Table 5.5.2-7. Contact rate of foundation.

Direction	Item	Earthquake			
		S_1		S_2	
		Long-distance	Short-distance	Long-distance	Short-distance
EW	Overturning moment (M) ($\times 10^6$ tf·m)	2.32	2.49	3.18	2.63
	Eccentric distance e (m)	9.82	10.5	13.5	11.1
	Eccentricity e/L	0.13	0.14	0.18	0.15
	Contact rate of foundation (%)	100	100	96	100

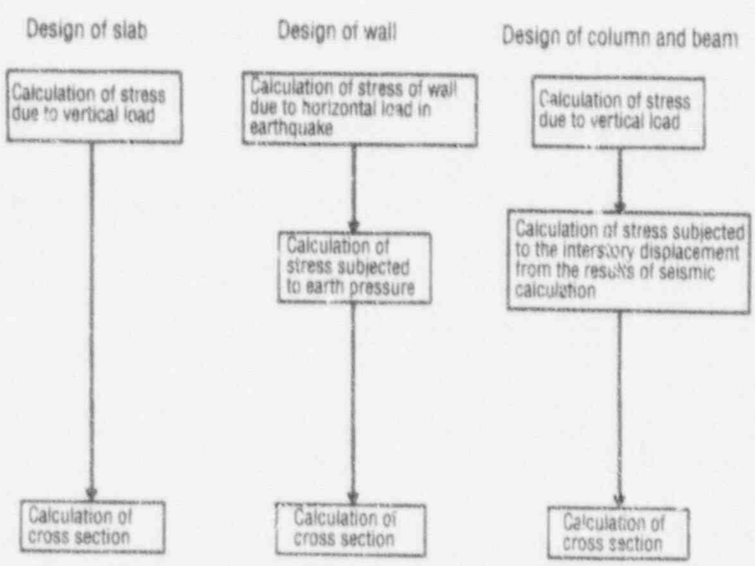


Figure 5.5.2-7. Design flow chart of the containment-vessel peripheral building.

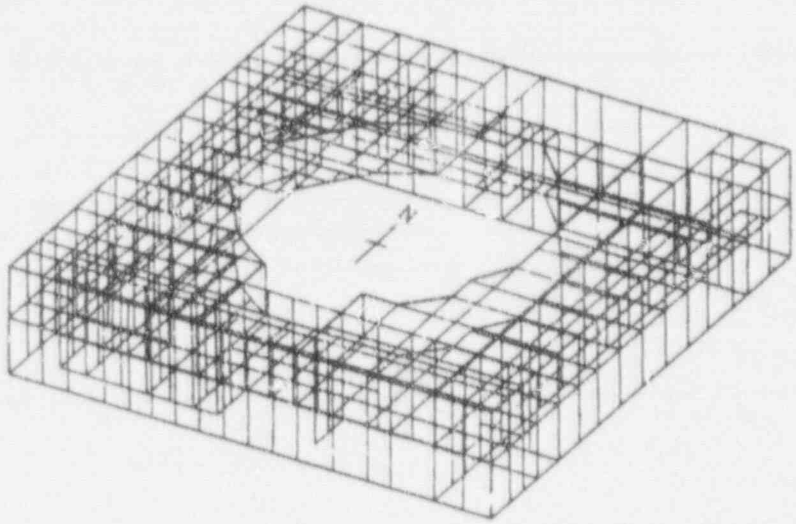


Figure 5.5.2-8. Analytical model diagram of contaiment-vessel peripheral building.

Table 5.5.2-8. List of load combinations.

	Conditions of external force	Conditions of loading
Long-term	Continuous	$G + P + SN_1 + E$
Short-term	Snow deposit	$G + P + SN_2 + E$
	Hurricane	$G + P + SN_3 + W + E$
	Earthquake	$G + P + SN_3 + K_1 + E$

G: Dead load
P: Live load
SN_1, SN_2, SN_3: Snow load
W: Wind load
K_1: Seismic load
E: Earth pressure load

$$\left.\begin{aligned}
A_{tn} &= \frac{N}{2\times f_t} \quad \text{(Number of reinforcing bars required for axial force in earthquake)} \\
A_{tg} &= \frac{Q}{2\times {}_wf_t} \quad \text{(Number of reinforcing bars required for in-plane shear force in earthquake)} \\
A_{tm} &= \frac{M}{f_t\times j} \quad \text{(Number of reinforcing bars required for earth pressure)} \\
A_t &= A_{tn}+A_{tg}+A_{tm}
\end{aligned}\right\} \qquad (5.5.2\text{-}1)$$

where N: axial force
- Q: shear force
- M: bending moment
- f_t: allowable tensile stress of reinforcing bars
- ${}_wf_t$: allowable tensile stress for shear reinforcing bars
- A_t: required number of reinforcing bars
- j: stress center distance of the member.

Calculation of the shear reinforcing bars is performed according to the "RC Standards." Figure 5.5.2-9 shows a schematic of the layout of reinforcing bars derived from the results of a cross-sectional calculation.

b. Foundation mat

The foundation mat in a reactor containment facility supports the upper structures, such as the reactor containment vessel, internal concrete, and containment-vessel peripheral building; in addition, it forms the bottom portion of the containment vessel itself. The foundation mat is a direct foundation made of reinforced concrete, and is directly supported on the hard rock ground.

The plan site at the bottom is 75 m in the EW direction, 80 m in the NS direction, and 8.0 m in thickness. The relation between the building and the foundation mat is illustrated by the cross-sectional view shown in Figure 5.5.2-1, with the PCCV, internal concrete, and containment-vessel peripheral building independently installed on the common foundation mat. Figure 5.5.2-10 shows a flow chart of the design of the foundation mat. Since the foundation mat forms the bottom portion of the containment vessel, the design should meet the requirements on the concrete containment vessel. The design method is described in Section 5.4 "Concrete containment vessel."

Figure 5.5.2-11 shows an analytical model using FEM for the foundation portion. The stress is calculated for a 3-D model with the foundation mat portion made of brick elements and the upper structure made of shell elements. Table 5.5.2-9 lists the load combinations which are taken into consideration in the design of the foundation mat.

As far as the cross-sectional design is concerned, the allowable-stress-level design method is used for load states I-III and the ultimate-strength design method is used for load state IV; the number of reinforcing bars laid out is calculated by dividing the total foundation mat into groups, with each group assumed to be a column acted upon by the unit-width membrane stress and flexural stress with respect to the dominant design stress. Figure 5.5.2-12 shows a schematic of the layout of reinforcing bars in the foundation mat as derived from the results of the cross-sectional calculation.

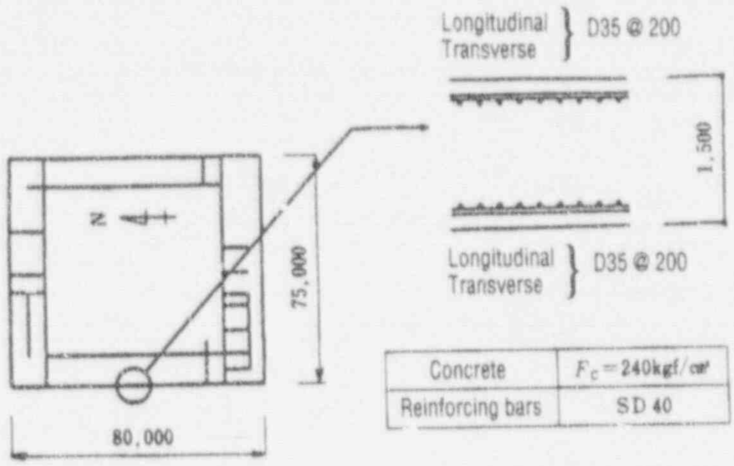


Figure 5.5.2-9. Schematic diagram of layout of reinforcing bars in primary walls of containment-vessel peripheral building.

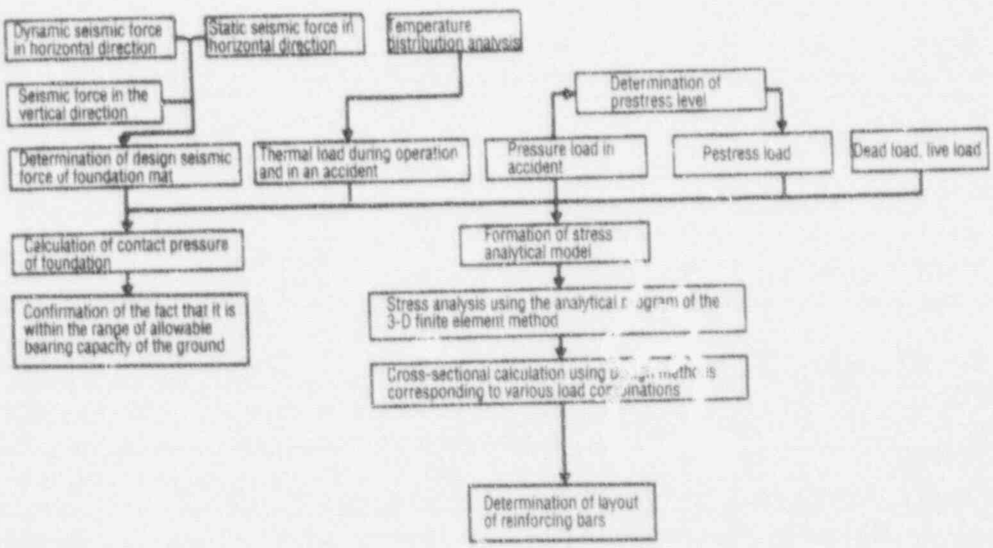


Figure 5.5.2-10. Flow chart of design of foundation mat.

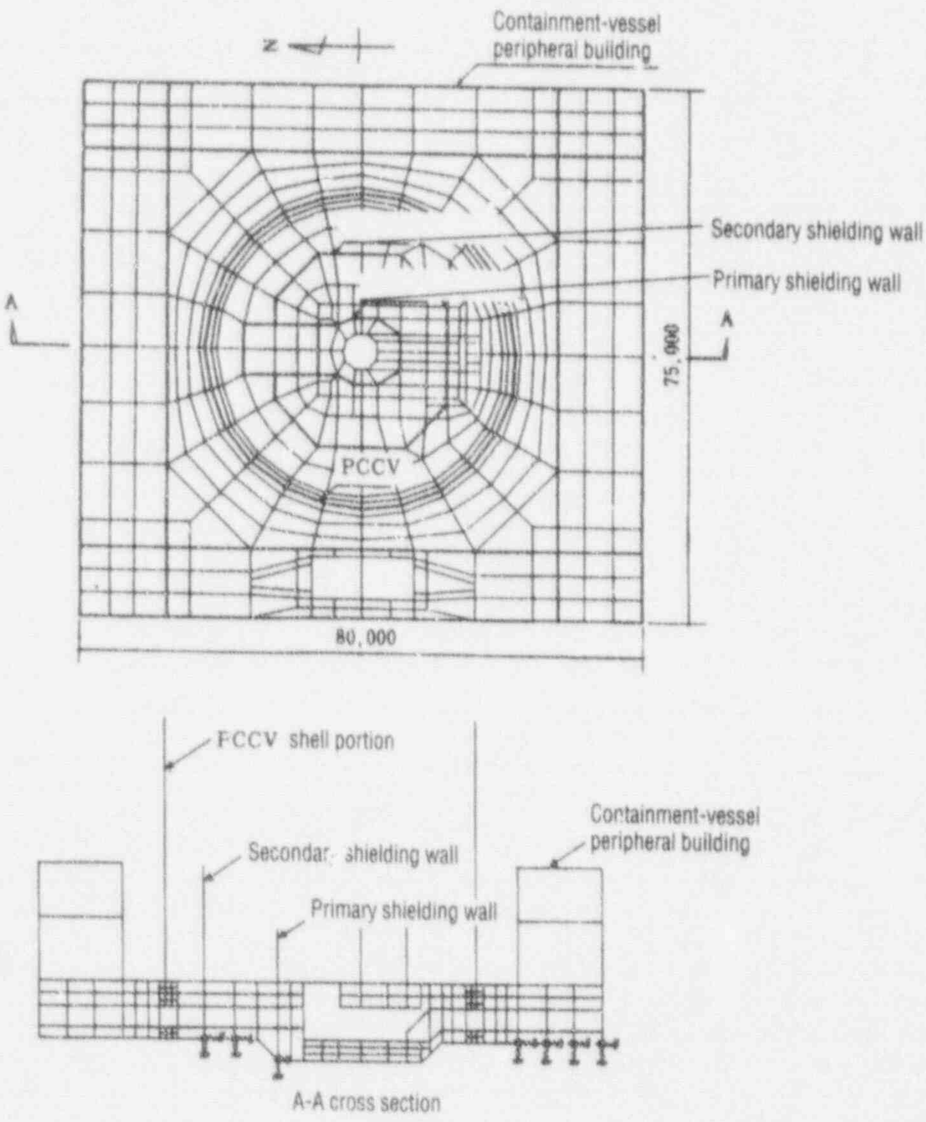


Figure 5.5.2-11. Analytical model of foundation mat.

Table 5.5.2-9. List of load combinations (foundation mat).

Load state	No.	Name	Load combination	
			Stress state 1	Stress state 2
I	1	During normal operation	$D+L+Fe$	$D+L+Fe+T_{1W}$
II	2	During test	$D+L+Fe+P_0$	—
III	3	During L-accident	$D+L+Fe+P_2(1)$	$D+L+Fe+P_2(1)+T_{2W}(1)$
	4		$D+L+Fe+P_2(24)$	$D+L+Fe+P_2(24)+T_{2W}(24)$
	5	During S_1-earthquake	$D+L+Fe+K_1SNU$	$D+L+Fe+K_1SNU+T_{1W}$
	6		$D+L+Fe+K_1NSU$	$D+L+Fe+K_1NSU+T_{1W}$
	7		$D+L+Fe+K_1NSU$	$D+L+Fe+K_1EWU+T_{1W}$
	8		$D+L+Fe+K_1EWD$	$D+L+Fe+K_1EWD+T_{1W}$
	9	L-accident + S_1 earthquake	$D+L+Fe+P_2(24)+K_1EWU$	$D+L+Fe+P_2(24)+K_1EWU+T_{2W}(24)$
IV	10	During L-accident	$D+L+Fe+1.5P_2(1)$	
	11	L-accident + S_1 earthquake	$D+L+Fe+P_2(1)+K_1EWU$	
	12	During S_2 accident	$D+L+Fe+K_2EWU$	

Stress state 1 refers to the stress state due to loads other than the thermal load.
Stress state 2 refers to the stress state due to all loads, including the thermal load.
D: dead load; L: live load; Fe: prestress load.

Seismic load
K_1SNU: S_1 earthquake, horizontal SN direction + vertical upward direction
K_1NSU: S_1 earthquake, horizontal NS direction + vertical upward direction
K_1EWU: S_1 earthquake, horizontal EW direction + vertical upward direction
K_1EWD: S_1 earthquake, horizontal EW direction + vertical downward direction
K_2EWU: S_2 earthquake, horizontal EW direction + vertical upward direction

Thermal load
T_{1W}: During normal operation
$T_{2W}(1)$: In accident (1 h later)
$T_{2W}(24)$: In accident (24 h later)

Pressure load
P_0: During test
$P_2(1)$: In accident (1 h later)
$P_2(24)$: In accident (24 h later)

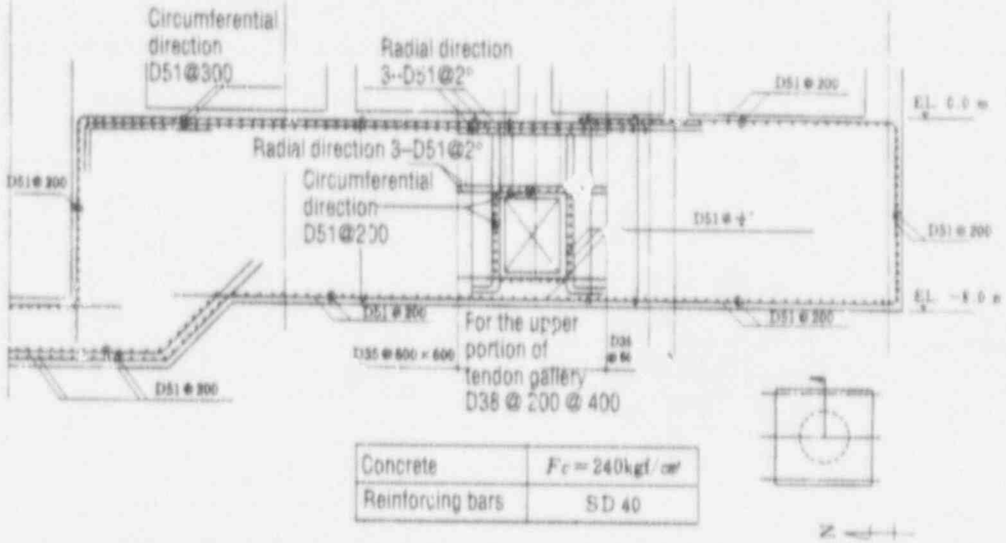


Figure 5.5.2-12. Schematic diagram of layout of reinforcing bars in foundation mat.

Table 5.5.2-10. Maximum contact pressure (units: tf/m^2).

	Load combination	Maximum contact pressure	Allowable bearing force
During normal operation	D+L+Fe	43	400
In S_1 earthquake	$D+L+Fe+K_1EWD$	89	800
In S_2 earthquake	$D+L+Fe+K_2EWU$	71	800

Table 5.5.2-10 lists the results of the maximum contact pressure derived from the FEM analysis of the foundation mat. As can be seen from this table, the contact pressure on the foundation has a sufficiently large margin with respect to the allowable bearing capacity.

c. Prestressed concrete containment vessel (PCCV)

The PCCV is made of an upper shell of a prestressed concrete portion, a bottom foundation of a reinforced concrete portion, and a steel liner portion which is applied on the inner surface of the concrete portion to ensure the leak-proof property. The upper shell portion of PCCV consists of a domed portion with a shell thickness of 1.1 m and a lower cylindrical portion with an inner diameter of 43 m and a thickness of 1.3 m; the inner height is about 64 m. In the concrete structural portion that provides the structural strength of the shell portion, a prestress is applied to reduce the membrane tensile stress.

The prestress force of the concrete portion is applied by stretching the inverted-U tendons and hoop tendons. The inverted-U tendons are arranged in grid form as they are projected from the upper dome portion, with the two ends of each tendon anchored on the upper portion of a tendon gallery mounted within the foundation mat. In addition, the hoop tendons are arranged in hoop form with a central angle of 240°. The two ends of each tendon are anchored on buttresses which are set for each 120°. The arrangement of the tendons is illustrated in Figure 5.5.2-13.

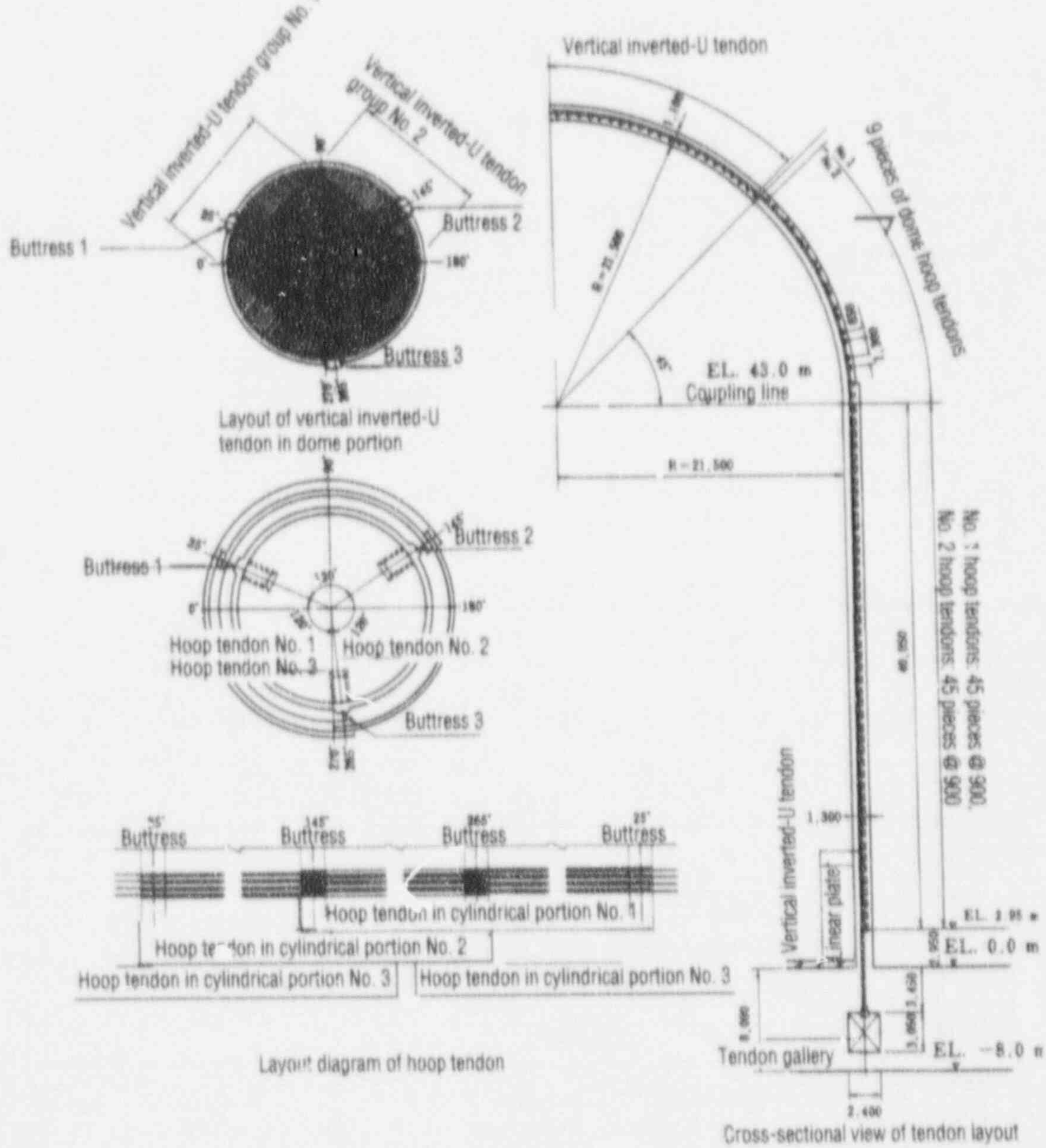


Figure 5.5.2-13. Layout diagram of tendons.

The design flow chart of PCCV is shown in Figure 5.5.2-14, with the design performed to meet the requirements on the concrete containment vessel. For details of the design, please see Section 5.4 "Concrete containment vessel."

Among the major external forces acting on the shell portion, the prestressed loads due to vertical inverted-U tendons and hoop tendons in the domed portion is considered to be a nodal load, and the pressure load for the radial direction at the cylindrical portion is considered as a pressure. For the thermal load, the results of the temperature distribution analysis is used. For the seismic load, the nodal load is determined to ensure the design shear is carried by various portions of the shell. In the 3-D FEM analysis model for stress analysis, the conventional portion of the shell is represented by shell elements and the buttress portion by beam elements. Figure 5.5.2-15 shows the analytical model. As far as the foot portion is concerned, it is analyzed using another model containing the foundation mat. In this way, its effect is taken into consideration. Table 5.5.2-11 lists the load combinations considered in the design of PCCV.

For load states I, II, and III, the allowable stress design is performed with respect to the stress calculated by the elastic analysis. Since the thermal stress due to the temperature load is a type of self-restricting stress, for the load combination containing the thermal stress, the cross-sectional calculation is performed by accounting for the reduction in the stiffness due to cracks. For load state IV, calculation of the stress is performed by elastic analysis. When the calculated stress is rather large, the cross section should be determined using the strength design method based on the ultimate strength capacity of the cross section to ensure the required safety margin in terms of the stress.

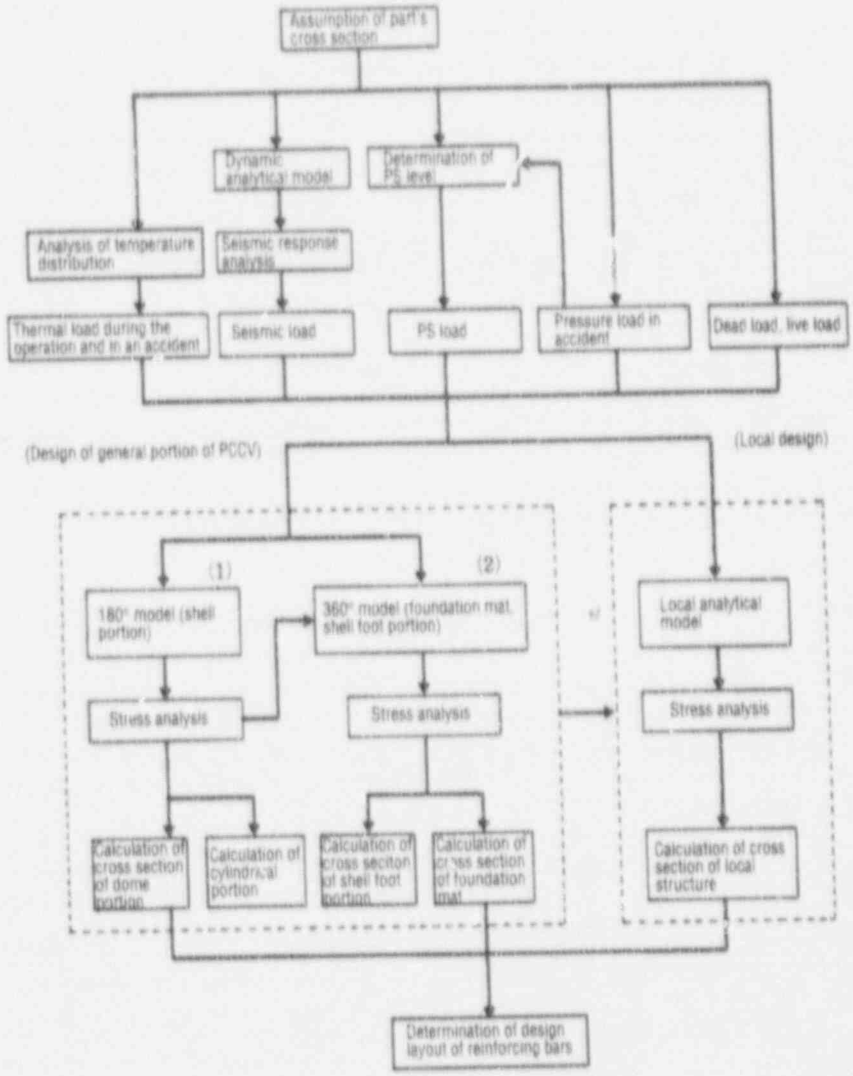
Calculation of the reinforcing bars in the longitudinal direction (meridian) and transverse direction (circumferential) is performed by taking it as an assumed column acted upon by the unit-width membrane stress and flexural stress. Figure 5.5.2-16 shows a schematic diagram of the layout of reinforcing bars of PCCV obtained in the calculation of the cross section. Figure 5.5.2-17 shows the allowable axial force-bending moment interaction curve which indicates the yield strength of a typical part of the cylindrical portion. Such a interaction curve is formed for each cross section, with the combination of all of the loads plotted on the diagram. It is confirmed that the part has a sufficiently high strength.

d. Spent-fuel pit

The spent-fuel pit is located in the lower portion of the fuel handling compartment of the containment-vessel peripheral building. Its primary structure is a reinforced concrete wall structure. As shown by the flow chart for the design in Figure 5.5.2-18, the design of the structural body include the horizontal load in an earthquake, temperature load in an accident, and a conventional load.

Figure 5.5.2-19 shows the FEM analysis model for the stress analysis. For the analytical model, EL +0.0 m is taken as fixed, and the model of the shear wall is formed using in-plane stress flat-plate elements. For the load in the horizontal direction, the shear stress in an earthquake is taken as the nodal load proportional to the concrete volume. Table 5.5.2-12 lists the load combinations considered in the design.

With respect to the stresses in the various portions obtained in the stress analysis, the longitudinal reinforcing bars in walls are determined by regarding them as a column. The required number of the reinforcing bars is determined as the sum of the required number of reinforcing bars for the vertical axial force and out-of-plane bending moment, and the required number of reinforcing bars for the in-plane shear stress. On the other hand, the transverse reinforcing bars in walls are determined by treating them as a beam element; the required number of reinforcing bars for the out-of-plane bending moment is then determined. When the direction of the horizontal seismic force is in agreement with the direction of the wall, the required number of reinforcing bars for the in-plane shear stress are added to give the required amount of the reinforcing bars.



Note (1): 180° model: model for stress analysis of shell portion
Note (2): 360° model: model for stress analysis of foundation mat containing shell foot portion

Figure 5.5.2-14. Flow chart of design of PCCV.

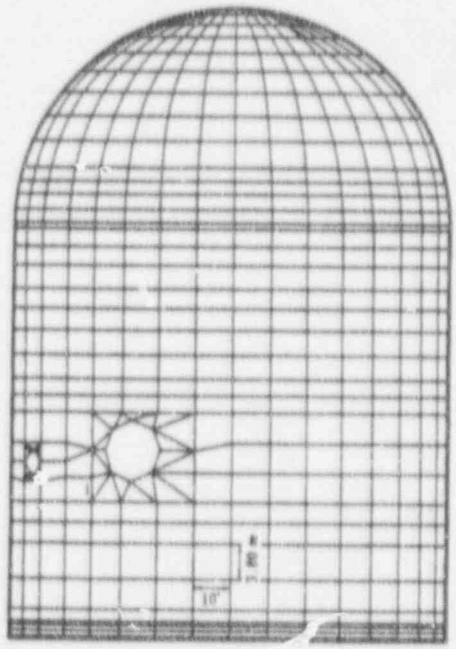


Figure 5.5.2-15. Model of PCCV analysis.

Table 5.5.2-11. List of load combination (PCCV).

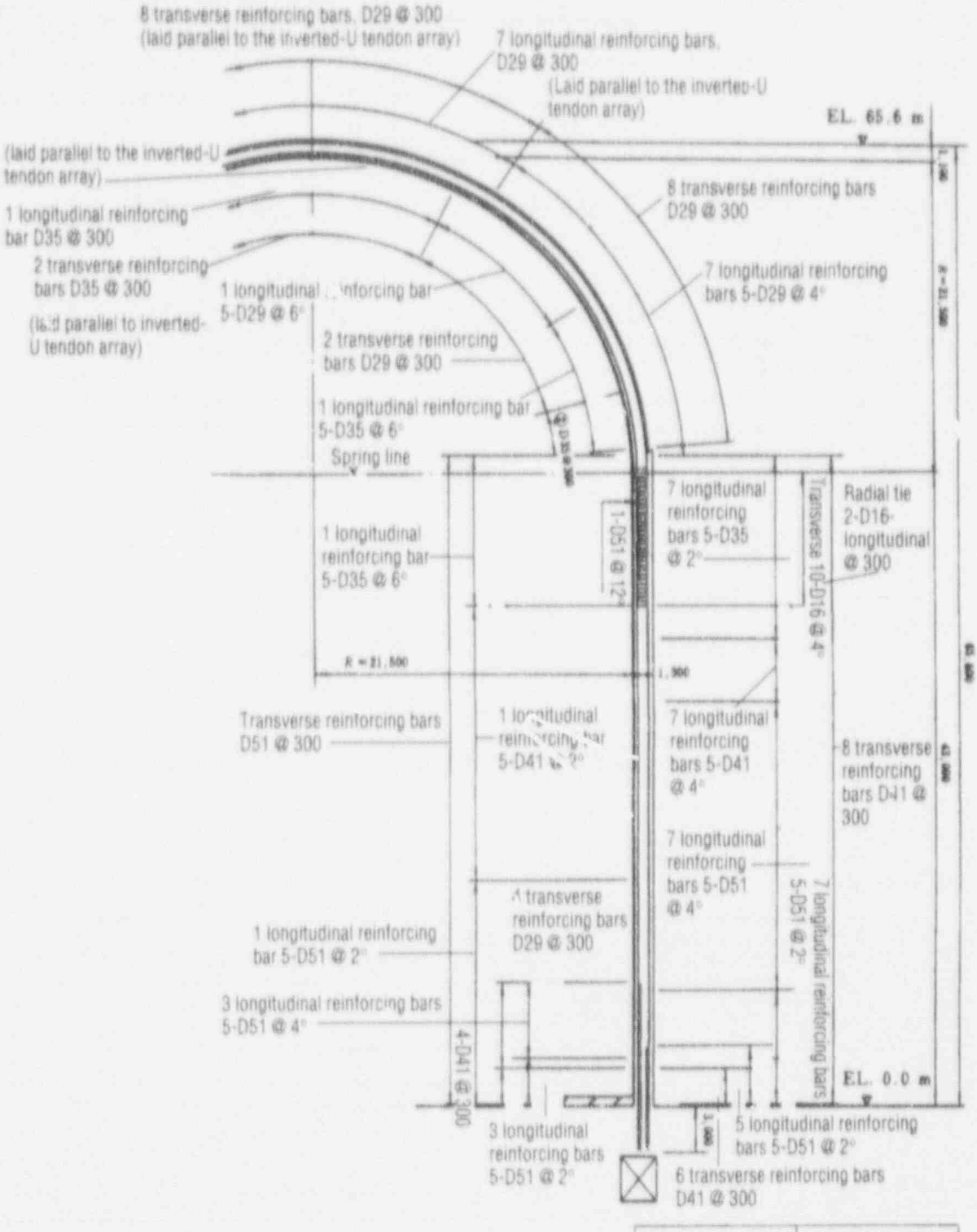
Load state	Name	Combination No.	Stress state 1	Stress state 2
I	In normal operation	1	$D+L+Fe+R_1$	$D+L+Fe+T_{1W}+R_1$
II	In test	2	$D+L+Fe+P_0$	—
III	In S_1 earthquake	3	$D+L+Fe+K_1+R_1$	$D+L+Fe+K_1+T_{1W}+R_1$
	In L-accident	4a	$D+L+Fe+P_2(1)+R_2$	$D+L+Fe+P_2(1)+T_{2W}(1)+R_2$
	In L-accident	4b	$D+L+Fe+P_2(24)+R_2$	$D+L+Fe+P_2(24)+T_{2W}(24)+R_2$
	L-accident + S_1 earthquake	5	$D+L+Fe+P_2(24)+K_1+R_2$	$D+L+Fe+P_2(24)+K_1+T_{2W}(24)+R_2$
IV	In S_2 earthquake	6	$D+L+Fe+K_2+R_1$	—
	In L-accident	7	$D+L+Fe+1.5P_2(1)+R_2$	—
	L-accident + S_1 earthquake	8	$D+L+Fe+P_2(1)+K_1+R_2$	—

Piping load

R_1: In normal operation

R_2: In L-accident

Table 5.5.2-9 lists the definitions of the other load symbols.



Concrete	$F_c = 420$ kgf/cm²
Reinforcing bars	SD 40

Figure 5.5 2-16. Schematic diagram of PCCV layout of reinforcing bars.

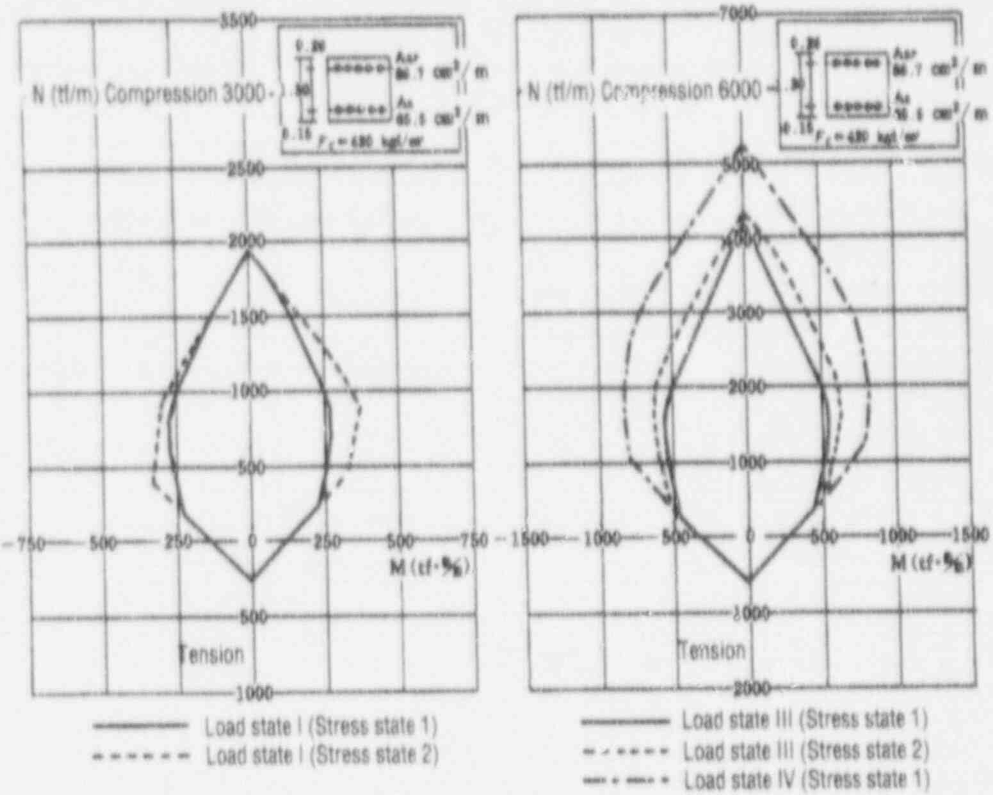


Figure 5.5.2-17. Allowable axial force-bending moment interaction curves.

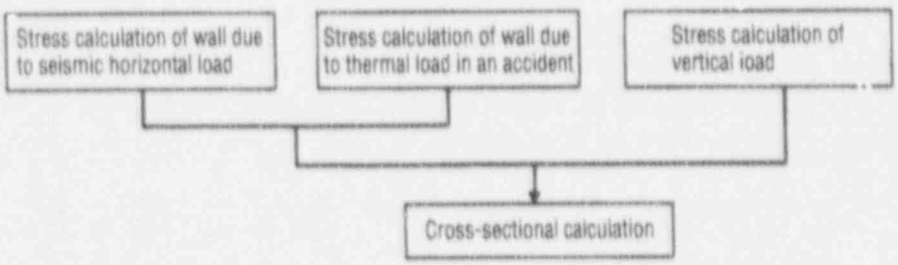


Figure 5.5.2-18. Design flow chart of spent-fuel pit.

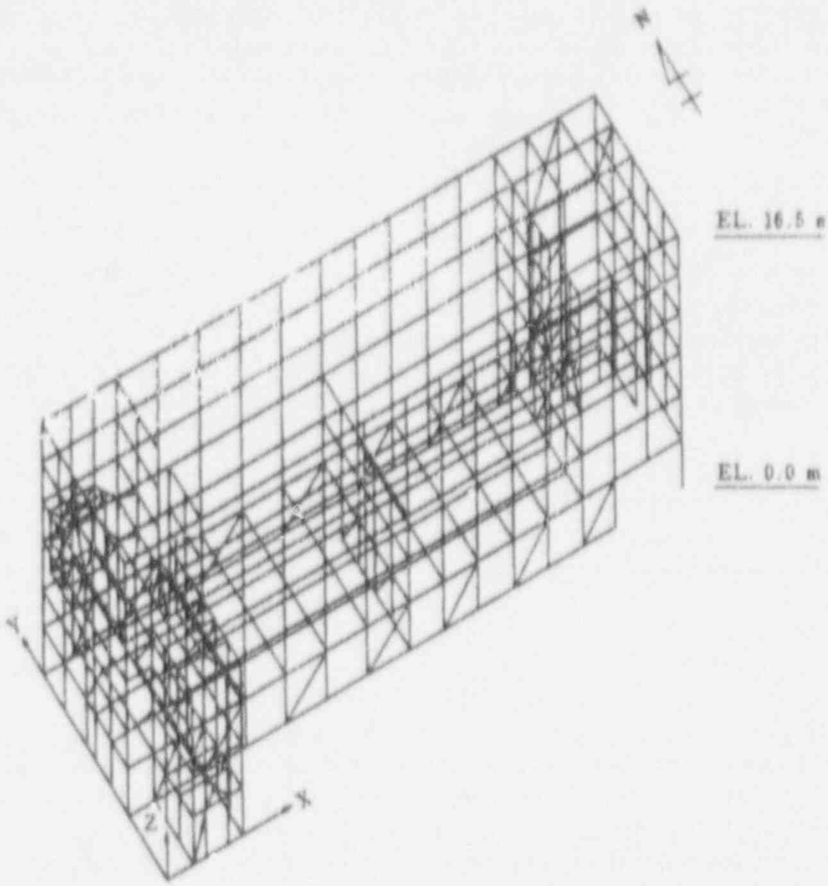


Figure 5.5.2-19. Analytical model diagram of spent-fuel pit.

Table 5.5.2-12. List of load combination (spent-fuel pit).

	State of external force	Load combination
Long-term	Normal time	G+P
	Normal time	$G+P+T_0$
Short-term	In S_1 earthquake	$G+P+K_1$
	In S_1 earthquake	$G+P+K_1+T_0$
	In accident	$G+P+T_a$
Ultimate	In S_2 earthquake	$G+P+K_2$

In an accident: 1 pit pump is out of order

G: dead load

P: live load

K_1: S_1 seismic load

K_2: S_2 seismic load

T_0: normal thermal load

T_a: accident thermal load

As far as the thermal stress is concerned, the bending cracks in the concrete are taken into consideration; the cross-sectional calculation is made by taking a 1/2 thermal stress in the long-term load combination and a 1/3 thermal stress in the short-term load combination. Calculation of the out-of-plane reinforcing bars is performed using the "RC Standards." Figure 5.5.2-20 shows a schematic diagram of the layout of reinforcing bars in the major portions determined from the results of the cross-sectional calculation.

e. Others

For PCCV, in many cases, high local stresses occur near the opening/penetrating hole, the portion with an abrupt change in shape and stiffness, and in the vicinity of the tendon anchorage portion. Hence, in order to ensure the safety of the structure, appropriate reinforcement should be provided. Hence, we will present a design example of the equipment hatch. Readers should refer to Figure 5.5.2-14 regarding the relative position of local designs in the design flow chart of the containment vessel.

The equipment hatch (E/H) is a circular opening with an inner diameter of 6.4 m arranged on the shell portion and used for carrying equipment into the reactor containment vessel. In the periphery of the opening portion, the shell thickness is increased to reduce the influence on the conventional portion of the cylindrical wall. The profile and dimensions are shown in Figure 5.5.2-21. The stress analysis is performed by 3-D FEM analysis for a semicylindrical partial model with an azimuthal angle of 180° with respect to the equipment hatch as the center, and from the EL +0.0 m foundation mat upper end to EL. 35.82 m on the vertical plane. The analytical model is shown in Figure 5.5.2-22.

As far as the finite elements used in the analytical model are concerned, for the vicinity of the opening portion, both the brick elements and the truss elements are used; in the range in which the effect of the stress concentration is small, shell elements are used. As the boundary conditions, the cylindrical foot portion is fixed, and the stress obtained from the stress analysis results of the general portion is used as the boundary condition for the upper horizontal plane and vertical planes on the two sides; in addition, the restraint is applied to ensure continuity.

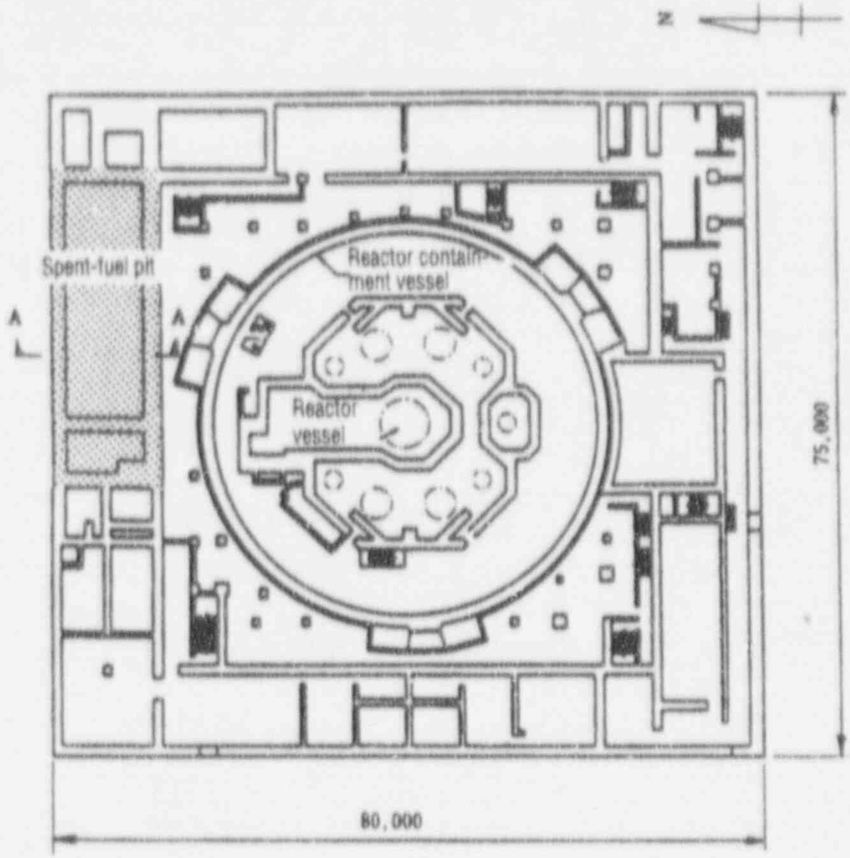
The cross-sectional calculation is performed using the same calculation method as that for PCCV general portion. For the ringbars around the opening, they are replaced by an equivalent concrete cross section. In this way, their effect is taken into consideration in the cross-sectional design. In this way, local stress analysis and cross-sectional design are performed to ensure that the strength of the entire structure is not reduced by the opening. Figure 5.5.2-23 shows a schematic diagram of the layout of reinforcing bars determined from the cross-sectional calculations.

f. Checks on horizontal strength capacity of building

The horizontal strength capacity (Q_u) of each story of the building is calculated on the basis of the following assumptions.

(i) The total strength capacity of each story in any given building (containment vessel, internal concrete, reactor external building) is the sum of the individual shear wall strength capacities.

(ii) The strength of the shear walls of the various buildings are calculated using the following formulas



Concrete	$F_c = 240$ kgf/cm²
Reinforcing bars	SD 40

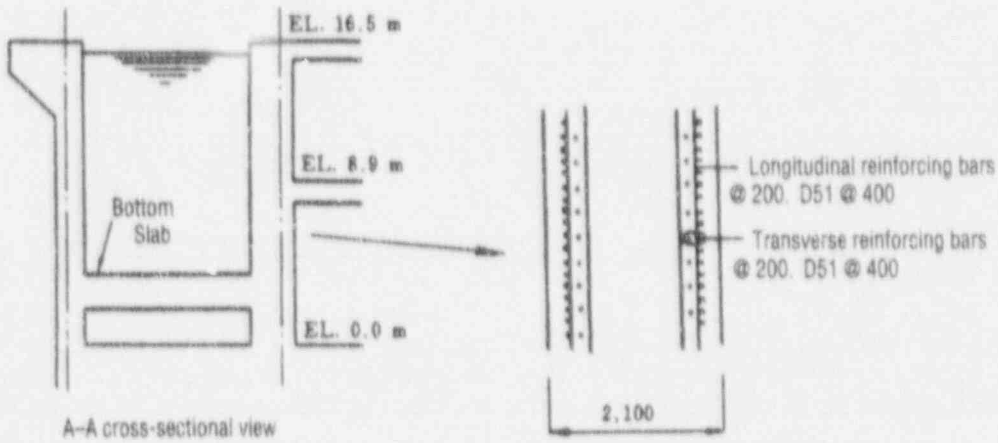


Figure 5.5.2-20. Schematic diagram of layout of reinforcing bars.

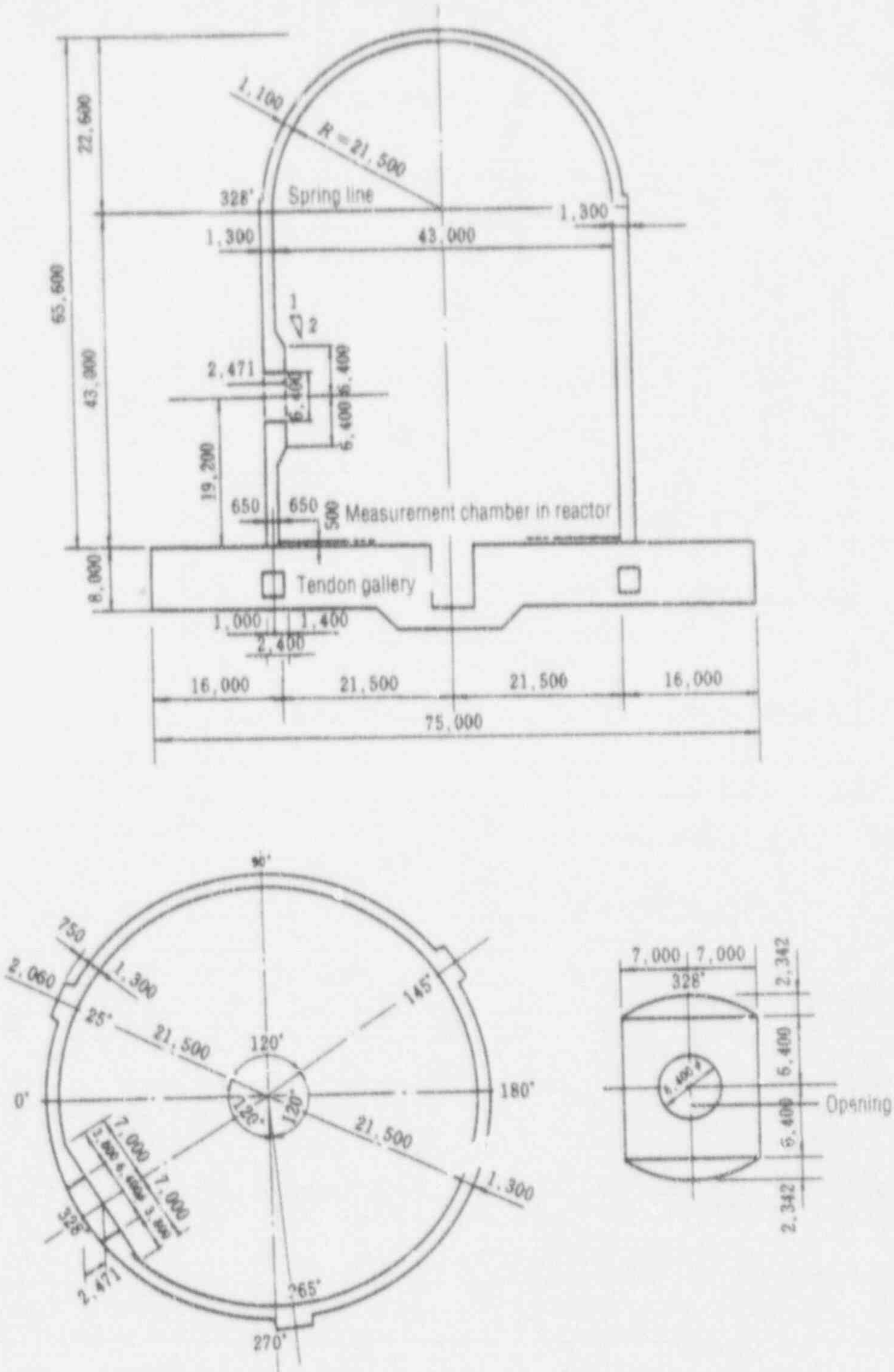


Figure 5.5.2-21. Diagram of shape and dimensions (units: mm).

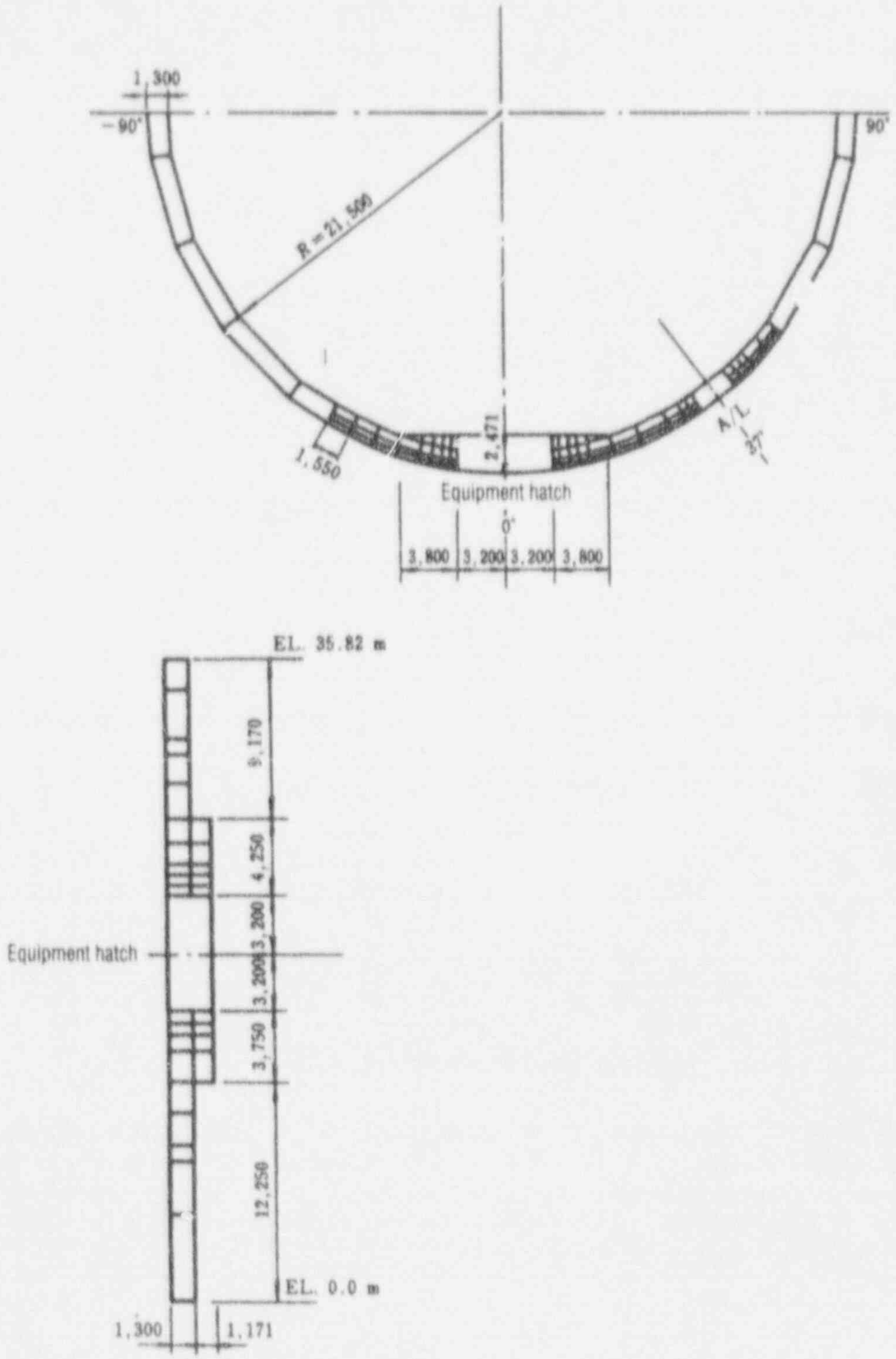


Figure 5.5.2-22. Analytical model.

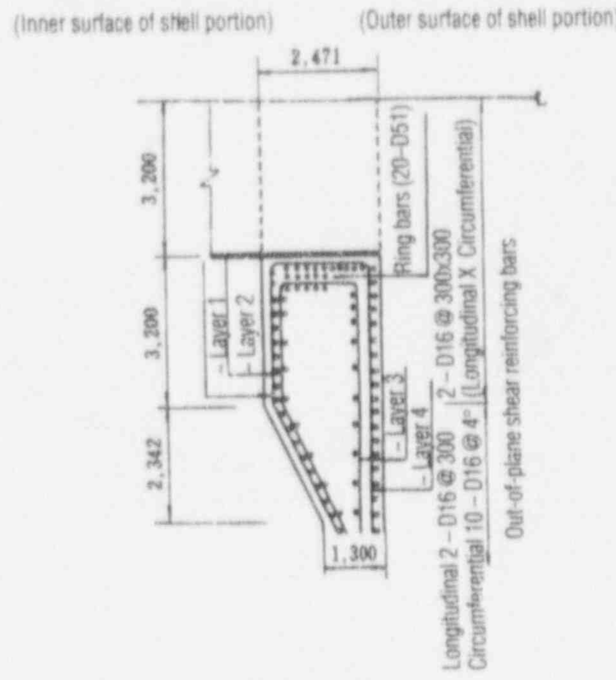


Figure 5.5.2-23. Schematic diagram of layout of reinforcing bars in the vicinity of the equipment hatch.

$$\left.\begin{aligned}
&Q_u = 0.216 F_c \cdot A_e \quad \text{(Containment vessel)}\\
&Q_u = 0.18 F_c \cdot A_e \quad \text{(Internal concrete)}\\
&Q_u = \left(\frac{0.0679 P_{te}^{0.23}(F_c + 180)}{\sqrt{M/(QD) + 0.12}} + 2.7\sqrt{\sigma_{wh} \cdot P_{wh}} + 0.1\sigma_0\right) \times A_e\\
&\qquad \text{(Containment-vessel peripheral building)}
\end{aligned}\right\} \qquad (5.5.2\text{-}2)$$

The required horizontal strength (Q_{un}) of each story of the building is calculated using the method described in Section 5.3.5(1) "Evaluation of static seismic force." In this case, the structural characteristic coefficient (D_S) is taken as 0.5. Table 5.5.2-13 lists the calculated results of the horizontal strengths of the various portions as compared with their required horizontal strengths.

Table 5.5.2-13. Comparison of horizontal strength.

	Portion	Horizontal strength Q_u ($\times 10^3$ tf)	Require. horizontal capacity strength Q_{uu} ($\times 10^3$ tf)
I/C	Upper portion pressurizing room	5.62	0.54
	Upper portion of steam generator chamber	12.10	0.97
	Lower portion of steam generator chamber	63.50	6.57
PCCV	Foot portion	81.65	11.30
REB	2nd floor	140.80	11.00
	1st floor	199.40	22.70

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Chapter 6. Aseismic design of equipment/piping systems

6.1 Basic items

6.1.1 Basic guidelines of aseismic design

(1) Structure plan and aseismic support plan

In principle, the equipment/piping system of a nuclear reactor facility is designed in such a way that it enters the rigid structure category. Since the earthquake strength of the equipment/piping system (which refers to the overall nuclear power equipment including the electrical instruments, etc.) depends significantly on the seismic support plan, an appropriate seismic support plan is important to ensure a sufficiently high earthquake strength. In the seismic support plan, it is important to arrange the seismic supporting devices in appropriate position and direction so that the thermal expansion of the equipment/piping system is restrained as little as possible and there is no excessively large seismic response during an earthquake. On the other hand, in the case when the thermal expansion is restrained, it is important to confirm that the thermal stress of the system is within the allowable limit. For the equipment for which the seismic support plan cannot be implemented appropriately, such as large-sized tanks, etc., it is necessary to reinforce the earthquake strength of the equipment itself.

The equipment/piping systems of nuclear reactor facilities, depending on the aseismic importance, are classified to Classes As, A, B, or C. For the important Classes As and A, in order to withstand both the static seismic force and the dynamic seismic force, it is important to implement the seismic support plan from the viewpoint of controlling the vibration frequency of the system, i.e., to make the system enter the rigid structure category. For Classes B and C, it is sufficient to design a seismic support which withstands the static seismic force. For Class B, however, in the case when the dynamic responses of the earthquake might be large, it is necessary to investigate the earthquake strength; if needed, the results should be reflected in the seismic support method.

For portions of the equipment/piping system with a certain degree of freedom in design, the position of the center of gravity should be made as low as possible, and the mounting should be as stable as possible. In the case when an equipment with a lower aseismic importance is closely located to an equipment with a higher aseismic importance, it is necessary to check the configuration plan once more to make sure that the damage in the equipment with a lower importance due to earthquake does not affect the equipment with a higher importance. In addition, so long as the configuration plan is appropriate, the seismic support plan should be made easier and simpler. Since the seismic support plan may cause trouble in the maintenance and service of the machines, a necessary and sufficient optimum plan for ensuring the aseismic safety of the system is preferred.

For a light water reactor or conversion reactor, as far as the structure of equipment/piping system itself is concerned, usually the plate thickness is not controlled by the seismic load, and the effect of the seismic force on the equipment is relatively small in comparison with the normal stress during operation. However, for the support structure, since the seismic force is dominant, appropriate strength design should be made in consideration of the uncertain factor of seismic force, e.g., enough stiffness should be ensured for the support points. In particular, the design of the anchorage, which is believed to be the most important portion in the aseismic design considering the likelihood of seismic damage. A sufficient attention should be paid to this portion since anchorage is on the boundary between the building/structure and an equipment.

(2) Seismic analysis and safety evaluation

Depending on the aseismic importance, the equipment/piping systems can be appropriately classified to aseismic Classes As, A, B, and C. For each specific aseismic class, it is necessary to make sure that it is safe with respect to the design seismic force.

The design seismic force is calculated from the horizontal static seismic coefficient corresponding to each aseismic class. In addition, for Classes As and A, the dynamic seismic force based on the appropriate seismic response analysis for the basic earthquake ground motions S_2, S_1, i.e., the extreme design earthquake and the maximum design earthquake, and the static seismic force due to the vertical seismic coefficient must be calculated.

The seismic safety evaluation of the equipment/piping system is based on the calculations (design by analysis) to confirm that the seismic stress based on the appropriate stress/intensity analysis using the above design seismic force combined with the stress caused by the other loads is within the allowable limit. However, when the ability of the equipment to maintain its function cannot be determined by the allowable stress calculations due to the complexity of the equipment or lack of reliability in analysis scheme, the confirmation may be made by performing vibration test, etc. (Evaluation using test).

The primary stress due to the design seismic force (the static seismic force for all the classes, and the dynamic seismic force based on the basic earthquake ground motion S_1 for Classes As and A) is limited within the yield point of the material used. When it is required to calculate the sum of the primary stress and secondary stress, the limit is set within a range without causing an excessively large strain. This is based on the premise that the seismic response of the system stays within the range of linear/elastic behavior from the macroscopic point of view. Hence, for the design by analysis, it is always necessary to calculate the primary seismic stress of the system appropriately. However, for the secondary stress, evaluation should be made when it has influence on the linear/elastic behavior of the system or on the low-cycle fatigue during earthquake. Of course, for aseismic Classes As and A, judging from their importance of structure, when significant secondary stress might take place, appropriate evaluation should be made of the secondary stress.

For the dynamic seismic force based on basic earthquake ground motion S_2 of aseismic Class As, it may enter the range of nonlinear/elastoplastic behavior. In this case, however, it is necessary to make sure that the ductility of the system is properly considered and there exists appropriate safety to maintain the ultimate strength of the system or the function.

In the case of an evaluation using a test, an appropriate vibration test or other type of equivalent tests should be performed by paying a due attention to the model scale law and the input motion characteristics at the support joints. It should be confirmed that there exists appropriate safety in strength and function with the effects of the other related loads taken into consideration.

6.1.2 Classification of aseismic imporiance

An equipment system that consists of several equipment/piping systems can be classified as a primary equipment or auxiliary equipment which directly or indirectly related to the requirement of function, or the direct support structures which directly support the loads of these equipment. Consequently, as described in Chapter 1, section 1.2.2 "Classification of aseismic importance," all the equipment corresponding to the same classification in function has the same aseismic importance.

6.1.3 Load combination and allowable limits

The guidelines of the load combination and allowable limits are as follows. For details, please see references [6.1.1-1], [6.2.2-1].

(1) Load combination

a. For phenomena which might be caused by seismic motion, their loads are combined.

b. For phenomena which are not caused by seismic motion, if the probability of simultaneous occurrence of the phenomena and earthquake is high, considering the probability of the phenomena and the duration of the load as well as the probability of earthquake, their combination should be taken into consideration.

(2) Allowable limits

a. Class As

(a) In the case when the seismic force calculated by basic earthquake ground motion S_1 or the static seismic coefficient is combined with the other load, in principle, an elastic state should be maintained.

(b) In the case when the seismic force calculated by the basic earthquake ground motion S_2 is combined with the other load, in principle, an excessively large deformation should be avoided.

b. Class A

Same as above a. (a).

c. Classes B and C

In the case when the seismic force calculated by the static seismic coefficient is combined with the other load, in principle, an elastic state should be maintained.

6.1.4 Design seismic force

The design seismic force is calculated on the basis of the basic earthquake ground motion and the static seismic coefficient corresponding to the aseismic importance of the equipment.

6.1.5 Earthquake response analysis

(1) General response analytical method

The equipment/piping system is designed to withstand the static seismic force corresponding to the aseismic importance. For aseismic Classes As and A, however, design is made to withstand both the static seismic force and the dynamic seismic force. For Class B, the safety of equipment which might resonate with the vibration of the support structure including the building is investigated using the dynamic seismic force corresponding to Class B. The dynamic seismic force is calculated by a seismic response analysis. The seismic response analysis of the equipment/piping system is usually performed by the spectral modal analysis method based on the design floor response spectrum at the installation floor. The design floor spectrum is usually taken as that of the most appropriate floor such as the floor near the center of gravity of the system or the floor with the most aseismic support points. However, in the case when a further seismic safety evaluation is needed, a multi-input analysis or similar approximate analytical method may be performed using the above design floor spectra. In using the spectral modal analysis method, all the modes should be considered whose mode participation factors are not negligible, the superposition is performed by the Square Root of the Sum of the Square method (referred to as "SRSS" method hereinafter) with respect to the necessary response calculation of acceleration, displacement, stress, reaction forces, etc. Combination of the response due to vertical seismic coefficient and the horizontal dynamic response is performed by adopting the absolute sum method. For the nuclear reactor containment vessel, nuclear reactor pressure vessel, and internal components, an analysis model coupled with the primary building, or an equivalent model using the substructuring method, should be adopted and a numerical time history analysis should be performed considering the size, complex seismic support systems and their importance in the relative displacement between the support joints. Of course, even if the structure is not very important, it is also possible to calculate the dynamic seismic forces using the time history response analysis method with the seismic response acceleration waveform and the displacement waveform at the support points as the inputs.

For aseismic Class As, it is acceptable to perform the elastic design for the basic earthquake ground motion S_2 using the linear spectral modal analysis method based on the floor response spectrum for S_2. In addition, the ductility of the system is evaluated, and the aforementioned nonlinear time history response analysis using inputs at the support points may be adopted. For aseismic Class B, if it is determined that the fundamental vibration frequency is resonant with the primary structure, dynamic evaluation using 1/2 of the floor response spectrum for S_1 design is performed to confirm the seismic safety. For the seismic response analysis, if it confirmed to be safer side, the approximate method or simple method may be used (constant pitch span method, response evaluation method using only fundamental vibration frequency).

(2) Analytical model

The containments are usually modeled as a one-dimensional multi-degree of freedom-bending/shear beam system; the piping are usually modeled as three-dimensional multi-degree of freedom-bending/torsion/shear beam system; and the other equipment may be modeled similarly. A more detailed model may be necessary to include the ovalization effects for the containment vessels and sloshing effects for large storage tanks. In addition, the discrete-mass system (concentrated constant system) may be replaced by a continuous system (distributed constant system) or a combined system. It may also be replaced by a finite elements model.

For the seismic support structures, since usually designed as a rigid structure, it is possible to assume the support points as being rigid. However, for a relatively large frame structure, if the stiffness is not higher enough than that of the supported equipment/piping system, the stiffness of the support should be considered. For anchor portions, their stiffness (such as elongation of anchor bolts, local bending of anchor plate, etc.) should be taken into consideration from mechanical point of view.

In a discrete-mass system model, the mass position is usually set at the center of gravity of each element obtained by subdividing the system. In the case of physically concentrated mass (a pump in a piping system, etc.), the concentrated mass is taken at that point. The number of masses should be large enough to have enough number of mode shapes to adequately express the vibration shape in seismic analyses. The points needed for stress evaluation are at mass points or nodes.

The range of the analytical model is usually from anchor (6 degree of freedom constrained) to anchor or free end. It is also possible to determine it by appropriately judging the boundaries (such as the nozzle end of a rigid piping container, the joint portion between a small-diameter pipe and a large-diameter pipe, etc.) believed to be distinguishable in a seismic analysis.

The properties of each element of the analytical model include the average moment of inertia, effective shear cross-sectional area of each element, and other geometrical characteristics of the system, as well as elastic coefficient and other material mechanical characteristics that depend on the operating temperature, etc. Each of these properties should be evaluated appropriately.

In principle, the damping is assumed based on the conventionally used design damping constants. In the case of a composite system with different portions having different damping constants (such as a composite system with 1.0% for containment, 2% for support frame, and 2.5% for piping), it is possible to calculate the modal damping constants for use. The design damping constants are usually determined from traditionally assumed values for elastic analysis at present. In this case, when the S_2 seismic response of Class As equipment is also in the elastic range from the macroscopic viewpoint, it is believed to be appropriate to use this design damping constants. Since the design damping constant is determined in a conservative way, in the case of special investigation and research, elastoplastic seismic response analysis, or equivalent elastic seismic response analysis, etc., it is possible to adopt the corresponding damping constant so long as it is proved to be appropriate.

(3) Earthquake response analysis and design seismic load

For the aseismic Class As and Class A equipment, such as the Type 1 equipment, the Type 2 containment, and the Type 3 equipment, the seismic load is determined on the basis of the seismic load due to S_2 and S_1 earthquake response analyses (moment, shear, axial force, etc.) and the seismic load due to the static seismic force.

Between the seismic load due to the S_1 earthquake and the seismic load due to the static seismic force, the more severe force is adopted in principle. However, at a site with high seismicity, if it is determined that the seismic load caused by the S_1 seismic force is clearly stricter than the static seismic load, calculation of the latter may be omitted.

For a simple Class A model with small number of masses, the S_1 seismic response acceleration and the static seismic coefficient are compared to each other, and the more severe one is selected and used to calculate the design seismic force.

For the static seismic force of the equipment system, in the case when the story shear coefficient of the building in which the equipment system is installed is determined (the aseismic class of the building is assumed as the same as that of the equipment system), 1.2 times the coefficient is taken as the design horizontal seismic coefficient in principle. However, for the equipment connected to the building (nuclear reactor containment vessel, etc.), outdoor equipment (tanks, etc.), etc., it is preferred that calculation be performed using modal analysis with the appropriate base shear coefficient which is determined corresponding to the aseismic class.

6.1.6 Stress/strength analysis

(1) Stress analysis of Class As and A equipment

In the case of an analytical model of discrete-mass and bending/shear beam system, the stresses in the equipment/piping system caused by static seismic force or dynamic seismic force are calculated in terms of bending moment (M), shear force (Q), and axial force (N).

For the Type 1 and Type 2 vessels, it is, in principle, to adopt the stress analysis methods such as the shell theory or finite element method, for simultaneous operating load conditions with seismic loads, $\pm$M, $\pm$Q, $\pm$N. However, it is also possible to calculate the stresses during earthquake and the stresses in the operating state separately, followed by adding them in consideration of the stress types and stress component directions.

For the Type 1 and Type 3 piping systems, the stress evaluation is usually performed by the absolute sum of M, Q, N calculated for various operating loads and the $\pm$M, $\pm$Q, $\pm$N due to an earthquake. When the absolute sum is calculated, the seismic force direction and the stress component directions should be taken into consideration. However, it is also possible to directly add up the stresses (σ = CM/Z, C = stress factor, Z = section modulus, etc.) during earthquake and the stress in the operating state.

For the other equipment/piping systems it is preferred that the stress analysis be performed by adding up M, Q, N caused by the operating loads which should be combined with $\pm$M, $\pm$Q, $\pm$N due to the seismic force, in consideration of the directions of the components. However, it is also possible to adopt the simpler method in which the stress analysis is performed independently for each case and the obtained stresses are then added up on the safer side.

For the pad portion, lug portion, nozzle portion, seismic support leg joint portion, and other portions of the primary equipment, local stress concentration should be evaluated using the finite element method or the Bijlaard method. Also, for the large-size self-supporting shell structures (containment vessels, tanks, etc.), it is necessary to perform the buckling safety evaluation due to $\pm$M, $\pm$Q, $\pm$N during earthquake.

The stresses caused by the seismic force are usually primary stresses. On the other hand, the stress caused by relative displacement between support points of a piping system, the stress in the containment vessel caused by the relative displacement of the upper building shear lug, etc., are secondary stresses. However, in the conventional building/equipment time history response analysis, it is impossible to make a distinction between the primary and secondary stresses $\pm$M, $\pm$Q, $\pm$N. As a result, additional special analysis for this distinction is needed. Attention should be paid to this point. However, it is conservative to regard these $\pm$M, $\pm$Q, $\pm$N as the primary stresses.

The primary stress due to the seismic force usually represents all the internal forces which are needed to meet the "equilibrium conditions of forces" against the external seismic forces (in the case of a leg joint portion of a containment, the local stress of the containment due to the leg reaction force = primary stress). Consequently, it is necessary to perform a detailed analysis of these stresses, and to evaluate the maximum stress.

The secondary stress due to the seismic force meets the "self-balance condition." When the portion where it takes place cannot be ignored from the viewpoint of maintaining the function of the equipment system (such as the joint portion between the vessel main body and the support skirt cylinder shell, etc.), it should be evaluated.

In the case of fatigue evaluation, which is required when the primary plus secondary stresses due to the seismic force exceed the limit value, it is necessary to know the number of cycles during an earthquake. This may be appropriately determined on the basis of the characteristics of the response seismic waveform of the installation floor and the seismic response characteristics of the system.

Combination of the stress due to the dynamic horizontal seismic force and the stress due to the static vertical seismic force of the primary equipment is performed in principle by making an absolute sum. However, "SRSS" method may be used to combine the stress due to the other dynamic loads (such as the dynamic load when the primary steam escape safety valve in the BWR containment vessel is activated, etc.) and the stress due to dynamic seismic force.

(2) Stress analysis of Class B and C equipment

Usually, Class B and C equipment is designed according to static seismic force. Since static seismic force is determined independent of the seismicity of the site, the design analysis evaluation method of the equipment is standardized. The main parts in this category include vessels, tanks, pumps, blowers, piping, and ducts. For the stress analysis/strength evaluation for different types of equipment, the precondition is based on calculation of the primary stress due to earthquake, with prescribed stress evaluation points, stress calculation formula, and calculation sheet form (see section 6.6.3 "Class B and C equipment"). Consequently, design and analysis of Class B and C equipment may be performed in this way if it is proved to be sufficient. For a Class B equipment which might have a resonance problem, a dynamic study is required. For this case, the natural period should be calculated. For an item which is not classified as a rigid structure, the evaluation using dynamic force is included in the format.

(3) Stress analysis of support structures

The seismic reaction of the support structure is calculated from the dynamic and static seismic forces for Class As and A equipment and mainly from the static seismic force for Classes B and C. The support structure must be designed to withstand the seismic reaction. For Class As and A support structures, it is necessary to ensure not only a high strength but also a necessary rigidity. However, for the support structures, too, there is a standard design analytical evaluation method based on the relationship of "seismic reaction = load" and the viewpoint of ensuring a necessary rigidity in the case of Classes As and A (see section 6.6.4 "Support structures").

Hence, the stress analysis evaluation of the support structures should be implemented using a sufficient method. An attention should be paid to the fact that the support structure design is closely related to the Steel Structure Design Standard of the Architecture Institute of Japan.

6.1.7 Seismic safety evaluation

As far as the seismic safety evaluation of the equipment/piping system of the nuclear reactor facilities is concerned, in the case of "design by analysis," depending on the aseismic importance, it is necessary to make sure that the various stresses caused by the other loads to be combined with the design seismic force are within the corresponding allowable stress limits. However, depending on the type of equipment system, the functional requirement may not be sufficiently evaluated by only performing strength calculation. Care should be exercised in this case. In this respect, the "evaluation by test," includes not only the strength evaluation, but also the evaluation from the viewpoint of function maintenance. It is, however, important to confirm the similarity of the test samples and the appropriateness of the seismic input characteristics.

For the seismic safety evaluation of Class As equipment during S_2 earthquake, if the elastoplastic response of the building is significant, attention should be paid to the effects on the elastoplastic response characteristics of the building itself, deformation characteristics and floor response, as well as the reliability of the elastoplastic behaviors.

6.1.8 Basic sequence of aseismic design

In this section, we will summarize section 6.1 "Basic items" in the form of a practical aseismic design sequence. Figure 6.1.8-1 shows the basic aseismic design sequence of the equipment/piping system from an overall macroscopic point of view, which corresponds to the explanation presented in section 6.1.1-6.1.4.

For nuclear power equipment, the seismic analyses (seismic response analysis, stress/strength analysis, etc.) which is important for the aseismic design has been explained in sections 6.1.5-6.1.7 with respect to the basic items. Figure 6.1.8-1 shows a partially enlarged form of the corresponding sequence. Figure 6.1.8-2 shows a block diagram concerning the seismic analysis. As further detailed blocks, we present Figure 6.1.8-3 (block diagram concerning seismic response analysis), Figure 6.1.8-4 (block diagram concerning stress/strength evaluation of Class As and A equipment), and Figure 6.1.8-5 (block diagram concerning stress/strength evaluation of Class B and C equipment).

These figures are sequential design diagrams corresponding to those already described in section 6.1 "Basic items." Consequently, explanation of these figures can be omitted. For details, please see the explanation in section 6.1 and in the following sections and the related flow charts. Table 6.1.8-1 summarizes the titles of these flow charts. In the table (and in text), "Notification No. 501" refers to Notification No. 501 of the Ministry of International Trade and Industry "Technical standards of structures related to nuclear power equipment for power generation" (October 30, 1980).

6.2 Importance classification

6.2.1 Basic guideline

The nuclear reactor facility for power generation must have a very high aseismic property so that it will not cause any major accident under any conceivable seismic force during the operation period. In order to reach this safety target, as shown in Chapter 1, "1.1.2 Aseismic design and safety design," the facilities should be classified according to importance from the safety point of view, and designed accordingly. Hence, for the equipment systems, that form of equipment/piping system, those which have the same functional requirements are classified to have the same aseismic importance irrespective of the form (direct or indirect) of the function.

6.2.2 Summary of importance classification

Definitions of importance classification and classification according to function are listed in Table 1.2.2-1 and Table 1.2.2-2 in Chapter 1. In addition, Table 6.2.2-1 lists the examples of the classification of aseismic importance for major equipment of the equipment/piping system. Several features in the importance classification will be presented in the following. For the background and details of importance classification, the readers are referred to "Technical Guidelines for Aseismic Design of Nuclear Power Plant: Classification of Importance Level/Allowable Stress Edition, JEAG 4601-Supplement-1984" by Nuclear Power Institute of the Japan Electrical Association (referred to as "JEAG 4601-Supplement-1984" hereinafter).

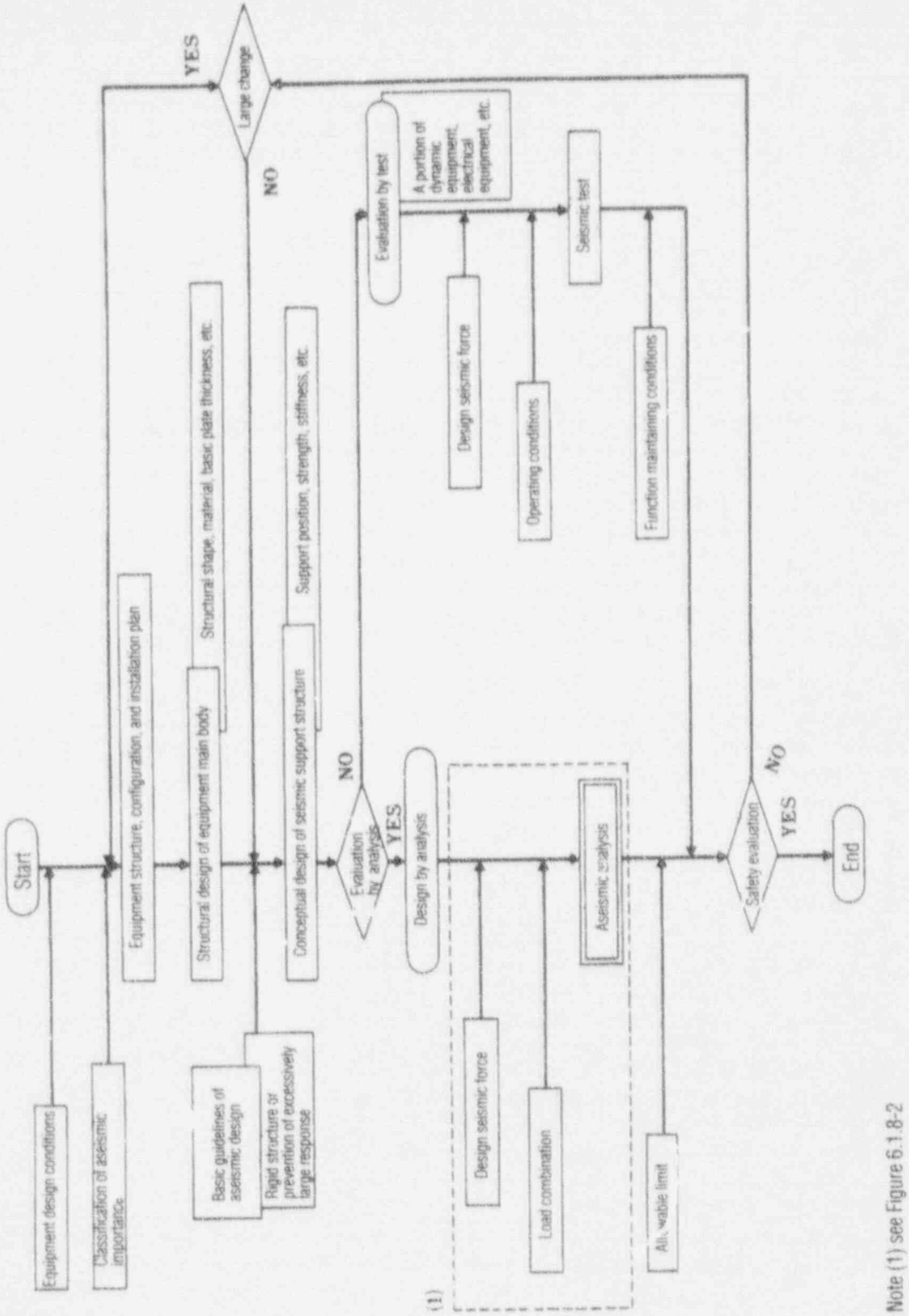
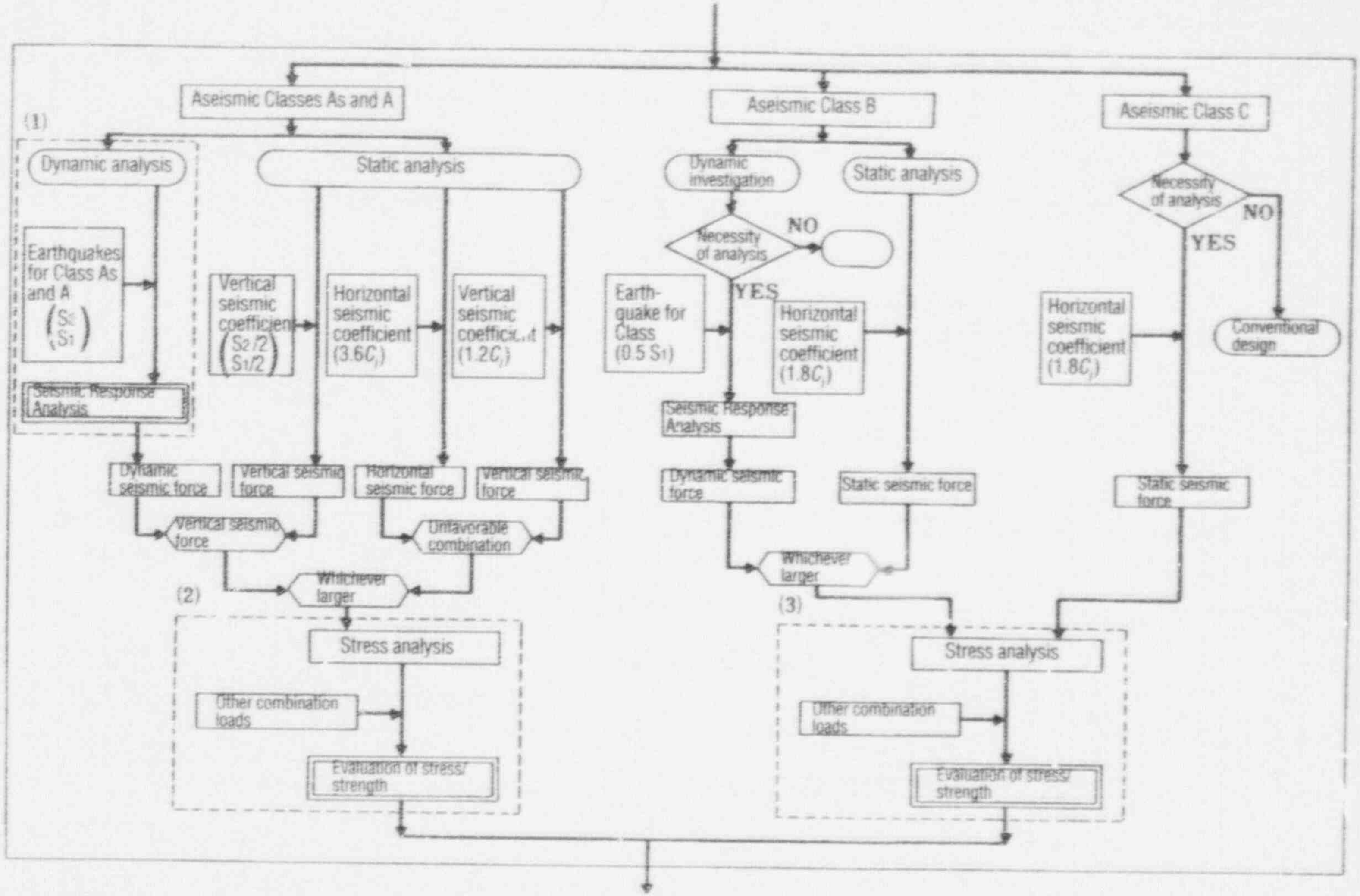


Figure 6.1.8-1. Basic sequence of aseismic design of equipment/piping system.



Note (1) See Figure 6.1.8-3.
Note (2) See Figure 6.1.8-4.
Note (3) See Figure 6.1.8-5.
Symbols in ( ) are the same as those in Table 6.4.1-1.

Figure 6.1.8-2. A partial detailed flow sheet of the basic sequence (aseismic analytical block diagram).

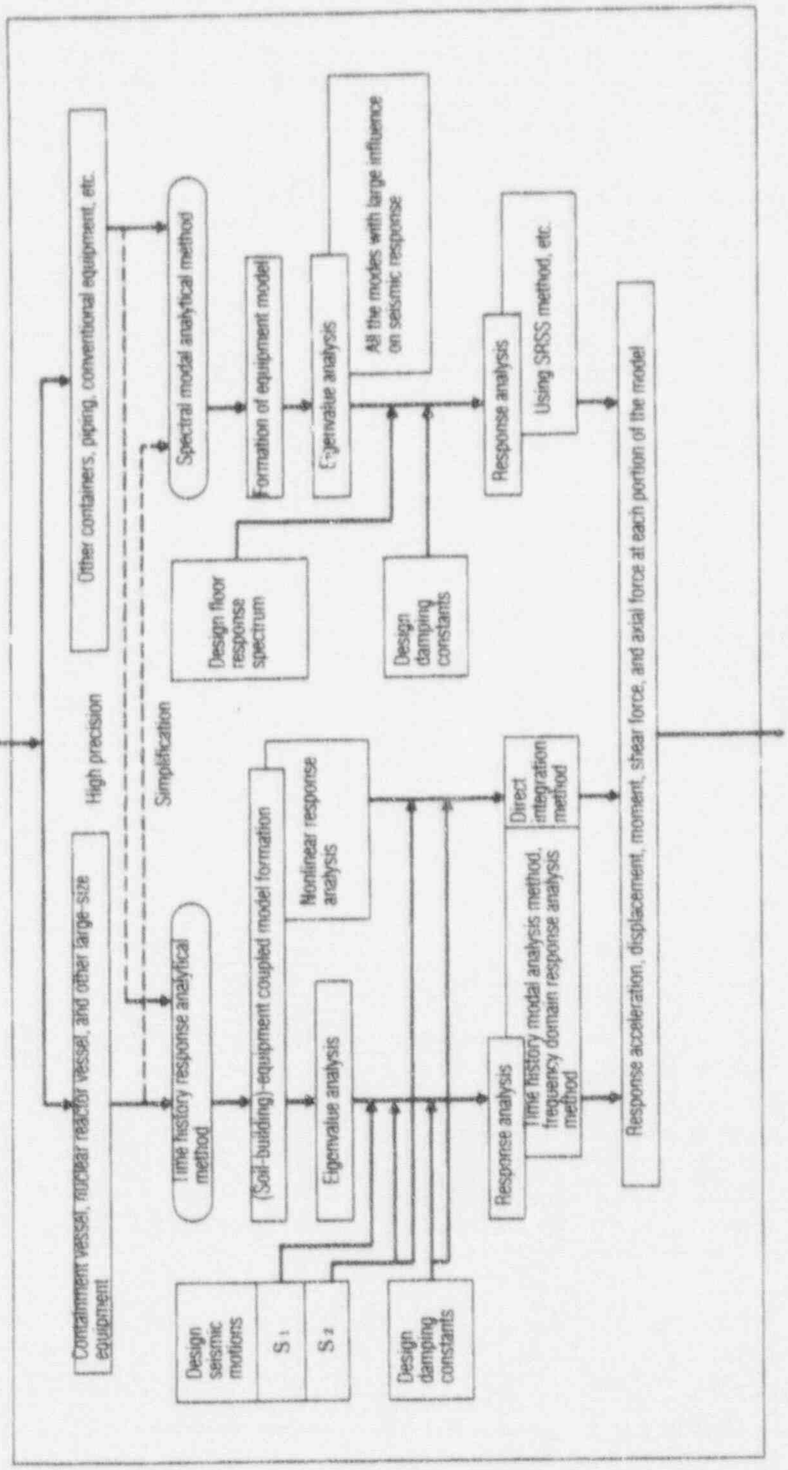
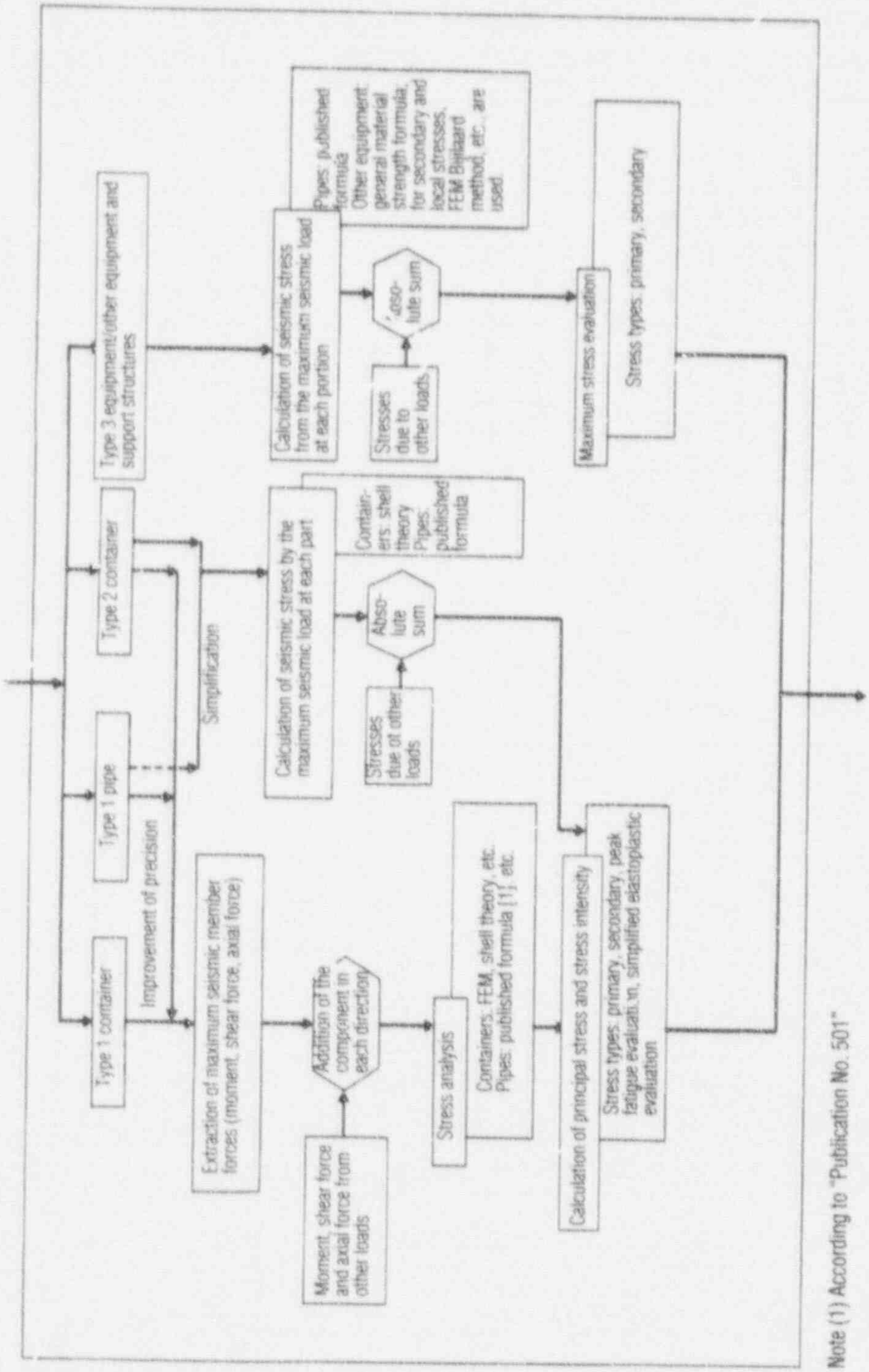


Figure 6.1.8-3. Partial detailed portion of the basic sequence (seismic response analysis block sheet).



Note (1) According to "Publication No. 501"

Figure 6.1.8-4. Details of a portion of the basic procedure (Classes As and A stress/strength evaluation block diagram).

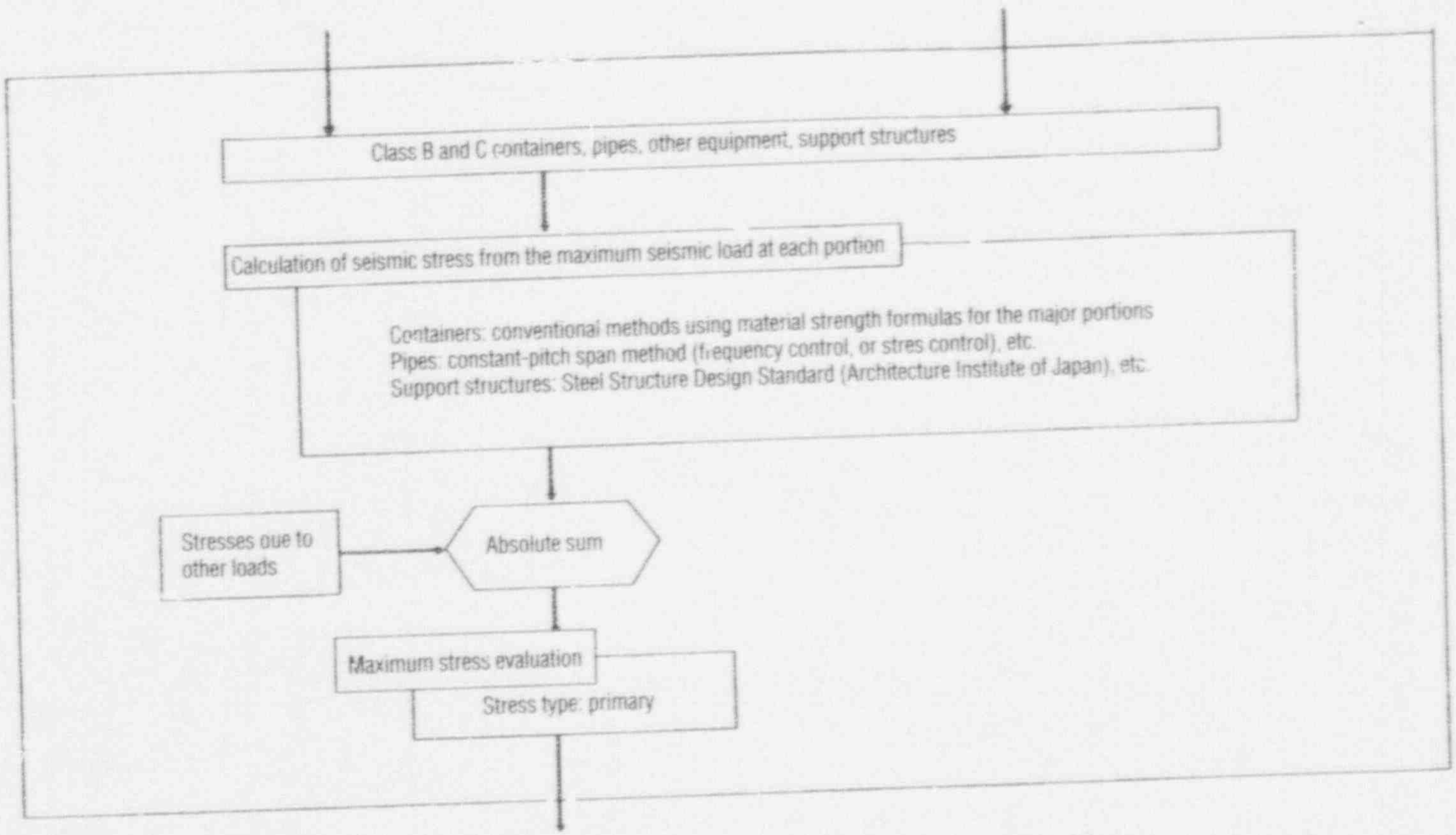


Figure 6.1.8-5. Details of a portion of the basic procedure (Class B and C stress/strength evaluation block diagram).

Table 6.1.8-1. List of design procedure drawings.

<table>
<tr><th>Class</th><th>Type</th><th>Common (basic)</th><th>Containers</th><th>Pipes</th><th>Pumps</th><th>Values</th><th colspan="4">Others</th></tr>
<tr><td rowspan="5">As

A</td><td>Type 1</td><td>6.1.8-1
6.1.8-2
6.1.8-3
6.1.8.4</td><td>6.6.2-3</td><td>6.6.2-12</td><td>6.6.2-13</td><td>6.6.2-14</td><td colspan="4">—</td></tr>
<tr><td>Type 2</td><td>6.1.8-1
6.1.8-2
6.1.8-3
6.1.8.4</td><td>6.6.2-15
6.6.2-16</td><td></td><td></td><td></td><td>Penetrat. portion
6.6.2-19</td><td colspan="3">—</td></tr>
<tr><td>Type 3</td><td>6.1.8-1
6.1.8-2
6.1.8-3
6.1.8.4</td><td>6.6.2-21
6.6.2-22</td><td>6.6.2-24
6.6.2-25</td><td>—</td><td>—</td><td colspan="4">—</td></tr>
<tr><td>Active equip.</td><td>6.1.8-1</td><td></td><td></td><td>—</td><td>—</td><td colspan="4">—</td></tr>
<tr><td>Electric. measure. equip.</td><td>6.1.8-1</td><td></td><td></td><td></td><td></td><td>Board
6.7-1</td><td>Tray
6.7-2</td><td>Device
6.7-3</td><td>Apparat.
6.7-4</td></tr>
<tr><td>B
·
C</td><td>—</td><td>6.1.8-1
6.1.8-2
6.1.8-5
6.6.3.1</td><td>6.6.3-2</td><td>6.6.3-40
6.6.3-44</td><td>—</td><td>—</td><td>Blower
6.6.3-45</td><td colspan="3">—</td></tr>
</table>

Table 6.2.2-1. Examples of aseismic importance classification of major equipment and piping system.

Aseismic importance	Major equipment	
	BWR	PWR
Class As	(i) Nuclear reactor pressure vessel; vessels, pipes, pumps, and valves within the nuclear reactor coolant pressure boundary (ii) Spent fuel pool (iii) Control rods, control rod driving mechanism, control rod driving hydraulic system (scram function) (iv) Residual heat removal system (cooling mode in shutdown state) (v) Nuclear reactor containment vessel; piping and valves within the boundary of the nuclear reactor containment vessel	(i) Nuclear reactor pressure vessel; vessels, pipes, pumps, and valves within the nuclear reactor coolant pressure boundary (ii) Spent fuel pool (iii) Control rods, control rod driving mechanism, control rod driving hydraulic system (scram function) (iv) Residual heat removal system (v) Nuclear reactor containment vessel; piping and valves within the boundary of the nuclear reactor containment vessel
Class A	(i) Emergency nuclear reactor core cooling system (ii) Standby gas treatment system (iii) Reactor internal structures	(i) Safety injecting system (ii) Annular air cleaning equipment (iii) Reactor internal structures
Class B	(i) Waste disposal system (ii) Steam turbine, condenser, feedwater heater (iii) Fuel pool cooling system	(i) Waste disposal system (ii) Spent fuel pit water cleaning system
Class C	(i) Sample collecting system, floor drainage system, etc. (ii) Main generator/transformer	(i) Sample collecting system, floor drainage system, etc. (ii) Turbine equipment, main generator/transformer

{1} For the main steam piping of a BWR, although the portion from isolation valve to main stop valve is classified as aseismic Class B, evaluation is made to ensure that it is not damaged by standard seismic motion S_1.

{2} For the fuel assembly, evaluation is made related to the control rod insertion function with respect to basic earthquake ground motion S_2.

{3} The structures in the reactor are classified as aseismic Class A; evaluation is also made to ensure that basic earthquake ground motion S_2 causes no problems in control rod insertion and reactor core cooling.

{4} In the case when a normally closed or separable valve is set in a small-diameter pipe directly connected to the system equipment, the valve and the pipe toward the equipment side from the valve are taken as the same aseismic class as that of the system equipment. For instrument pipes, etc., without the aforementioned valves, the portion where the fluid is contained is taken as the same aseismic class as that of the system equipment.

6.3 Load combination and allowable limits

The detailed content of this section is described in "JEAG 4601-Supplement-1984." A summary is presented here.

6.3.1 Basic guideline

(1) Explanation of symbols

III_AS: Allowable stress state with a special limit with respect to the stress generated by earthquake, with the allowable stress corresponding to operating state III in "Notification No. 501" taken as the standard.
IV_AS: Allowable stress state with a special limit with respect to the stress generated by earthquake, with the allowable stress corresponding to operating state IV in "Notification No. 501" taken as the standard.
B_AS: Allowable stress state of aseismic Class B equipment in earthquake
C_AS: Allowable stress state of aseismic Class C equipment in earthquake
D: Dead load
P: Pressure load in operating state of the plant (except the state after coolant loss accident) that should be combined with the earthquake[1]
M: Mechanical load, other than earthquake load and dead load, which acts on the equipment in the operating state of the plant (except the state after coolant accident loss) and should be combined with the earthquake[2]
P_L: Pressure load generated after a coolant loss accident excluding the period just after the accident
M_L: Mechanical load, other than dead load and seismic load, generated after a coolant loss accident, excluding the period right after the accident.
P_D: Mechanical load which is caused in plant operating state I or II (operation state III is also included in some cases) or by the maximum allowable working pressure set in design of the equipment, and should be combined with the earthquake.
M_D: Mechanical load which is caused in plant operating state I or II (operating state III is also included in some cases) or determined in design of the equipment, and should be combined with the earthquake.
P_d: Design load due to the maximum allowable working pressure
M_d: Design mechanical load
S_1: Seismic force or static seismic force set according to basic earthquake ground motion S_1
S_2: Seismic force set according to basic earthquake ground motion S_2
S_B: Seismic force (Note 2) derived from the seismic motion suitable for aseismic class B equipment or static seismic force
S_C: Static seismic force suitable for aseismic Class C equipment
S_y: Design yield strength, values defined in Appendix Table 9 in "Notification No. 501"
S_u: Design tensile strength, values defined in Appendix Table 10 in "Notification No. 501"
S_m: Design stress strength, values defined in Appendix Table 2 in "Notification No. 501"
For tension bolts at pressure portion, values defined in Appendix Table 3 are used.
S: Allowable tensile stress, values defined in Appendix Table 6 or 7 in "Notification No. 501."
For Type 2 vessels, values defined in Appendix Table 4 are used. For Type 2 tension bolts in the pressure portion, values defined in Appendix Table 5 are used. For other tension bolts in the pressure portion, values defined in Appendix Table 8 are used.

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[1]In each operating state, for P and M, the values set on the safe side (such as the maximum allowable working pressure, design mechanical load) may be used.

[2]The seismic force derived from the seismic motion used for the aseismic Class B equipment may be taken as 1/2 the value of the seismic force derived from basic earthquake ground motion S_1.

f_t: Allowable tensile stress. For support structures (except bolts, etc.), values defined in Article 88-3-1-A in "Notification No. 501" are used. For bolts, etc., values defined in Article 88-3-2-A in "Notification No. 501" are used.

f_s: Allowable shear stress. Same as above.

f_c: Allowable compression stress. For support structures (except bolts, etc.) values defined in Article 88-3-1-A in "Notification No. 501" are used.

f_b: Allowable bending stress. Same as above.

f_p: Allowable bearing stress. Same as above.

f_t*, f_s*, f_c*, f_b*, f_p*: Values are calculated by replacing the phrase of "the value defined in Appendix Table 9" by the phrase "1.2 times the value defined in Appendix Table 9" (in Articles 88-3-1C and 88-3-2C in "Notification No. 501") when said f_t, f_s, f_c, f_b, and f_p are calculated. For the other support structures, for the aforementioned $f_t - f_p$*, the value of F in Article 88-3-1A(A) of "Notification No. 501" is selected as follows. That is, it is taken as 0.7 times the value defined in Appendix Table 9 or the value defined in Appendix Table 10, whichever is smaller. For austenitic stainless steel and high-nickel alloy with normal temperature higher than 40°C, the value is taken as 1.35 times the value defined in Appendix Table 9, 0.7 times the value defined in Appendix Table 10, or the value at room temperature defined in Appendix Table 9, whichever is smallest.

ASS: Austenitic stainless steel

HNA: High-nickel alloy

(2) Aseismic Class As and A facilities

In the state when the load due to basic earthquake ground motion S_1 and the load in operating state I are combined, the facility (equipment/piping system) should stay in elastic state in principle. When the operating state IV(L) is considered as the design condition of the equipment, such as ECCS, the load due to basic earthquake ground motion S_1 is combined with the load of either operation state I or operation state IV (L), and the facility (equipment/piping system) should stay in elastic state in principle. In this case, the allowable stress is determined by adding the special limitation on the seismic stress to the basic allowable stress at operation state III defined in "Notification No. 501."

When the load due to basic earthquake ground motion S_2 is combined with the load in operation state I, the facility (equipment/piping system) should not have an excessive deformation with degradation in the functions. The allowable stress in this case is determined on the base of the allowable stress in operation state IV defined in the notification, with the special stress limitation for the stress generated by earthquake taken into consideration.

For phenomena in operation state II or III, if the phenomenon is associated with the earthquake or if the phenomenon lasts for a relatively long time, its combination with the earthquake should be taken into consideration.

(3) Aseismic Class B and C facilities

For aseismic Class B facilities, elastic design is performed with respect to the aseismic Class B design seismic load. As far as the load combination is concerned, the combination of the load in the normal operation state or abnormal transition period of the facility and the seismic load is taken into consideration.

For aseismic Class C facilities, consideration is made on the base of the combination of the loads of an aseismic Class B facility and allowable stress.

6.3.2 Load combination

(1) Operating state and combination with seismic loads

The seismic loads are combined with the loads in the following states. When evaluation is to be made of the load combination, it is only necessary to evaluate the load combination which would give the most serious result among the various load combinations to be considered, while the other load combinations with less serious consequence may be ignored. For specific examples of loads with serious results, please see section "6.6.1 Load/stress combination."

a. Operation state I

b. In operation state II, the plant states induced by the following phenomena as the dependent phenomena of earthquake

(For BWR)

(1) Offsite power loss
(2) Loss of feed water heating
(3) Erroneous operation of recirculation flow rate control system
(4) Failure in recirculation pump
(5) Loss of overburden load
(6) Closure of main steam isolating valve
(7) Failure in feed water control system
(8) Failure in pressure control apparatus
(9) Loss of total feed water flow
(10) Turbine trip
(11) Scram

(For PWR)

(1) Fall of control rod cluster
(2) Partial loss of primary coolant flow
(3) Rapid increase in steam load
(4) Loss of main feed water to steam generator
(5) Offsite power loss
(6) Loss of overburden load
(7) Nuclear reactor trip

c. In operation state II, the plant states induced by the following phenomena which are not associated with the earthquake but may last for a relatively long time:

(For BWR)

(i) Combination with basic earthquake ground motion S_1
  (1) Erroneous operation of safety relief valve
(ii) Combination with basic earthquake ground motion S_2
  None

(For PWR)

None

d. Among operation states III, the plant states induced by the following phenomena as phenomena associated with earthquake

(For BWR)
None

(For PWR)
(1) Abnormal pressure loss in the secondary cooling system
(2) Loss of primary coolant flow accident

e. In operation state IV, the plant states which are induced by the following phenomena which are not associated with the earthquake but may last for a long time; the states immediately after the following phenomena, however, are excluded.

(For BWR)
(i) Combination with basic earthquake ground motion S_1
(1) Loss of coolant accident
(ii) Combination with basic earthquake ground motion S_2
None

(For PWR)
(i) Combination with basic earthquake ground motion S_1
(1) Loss of primary coolant accident
(ii) Combination with basic earthquake ground motion S_2
None

(2) Load combination and allowable stress state

Table 6.3.2-1 lists the combinations of seismic load and other loads and the corresponding allowable stress states.

6.3.3 Allowable stresses of major equipment

(1) Allowable stresses of aseismic Class As and A facilities

In aseismic Class As and A facilities, the allowable stresses of vessels, pipes, pumps, valves, reactor core support structures, reactor internal structures, support structures, and tension bolts of pressure portion are listed in Tables 6.3.3-1, 6.3.3-2, 6.3.3-3, 6.3.3-4, 6.3.3-5, 6.3.3-6, and 6.3.3-7, respectively.

(2) Allowable stresses of aseismic Class B and C facilities

Table 6.3.3-8 lists the allowable stresses of the major equipment in aseismic Class B. The allowable stresses of the aseismic Class C facilities are set just as those of aseismic Class B facilities.

Table 6.3.2-1. Combinations of seismic load and other loads and corresponding allowable stress states.

Aseismic class	Load combinations	Type[(1)]								
		Type 1	Type 2	Type 3	Type 4	Type 5	Core support structure	Others		
		Equipment support structures	Vessel support structures	Equipment support structures	Vessels piping	Piping		Pumps valves	Reactor internal structure	Support structures
As	$D+P+M+S_1$	III_AS	III_AS	—	—	—	III_AS	—	—	—
	$D+P_D+M_D+S_1$	—	—	III_AS	III_AS	—	—	III_AS	III_AS	III_AS
	$D+P_L+M_L+S_1$	IV_AS[(2)]	III_AS[(3)]	—	—	—	IV_AS		—	—
	$D+P+M+S_2$	IV_AS	IV_AS	—	—	—	IV_AS		—	—
	$D+P_D+M_D+S_2$	—	—	IV_AS	IV_AS	—	—	IV_AS	IV_AS	IV_AS
A	$D+P_D+M_D+S_1$	—	—	III_AS	III_AS	III_AS	—	III_AS	III_AS	III_AS
B	$D+P_d+M_d+S_B$	—	—	B_AS	B_AS	B_AS	—	B_AS	—	B_AS
C	$D+P_d+M_d+S_C$	—	—	—	C_AS	C_AS	—	C_AS	—	C_AS

[(1)]In principle, the equipment types are defined in "Notification No. 501." Vessels/piping not defined in the Notification are as follows:

1) For vessels/piping associated with the emergency reserve power generator facility classified as aseismic Class A or As, the Type 3 definition is applied.
2) For ducts not classified as Type 5 piping, the definition of the Type 5 piping is still applied.
3) For vessels/piping other than said 1), 2) and not defined in the Notification, the definition of Type 4 is applied.

[(2)]For ECCS and related [equipment] needed for operation in an accident, III_AS is applied.

[(3)]1) For the Class 2 vessels, P_L of the load combination $(D+P_L+M_L+S_1)$ in allowable stress state III_AS is taken as the pressure in the nuclear reactor containment vessel at the time 10^{-1} year after LOCA.

2) Since the nuclear reactor containment vessel is the ultimate barrier after LOCA, in order to assure the safety margin for the overall structure, the combination of the maximum internal pressure after LOCA and the S_1 seismic motion (or static seismic force) is taken into consideration. Evaluation in this case is performed using the allowable limit of allowable stress state IV_AS.

Table 6.3.3-1. Allowable stress of vessels.

<table>
<tr><th rowspan="3">Type</th><th rowspan="3">Allowable stress state</th><th colspan="6">Stress class</th></tr>
<tr><th rowspan="2">Primary general membrane stress</th><th rowspan="2">Primary membrane stress + primary bending stress</th><th rowspan="2">Primary + secondary stress</th><th rowspan="2">Primary + secondary + peak stress</th><th colspan="2">Special stress limit</th></tr>
<tr><th>Pure shear stress</th><th>Bearing stress</th></tr>
<tr><td rowspan="2">Type 1</td><td>III_AS</td><td>$\begin{pmatrix} S_y \\ \frac{2}{3} S_u \end{pmatrix}^{(1)}$
For ASS and HNA;
1.2 S_m</td><td>1.5 times the left column</td><td rowspan="2">3 S_m (2)</td><td rowspan="2">Fatigue usage factor $\leq 1.0^{(3)}$</td><td>[illegible] 5 S_m</td><td>S_y (5)
(1.5 S_y)</td></tr>
<tr><td>IV_AS</td><td>$\frac{2}{3} S_u$
For ASS and HNA;
$\begin{pmatrix} \frac{2}{3} S_u \\ 2.4 S_m \end{pmatrix}^{(1)}$</td><td>1.5 times the left column</td><td>0.4 S_u</td><td>S_u (5)
(1.5 S_u)</td></tr>
<tr><td rowspan="2">Type 2</td><td>III_AS</td><td>$\begin{pmatrix} S_y \\ 0.6 S_u \end{pmatrix}^{(1)}$
For ASS and HNA;
1.2 S</td><td>1.5 times the left column</td><td rowspan="2">3 S (2)</td><td rowspan="2">Fatigue usage factor $\leq 1.0^{(3)}$</td><td>0.6 S</td><td>S_y (5)
(1.5 S_y)</td></tr>
<tr><td>IV_AS</td><td>Continuous portion
0.6 S_u
Discontinuous portion
$\begin{pmatrix} S_y \\ 0.6 S_u \end{pmatrix}^{(1)}$
For ASS and HNA;
Continuous portion
$\begin{pmatrix} 2 S \\ 0.6 S_u \end{pmatrix}^{(1)}$
Discontinuous portion
1.2 S</td><td>1.5 times the left column</td><td>0.4 S_u</td><td>S_u (5)
(1.5 S_u)</td></tr>
</table>

Table 6.3.3-1 *(Cont'd)*. Allowable stress of vessels.

<table>
<tr><th rowspan="3">Type</th><th rowspan="3">Allowable stress state</th><th colspan="6">Stress class</th></tr>
<tr><th rowspan="2">Primary general membrane stress</th><th rowspan="2">Primary membrane stress + primary bending stress</th><th rowspan="2">Primary + secondary stress</th><th rowspan="2">Primary + secondary + peak stress</th><th colspan="2">Special stress limit</th></tr>
<tr><th>Pure shear stress</th><th>Bearing stress</th></tr>
<tr><td rowspan="2">Types 3 and 4</td><td>$III_A S$</td><td>$\begin{pmatrix} S_y \\ 0.6\ S_u \end{pmatrix}^{(1)}$
For ASS and HNA; 1.2 S</td><td>1.5 times the left column</td><td colspan="2" rowspan="2">Fatigue usage factor ≤ 1.0,[4] if primary + secondary stresses $\leq 2\ S_y$,[2] the fatigue analysis is not needed.</td><td rowspan="2">—</td><td rowspan="2">—</td></tr>
<tr><td>$IV_A S$</td><td>$0.6\ S_u$</td><td>1.5 times the left column</td></tr>
</table>

(1)The smaller value in parentheses.

(2)Evaluation is made of stress range by seismic motion only.

(3)Fatigue usage factor is derived from only the seismic motion, and it is added to the fatigue usage factor in operation state I or II for evaluation.

(4)Evaluation is made of stress by seismic motion only.

(5)Data in parentheses refer to the case when the distance between the acting end of bearing load and the free end is longer than the acting width of the bearing load.

Table 6.3.3-2. Allowable stress of piping.

<table>
<tr><th rowspan="2">Type</th><th rowspan="2">Allowable stress state</th><th colspan="4">Stress class</th></tr>
<tr><th>Primary general membrane stress</th><th>Primary stress (including bending stress)</th><th>Primary + secondary stress</th><th>Primary + secondary + peak stress</th></tr>
<tr><td rowspan="2">Type 1</td><td>$III_A S$</td><td>$1.5\ S_m$</td><td>$2.25\ S_m$
When stress by torsion $> 0.55\ S_m$, the flexural stress + torsional stress is $1.8\ S_m$</td><td rowspan="2">$3\ S_m$ [2]</td><td rowspan="2">Fatigue usage factor ≤ 1.0 [3]</td></tr>
<tr><td>$IV_A S$</td><td>$2\ S_m$</td><td>$3\ S_m$
When stress by torsion $> 0.73\ S_m$, the flexural stress + torsional stress is $2.4\ S_m$</td></tr>
<tr><td rowspan="2">Types 3 and 4</td><td>$III_A S$</td><td>$\begin{pmatrix} S_y \\ 0.6\ S_u \end{pmatrix}$ [1]
For ASS and HNA; $1.2\ S$</td><td>S_y
For ASS and HNA; $1.2\ S$</td><td colspan="2" rowspan="2">Fatigue usage factor ≤ 1.0 [4]
If primary + secondary stresses $\leq 2\ S_y$, [2] fatigue analysis is not needed.</td></tr>
<tr><td>$IV_A S$</td><td>$0.6\ S_u$</td><td>1.5 times the left column</td></tr>
<tr><td rowspan="2">Type 5</td><td>$III_A S$</td><td colspan="4">The span length of support should be kept smaller than the maximum allowable pitch to ensure the functions with respect to acceleration and relative displacement in earthquake.</td></tr>
<tr><td>$IV_A S$</td><td colspan="4">Same as above</td></tr>
</table>

[1] Smaller value in parentheses.

[2] Evaluation is made of the stress range caused by seismic motion only.

[3] The fatigue usage factor by earthquake only is derived, and it is added to the fatigue accumulation coefficient in operation state I or II for evaluation.

[4] Evaluation on stress caused only by seismic motion.

Table 6.3.3-3. Allowable stress of pumps.

<table>
<tr><th rowspan="2">Type</th><th rowspan="2">Allowable stress state</th><th colspan="4">Stress class</th></tr>
<tr><th>Primary general membrane stress</th><th>Primary stress (including bonding stress)</th><th>Primary + secondary stress</th><th>Primary + secondary + peak stress</th></tr>
<tr><td rowspan="2">Type 1</td><td>III_AS</td><td>$\begin{pmatrix} S_y \\ \frac{2}{3} S_u \end{pmatrix}^{(1)}$
For ASS and HNA;
$1.2\ S_m$</td><td>1.5 times the left column</td><td rowspan="2">$3\ S_m$ (2)</td><td rowspan="2">Fatigue usage factor ≤ 1.0 (3)</td></tr>
<tr><td>IV_AS</td><td>$\frac{2}{3} S_u$
For ASS and HNA;
$\begin{pmatrix} \frac{2}{3} S_u \\ 2.4\ S_m \end{pmatrix}^{(1)}$</td><td>1.5 times the left column</td></tr>
<tr><td rowspan="3">Type 3 and other pumps</td><td>III_AS</td><td>$\begin{pmatrix} S_y \\ 0.6\ S_u \end{pmatrix}^{(1)}$
For ASS and HNA;
$1.2\ S$</td><td>1.5 times the left column</td><td colspan="2" rowspan="2">Fatigue usage factor ≤ 1.0 (4)
However, if primary + secondary stress $\leq 2\ S_y$,(2) fatigue analysis is not needed</td></tr>
<tr><td>IV_AS</td><td>$0.6\ S_u$</td><td>1.5 times the left column</td></tr>
<tr><td colspan="5">Evaluation on maintenance of operating functions
A. Evaluation of function maintenance by calculation
The load acting on the bearing due to the seismic load obtained from static/dynamic analysis should be confirmed as within the allowable load. If needed, other functions should also be confirmed by calculation.
B. Evaluation of function maintenance by experiment
The ability to maintain function is confirmed by vibration experiment simulating an earthquake or by static experiment with simulated load equivalent to the load acting in earthquake.</td></tr>
</table>

(1)Smaller value in parentheses.

(2)Evaluation is made of the stress range by seismic motion only.

(3)The fatigue usage factor is derived from the seismic motion only, and it is added to the fatigue usage factor in operation state I or II for evaluation.

(4)Evaluation is made of the stress by seismic motion only.

Table 6.3.3-4. Allowable stress of valves.

Type	Allowable stress
Type 1	(a) Evaluation of pressure resistance function (valve) Among the valves connected to pipes with outside diameters smaller than 115 mm, for the electrical valves and pneumatic valves with particularly large driving portions, evaluation is performed according to Article 81-1-1-B in "Notification No. 501." However, this does not apply to those for which appropriate measures have been taken to prevent an excessively large stress during earthquake. (b) Evaluation of maintenance of operating functions For valves which are required to maintain operating functions during and after earthquake, confirmation should be made by any of the following measures: A. Evaluation of function maintenance by calculation The design load of the valve is derived by any of the following schemes: (A) The maximum acceleration of the valve is obtained by analysis of the piping system. (B) An allowable design acceleration is determined beforehand for the valve. From the design load given by any of these schemes, evaluation is made to ensure that the stress of the most-affected portion (usually the base of the bonet) among the various parts, such as yoke, valve body, stem, etc., is not higher than the yield point or the limit value needed for maintaining the functions. B. Evaluation of maintenance of functions by experiment. The maintenance of function is evaluated by vibration experiment that simulates an earthquake or static experiment that simulates the load acting during earthquake.
Type 3 and other valves	(a) Evaluation of pressure function maintenance (valve) In the case when the wall thickness of the valves is identical to that of the connecting piping, in particular, for the electrical valves and pneumatic valves that have large driving portions, evaluation is performed according to Article 81-1-1-B in "Notification No. 501." However, this does not apply to those for which appropriate measures have been taken to prevent an excessively large stress during earthquake. (b) Evaluation of operating function maintenance It is performed according to the rule for Type 1 valves.

Table 6.3.3-5. Allowable stress of reactor core support structures and reactor internal structures.

Element	Allowable stress state	Stress class						
		Primary general membrane stress	Primary membrane stress + primary bending stress	Primary membrane stress + secondary membrane stress	Primary + secondary stress	Special stress limit		
						Pure shear stress	Bearing stress	Torsional stress
Structures other than bolts	III_AS	$1.5\ S_m$	1.5 times left column	—	—	$0.9\ S_m$	$1.5\ S_y$ [2] $(2.26\ S_y)$	$1.2\ S_m$
	IV_AS	$\frac{2}{3}\ S_u$ [1] For ASS and HNA; $\begin{pmatrix} \frac{2}{3}\ S_u \\ 2.4\ S_m \end{pmatrix}$	1.5 times left column	—	—	$1.2\ S_m$	$2\ S_y$ [2] $(3\ S_y)$	$1.6\ S_m$
Bolts, etc.	III_AS	$1.5\ S_m$	1.5 times left column	$\begin{pmatrix} 0.9\ S_y \\ \frac{2}{3}\ S_u \end{pmatrix}$ [1]	$0.9\ S_y$	—		
	IV_AS	$\frac{2}{3}\ S_u$ [1] For ASS and HNA; $\begin{pmatrix} \frac{2}{3}\ S_u \\ 2.4\ S_m \end{pmatrix}$	1.5 times left column	—	—			

[1] Smaller value in parentheses.

[2] Data in parentheses refer to the case when the distance between the acting end of the bearing load and the free end is larger than the acting width of the bearing load.

Table 6.3.3-6. Allowable stress of support structures.

Element	Allowable stress state	Stress class									
		Primary stress					Primary + secondary stress				
		Tensile	Shear	Compression	Bending	Bearing	Tensile/Compression	Shear	Bending	Bearing	Buckling
Structures other than bolts	III_AS	$1.5f_t$	$1.5f_s$	$1.5f_c$	$1.5f_b$	$1.5f_p$	$3f_t^{(1)}$	$3f_s^{(1,2)}$	$3f_b^{(1,3)}$	$1.5f_p^{(4)}$	(3,4) $1.5f_b$ $1.5f_s$ or $1.5f_c$
	IV_AS	$1.5f_t^*$	$1.5f_s^*$	$1.5f_c^*$	$1.5f_b^*$	$1.5f_p^*$				$1.5f_p^{*(4)}$	
Bolts, etc.	III_AS	$1.5f_t$	$1.5f_s$	—	—	—	—	—	—	—	—
	IV_AS	$1.5f_t^*$	$1.5f_s^*$	—	—	—	—	—	—	—	—

(1) Evaluation is made of the stress range due to seismic motion only.

(2) At the tag weld portion, it is taken as $1.5f_s$ with respect to the maximum stress.

(3) f_b is derived according to Article 88-3-1-A(D) in "Notification No. 501."

(4) Evaluation is made of the maximum compression value of the stress obtained by adding the load due to the seismic motion to the stationary load due to self-weight, thermal expansion, etc.

Table 6.3.3-7. Allowable stress of tension bolts at pressure portion.

Type	Allowable stress state	Stress class		
		Average tensile stress	Average tensile stress + bending stress	Primary + secondary + peak stress
Class 1 (Vessels)	III_AS	$2\ S_m$	$3\ S_m$	Fatigue usage factor ≤ 1.0 (2)
	IV_AS	$\begin{pmatrix} 2.4\ S_m \\ \frac{2}{3}\ S_u \end{pmatrix}^{(1)}$	1.5 times the left column	
Class 1 (other than Vessels)	III_AS	$1.5\ S_m$	—	
	IV_AS	$2\ S_m$		
Type 2	III_AS	$2\ S$	$3\ S$	Fatigue usage factor ≤ 1.0 (2)
	IV_AS	$\begin{pmatrix} 2.4\ S \\ \frac{2}{3}\ S_u \end{pmatrix}^{(1)}$	1.5 times the left column	
Types 3 and 4	III_AS	$1.5\ S$	—	
	IV_AS	$2\ S$		

(1) Smaller value in parentheses.

(2) The fatigue usage factor due to seismic motion only is derived, and it is added to the fatigue usage factor in operation state I or II for evaluation.

Table 6.3.3-8. Allowable stress of vessels, piping, and pumps.

Type	Allowable stress state	Stress class	
		Primary general membrane stress	Primary stress
Type 3, 4 vessels	B_AS	$\begin{pmatrix} S_y \\ 0.6\ S_u \end{pmatrix}^{(1)}$ For ASS and HNA, 1.2 S	S_y For ASS and HNA, 1.2 S
Type 3, 4 piping		$\begin{pmatrix} S_y \\ 0.6\ S_u \end{pmatrix}^{(1)}$ For ASS and HNA, 1.2 S	S_y For ASS and HNA, 1.2 S
Type 5 piping		The span length of the support should be maintained smaller than the maximum allowable pitch to ensure that the function can be maintained with respect to the acceleration and relative displacement in earthquake	
Type 3 and other pumps		$\begin{pmatrix} S_y \\ 0.6\ S_u \end{pmatrix}^{(1)}$ For ASS and HNA, 1.2 S	S_y For ASS and HNA, 1.2 S

(1)Smaller value in parentheses.

6.4 Design seismic force

6.4.1 Aseismic classification and design seismic force

For the design seismic force of equipment/piping system (referred to as "equipment" hereinafter), either the static seismic force or the dynamic seismic force is adopted according to the natural frequency. They are determined according to the aseismic class of the equipment as shown in Table 6.4.1-1.

As far as the static seismic force of the equipment is concerned, the story shear force coefficient of the building in which the equipment is installed is taken as the seismic coefficient times 1.2, followed by a certain degree of additional increase corresponding to the importance. For the additional increase corresponding to the importance, the horizontal seismic force of 1.2 C_1 is applied to Class C equipment; the horizontal seismic force for Class As and Class A equipment are increased by a factor of 3, and the horizontal seismic force for Class B are increased by a factor of 1.5. In addition, when the standards other than those for a nuclear power facility are required for Class C equipment, it is necessary to make corresponding evaluation.

On the other hand, the dynamic seismic force is calculated by performing dynamic analysis using the basic earthquake ground motions for Class As and Class A. Among Class B equipment, for those which might become resonant with the vibration of the support structure, the dynamic seismic force considered is taken as 1/2 the seismic force determined by basic earthquake ground motion S_1.

6.4.2 Static seismic force

(1) General indoor equipment

The static horizontal seismic force applied on the general equipment installed on the various floors of the building is calculated using the story shear coefficient of the building as the seismic coefficient. In this case, application of the seismic coefficient is performed as follows. For the equipment installed below the reference surface, the static horizontal seismic force is calculated on the base of the underground seismic coefficient below the standard surface determined for the building (see Figure 6.4.1-1).

a. Equipment supported on floor
The story shear coefficient of the story beneath this floor is used as the seismic coefficient.

b. Equipment supported by the wall
The story shear force coefficient of this story is used as the seismic coefficient.

c. Equipment installed on ceiling
The story shear force coefficient of this story is used as the seismic coefficient.

d. Equipment installed beneath the reference surface
The underground seismic coefficient at the vertical position where the equipment is set is used.

(2) Equipment-building interaction

For the equipment and structures in the building-equipment coupling model (primary containment vessel, reactor pressure vessel, and reactor shield wall of BWR; nuclear reactor containment vessel, steam generator, internal concrete, etc., of a PWR), the static seismic force is calculated using formula (6.4.1-1) in the "Regulatory Guide" [6.4.1-1], with the story shear coefficient used as the seismic coefficient. In this case, the distribution coefficient of the story shear force in the height direction (A_i) is derived using any of the following methods [H-K-1].

Table 6.4.1-1. Importance classification and design seismic force.

Aseismic importance class	Equipment/piping system			
	Static seismic force		Dynamic seismic force	
	Horizontal	Vertical	Horizontal	Vertical
As	—	—	$K_h(S_2)$ (3)	$K_v(S_2)$ (5)
As, A	$K_h(3.6\ C_I)$ (1)	$K_v(1.2\ C_V)$ (2)	$K_h(S_1)$ (4)	$K_v(S_1)$ (6)
B	$K_h(1.8\ C_I)$	—	$1/2\ K_h(S_1)$ (7)	—
C	$K_h(1.2\ C_I)$	—	—	—

(1) $K_h(3.6\ C_I)$ is the horizontal seismic force of the equipment/piping system determined according to 3.6 C_I. C_I is the story shear coefficient determined on the base of the standard shear force coefficient ($C_0 = 0.2$) with the vibration characteristics of building/structure and soil type taken into consideration; it is used as a substitute for the seismic coefficient. In the case when the foundation of the building/structure is set directly on bedrock, it is decreased to 0.8 times the original value.

(2) $K_v\ (1.2\ C_V)$ is the vertical seismic force of equipment/piping system determined from 1.2 C_V. C_V is determined regarding the seismic coefficient of 0.3 as a standard value, and in consideration of the vibration characteristics of building/structure and the type of soil. In the case when the foundation of the building/structure is set directly on the bedrock, it is decreased to 0.8 times the original value. C_V is taken as constant in the vertical direction.

(3) $K_h(S_2)$ is the horizontal seismic force of the equipment/piping system based on basic earthquake ground motion S_2.

(4) $K_h(S_1)$ is the horizontal seismic force of the equipment/piping system based on basic earthquake ground motion S_1.

(5) $K_v(S_2)$ is the vertical seismic force of the equipment/piping system calculated by regarding 1/2 the maximum acceleration of basic earthquake ground motion S_2 as the vertical seismic coefficient.

(6) $K_v(S_1)$ is the vertical seismic force of the equipment/piping system calculated by regarding 1/2 the maximum acceleration of basic earthquake ground motion S_1 as the vertical seismic coefficient.

(7) Only those which might become resonant with the vibration of the support structure are taken into consideration. The design basis earthquake ground motion is taken as 1/2 the seismic force determined from the basic earthquake ground motion S_1.

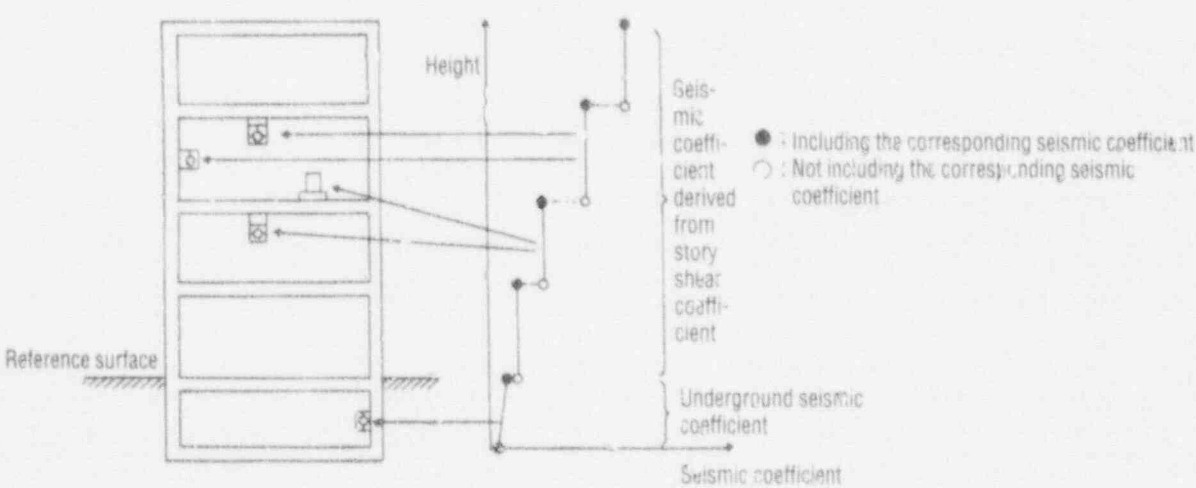

Figure 6.4.1-1. Determination of seismic coefficients for conventional indoor equipment.

$$C = nzR_tA_iC_0 \tag{6.4.1-1}$$

where n: importance factor (Class A: 3.0; Class B: 1.5; Class C: 1.0); z: zone coefficient (1.0); R_t: vibration characteristic coefficient; C_0: basic seismic coefficient (0.2).

a. Modal analysis method[1]

According to the "modal analysis method" (Note 1), A_i is calculated using formulas (6.4.1-2) and (6.4.1-3) on the base of the results of eigenvalue analysis of the building-equipment interaction system.

$$A_i = q_i/q_B \tag{6.4.1-2}$$

$$q_i = \frac{\sqrt{\sum_{j=1}^{l}\left(\sum_{s=i}^{n} W_s \beta_j U_{sj} R_{tj}\right)^2}}{\sum_{s=i}^{n} W_s} \tag{6.4.1-3}$$

where q_B: reference value of q_i for discrete mass used in calculating A_i; W_s: weight of point s (s = 1 ~ i ~ n); β_j: jth participation factor; $\beta_j U_{sj}$: jth participation factor at point s; R_{tj}: value of R_t with respect to jth natural period; l: mode number taken into consideration.

b. Static seismic coefficient method

According to the "Static seismic coefficient method" (Note 1), A_i is calculated according to formula (6.4.1-1) by modal analysis of the building only or by using the formula described in the guidelines. In this case, the static horizontal seismic coefficient applicable for the machine is taken as 1.2 times the story shear force coefficient of the building.

c. Method using dynamic analysis

According to the "method using dynamic analysis" (Note 1), A_i is calculated using formula (6.4.1-4) based on dynamic analysis of the building-equipment interacting system subjected to the design seismic motions;

$$A_i = \frac{Q_i}{\sum W_s} \Big/ \frac{Q_l}{W_t} \tag{6.4.1-4}$$

where Q_i: response shear force at point i; Q_l: response shear force of building at reference story; W_s: weight of the portion supported at point i; W_t: total weight.

(3) Outdoor equipment

For the static horizontal seismic force of the outdoor equipment, when the structure or foundation is buried, the seismic coefficient is calculated as 1.2 times the underground seismic coefficient or the seismic coefficient set for the structure in "Examination Guidelines"; when they are installed on the ground, just as the conventional indoor

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[1]These names are not formal. They are here only for convenience.

equipment, the story shear force coefficient at the installation position or foundation is taken as the seismic coefficient, which is multiplied by a factor of 1.2 and the obtained seismic coefficient is used for calculation in principle.

For equipment directly installed outdoors via foundation or support legs, etc., the story shear force coefficient is calculated on the base of the distribution of A derived using any of the following methods, and the result is taken as the seismic coefficient in calculating the static horizontal seismic force.

{1} Formula described in the "Guideline" (using the fundamental period of the equipment)
{2} Method using modal analysis
{3} Method using dynamic analysis (Note 1)

These specific calculation methods are based on above section (2).

6.4.3 Summary of dynamic seismic force

The dynamic seismic force used in the aseismic design of equipment/piping systems is determined on the base of the seismic force of Class As equipment with a high importance. In this section, we will introduce a summary in this respect. For details of calculation of the seismic force, please see section "6.5 Seismic response analysis."

(1) Class As and A equipment

For Class A equipment, the horizontal seismic force calculated according to the dynamic analysis for basic earthquake ground motion S_1 is adopted (such as analysis of ground-building-equipment interaction or analysis using design floor response spectrum at the installation position). For Class As equipment, which is particularly important among Class A equipment, the horizonal seismic force derived in the dynamic analysis for basic earthquake ground motion S_2 is also used. However, when it is determined that the equipment is a rigid structure (such as in the case when the fundamental natural frequency of the equipment is higher than 20 Hz, or in the case when the natural vibration frequency is higher than the region of dominant design floor response spectrum), the seismic force is calculated from the seismic coefficient determined on the base of the response acceleration of the building at the installation of the equipment. The vertical seismic force is also taken into consideration for Class As and A equipment. In this case, the vertical seismic force is determined from the vertical seismic coefficient which is 1/2 the maximum acceleration of the basic earthquake ground motion (constant in the vertical direction) and is supposed to act simultaneously with the horizontal seismic force in the unfavorable direction.

(2) Class B equipment

Among Class B equipment, for that which might become resonant with the vibration of the support structure (including building/structure, etc.), the dynamic seismic force is taken as 1/2 the seismic force determined from basic earthquake ground motion S_1. The vertical seismic force is not taken into consideration. Here, the equipment which might become resonant with the vibration of the support structure refer to that which has the natural vibration frequency of the equipment in the dominant region of the design floor response spectrum.

6.5 Earthquake response analysis

6.5.1 *Floor response spectrum*

(1) Outline of determining floor response spectrum

Among the equipment/piping systems under evaluation for aseismic design, for those which can be evaluated without considering the interaction with the building, the floor response spectrum can be used. Usually, the floor response spectrum can be formed on the concept shown in Figure 6.5.1-1, i.e., the time history of the response acceleration on the building floor or on the installation location of the equipment/piping system is calculated by dynamic analysis of the building or the building-equipment interaction system, and is used as the input wave. For this input wave, the response of the single discrete mass system is calculated. The results are represented as the floor response spectrum in a diagram with the natural period as abscissa and the damping constant as parameters.

Figure 6.5.1-3 shows an example of the calculation procedure of the floor response spectrum. According to this diagram, for the nuclear reactor building analysis model, dynamic analysis is performed for a simulated seismic wave with a magnitude of 6.5 and an epicentral distance of 1 km. The time history response wave at discrete mass No. 7 is calculated and used as the input wave for calculation of the floor response spectrum with natural period in the range of 0.05-1.0 sec for a damping constant h = 1, 3, and 4%. It can be seen that the floor response spectrum has a peak that reflects the vibration characteristics of the nuclear reactor building.

The floor response spectrum can be calculated according to the following procedure. Suppose the equipment is represented by a model of a one-degree-of-freedom system (a single discrete mass), the equation of motion becomes

$$m\ddot{x} + c\dot{x} + kx = m\ddot{y}_0 \tag{6.5.1-1}$$

where m: mass; c: damping coefficient; k: spring constant; x: relative displacement; $\ddot{y}_0$: time history of floor response acceleration.

Equation (6.5.1-1) can be rearranged to

$$\ddot{x} + 2h\omega\dot{x} + \omega^2 x = -\ddot{y}_0 \tag{6.5.1-2}$$

where h: damping constant $[=c/2(mk)^{1/2}]$; ω: natural radial frequency of vibration $[(k/m)^{1/2} = 2\pi/T$, T: natural period].

Equation (6.5.1-2) can be used to calculate the acceleration for various values of h:

$$S_a(T,h) = \max(\ddot{x} + \ddot{y}_0) \tag{6.5.1-3}$$

As shown in Figure 6.5.1-2, with h as the parameter, S_a is plotted against T, forming the floor response spectra.

Since the equipment is represented by a single discrete mass model with characteristic period of T_1 and damping constant of h_1, the maximum response acceleration during the period when said seismic motion $\ddot{y}_0$ acts can be derived from the floor response spectrum (Figure 6.5.1-2). Similarly, when the same seismic motion acts on a system having several different natural vibration frequencies and damping constants, the maximum response acceleration can also be derived from the floor response spectrum corresponding to the design damping constant.

Usually, the floor response spectrum is calculated on the basis of the aforementioned time history waveform. There are, however, other methods, such as the method in which the transfer function of the building-equipment interaction system is determined, and the floor response spectrum [6.5.1-2] is directly calculated from the target spectrum [6.5.1-3]. Following are several items to which attention should be paid when the floor response spectrum is calculated on the base of the time history waveform.

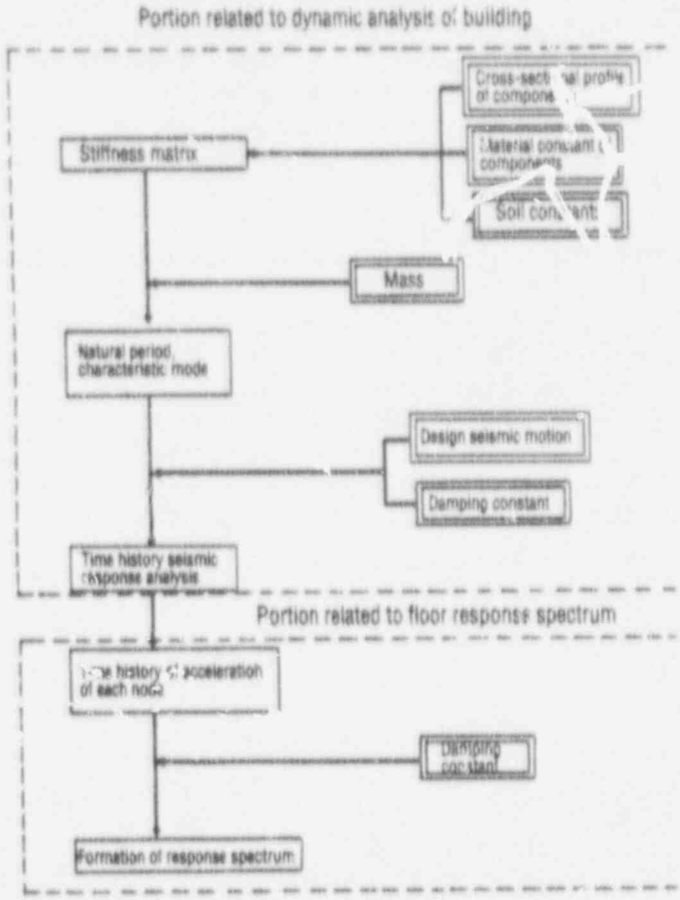


Figure 6.5.1-1. Analysis flow chart.

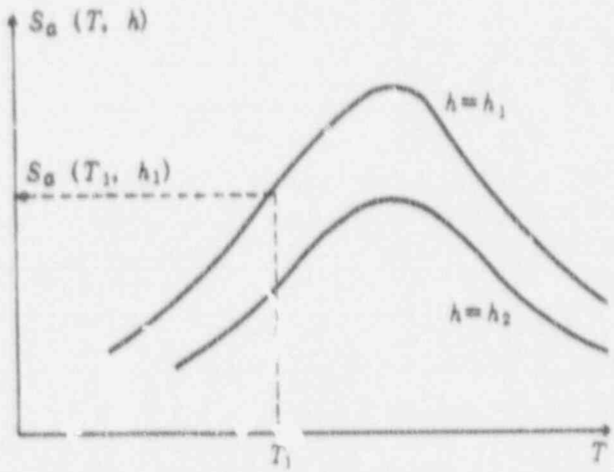


Figure 6.5.1-2. Floor response spectrum.

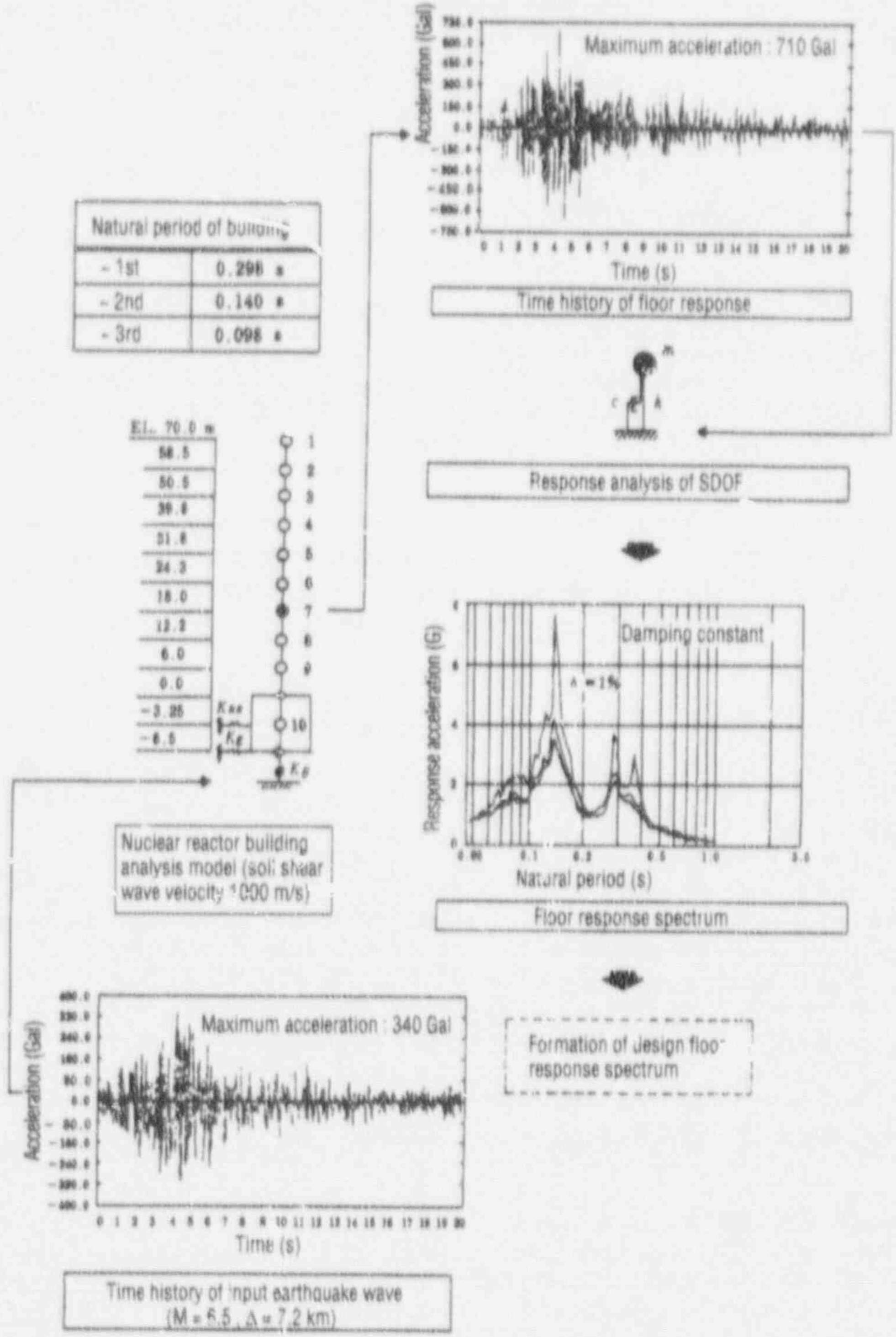


Figure 6.5.1-3. Example of formation of floor response spectrum.

a. Time increment of time history waveform and numerical integration method

The calculation precision of the floor response spectrum is related to the time increment of the input time history waveform. In particular, the influence of the time increment tends to increase in the short period. It is also related to the numerical integration method in this case. Consequently, in order to ensure sufficient accuracy for the lower limit value of the period region needed, it is necessary to combine the time increment of the time history waveform and the numerical integration method. The numerical integration methods that may be used include the Newmark-β method, the Runge-Kutta-Gill method, and the Nigam's method.

b. Calculation interval of natural periods

When the floor response spectrum is to be formed, the calculation interval of the natural periods usually should be determined for each plant considering the soil conditions, etc., to ensure an appropriate representation of the spectral characteristics in the predominant period of the building. Tables 6.5.1-1 and 6.5.1-2 lists examples of the calculation interval of natural periods actually adopted.

c. Others

The floor response spectrum is only the calculation result of the maximum value of the response values of a single discrete mass system. It does not mean that the maximum values for the various natural periods take place at the same time point. Attention should be paid to this feature since the information concerning the time axis is lost.

(2) Design floor response spectrum

For the design floor response spectrum (see Figure 6.5.1-4), it is important to evaluate the factors that influence the floor response spectrum and their possible range as well as the variation amplitude in the floor response spectrum caused by these factors. It is also important to take the deviation in the natural period of the equipment into consideration. In past research work [H-K-2], evaluation has been performed according to the procedure shown in Figure 6.5.1-5 for the soil properties, building stiffness, and calculation formula of soil spring constant, which affect the variation in the floor response spectrum, as well as the damping constant, phase characteristics of the simulated earthquake wave, etc. As a result, it is found that variations in these factors can be covered if the floor response spectrum is shifted by ±10% in the period axial direction.

Consequently, in principle, the design floor response spectrum is taken as the floor response spectrum broadened by ±10% in the period axial direction. However, when the variation amplitudes of the aforementioned factors are decreased due to site conditions, building conditions, etc., it is also possible to reduce the aforementioned shift rate (±10%) for the floor response spectrum. This method has been pointed out in past research work [H-K-2].

6.5.2 Dynamic analysis model

The seismic response analysis of equipment can be roughly classified into the following two types: analysis performed with direct interaction with the ground/building system and analysis performed with the indirect influence of the ground/building system via the floor response spectrum taken into consideration. The former analysis method is usually used for larger-size important equipment such as the nuclear reactor containment vessel, nuclear reactor pressure container and core internals. On the other hand, the latter analysis method is usually used for tanks, heat exchanger, pumps, piping and much other equipment. Consequently, analysis of most of the equipment is performed using the floor response spectrum.

Table 6.5.1-1. Example of calculation intervals of radial frequency of vibration.

Natural period	Calculation interval ($\Delta\omega$: rad/s)
0.05 ~ 0.10	4.0
0.10 ~ 0.15	1.5
0.15 ~ 0.30	0.8
0.30 ~ 0.60	0.6
0.60 ~ 1.00	0.5

Table 6.5.1-2. Example of calculation intervals of period.

Natural period	Calculation interval (ΔT: s)
0.05 ~ 0.10	0.002
0.10 ~ 0.20	0.005
0.20 ~ 0.30	0.010
0.30 ~ 0.40	0.020
0.40 ~ 0.70	0.050
0.70 ~ 1.00	0.100

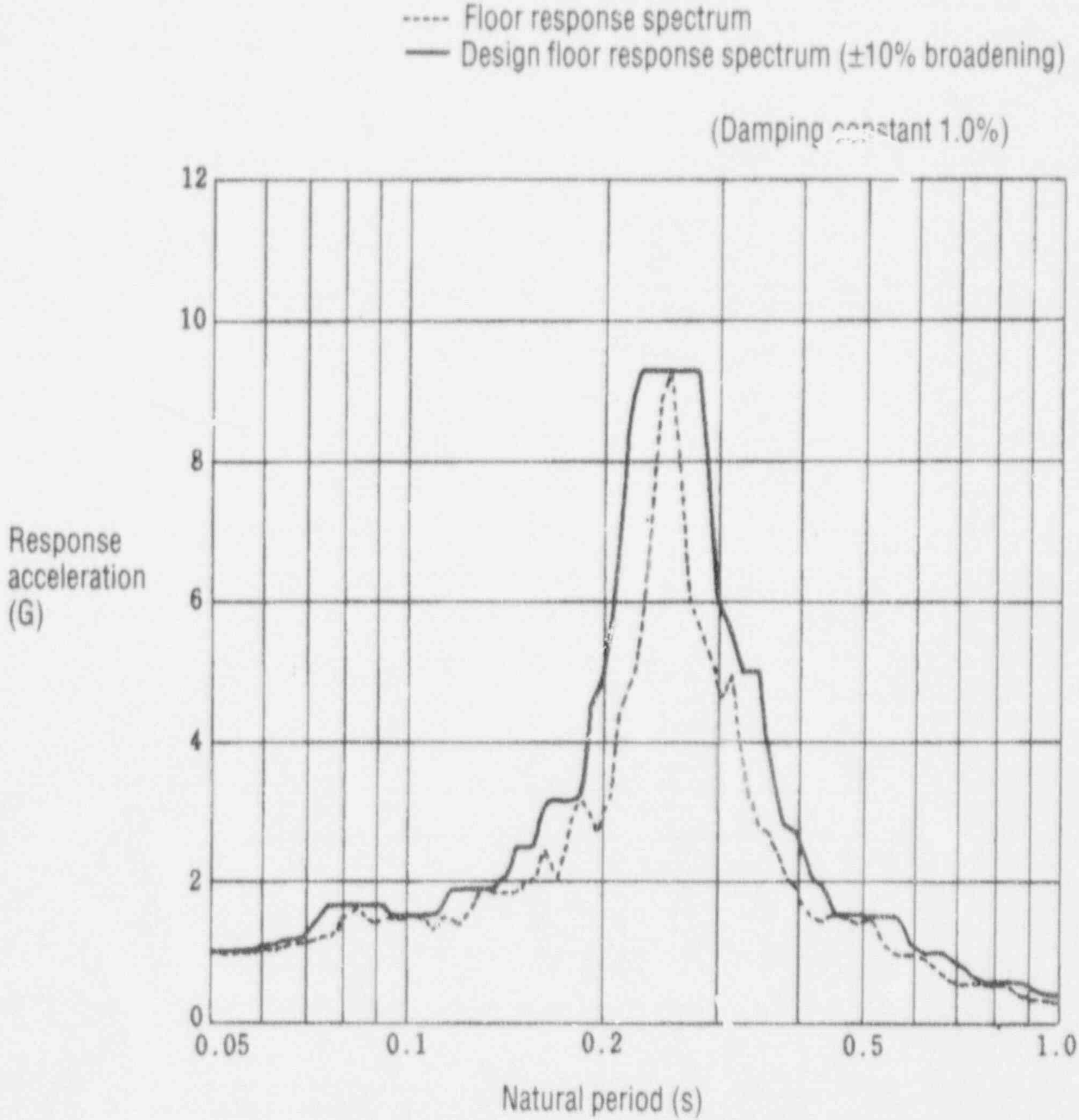


Figure 6.5.1-4. Example of design floor response spectrum.

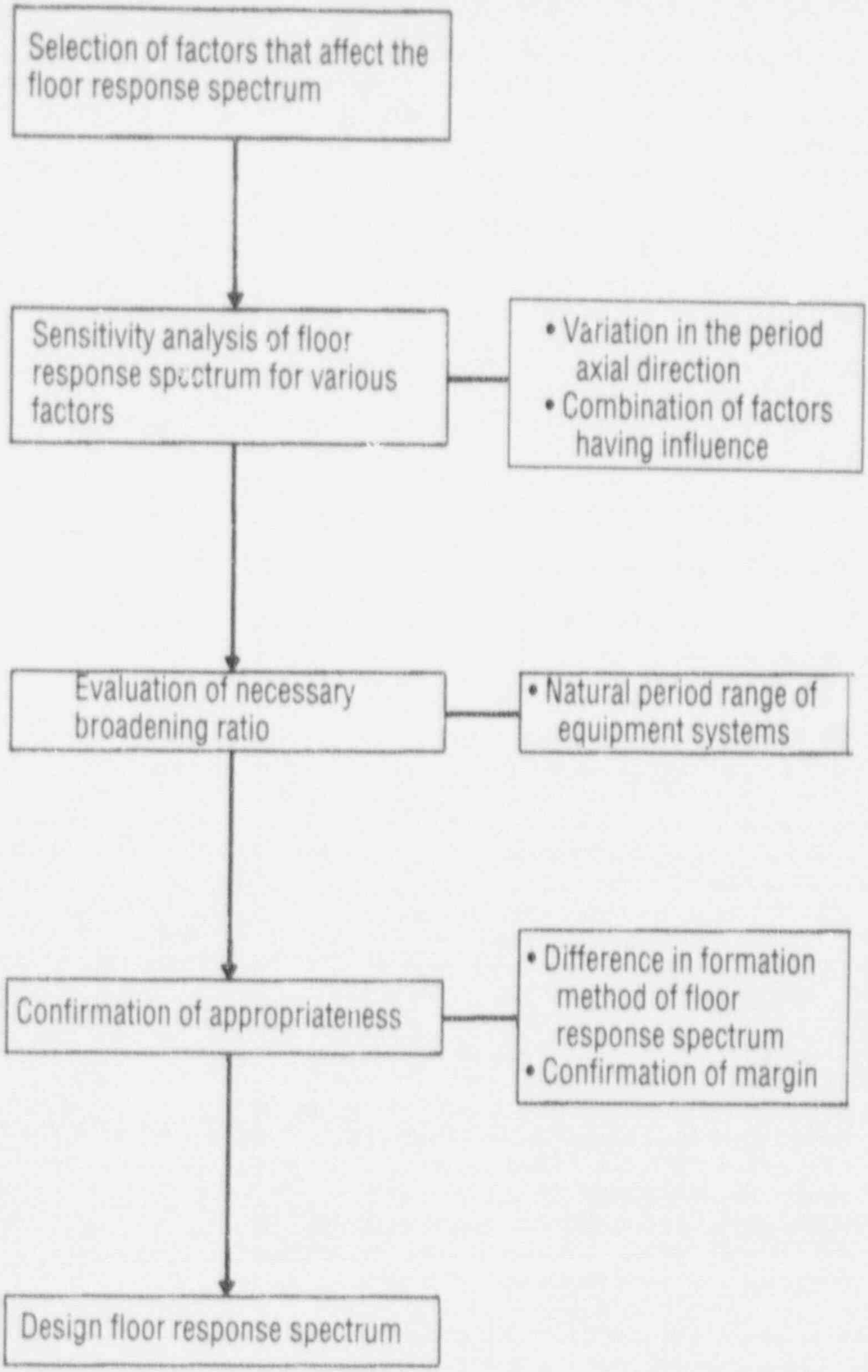


Figure 6.5.1-5. Ideas for forming design floor response spectrum.

As pointed out above, for the important equipment, analysis is performed using an analysis model in which the equipment interacts with the ground/building model. Details of the ground/building analysis model and the analysis method are described in Chapter 5, "Aseismic design of the buildings and structures." Here, we will discuss only the analysis method characteristic of the equipment. In addition, we will discuss guideline for forming models for equipment not interacting with the building, and will present some examples in this respect.

(1) Basic guidelines of formation of models for equipment/piping system

In the following, we will classify the equipment into vessels, piping, and others for a discussion of the basic guideline of the formation of models.

a. Vessels

In principle, the seismic response analysis model of a container is formed as a multiple lamped mass beam model with the following basic principles.

(a) General principles

(i) The model is formed corresponding to its vibration characteristics.

(ii) The discrete masses and nodes are determined in consideration of the major points in stress analysis and to enable appropriate representation of the typical vibration models with attention paid to the mass distribution, stiffness variation, dimensions, etc., of the container.

(iii) As far as the stiffness evaluation of the beam between discrete masses is concerned, in principle, evaluation is performed of flexural, shear, and, if needed, torsional behavior, as well as stiffness in axial direction.

(iv) When the mass of the container is to be evaluated, in addition to the mass of the container itself, the masses of the attached equipment, thermal insulation material, contained fluid, etc., are also evaluated as a discrete mass system or a distributed mass system.

(v) In principle, the values of longitudina. modulus of elasticity and shear modulus of elasticity of the material used for stiffness evaluation of the part are taken as those at the operating temperature of the element when the seismic motion takes place.

(b) Evaluation on interaction with connecting piping

As far as the necessity for considering the interaction between the model of a container and the model of the piping connected to it is concerned, the mass and stiffness of the container are compared with those of the piping connected to the container. If it is determined that the interaction between them can be ignored, they can be separated from each other with a model formed for the container only. In other cases, an interaction model is formed by including piping up to a range with a rather high piping stiffness.

(c) Evaluation on interaction between vessels and building

From the interaction between a vessel and the building/structure supporting it, in the case when the mass and stiffness of the structure that directly supports the vessel are similar to those of the container itself and there exists a mutual influence between them, the two portions are taken as interactive in the model.

(d) Evaluation of support structure

In principle, the support structure of a vessel is represented by springs or elements by evaluating its stiffness.

b. Piping

The model of piping is formed in consideration of the following principles.

(a) General principles

(i) The piping is represented by a three-dimensional model with stiffnesses determined by accounting for flexural, shear, torsional and axial forces.

(ii) When there exist valves or other eccentric weights, the model should enable evaluation of their influences.

(iii) The range contained in a single model is defined as from one anchor point to another anchor point in principle.

(iv) When there exists branched piping, the model formed should enable its influence to be taken into consideration. However, when the diameter of the branched pipe is much smaller than that of the mother pipe and the vibration of the branched pipe has little influence on the mother pipe, this rule does not apply.

(v) The nodal points are set at points where the stress is believed high and they are set with an appropriate interval to enable full representation of the typical vibration modes.

(vi) In principle, the piping support structure is handled with the following boundary conditions.

Restraint: The stiffness in the restraining direction is considered.
Snubber: The stiffness in the restraining direction is considered.
Anchor: The six directions are taken as fixed.
Hanger: Restraint is not considered.

(vii) As far as the mass of the piping is concerned, in addition to the mass of the piping itself, the concentrated masses of valves, and the masses of thermal heat insulator and fluid in the pipes are also taken into consideration as a discrete mass system or a distributed mass system.

(viii) In principle, the values of the longitudinal modulus of elasticity and shear modulus of elasticity used in the stiffness evaluation of piping are taken as those at the temperature of the piping when the seismic motion is applied. However, in order to simplify the design, the following rules for the temperature may also be adopted.

- Temperature of the fluid in the pipe when the seismic motion is applied.
- The highest temperature of the pipe or fluid in the operation state of plant or phenomenon in combination with the seismic force.
- The highest application temperature or the highest temperature in normal operation.

c. Others

For equipment (core internals, support structures, etc.) other than aforementioned a. containers and b. piping, the model is also formed mainly using a multiple discrete mass or single discrete mass beam model.

Depending on its vibration characteristics, the flexural deformation, shear deformation, or both are taken into consideration.

(2) Vessels

As pointed out above, the vessels in the nuclear power facilities can be classified as Type 1 to Type 4 vessels. The Type 3 and Type 4 vessels are mainly the tanks which will be explained later. Here, we will discuss Type 1 and Type 2 vessels which are believed to be the most important and are usually analyzed by accounting for the interaction with other structures.

a. Class 1 vessel

(a) BWR

The nuclear reactor pressure vessel is the only BRW Type 1 vessel, which is illustrated schematically in Figures 6.5.2-8 and 6.5.2-16. Based on the aforementioned basic guidelines, the model of the nuclear reactor pressure vessel is composed of flexural shear beams, with the model interacting with the building and nuclear reactor containment vessel. Figure 6.5.2-3 illustrates an example of the building/nuclear reactor containment vessel interaction analysis model.

(b) PWR

Typical PWR Type 1 vessels include the nuclear reactor vessel, steam generator and primary coolant pump which form the primary cooling equipment, which is illustrated schematically in Figures 6.5.2-11 and 6.5.2-19. Generally speaking, the model is formed on the base of the aforementioned basic guidelines. As a conventional seismic response analysis model, the center of the nuclear reactor container is taken as the fixed end, while the steam generator, primary coolant pump and piping connecting them are represented by a beam/discrete mass system, with the support structure represented by elements or springs. Figure 6.5.2-12 illustrates an example of the model.

Recently, in order to make the analysis more realistic, as shown in Figure 6.5.2-14, the building and the primary cooling equipment are coupled, and the building/equipment interaction model is used for performing the time history analysis.

b. Type 2 vessels

(a) BWR

The Type 2 vessel of BWR is the nuclear reactor containment vessel. Figure 6.5.2-1 illustrates the structure of the BWR nuclear reactor containment vessel. The nuclear reactor containment vessel has a shell structure and is supported by the reactor building through shear lugs. It has been found that the stress due to the ovalization vibration generated in earthquake is much smaller than the stress due to the internal pressure etc.; hence, the model is formed using flexural shear beams according to the aforementioned basic guidelines. Figure 6.5.2-2 illustrates an example of the ovalization vibration. Figure 6.5.2-3 illustrates an example of the analysis model, in which the containment vessel is modeled by bending-shear beams, and the interaction between ground, nuclear reactor building and nuclear reactor pressure vessel is considered. As shown in this figure, the nuclear reactor containment vessel is represented by a model having many discrete masses, which are located at the seam portion, hatch portion, wall-thickness changing portion, inflection point and piping mounting portion of the containment vessel. In this way, the seismic force can be calculated easily. Figure 6.5.2-4 illustrates an example of the eigenvalue analysis results obtained using the analysis model shown in Figure 6.5.2-3.

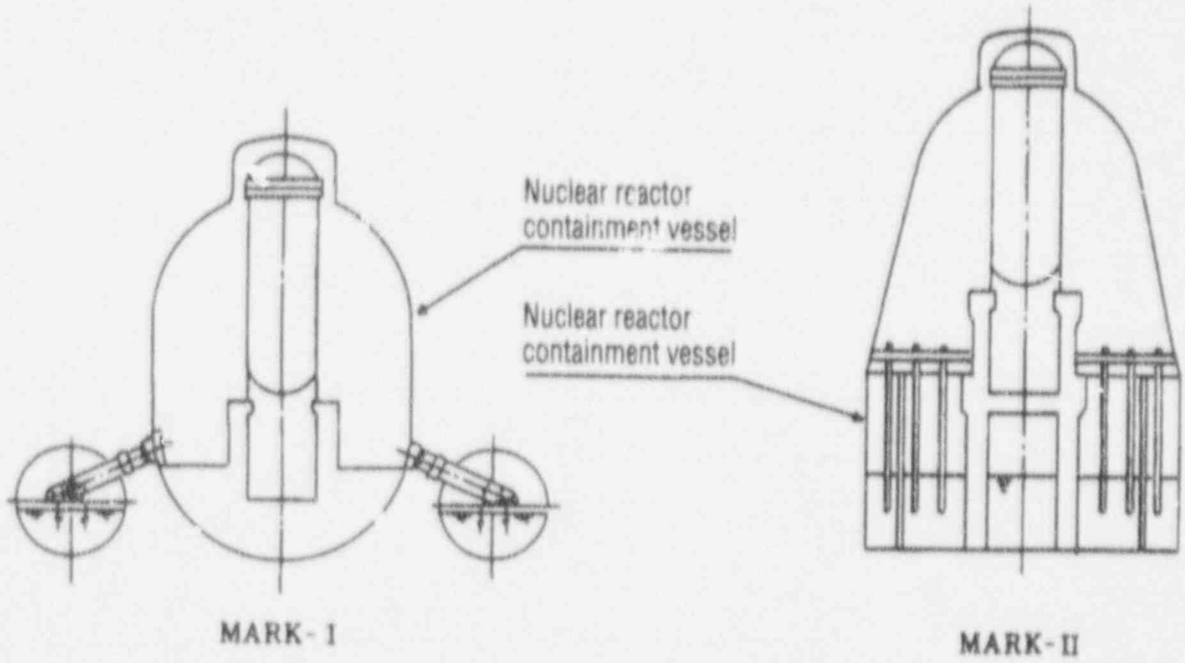


Figure 6.5.2-1. BWR nuclear reactor containment vessel.

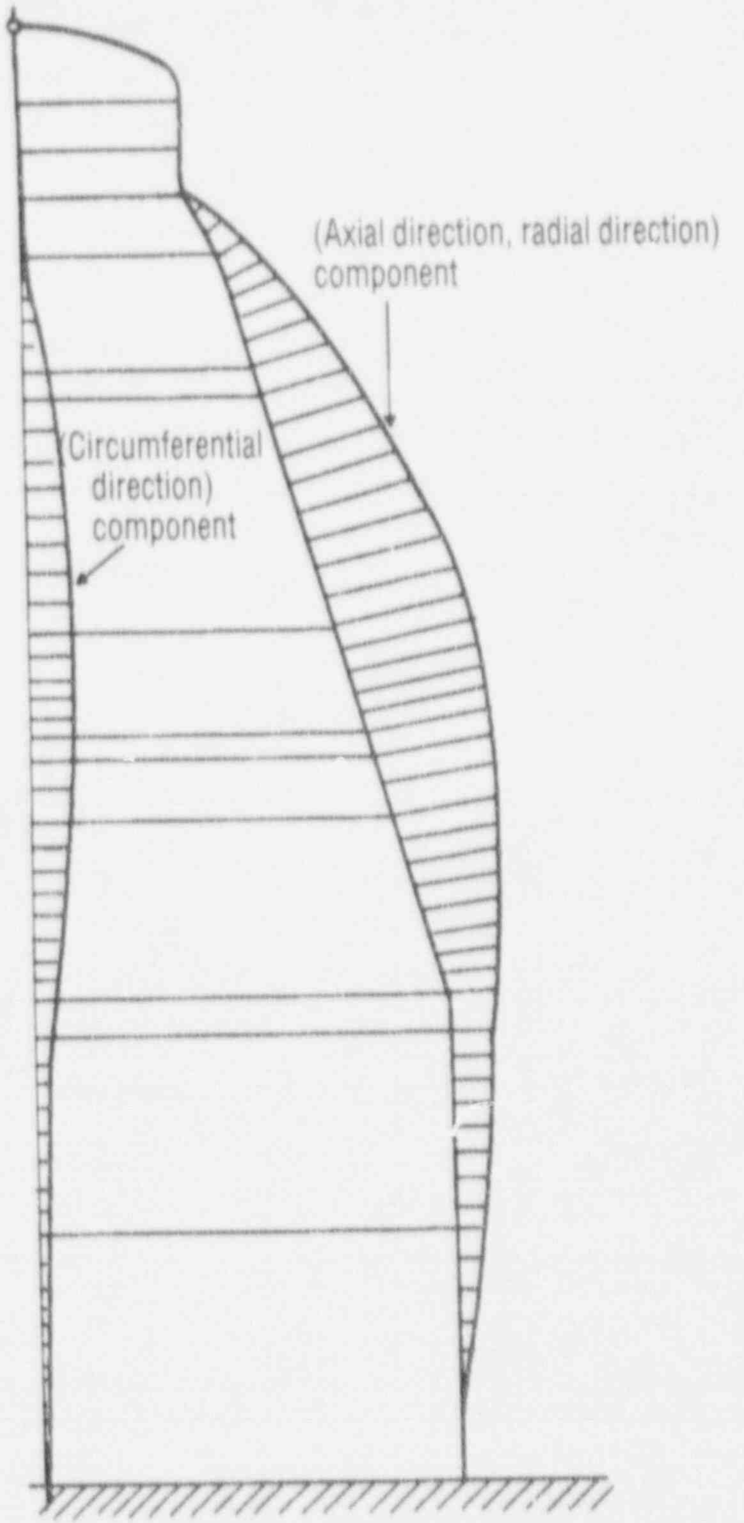


Figure 6.5.2-2. Example of analysis of ovalization vibration of nuclear reactor containment vessel.

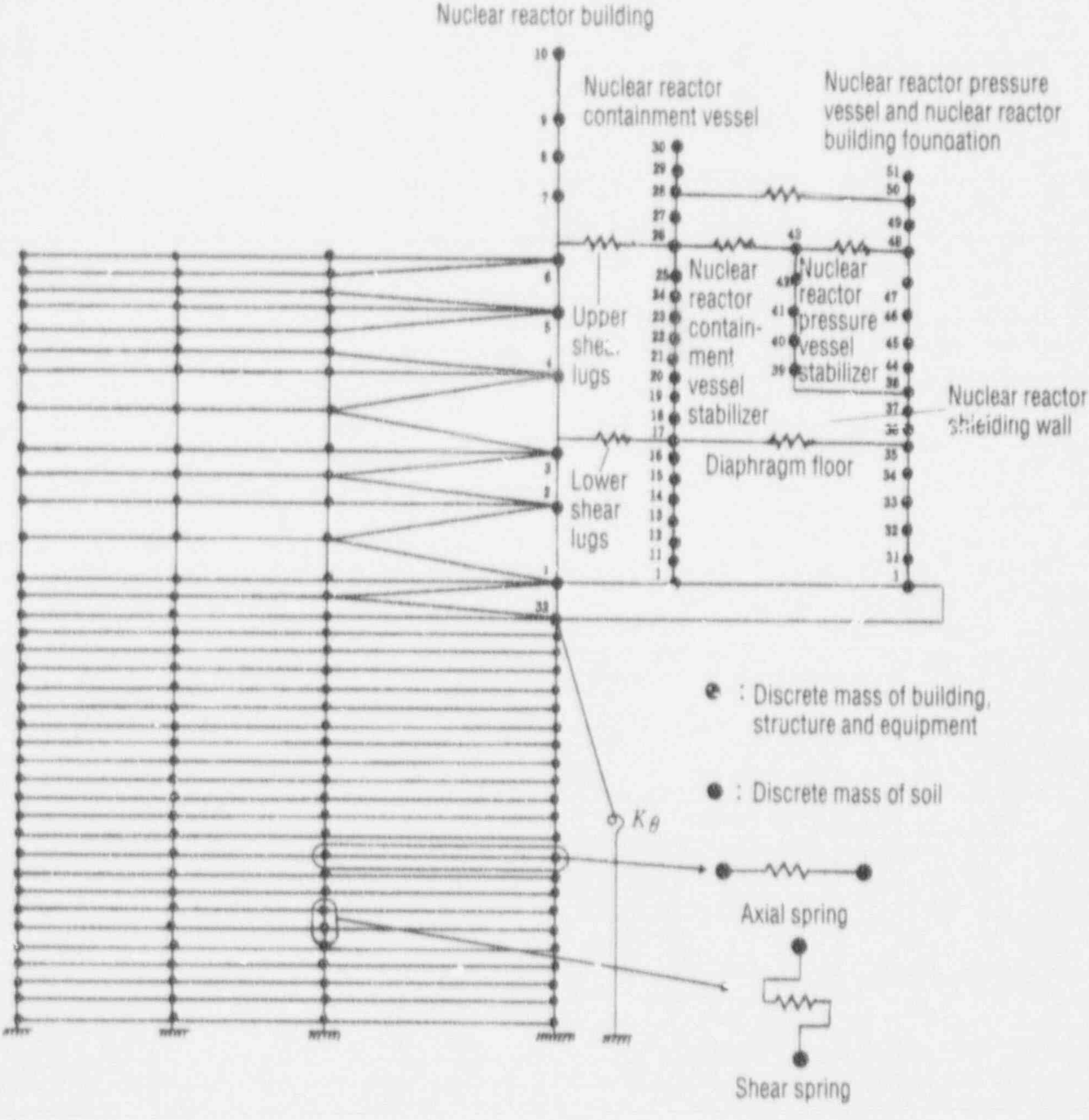
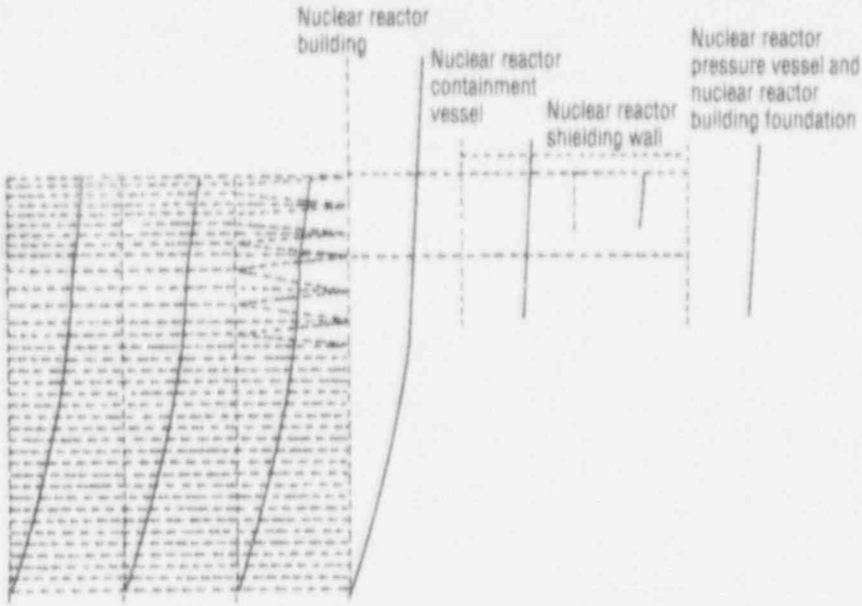
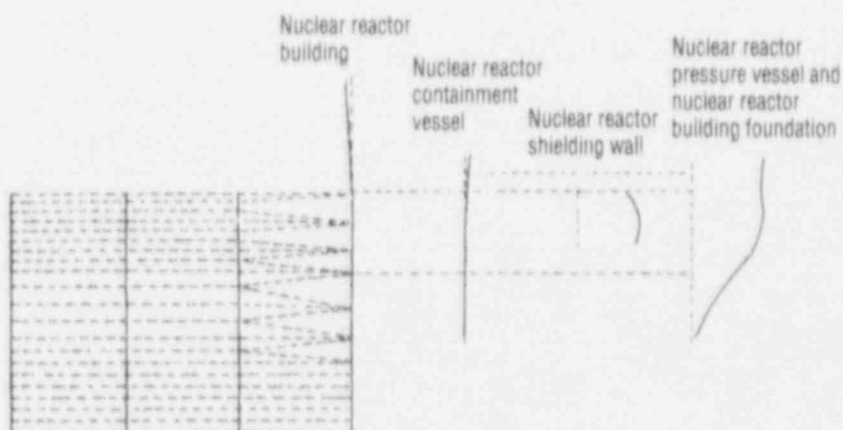


Figure 6.5.2-3. Example of seismic response analysis model of BWR nuclear reactor containment/nuclear reactor pressure vessel.



First vibration mode



19th v bration mode

Figure 6.5.2-4. Example of chara. .c modes of BWR nuclear reactor containment vessel/nuclear reactor pressure vessel interaction model.

As shown in the example, for a BWR, the nuclear reactor containment vessel and the nuclear reactor pressure vessel are directly interactive with the ground and building in the model. That is not because the equipment and building are interactive with each other with respect to their dynamic characteristics; instead, this is because the structure has a large scale in this case and is supported from various points on the building, and a more accurate evaluation can be performed regarding the inputs from the reactor building.

(b) PWR

As an example of the PWR containment vessel, the vibration analysis model of a cylindrical steel containment vessel is to be discussed in this section. For the PCCV (prestressed concrete nuclear reactor containing vessel), please see section 5.1.5(4)c. As shown in Figure 6.5.2-5, the nuclear reactor containment vessel is made of a hemispherical head portion, a cylindrical barrel portion, and a dish-like bottom portion. The lower portion is fixed in the foundation concrete of the nuclear reactor building. Consequently, an interaction model with the nuclear reactor building is used for the time history analysis. Also in this case, the model of the nuclear reactor containment vessel is a multiple discrete mass beam model. As shown in Figure 6.5.2-6, the equivalent stiffness of the model is determined on the bais of the vibration frequencies and modes of the axisymmetric model. Although an ovalization vibration is excited in the nuclear reactor containment vessel during earthquake due to equipment hatch, air lock, and other axially asymmetric masses, it has been found that the stress caused by this ovalization vibration is much smaller than the stress caused by the internal pressure. The ovalization vibration has two series of vibration modes: N (vibration in circumferential direction) and M (beam-type vibration in axial direction). The characteristic vibration frequencies are shown in Figure 6.5.2-7.

(3) Piping

For the piping systems, a three-dimensional model including valves and pumps is used on the base of the aforementioned basic guidelines.

a. BWR [H-K-10]

As an example of the model of the piping system, a model of the primary loop recirculation system piping analysis model will be discussed here. Figure 6.5.2-8 illustrates schematically the BWR circulating system piping. It's model is shown in Figure 6.5.2-9. For the BWR, circulating system's piping model, the discrete mass points are determined at piping support installing points, locations of pumps, valves, etc., points of change in piping diameter, nozzle positions, and other important points for stress evaluation. Also, at the pipebends, evaluation of the stiffness change should be considered; and, for the valves, pumps, etc., equivalent stiffness and mass are evaluated in the model.

As will be explained later in section "6.5.4 Earthquake response analysis method," the piping design is usually performed using the spectral modal analysis method. The examples of the analysis results are as follows. Table 6.5.2-1 and Figure 6.5.2-10 show the input accelerations and vibration modes for the BWR circulation system piping. Table 6.5.2-1 lists the horizontal seismic coefficients and vertical seismic coefficients for each vibration mode, with the horizontal seismic coefficients calculated from the floor response spectrum. Table 6.5.2-2 lists the analysis results of seismic force (SS in the table) with respect to the reaction force and moment. W is the dead load due to the self-weight of the piping.

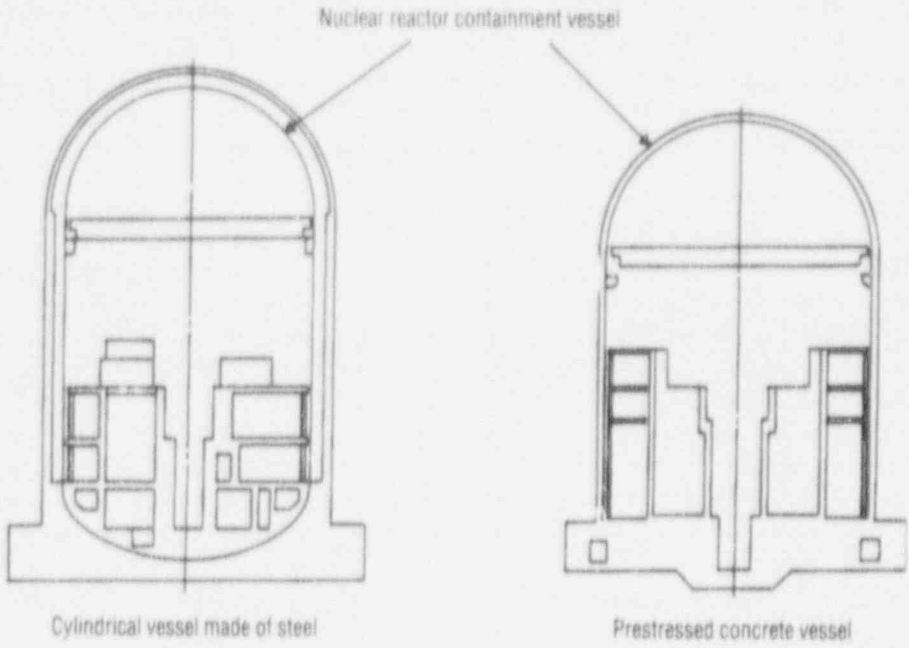


Figure 6.5.2-5. PWR nuclear reactor containment vessel.

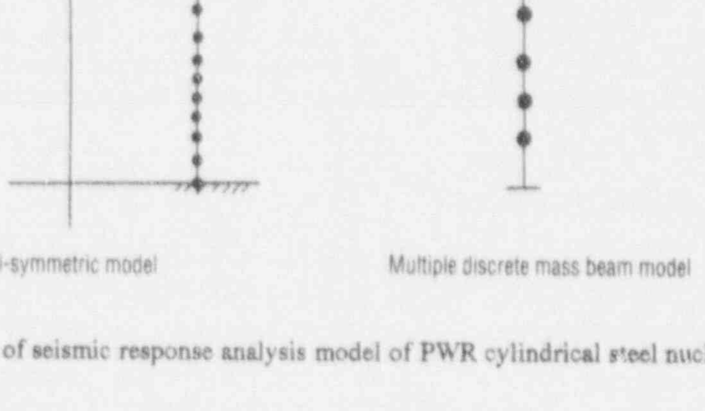


Figure 6.5.2-6. Examp' . of seismic response analysis model of PWR cylindrical steel nuclear reactor containment vessel.

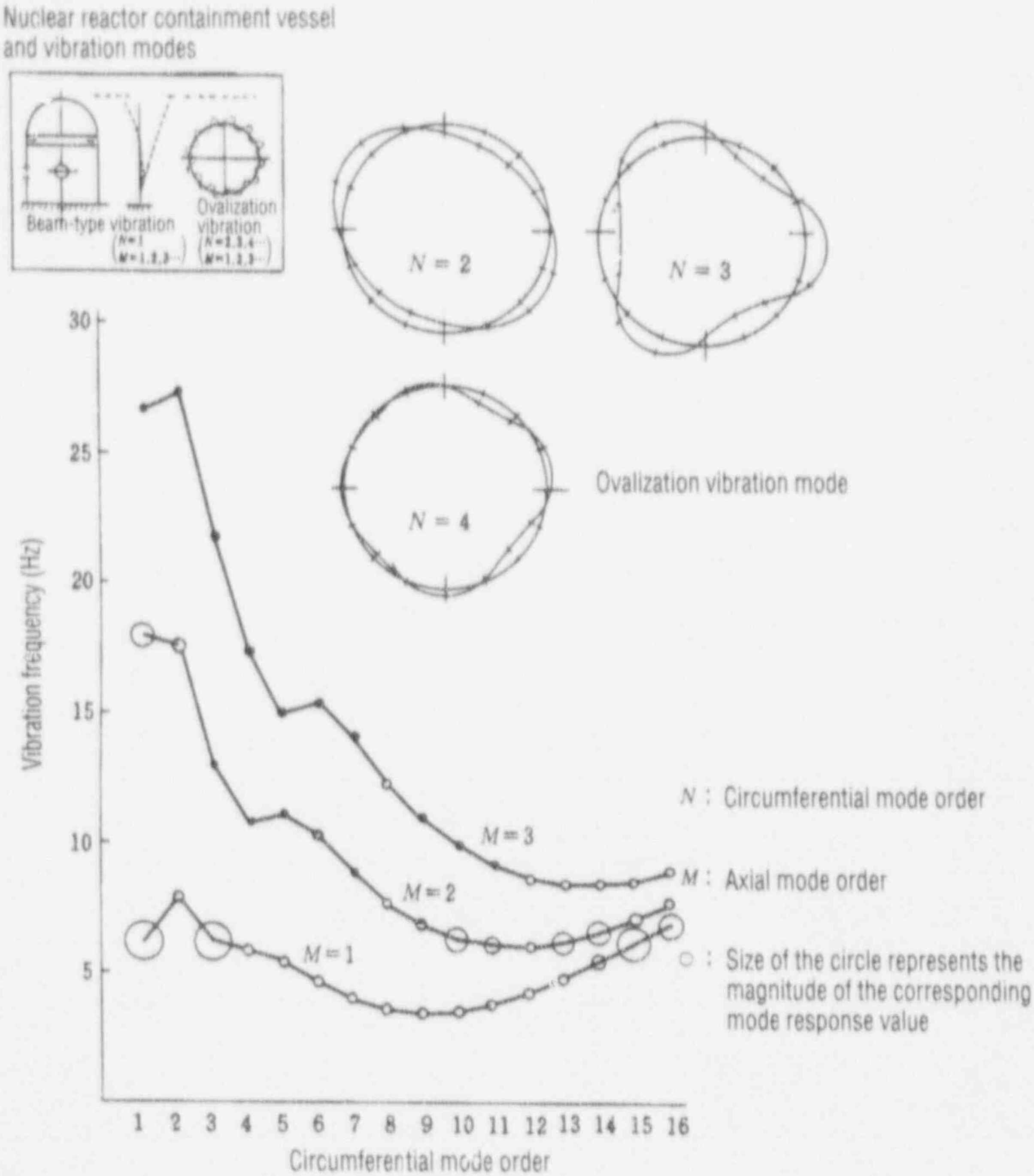


Figure 6.5.2-7. Example of seismic response of ovalization vibration of PWR nuclear reactor containment vessel.

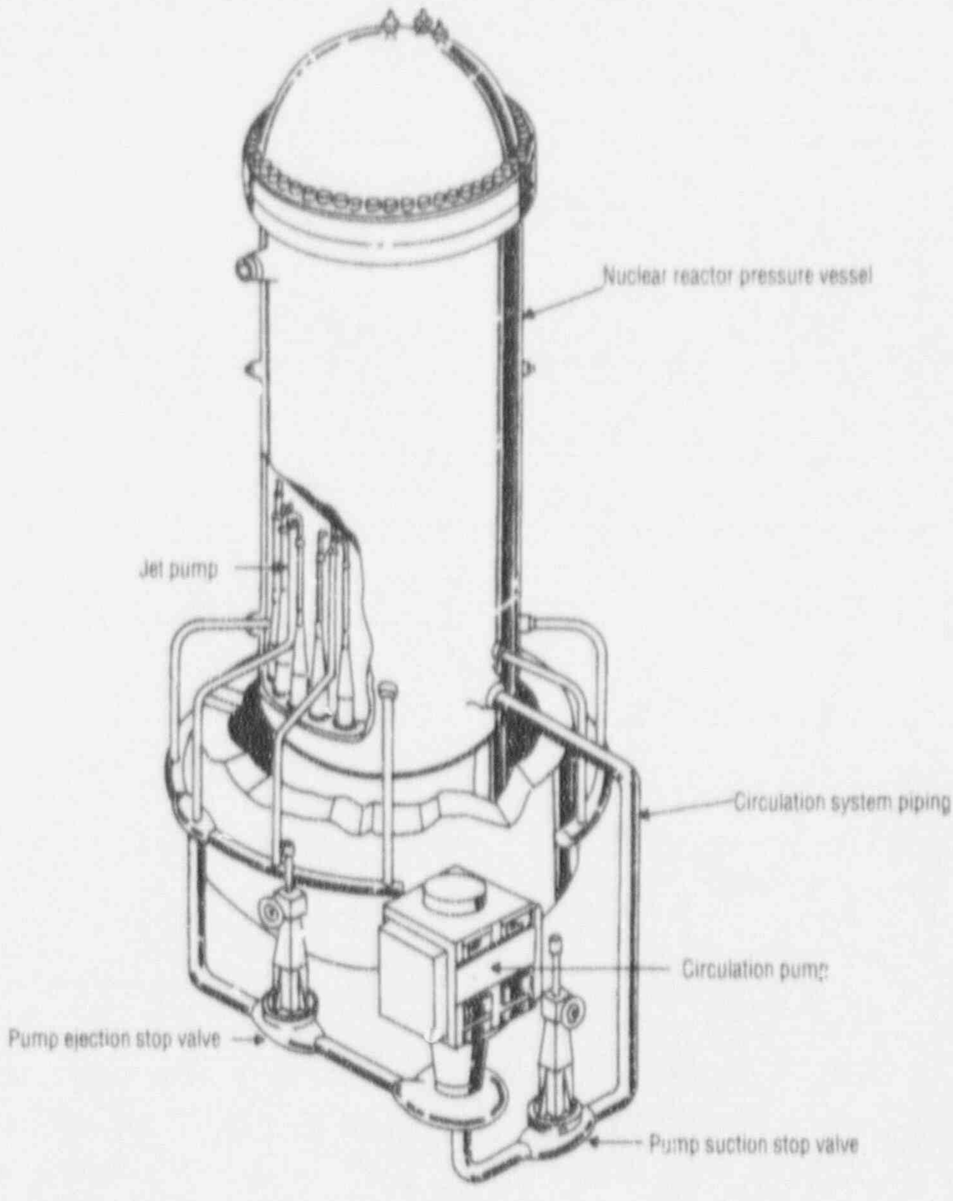


Figure 6.5.2-8. BWR circulation system piping (example).

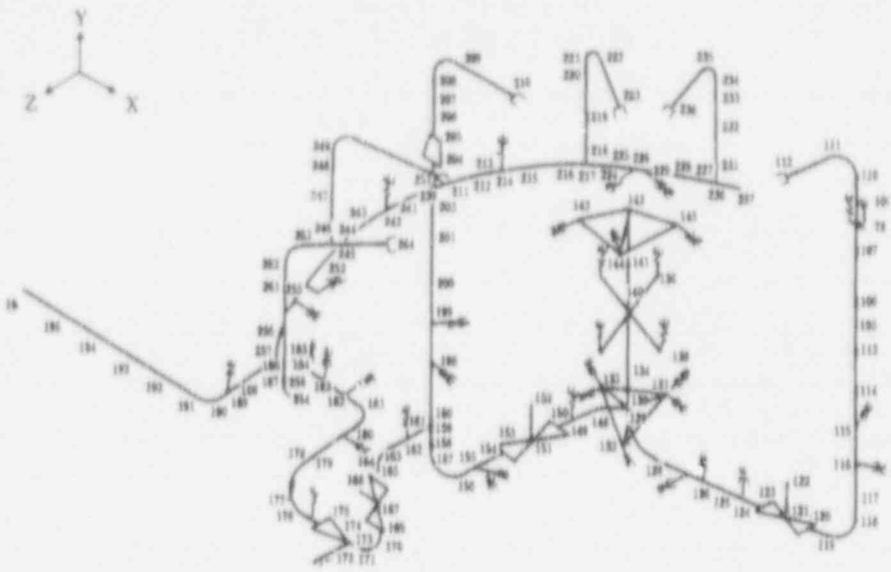


Figure 6.5.2-9. BWR circulation system piping analysis model (example).

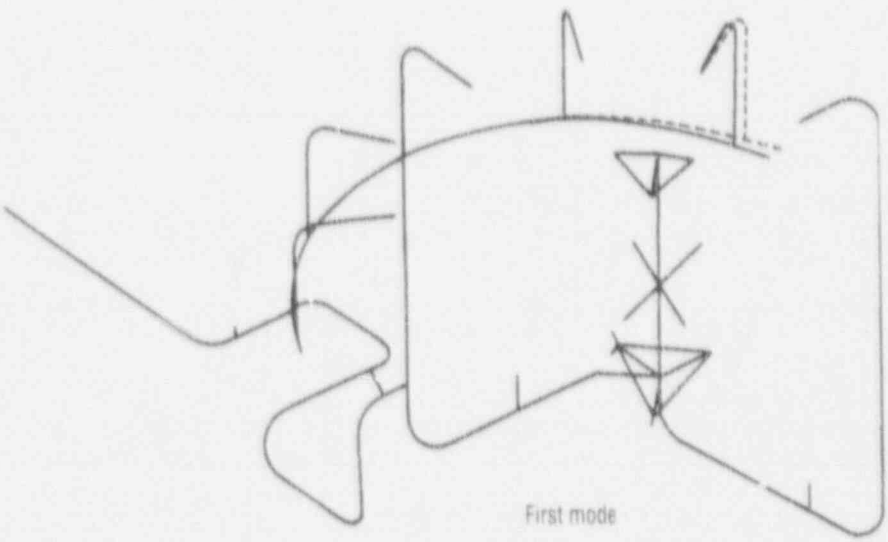


Figure 6.5.2-10. BWR circulation system piping vibration modes.

Table 6.5.2-1. Input acceleration.

Mode	Natural period (s)	Response horizontal acceleration (G)	Vertical seismic coefficient
First	0.0787	0.82	0.16
Second	0.0751	0.74	0.16
Third	0.0608	0.65	0.16
Fourth	0.0511	0.66	0.16
Fifth	0.0481	0.66	0.16

Table 6.5.2-2. BWR circulation system piping seismic response analysis results.

Nodal point no.	Load		Reaction force (kgf)			Moment (kgf·m)		
			X-direction	Y-direction	Z-direction	Around X-axis	Around Y-axis	Around Z-axis
112	W		87.0	898.8	−16.5	459.8	−196.4	237.8
	SS	SXY	147.7	883.5	533.1	1708.4	204.5	146.0
		SYZ	105.4	626.2	378.4	1211.2	147.1	103.9
	W + SS		234.7	1782.3	−549.6	2168.2	−400.9	383.8
210	W		146.5	432.9	64.2	−21.5	90.5	−335.0
	SS	SXY	573.4	186.3	453.5	120.7	659.6	135.7
		SYZ	545.5	178.1	598.8	161.5	869.4	137.1
	W + SS		719.9	619.2	663.0	−183.0	959.9	−472.1
223	W		−22.8	806.4	238.3	433.6	392.1	−807.8
	SS	SXY	170.2	374.8	223.5	91.9	65.1	353.8
		SYZ	219.4	519.8	305.6	127.4	80.7	491.3
	W + SS		−242.2	1326.2	543.9	561.0	472.8	−1299.1
236	W		−51.1	1422.0	171.5	1611.6	276.9	−1001.8
	SS	SXY	436.0	421.6	363.9	210.7	419.3	46.4
		SYZ	596.8	586.3	500.2	279.8	565.8	49.6
	W + SS		−647.9	2008.3	671.7	1891.4	842.7	−1051.4
251	W		−12.7	851.8	−206.0	−497.3	−346.0	−868.6
	SS	SXY	118.5	106.1	162.2	16.2	261.8	175.5
		SYZ	112.7	141.8	212.7	13.4	326.9	2141.0
	W + SS		−131.2	993.6	−418.7	−313.5	−672.9	−1082.7
264	W		−39.4	1452.0	−149.8	−1674.2	−243.3	−1027.8
	SS	SXY	106.0	171.5	107.3	301.4	184.5	82.9
		SYZ	145.2	178.2	105.5	306.4	249.6	79.9
	W + SS		−184.6	1630.2	−257.1	−1980.6	−492.9	−1110.7

Note—W: self-weight; SXY: earthquake in X-direction; SYZ: earthquake in Z-direction; SS: seismic coefficient

b. PWR

As an example of the model of the PWR piping, we will discuss the model of the primary coolant equipment. Figure 6.5.2-11 shows schematically the PWR primary coolant equipment. Figure 6.5.2-12 shows the model of this equipment. As can be seen from Figure 6.5.2-12, the seismic analysis of the PWR primary coolant equipment is performed using a three-dimensional model. It consists of primary coolant pipe, steam generator, primary coolant pump, and support structure, with the primary coolant pipe having its fixed end at the center of the nuclear reactor vessel. For the internal structure of the equipment, the weight is distributed in the various discrete mass points, and the stiffness refers to the stiffness of the container barrel, pump casing, motor stand, etc. Depending on the specific shape, the support structure's model is formed by equivalent beams or spring elements. Just as in the BWR case, the vibration modes of the PWR primary cooling equipment are illustrated in Figure 6.5.2-13. Table 6.5.2-3 illustrates the response acceleration at the various nodal points obtained from the analysis results.

(4) Other equipment

In the above, we have discussed several important equipment and piping items with analysis performed in consideration of their interaction with the building. However, in the nuclear power plant, there are many other important machines/equipment in addition to these vessels and piping, such as the core internals, fuel assembly, heat exchanger, pump, tank, etc. In the following, we will discuss the models and analysis guidelines for these machines/equipment.

a. Core internals and fuel assembly

(a) BWR [6.5.2-1], [6.5.2-2], [6.5.2-3]

Figure 6.5.2-16 shows the configuration of the BWR nuclear reactor pressure vessel and core internals. As can be seen from Figure 6.5.2-16, the interior of the nuclear reactor pressure vessel consists of reactor core fuel assembly, reactor core support structure, control rods, control rod guide tube, etc. For the fuel assembly that forms the reactor core, each group includes four pieces, with their upper portion supported by the upper grid plate and with their lower portion supported by the reactor core support plate and control rod guide tubes. The upper grid plate and reactor core support plate are supported by a shroud that surrounds the fuel assembly. The shroud is supported by a baffle plate and shroud support legs in the nuclear reactor pressure vessel. The control rods can be inserted into the reactor core or removed from it through the control rod tube at the lower portion of the reactor core. Each control rod is connected to the driving mechanism main body via a coupling mechanism. On the other hand, the main body of the driving mechanism is contained in the control rod driving mechanism housing welded to the bottom portion of the nuclear reactor pressure vessel. The control rod drive is further connected to the accumulator of the scram system as the power source through the CRD piping system.

The seismic analysis of the nuclear reactor pressure vessel and core internals is usually performed using the so-called multiple discrete mass beam model, according to which the stiffness is evaluated by the bending-shear beam model and the weight is taken as concentrated mass in a discrete mass point system. Also, with the same purpose as the containment vessel, an interactive with the building is considered. Figure 6.5.2-17 shows an example of the seismic response analysis model of the nuclear reactor building–nuclear reactor pressure vessel–core internal coupled system. Since the fuel assembly that forms the reactor core, shroud, etc., are in water, the increase in the virtual mass is taken into consideration when the model is to be formed. As an example of the model in consideration of the effect of vibration in water, the model of the fuel assembly is described. In the seismic response analysis, the fuel assembly is taken as elastic beams which are simply supported by the upper grid plate at the upper end and by the reactor core support plate and control rod guide tubes at the lower end.

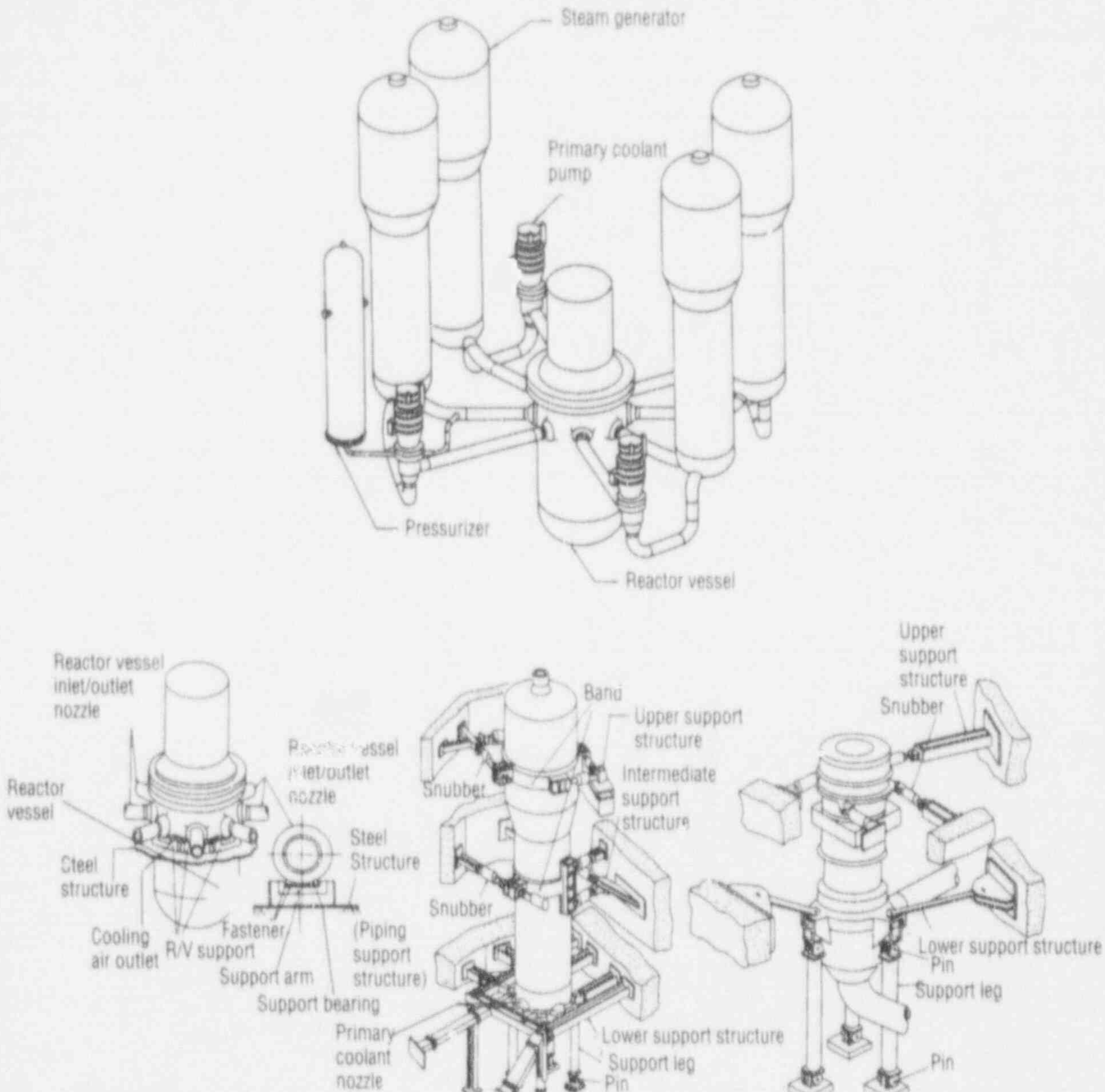


Figure 6.5.2-11. Schematic of PWR primary coolant equipment.

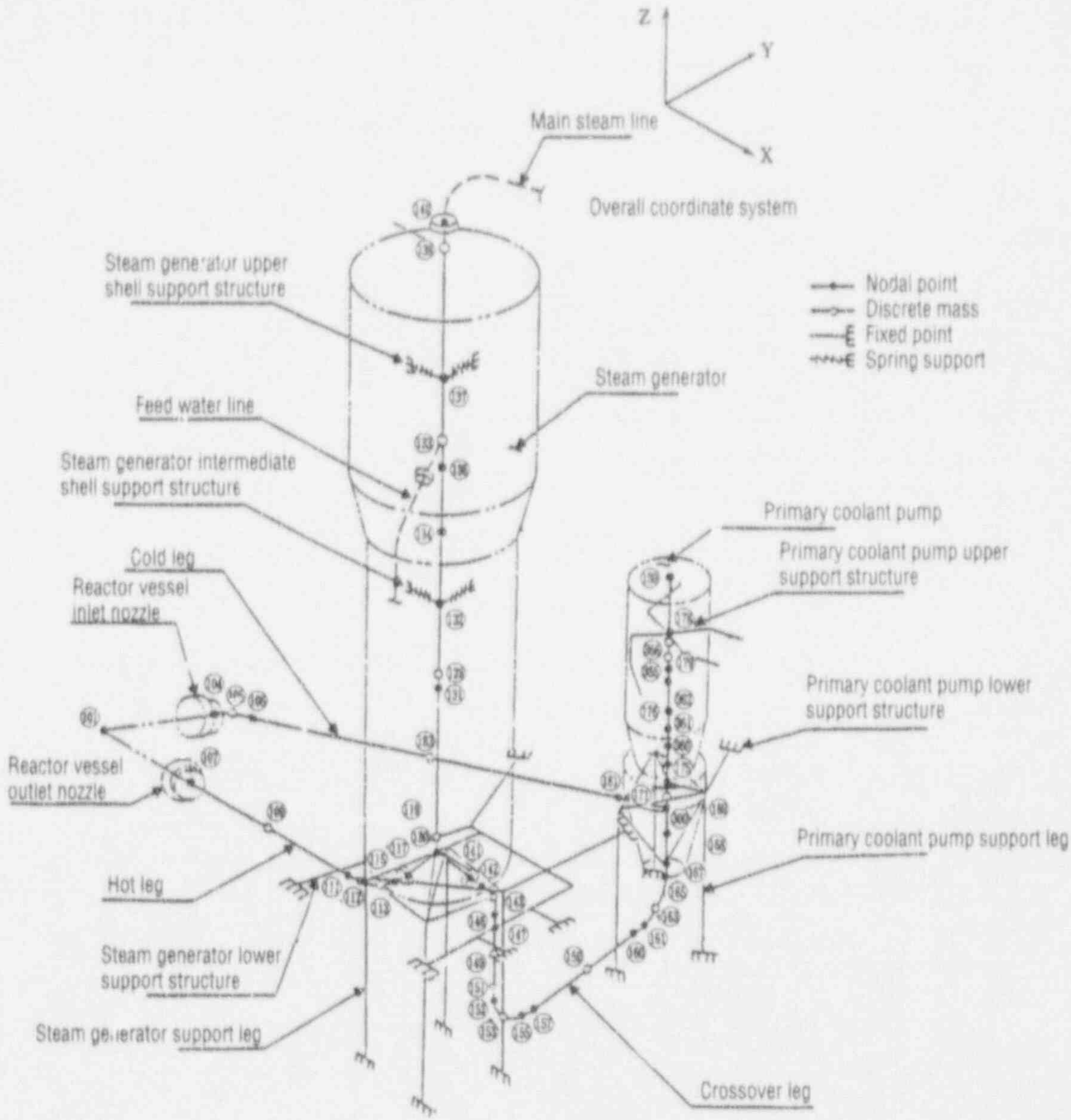


Figure 6.5.2-12. Example of seismic response analysis model of PWR primary cooling equipment.

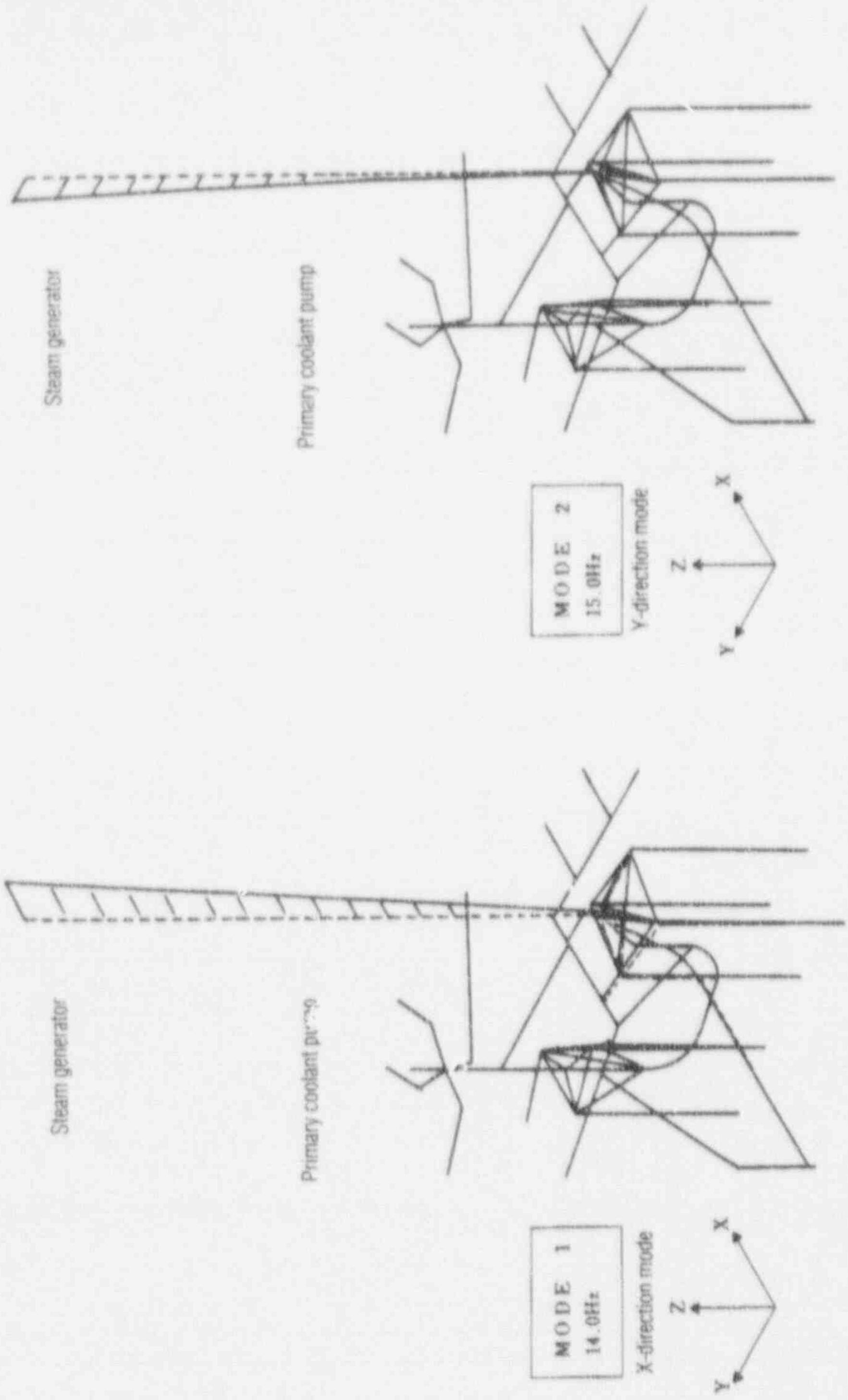


Figure 6.5.2-13. Example of vibration modes of PWR primary coolant equipment.

Table 6.5.2-3. Examples of response acceleration of PWR primary coolant equipment.

(During S_1 earthquake, units: G)

Nodal point no.	X-direction earthquake			Y-direction earthquake		
	X-direction acceleration	Y-direction acceleration	Z-direction acceleration	X-direction acceleration	Y-direction acceleration	Z-direction acceleration
109	0.12	0.11	0.22	0.07	0.22	0.10
113	0.29	0.30	0.34	0.18	0.53	0.14
119	0.75	0.33	0.07	0.32	0.65	0.05
129	1.65	0.41	0.07	0.43	1.10	0.06
133	2.69	0.07	0.08	0.10	1.99	0.07
139	3.73	0.40	0.03	0.53	3.14	0.07
143	0.41	0.30	0.15	0.27	0.54	0.21
149	0.51	0.52	0.16	0.51	0.85	0.22
153	0.91	0.90	0.18	1.02	1.40	0.25
159	0.83	0.95	0.20	1.00	1.47	0.24
163	0.53	1.01	0.22	0.69	1.55	0.16
169	0.35	0.63	0.24	0.52	1.03	0.19
366	1.21	0.42	0.28	1.31	2.09	0.21
179	1.05	0.35	0.27	1.16	1.80	0.20
183	0.46	1.01	0.31	0.67	1.50	0.35
189	0.01	0.02	0.01	0.02	0.03	0.01

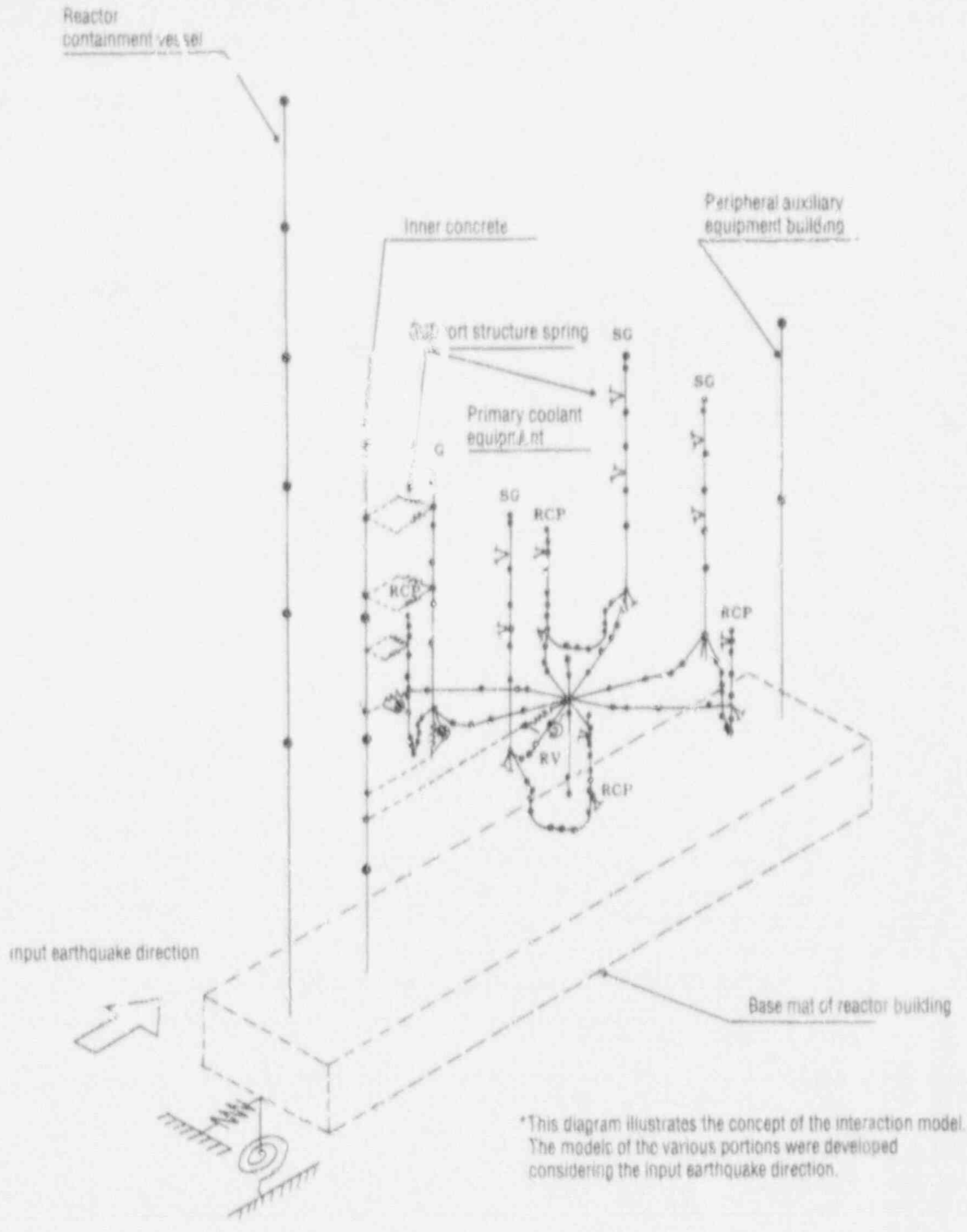


Figure 6.5.2-14. Example of PWR coolant equipment/building interaction model.

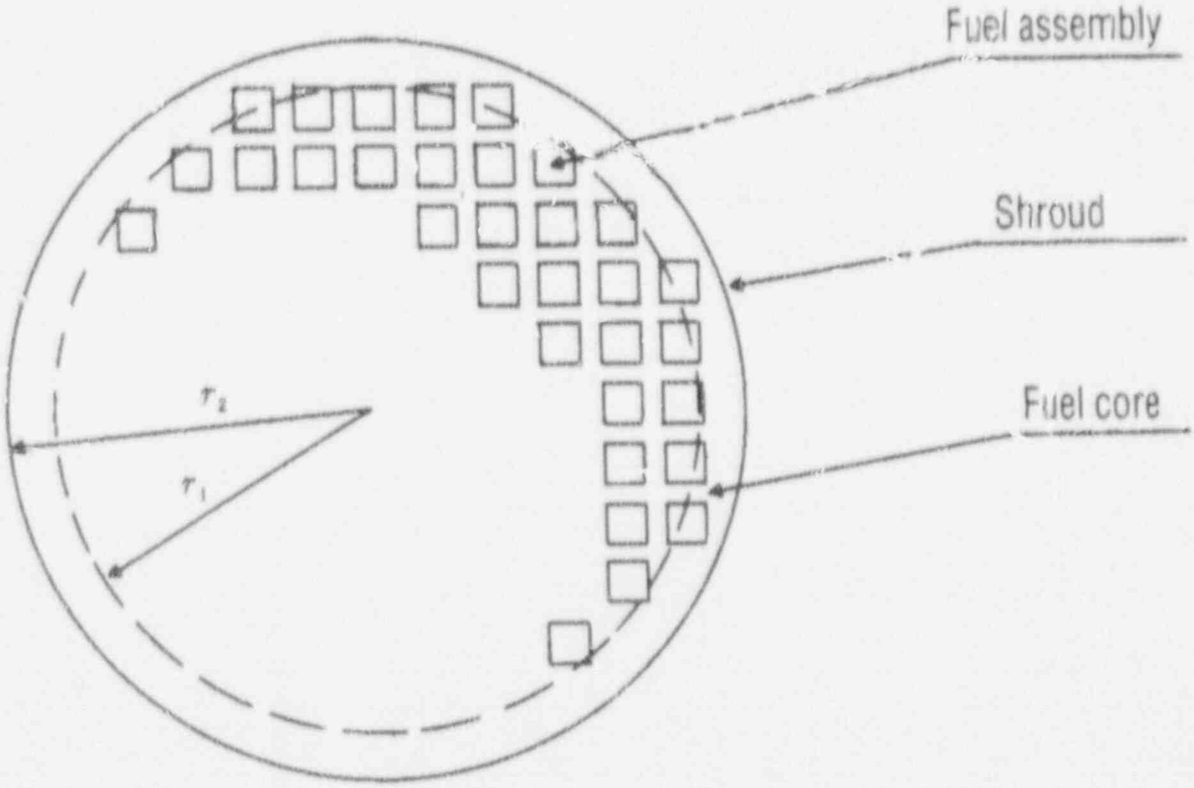


Figure 6.5.2-15. Cross-sectional view of shroud case.

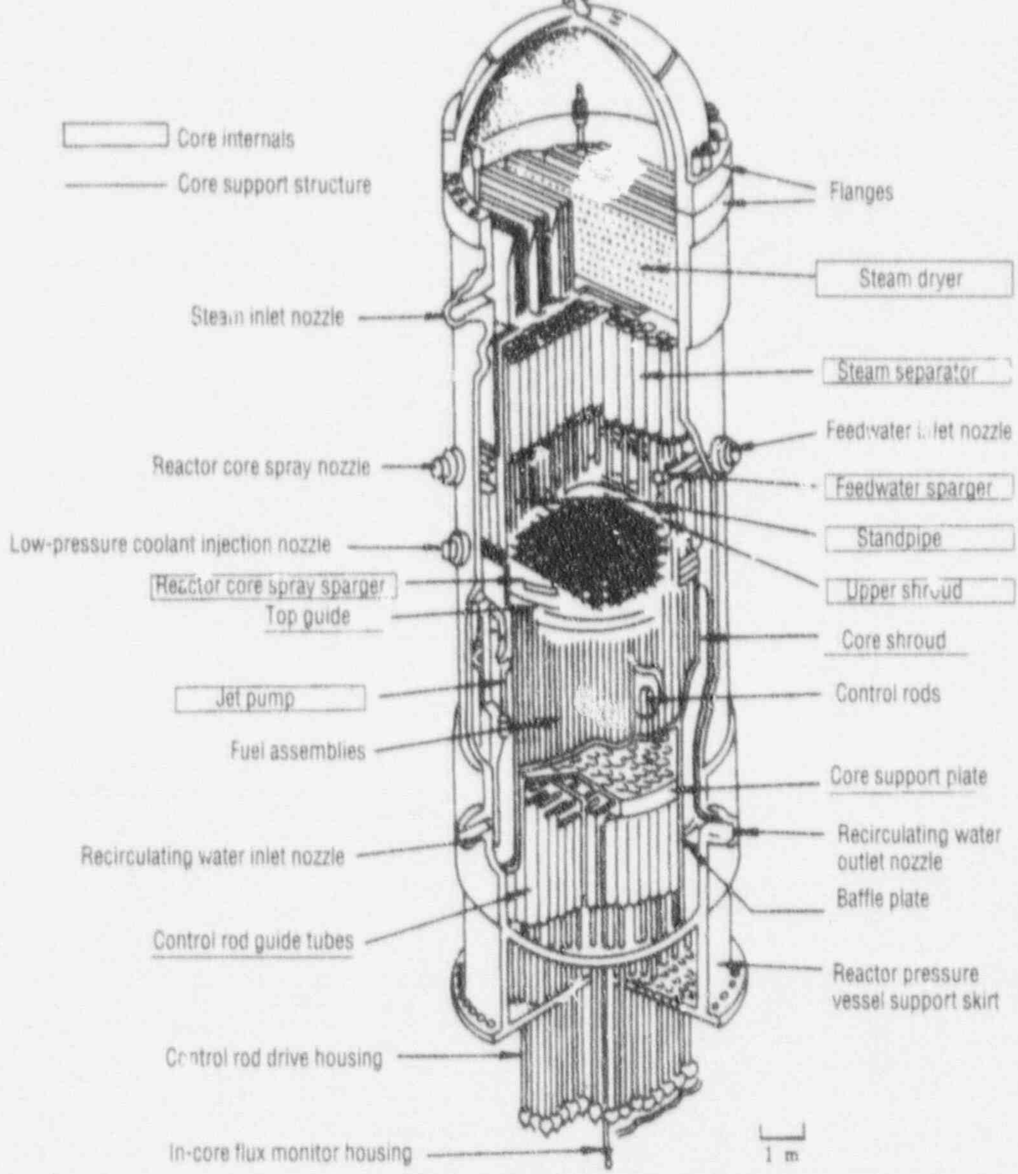


Figure 6.5.2-16. Schematic diagram of internal structure of nuclear reactor pressure vessel (BWR, as an example).

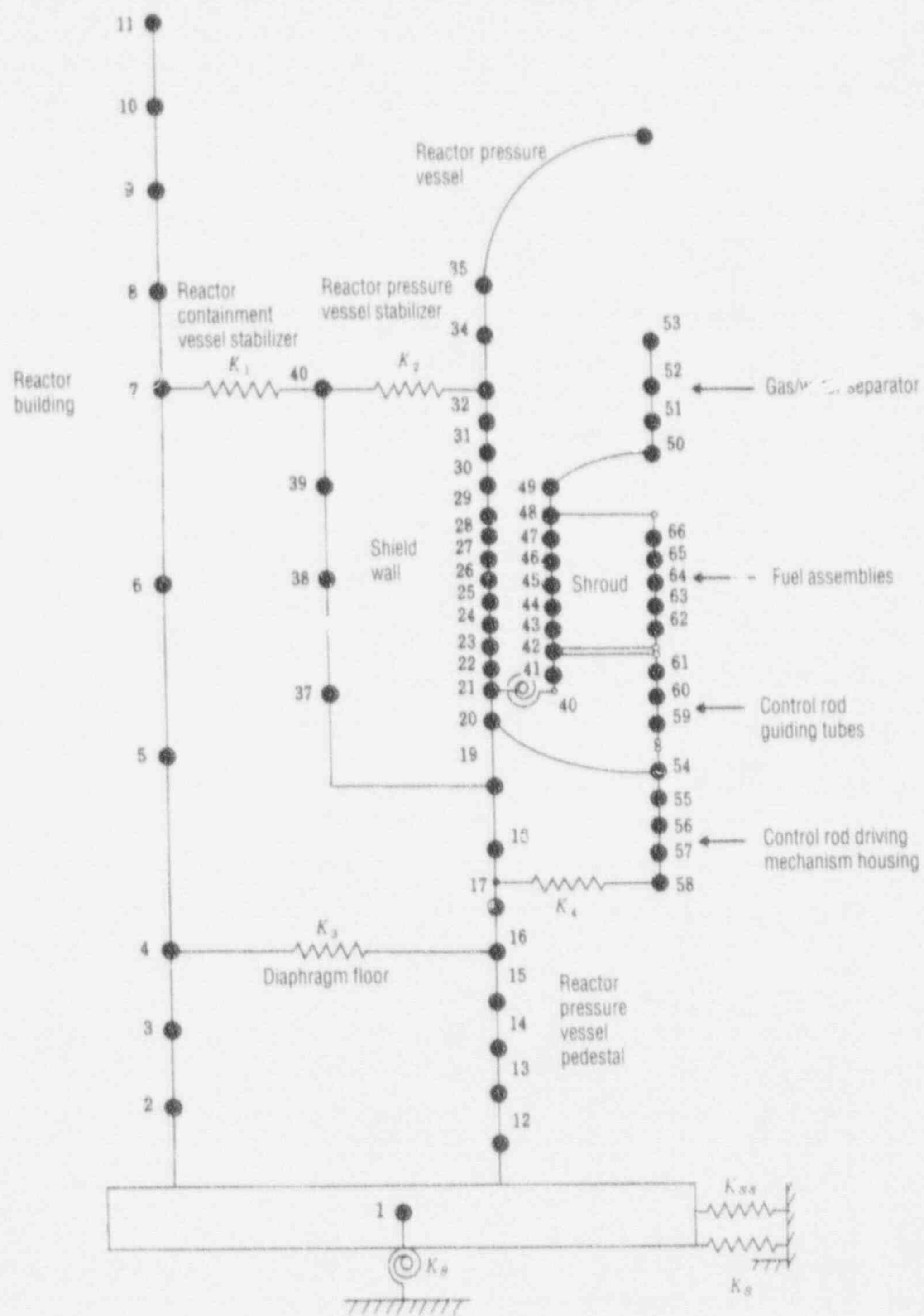


Figure 6.5. 17. Example of seismic response analysis model of BWR nuclear reactor pressure container/structures in reactor.

Although the actual reactor core is an assembly of several hundred fuel assemblies, since all of the fuel assemblies vibrate in the same phase, the reactor core can be represented by a model composed of a single elastic beam. The relation between the fuel assembly and shroud can be taken as an equivalent double cylinder as shown in Figure 6.5.2-15. The inertial force of the mass matrix due only to water can be represented as follows. In the actual design model, the pressure vessel is further added to the aforementioned model, and the additional mass matrix is formed for a three-layer cylinders, and the overall equation of motion is formed by adding the mass matrix to the structural weight. Figure 6.5.2-18 illustrates an example of the vibration mode of the model formed in this way (Figure 6.5.2-17).

$$\begin{pmatrix} M^V & -M^V - M^D \\ -M^V - M^D & M^V + 2M^D + M^F \end{pmatrix} \begin{Bmatrix} \ddot{y}_1 \\ \ddot{y}_2 \end{Bmatrix} \qquad (6.5.2\text{-}1)$$

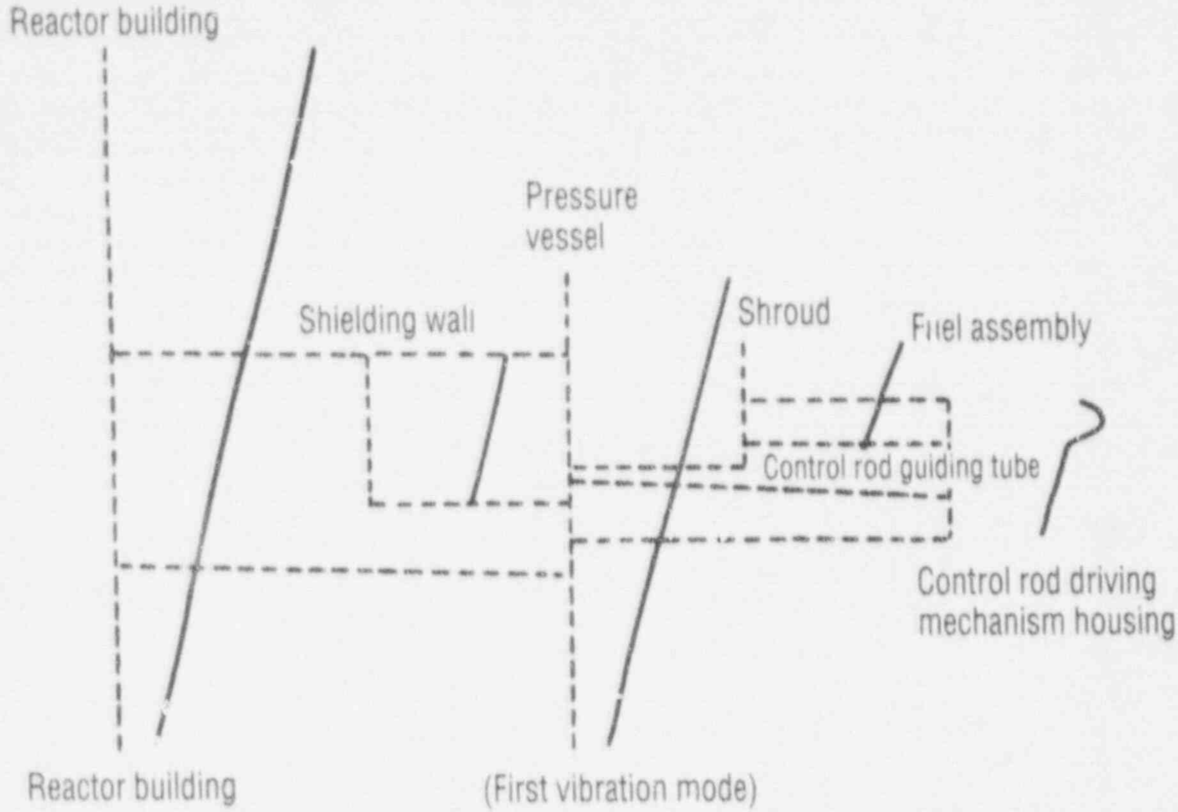
M^V: virtual mass
M^D: excluded mass
M^F: mass of water between cylinders
$\ddot{Y}_1$: displacement of fuel core
$\ddot{Y}_2$: displacement of shroud
r_1: equivalent radius of fuel assembly
r_2: radius of shroud

(b) PWR

The PWR nuclear reactor main body is composed of the reactor vessel, internal structures, and fuel assembly. For the reactor vessel, the nozzle lower portion is supported by internal concrete via a support structure. Figure 6.5.2-19 shows the configuration of the reactor vessel and internal structures. The corresponding analysis model is shown in Figure 6.5.2-20.

The internal structures can be divided into upper core internal and lower core internal. The former structures include upper support plate, upper support column, upper core plate, control rod cluster guide tubes, etc. The latter structures include core barrel, lower core plate, lower core support plate, lower core support column, core baffle, etc. The fuel assembly that forms the reactor core has its upper portion supported by the upper core plate and its lower portion supported by the lower core plate. The upper core plate is supported by upper core support column and upper core support plate. The lower core plate is supported by lower core support column, lower core support plate and core barrel. The core barrel and the upper core support plate are supported by reactor vessel flange portions. The control rods can be inserted into or removed from the core through control rod cluster guiding tubes at the upper portion of the core. Each control rod is connected to the driving unit via a coupling mechanism. On the other hand, the driving unit is fixed to the nuclear reactor containment's upper portion. The control rod driving unit uses a magnetic jack method. That is, during the operating state, the control rods are held above the core portion; during an earthquake, since the power source of the control rod driving unit is cut off, the control rods fall by their own weight.

For the analysis of the internal structure, spectral modal analysis is usually performed using the floor response spectrum at the mounting position of the nuclear reactor support structure. As will be explained later, for analysis of the fuel assembly, nonlinear analysis should be performed. As a result, according to the model shown in Figure 6.5.2-20, the floor response time history at the internal concrete is used to perform the time history analysis. In this way, the floor response time histories are obtained for the upper and lower core plates used as the supports of the fuel assembly. The results are used as the inputs to the fuel assembly in the response analysis performed separately. Figure 6.5.2-21 shows an example of the vibration modes of the reactor vessel and internal structures.



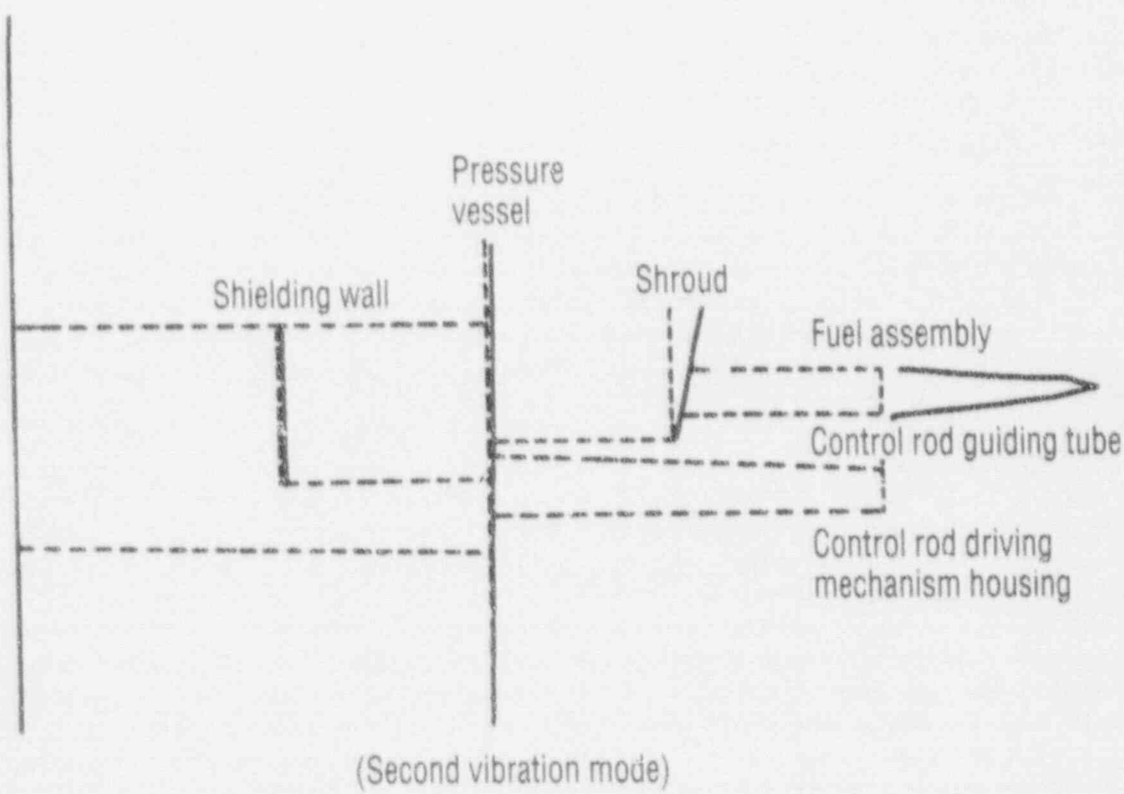


Figure 6.5.2-18. Example of characteristic vibration mode of BWR nuclear reactor pressure vessel/core internals.

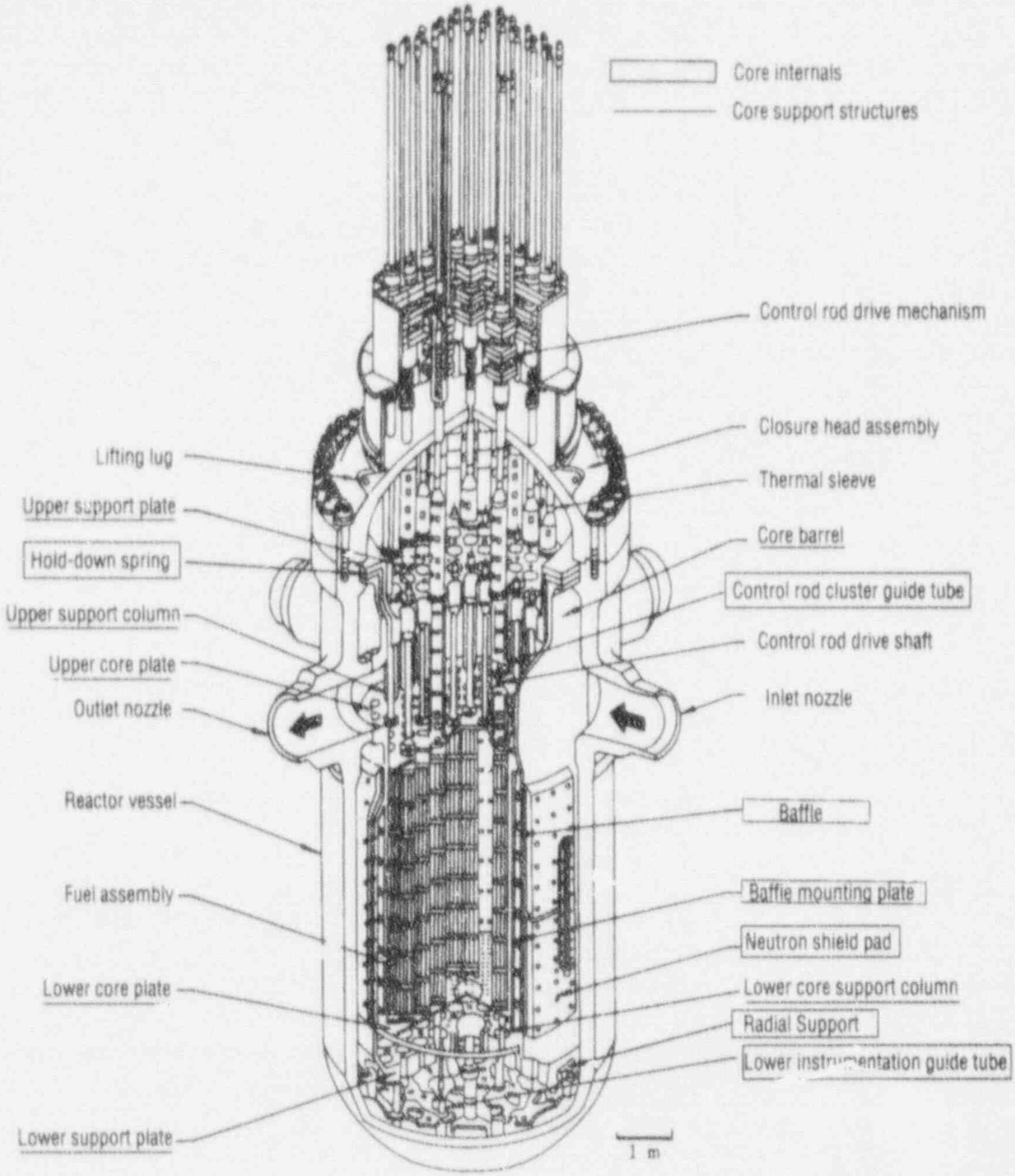


Figure 6.5.2-19. Schematic diagram of structures in nuclear reactor containment (PWR, as an example).

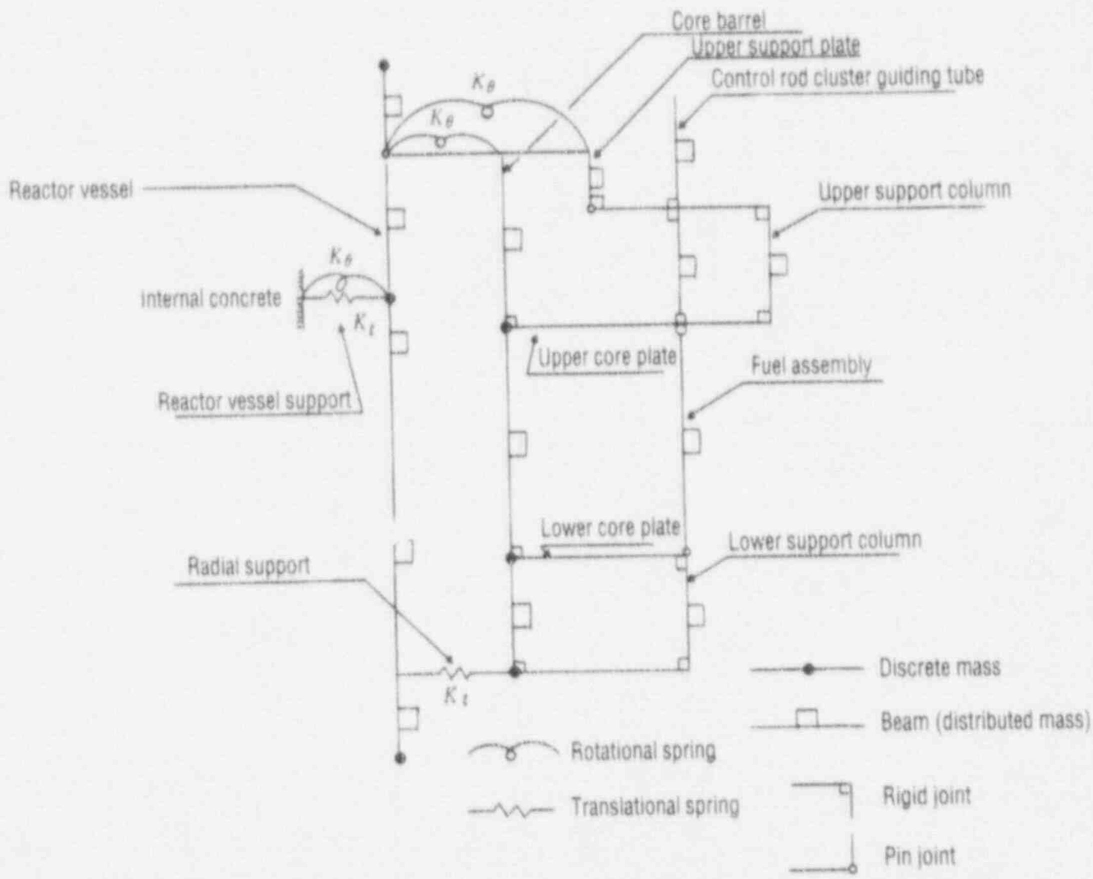


Figure 6.5.2-20. Example of seismic response analysis model of PWR nuclear reactor vessel/internal structures.

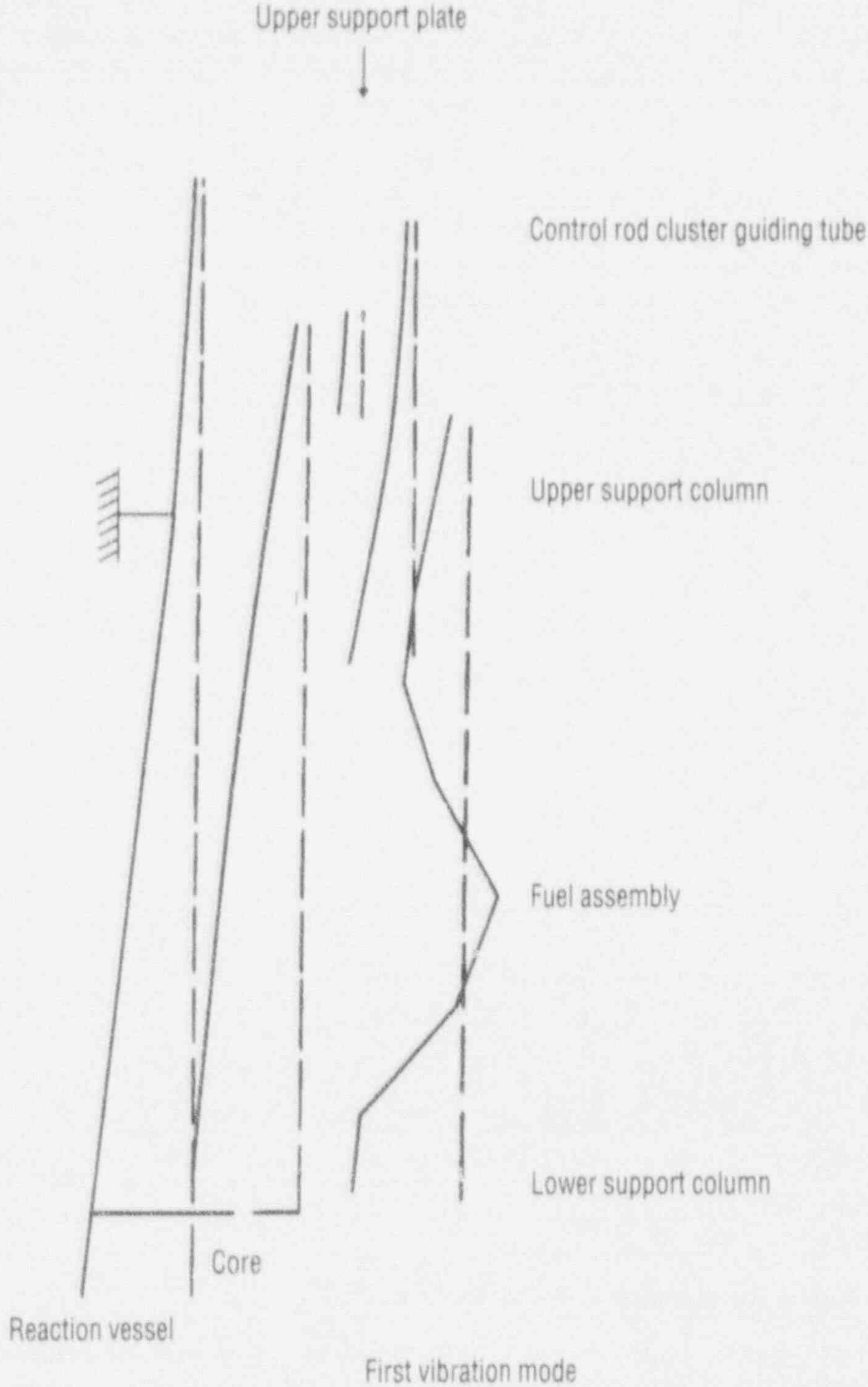


Figure 6.5.2-21. Example of vibration models of PWR nuclear reactor vessel/internal structures.

For the fuel assembly, in the core, the upper and lower nozzles are supported by upper and lower core plates separated from each other by a distance of δ. However, in the case when the vibration amplitude of the fuel assembly is larger than δ during earthquake, collision would take place between the baffle plate and the support grid of the assembly, and the collisions propagates to the adjacent assemblies in sequence, with a complicated nonlinear group vibration response pattern.

As shown in Figure 6.5.2-22, in the analysis model, among the fuel assembly group arranged in the core vessel, one row with the largest rod number is selected (for example, 15 rods in the case of 3 LOOP), a group vibration analysis model with impact spring and energy absorbing element set at the assembly support grid position where the collision takes place is used to perform nonlinear analysis using two-point input at the upper and lower core plates.

The vibration equation is shown as Equations (6.5.2-2) and (6.5.2-3). As far as the effect of water is concerned, the mass matrix is formed with the displaced mass accounted for in terms of the added mass.

$$M\ddot{X} + C\dot{X} + KX = -M\ddot{X}_0 + F \qquad (6.5.2\text{-}2)$$

$$X = \begin{Bmatrix} x_1 \\ \theta_1 \\ x_2 \\ \theta_2 \\ \vdots \\ x_n \\ \theta_n \end{Bmatrix} \quad X_0 = \begin{Bmatrix} x_{01} \\ \theta_0 \\ x_{02} \\ \theta_0 \\ \vdots \\ x_0 n \\ \theta_0 \end{Bmatrix} \quad F = \begin{Bmatrix} f_1 \\ 0 \\ f_2 \\ 0 \\ \vdots \\ f_n \\ 0 \end{Bmatrix} \qquad (6.5.2\text{-}3)$$

where X_0: Absolute displacement of water vessel

X_i: Relative translational displacement with respect to water vessel at nodal point i

θ_i: Relative rotational displacement with respect to water vessel at nodal point i

n: Total number of nodal points

f_i: Impact force or support reaction force

M: Mass matrix

K: Stiffness matrix

C: Viscosity matrix

b. Vertical pumps, heat exchanger [H-K-10], etc.

The heat exchanger, pumps, and tanks used as nuclear power generation equipment have various different forms depending on vertical, horizontal type and installation method. Among this equipment, we will discuss the following items:

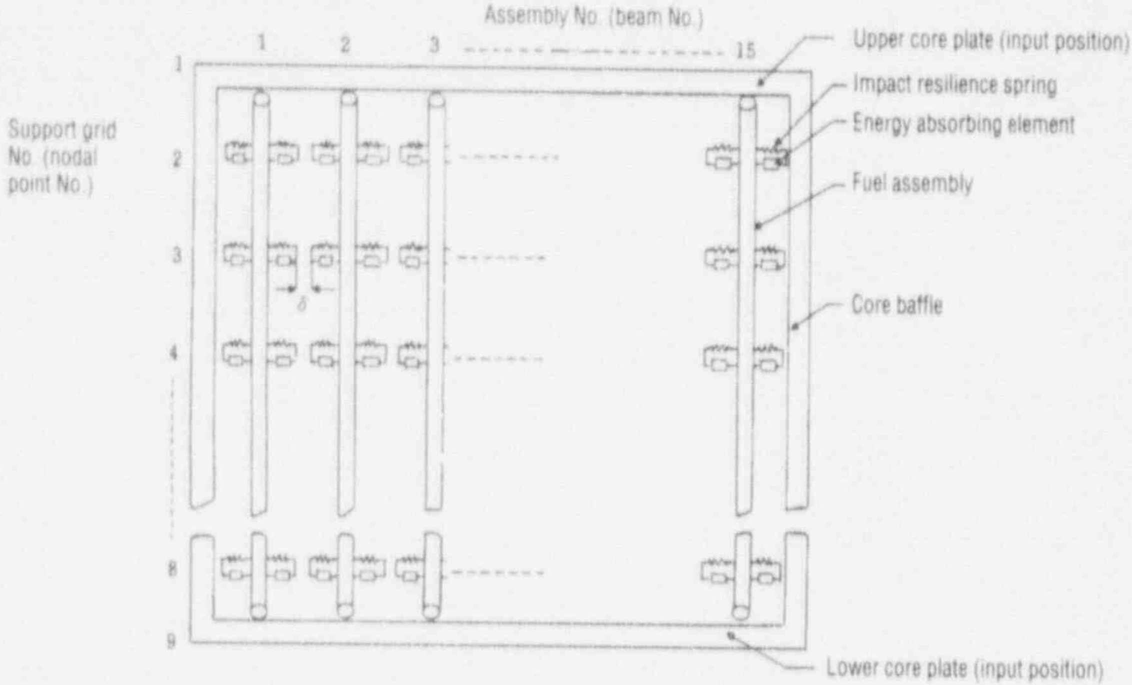


Figure 6.5.2-22. Example of vibration analysis model of PWR fuel assembly group.

(a) Vertical cylindrical tanks with legs
(b) Horizontal heat exchanger
(c) Vertical pumps
(d) Emergency diesel main body and generator, cable tray, electric board, duct, and tanks needed for evaluation of sloshing.

(a) Veritcal cylindrical tank

The modeling and calculation of natural period are performed according to the following guidelines.

(i) Assumption

To obtain the deformation mode, since the mounting positions of legs deviate from the center of gravity, the legs' flexural and shear deformation (Type A) and the overall flexural and shear deformation of the entire tank considered as a beam (Type B) are considered (see Figure 6.5.2-23).

(ii) Calculation model

According to the assumption in (i), the tank is taken as a single discrete mass vibration system with a fixed lower end. However, when the anchor bolts of the legs form one line (viewing from the direction perpendicular to horizontal force F) for each leg, the lower end of the leg is considered as simply supported.

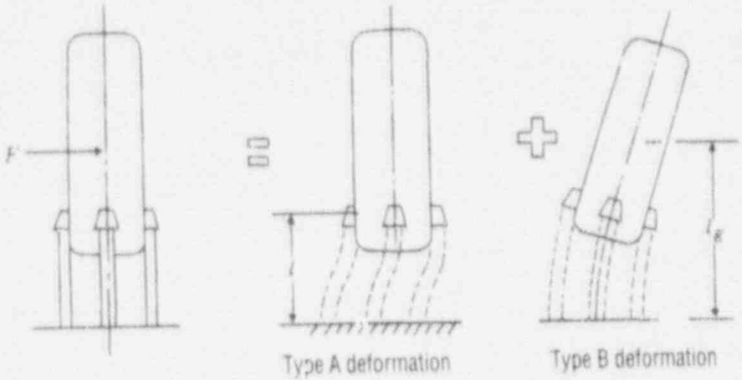


Figure 6.5.2-23. Deformation modes.

(b) Horizontal heat exchanger (see Figures 6.5.2-24 through 6.5.2-30)

(i) Assumptions

{1} The heat exchanger is considered as in operating state.
{2} The barrel of the heat exchanger is taken as rigid.
{3} The first leg is mounted on the foundation by anchor bolts. The mounting portion is taken as fixed.
{4} For the heat exchanger which takes thermal deformation into consideration, since the second leg can slide in the longitudinal direction, all the force in this direction is assumed to act on the first leg; for the forces other than the aforementioned, it is taken as fixed.

(ii) Calculation model

According to assumptions {2} and {3} in (i), the heat exchanger is considered as a single discrete mass vibration system with fixed lower end as shown in Figures 6.5.2-29 and 6.5.2-30.

For the heat exchanger as described in {4} above with a second leg able to slide in the longitudinal direction, evaluation is made only for the first leg (see Figure 6.5.2-27).

(c) Vertical pump [K-K-8]

Figures 6.5.2-31 and 6.5.2-32 illustrate the analysis model of a vertical pump. The model used is a one-dimensional model which ignores the interaction effect in the two horizontal directions at the bearing portion, etc. As far as the pump structure is concerned, motor casing, column, barrel, shaft, etc., are taken as elastic beams with flexural deformation and shear deformation taken into consideration; for the impeller, only the mass is taken into consideration, while the gyro effect and rotational inertia are ignored. For the in-water bearing, the model is formed as an equivalent spring; the motor bearing is taken as a spring in the model with the spring constant determined with reference to the experimental value and ball-and-roller bearing theory. The mounting flange portion is taken as a rotational spring with its stiffness taken into consideration. The water within the barrel casing is evaluated as apparent mass.

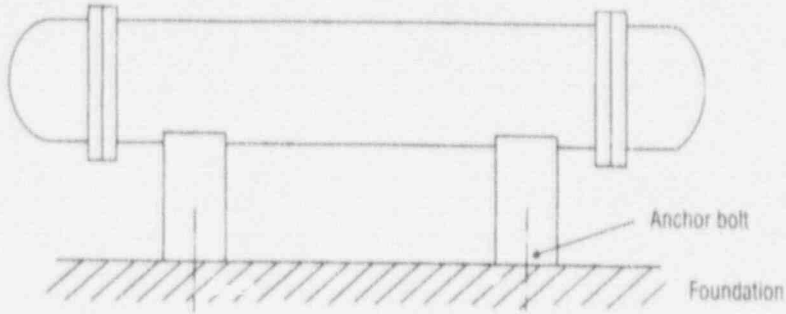


Figure 6.5.2-24. Schematic diagram of structure.

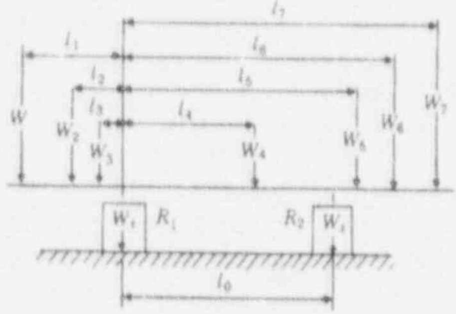


Figure 6.5.2-25. Load state.

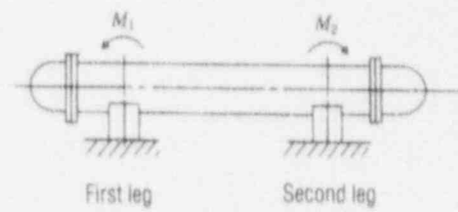


Figure 6.5.2-26. Bending movement at leg position.

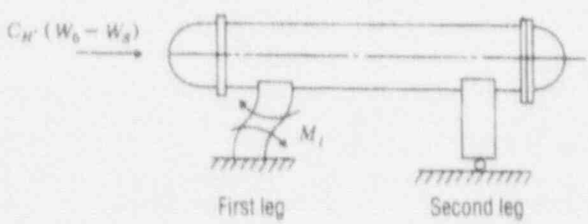


Figure 6.5.2-27. Local moment due to load in the longitudinal direction acting on the barrel.

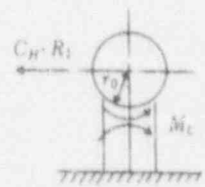


Figure 6.5.2-28. Local moment due to load in the transverse direction acting on the barrel.

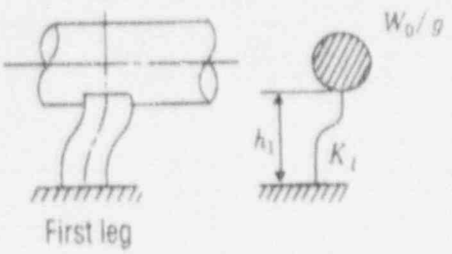


Figure 6.5.2-29. Calculation model of natural period in the longitudinal direction.

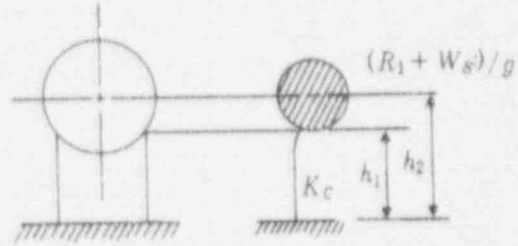


Figure 6.5.2-30. Calculation model of natural period in the transverse direction.

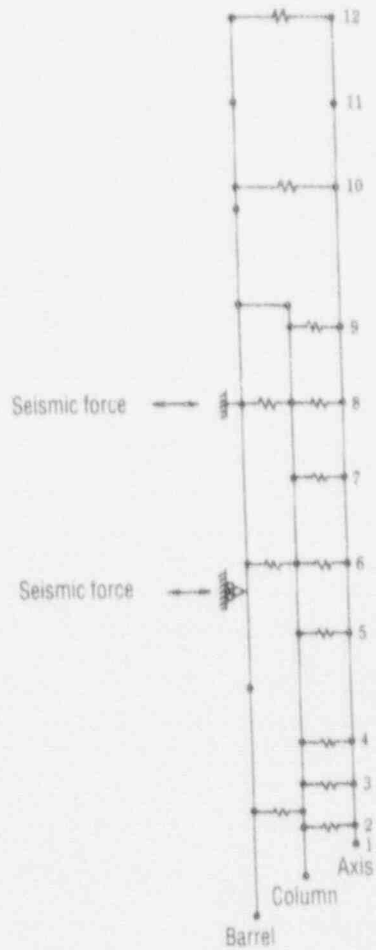


Figure 6.5.2-31. Example of analytical model of a multiple discrete mass system.

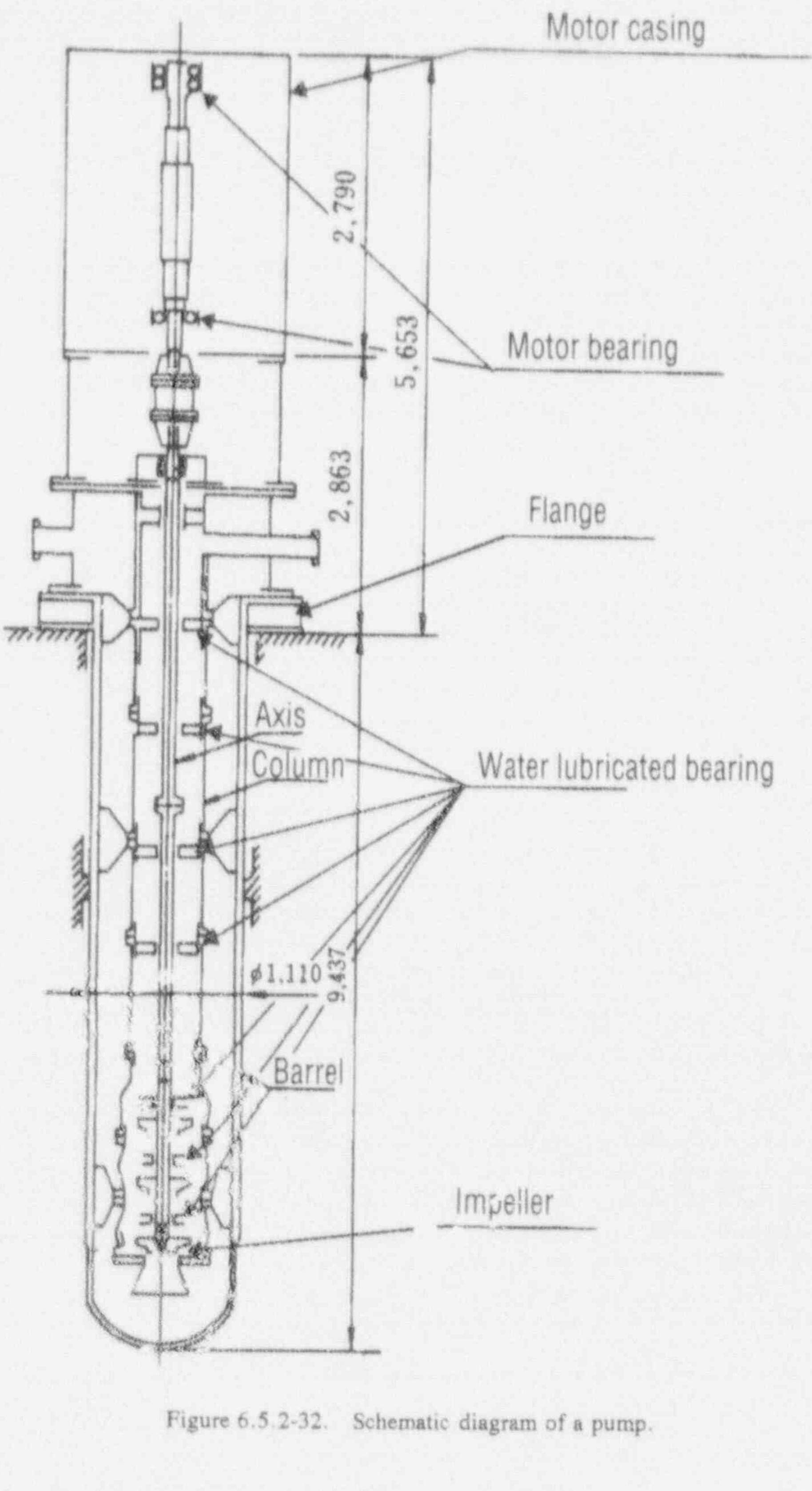


Figure 6.5.2-32. Schematic diagram of a pump.

(d) Emergency diesel body and generator, cable tray, electric board, duct, and tanks needed for evaluation of sloshing

The natural period of the diesel body and generator in the emergency diesel power generation equipment is calculated considering the flexural shear beam model of a single discrete mass system. In this case, for the model, the smallest cross section is taken into consideration, and the eigenvalue analysis is performed in the direction of the smallest stiffness. Figures 6.5.2-33 to 6.5-2-36 illustrates the schematic structure of the diesel body and generator as well as the eigenvalue calculation model, respectively.

For board, rack, etc., since they have been identified as rigid in the test, no model is formed for eigenvalue analysis. Instead, static analysis is performed to identify the strength of the anchor bolts. For air conditioning system duct and cable tray, in the case when they are designed as support structures with a rigid configuration, no eigenvalue calculation is performed. However, recently, some of them are designed not as a rigid structure. In this case, the seismic performance should be confirmed by analysis or test of the board, etc., and dynamic analysis is being performed using the model of simple support beams for the cable tray.

Figure 6.5.2-37 shows the appearance of the electrical board. Figure 6.5.2-38 shows an example of the model using the finite element method. Figures 6.5.2-39 and 6.5.2-40 show examples of support of the cable tray. Figure 6.5.2-41 shows an example of the analytical model [H-K-3].

Among tanks and containers containing liquid, when a large influence of sloshing is considered during earthquake, an evaluation is also performed for sloshing (see Figure 6.5.2-42). In the conventional schemes, sloshing of liquid during earthquake is analyzed using Housner's theory. Recently, however, evaluation is also performed using the velocity potential theory. The container shape may be either cylindrical or rectangular. Since the container wall is assumed to be a rigid body, the interaction effect with the container wall is usually ignored. However, there are some design cases in which the mass-spring model of the tank itself is combined with the mass-spring model of sloshing (determined using Housner's method) to account for the fluid-interaction effect.

(5) Support structures

In this section, as examples of the models for the support structure, we will discuss "a. Horizontal three-barrel cylindrical container" and "b. Vertical cylindrical equipment supported at the waist portion." For these support structures, the scheme of modeling for conventional equipment described in section "6.6.3 Class B and C equipment" cannot be applied. As a result, modeling is performed in a unique way.

a. Three-barrel cylindrical container (see Figure 6.5.2-43)

(a) Calculation conditions

(i) The weight of the heat exchanger is taken as a concentrated load on the central axis of the barrel.
(ii) Each of the barrels of the heat exchanger is supported by two legs. The barrels are fixed to the frame by leg-fastening bolts. Of the two legs, barrels can slide in the longitudinal direction.
(iii) The frame is supported by four legs, each of which is fixed to the foundation by the anchor bolts.

(b) Calculation method of natural period

(i) Assumptions

{1} The heat exchanger is taken as in the operating state.
{2} The barrels are fixed to the first legs.
{3} Since the joints at the second leg of each barrel can slide in the longitudinal direction, the force in this direction is taken as totally acting on the first leg.
{4} The legs of the frame mounted by the anchor bolts are assumed to be fixed.

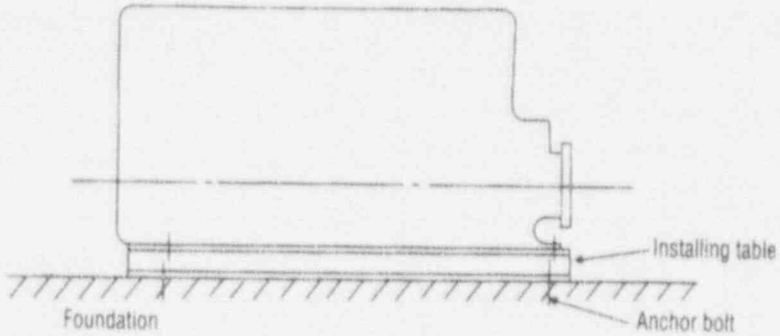


Figure 6.5.2-33. Schematic diagram of emergency diesel body.

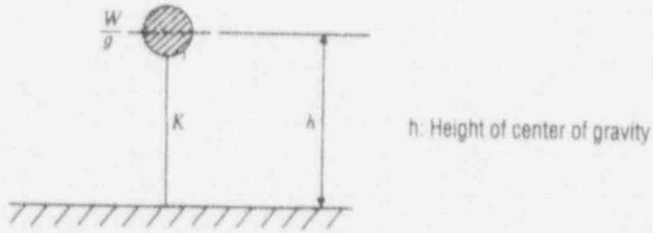


Figure 6.5.2-34. Example of calculation model of natural period.

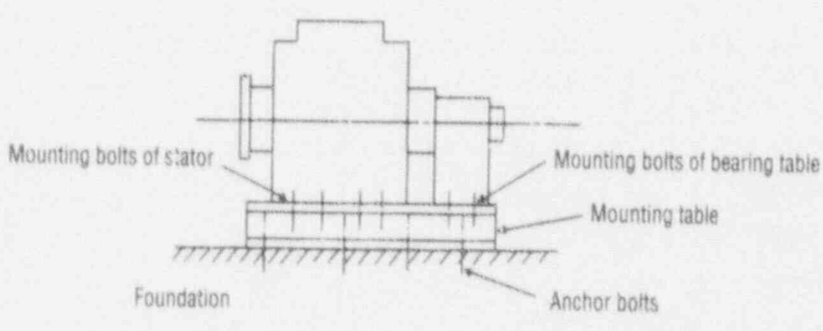


Figure 6.5.2-35. Schematic diagram of power generator.

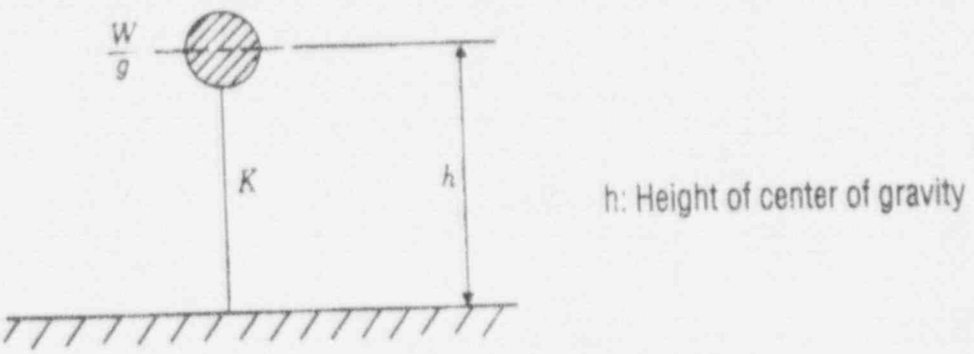


Figure 6.5.2-36. Example of calculation model of natural period.

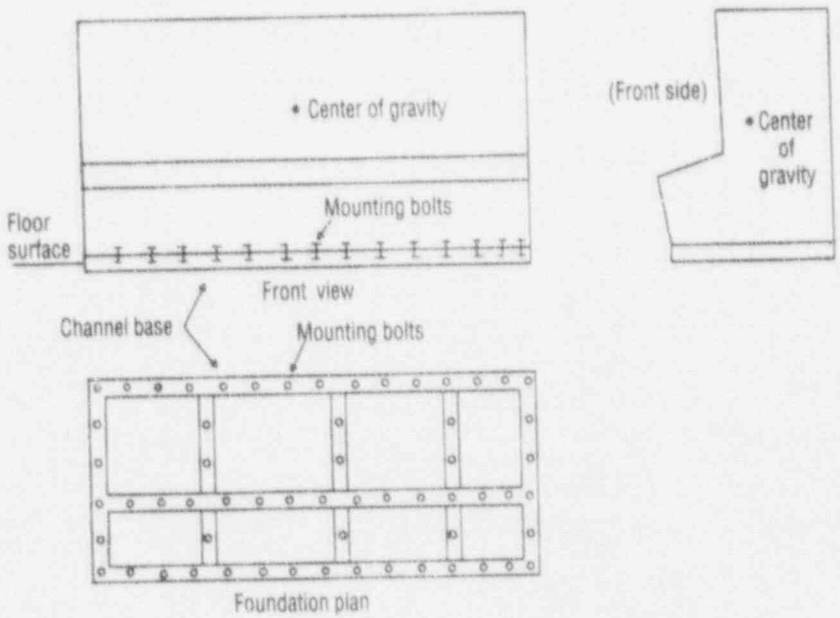


Figure 6.5.2-37. Appearance and foundation diagrams.

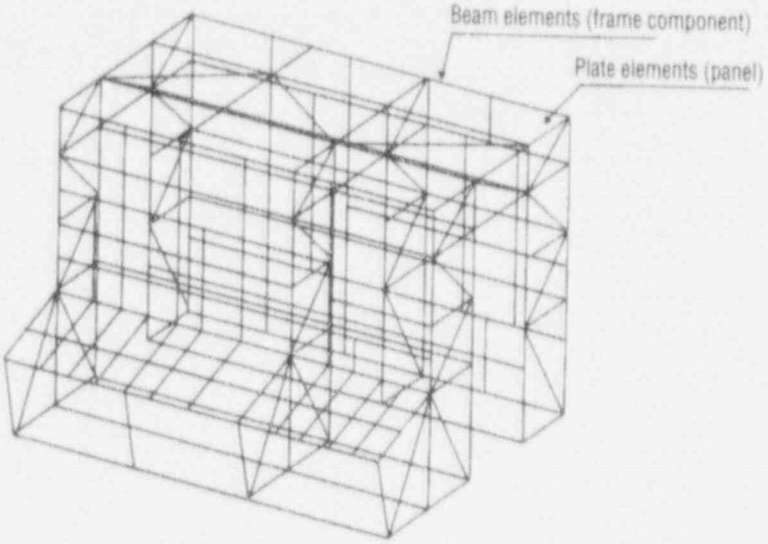


Figure 6.5.2-38. Example of finite element model for seismic calculation of electric board.

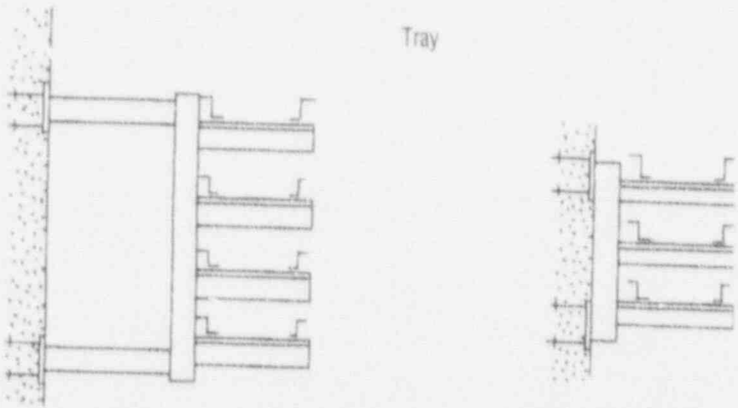


Figure 6.5.2-39. Cable tray support conceptual diagram (example of horizontal portion).

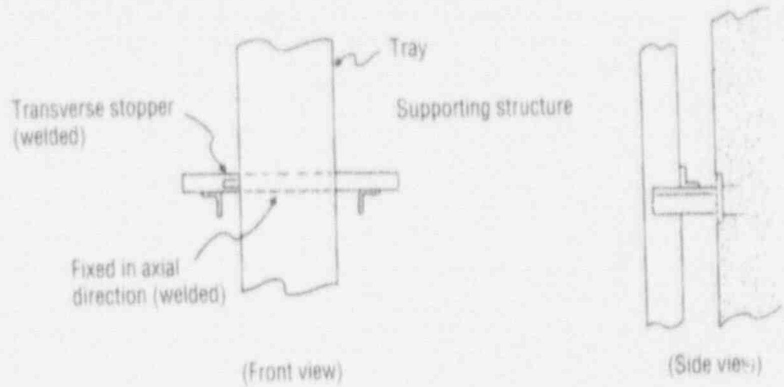


Figure 6.5.2-40. Cable tray support conceptual diagram (example of vertical portion).

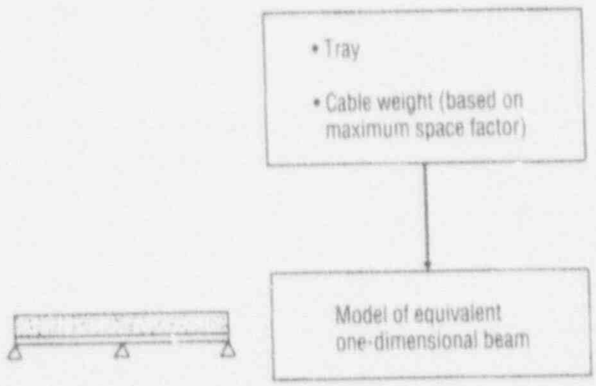


Figure 6.5.2-41. Example of model of cable tray.

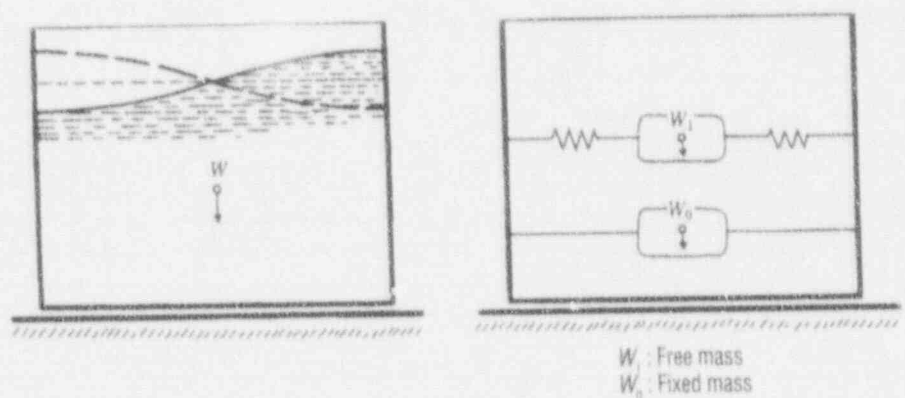


Figure 6.5.2-42. Formation of model of liquid sloshing according to Housner's theory [6.5.4-9].

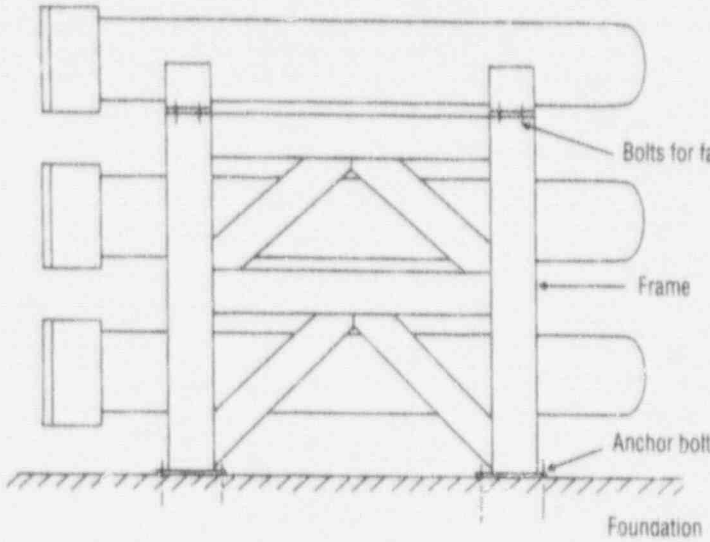


Figure 6.5.2-43. Schematic structural diagram.

(ii) Calculation model

According to assumptions {2}-{4} in (i), the model of this heat exchanger is made as a multiple discrete mass model as shown in Figure 6.5.2-44.

b. Vertical cylindrical container supported at the waist portion (see Figure 6.5.2-45)

(a) Calculation conditions

(i) The weight of the heat exchanger is taken as concentrated on the central axis of the barrel.
(ii) The barrel of the heat exchanger is supported by four legs, which are fixed to the foundation by anchor bolts. The four legs can slide in the radial direction of the barrel with respect to the foundation.

(b) Calculation method of natural period

(i) Assumptions

{1} The heat exchanger is assumed to be in the operating state.
{2} Legs, support frame, and foundation concrete are assumed as rigid.
{3} Legs are mounted to the foundation by anchor bolts. This portion is modeled as spring support.
{4} The support frame can slide in the direction of displacement caused by thermal expansion of the barrel.
{5} The portion of the support frame mounted on the wall is considered as fixed.

(ii) Analysis model

According to assumptions {2}–{5} in (i), this heat exchanger is modeled using a multiple discrete mass model shown in Figure 6.5.2-46.

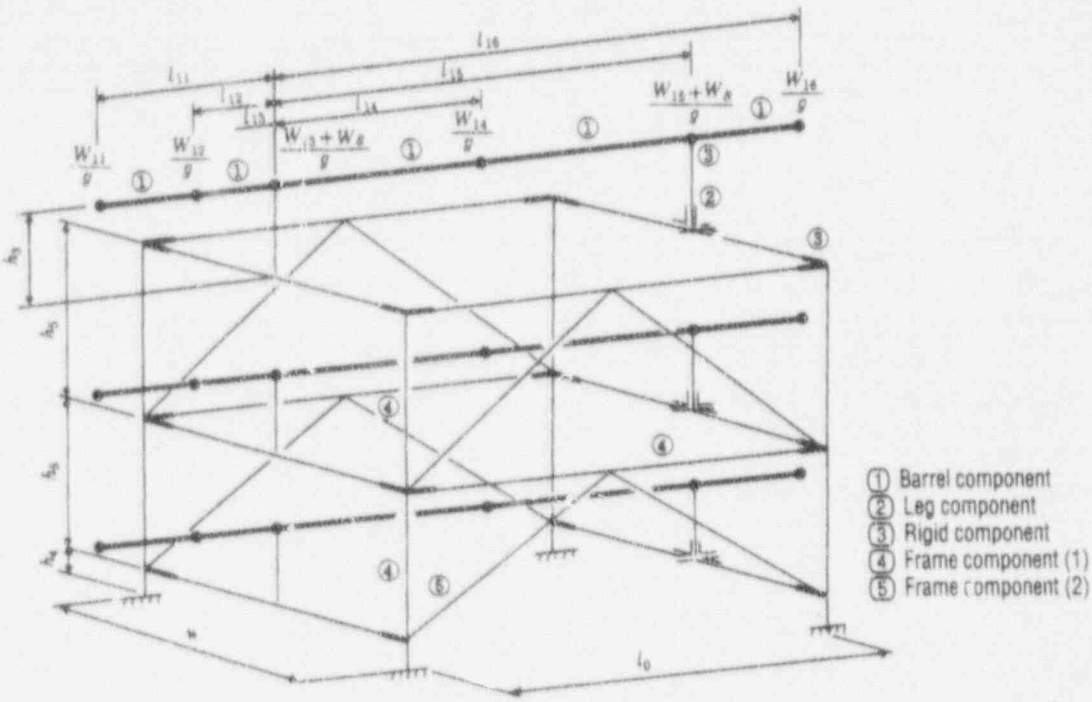


Figure 6.5.2-44. Analysis model of natural period.

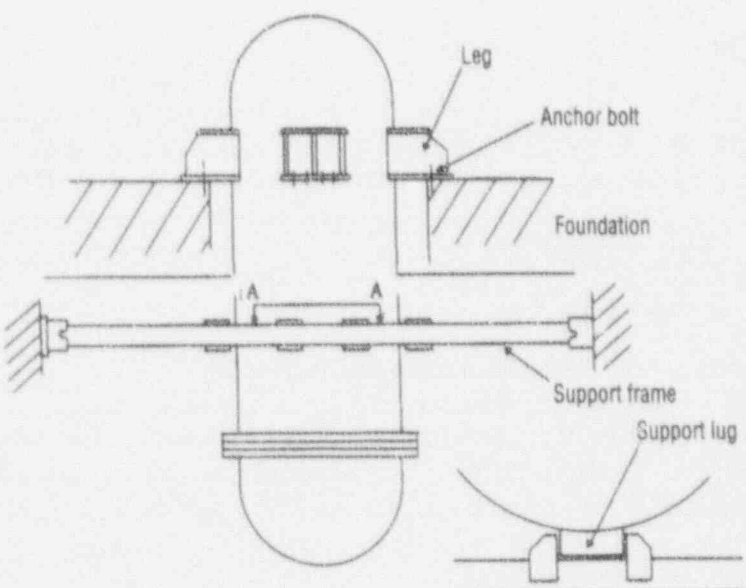


Figure 6.5.2-45. Schematic structural diagram.

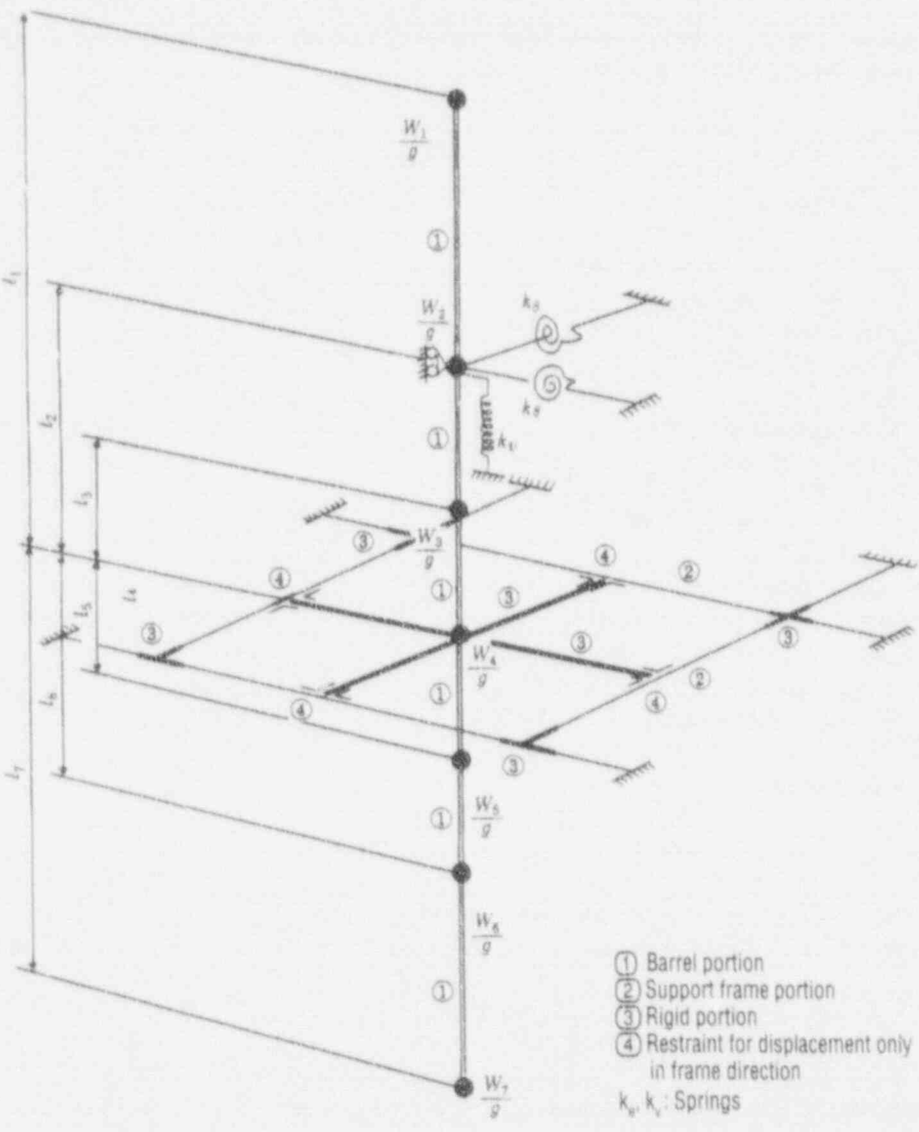


Figure 6.5.2-46. Analysis model for natural period.

6.5.3 Design damping constants

For the dynamic seismic response analysis of machines/equipment described in "6.5.2 Dynamic analysis model," the following values are usually used.

(1) For S_1 seismic response

Equipment	Damping constant (%)
Reinforced concrete structure (Note 1)	5.0
Steel frame structure	2.0
Welded structure	1.0
Bolt/rivet structure	2.0
Piping (Note 2)	0.5-2.5
Air conditioning duct	2.5
Cable tray (Note 3)	5.0
Pump, fan, and other mechanical equipment	1.0
Electrical board (Note 3)	4.0
Liquid sloshing	0.5
Fuel assembly (BWR)	7.0
Fuel assembly (PWR) (Note 4)	10.0-15.0
Control rod driving mechanism (BWR)	3.5
Control rod driving mechanism (PWR)	5.0
Primary coolant equipment (PWR) (Note 5)	3.0

(2) For S_2 seismic response

The values in (1) for S_1 seismic response are also used for S_2 seismic response.

Note 1: Damping constant of reinforced concrete structure

This value is traditionally used as the damping constant of reinforced concrete structures.

Note 2: Damping constant for piping design [6.5.3-1], [6.5.3-2], [6.5.3-3], [6.5.3-4], [6.5.3-6]

In (1), the damping constant for piping design is given as in the range of 0.5-2.5%. However, if certain special conditions are met, it is possible to use the values listed in Table 6.5.3-1.

For all the vibration modes, the damping constants for design of piping are listed in Table 6.5.3-1. However, if the specified conditions are not met, a damping constant of 0.5% should be used.

For PWR in-core instrumentation output pipe, [the damping constant] is taken as 2.5% irrespective of the next table and the application conditions.

Note 3: Damping constants of electric board and cable tray [H-K-3]

The damping constants of the electric board and cable tray are as follows.

(i) Damping constant for design of electric board

The damping constant for design of the self-supporting locked electric board used in the nuclear power plant is taken as 4.0%.

<u>Items for attention when the damping constant is used</u> When the aforementioned design damping constant is used, attention should be paid to the following items:

{1} When the structure of the electric board or the damping mechanism varies significantly in the range surveyed, it is necessary to use an appropriate method to assess the damping.

{2} In the electric board structural design, for the anchor portion, welded portion and other portions where a concentrated load is expected, appropriate margin should be ensured.

Table 6.5.3-1. Damping constants for design of piping system.

Piping type		Damping constant for design (%) With thermal insulation	Damping constant for design (%) Without thermal insulation
I	Piping system supported mainly by snubbers and frame restraints, with four or more supports (snubbers or frame restraints)	2.5	2.0
II	Piping system having snubbers, frame restraints, rod restraints, hangers, etc., with four or more supports (excluding anchors and U-bolts); not belong to piping Type I	1.5	1.0
III	Not belong to piping Type I or II	1.0	0.5

Application conditions:

{i} The design damping constants listed in the table are applicable to piping systems which are independent vibration systems from one anchor to another anchor.

{ii} The design damping constants listed in the table are applied in the case when the piping system is designed to have a period shorter than the undamental period of the building to which it is installed.

{iii} The number of supports in the table is counted as follows: When there is a number of supports to support in the same direction at the same support point, they are taken as one support. On the other hand, when there are several supports supporting in two directions at the same support point, they are counted as two supports.

{iv} For the supports, as viewed from the overall piping system, the position and direction should not be locally concentrated.

{v} For the interval between support points, the following conditions should be met:

$$\frac{\text{Total length of piping system}}{\text{Number of support points of support fixtures determined for each piping unit}} \leq 15 \text{ (m/support point)}$$

Here, the so-called support points refer to the location where the support fixtures are mounted. Even when a number of support fixtures are mounted, they are still taken as one support point.

{vi} Based on the construction management rules of the support fixtures, management of the construction should be performed carefully.

(ii) Damping constant of cable tray

For the solid-type or ladder-type cable tray used in the nuclear power plant, the design damping constant is taken as 5.0%.

Note 4: Damping constant for fuel assembly (PWR).

The damping constant of the fuel assembly in a PWR is determined experimentally. It is taken as 10% for the 17 x 17 type fuel assembly and 15% for the 14 x 14 fuel assembly.

Note 5: Damping constant for primary coolant equipment (PWR)

A damping constant of 3.0% is used for the primary coolant equipment of a PWR, including steam generator, primary coolant pump, and primary coolant piping.

6.5.4 Earthquake response analysis method [6.5.4-1], [6.5.4-2], [6.5.4-4]

When dynamic analysis is performed for the analysis model described in Section "6.5.2 Dynamic analysis model" to calculate the absolute acceleration, relative acceleration, maximum relative displacement, maximum shear force, maximum moment, etc., and to evaluate the design seismic forces, the most frequently used methods are time history analysis and spectral modal analysis. In time history analysis, the response of the system is calculated as a function of time directly or after transformation to the modal coordinate system. On the other hand, according to spectral modal analysis, the maximum response of each mode is determined directly using the design floor response spectrum, and the maximum response of the system is derived by superposing the response of the modes. Several schemes have been proposed to obtain the maximum responses to reasonably account for the correlation among the modes. At present, the maximum response is usually obtained by "SRSS" of the responses of the modes. However, when the equipment is coupled with the building analysis model shown in Chapter 5, "5.2.3 Soil-structure interaction," in which the soil model is formed by complex springs, the analysis is performed in the frequency domain with the input seismic motion treated by Fourier transformation.

The machines/equipment systems include those that are coupled with soil/building and those which are analyzed without such interaction. Since the input seismic motion to the soil/building/equipment interaction model is derived in time history form from the basic earthquake ground motion defined at the rock outcrop surface by free field analysis, the time history analysis method is usually used for the equipment coupled with the soil/building system. On the other hand, for the equipment/piping analyzed without interaction with the soil/building model, the spectral modal analysis method is usually used to determine the seismic load. This is based on the following principle: in the practical design, only the maximum value is needed without considering the entire time history. Hence, according to a representation with the natural period as variable and the damping constant as parameters, it is possible to derive immediately the maximum response value (usually, acceleration) with respect to a single discrete mass model. Also, for the equipment represented by a multiple discrete mass model, usually "SRSS" of the response of each mode is used [6.5.3-5].

Figure 6.5.4-1 illustrates the relation among the response analysis methods. The member forces shown in Figure 6.5.4-1 include shear force and bending moment. For these member forces, stress evaluation is performed according to the procedure explained in section "6.6 Stress/strength analysis." In this way, the aseismic design of the equipment/piping is performed.

In the following, the analysis methods corresponding to the methods of model formation described in section "6.5.2 Dynamic analysis model" will be explained.

(1) Equipment-building interaction

Class A equipment of the equipment-building-equipment interaction system is analyzed using the models explained in section "6.5.2 Dynamic analytical model," i.e., for BWR, the model for reactor vessel/internal structures shown in Figure 6.5.2-19 and the model of reactor containment/reactor vessel in Figure 6.5.2-3 are used; for PWR, the equipment-building interaction model shown in Figure 6.5.2-8 can be used for seismic response analysis. For these models of equipment coupled with building, time history analysis is usually used in the conventional design. That is, as shown in Figure 6.5.4-1, the results of direct integration or modal analysis are used. Details of these analysis methods were described in section "5.2.4(3) Response analysis method." They will not be repeated here. In the case of interaction equipment model, the following schemes are often used.

In the case when direct integration is used, the Newmark-β method is usually used as the integration method with β taken as 1/4 or 1/6. For the eigenvalue analysis method in the modal analysis method, when it is solved as the general eigenvalue problem;

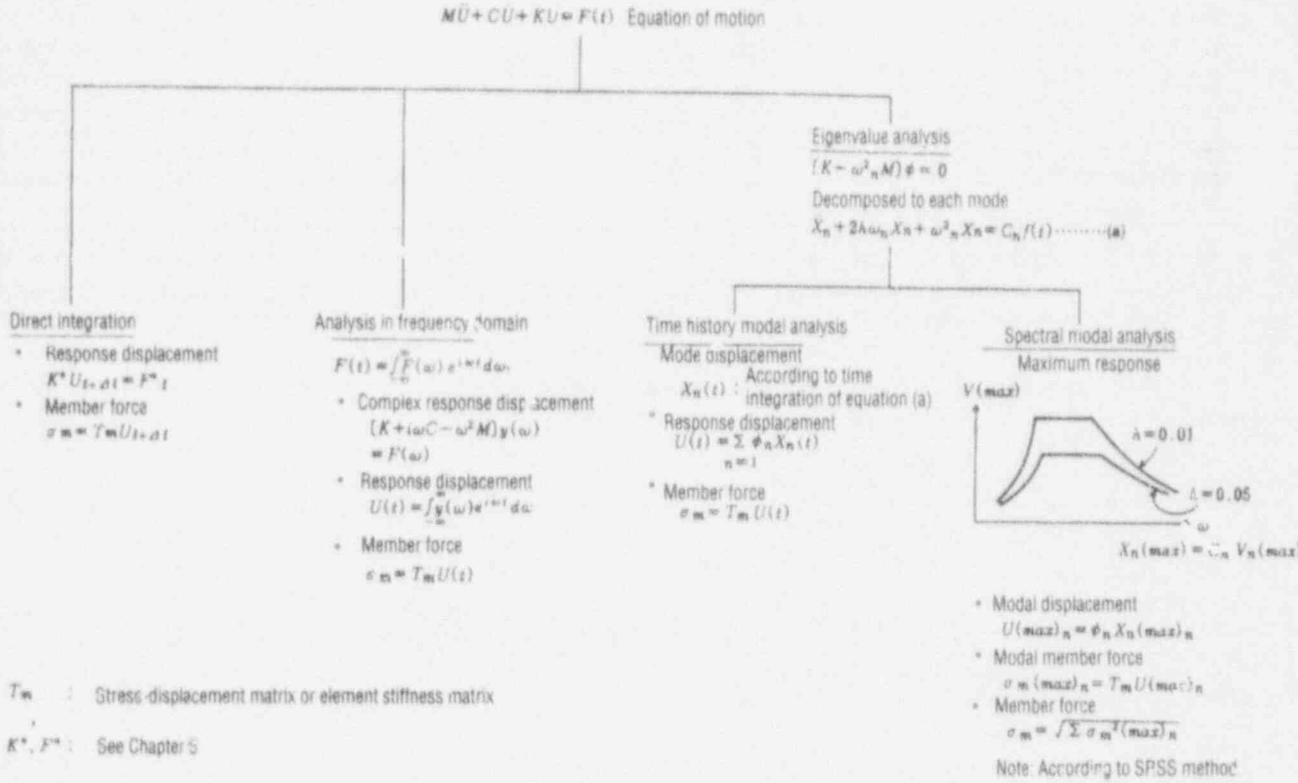


Figure 6.5.4-1. Relation among response analysis methods.

{1} Conventional Jacobi method etc., are used; when it is solved as a standard eigenvalue problem;
{1} QR method
{2} Jacobi method and other transformation methods are usually used. Time integration for each mode is usually performed by using;
{1} Runge-Kutta-Gill method
{2} Duhamel integration method (Nigam's method), etc.

Vibration modes for the equipment-building interaction model are shown in Figure 6.5.2-20 for reactor vessel/internal structures, and in Figure 6.5.2-4 for the equipment-building interaction model of the nuclear reactor containment/reactor vessel.

(2) Vessels

For Class A vessels, as pointed out in section "6.5.2 Dynamic analysis model," if analysis is performed using the equipment-building interaction model, the maximum shear force and maximum bending moment obtained from the bending-shear beam model are taken as the seismic load. On the other hand, for the vessel without building interaction, if it is not rigid, seismic response analysis is usually performed using the design floor response spectrum at the installation position. Spectral modal analysis is usually used as the analysis method. The general features of spectral modal analysis are described in section "5.2.4(3) Response analysis method," its basic principle is described in section "6.5.1 Floor response spectrum." Hence, we will discuss the specific procedure of the design.

After the eigenvalue analysis, for each mode, the equation of motion of the normal coordinate system is determined as

$$\ddot{x}_i + 2h_i\omega_i\dot{x}_i + \omega_i^2 x_i = -\beta_i\ddot{y}_0 \tag{6.5.4-1}$$

where h_i: ith damping constant; ω_i: ith natural frequency of vibration: β_i: ith participation factor.

Then, acceleration α_i is obtained for each mode with natural froquency ω_i from the floor response spectrum of the input seismic motion $\ddot{y}_0$. As shown in Figure 6.5.4-2, the ith natural period is calculated and then α_i is determined. The spectrum shown here is obtained by broadening in the period direction as described in section "6.5.1 Floor response spectrum."

Then, transformation is made from the modal coordinate system to the original coordinate system as follows.

$$\left\{\ddot{X} + \ddot{y}_0\right\}_i \quad i\text{th acceleration} \quad \beta_i\alpha_i\{\bar{X}\}_i \tag{6.5.4-2}$$

$$\{X\}_i \quad i\text{th displacement} \quad \beta_i\frac{\alpha_i}{\omega_i^2}\{\bar{X}\}_i \tag{6.5.4-3}$$

$$\begin{Bmatrix} S \\ M \end{Bmatrix}_i \quad i\text{th member force} \quad (K)\{\bar{X}\}_i \tag{6.5.4-4}$$

S: shear force; M: bending moment; $\{\bar{X}\}_i$: ith mode vector (characteristic mode).

From the responses of each mode, the square root of the sum of the squares is derived using the "SRSS" method for acceleration, displacement, and member force.

$$\text{Maximum response acceleration} = \sqrt{\sum_{i=1}^{N}\left\{\ddot{X} + \ddot{y}_0\right\}_i^2} \tag{6.5.4-5}$$

$$\text{Maximum response displacement} = \sqrt{\sum_{i=1}^{N}(X)_i^2} \tag{6.5.4-6}$$

$$\text{Maximum shear and bending moment, etc.} = \sqrt{\sum_{i=1}^{N}\begin{Bmatrix} S \\ M \end{Bmatrix}_i^2} \tag{6.5.4-7}$$

N: Number of modes up to the vibration mode considered to be rigid.

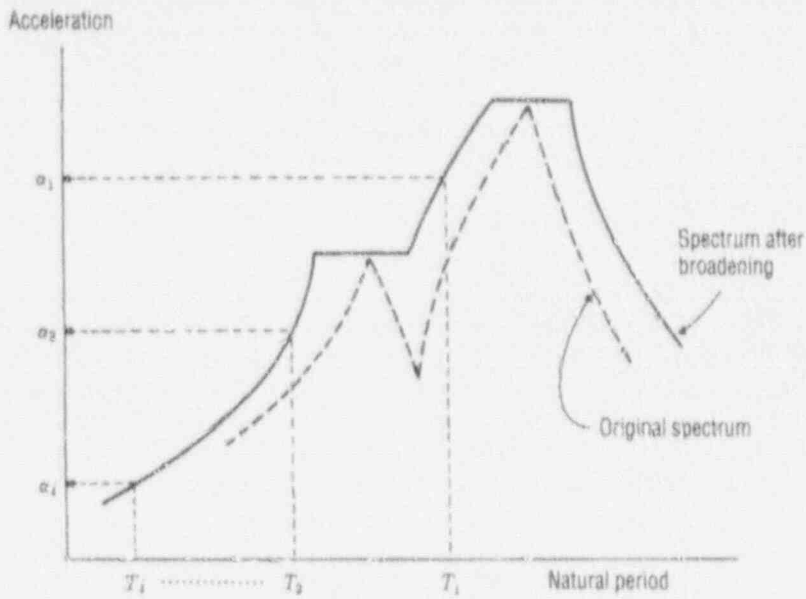


Figure 6.5.4- Floor response spectrum.

(3) Piping

For Class A piping, the most often used seismic response analysis method is the spectral modal analysis method described in section "6.5.4(2) Vessels." However, the time history analysis method described in section "6.5.4(1) Equipment-building interaction" may also be used in some cases. Since the piping system is supported at the multiple support points, it is believed to be appropriate to use the multi-input analysis method. Several methods have been proposed for multi-input analysis. Usually, however, the method proposed by Clough [6.5.4-2] is used in the case of multi-input analysis of piping. Clough's method is characterized by the feature that the system is defined by a sum of the pseudostatic displacement obtained from the static equilibrium relationship and the dynamic displacement. Finally, the equation of motion of the system becomes equation (6.5.4-8).

$$M\ddot{U} + C\dot{U} + KU = -MH\ddot{U}_b \tag{6.5.4-8}$$

M: mass matrix; C: damping matrix; K: stiffness matrix; $\ddot{U}_b$: absolute acceleration at support point; H: transform matrix obtained from static equilibrium relation.

Since equation (6.5.4-8) has the same form as that of the equation of motion treated up to ..., in addition to the multi-input analysis by the time history scheme, spectral multi-input analysis using the response spectrum at the support point may also be used. In the design of piping with a huge degrees of freedom as a simpler and yet conservative way, the analysis method using a single input which envelopes the inputs at the various support portions is preferred in many cases.

That is, for the seismic response analysis of the piping system, the analysis method mainly used is to determine the seismic forces by spectral model analysis using the three-dimensional model formed with the bending-shear beams (beams, piping elements) as described in section "6.5.2(3) Piping." In the spectral model analysis method, for the mode superposition of the member forces, the "SRSS" method is usually used, and the absolute value sum is usually used to combine the seismic input directions.

As an example of the result of spectral modal analysis, section "6.5.2 Dynamic analysis model" presents the vibration modes and response values for the BWR circulation system piping and the PWR primary coolant equipment.

(4) Other equipment

The analysis methods that are applicable for the heat exchanger, pumps, tanks and other equipment for which the models are described in section 6.5.2(4) "Other equipment" are similar to those of the Class A vessels and Class A piping. That is, if resonance might take place, response spectral analysis or time history analysis may be performed. Also, for rigid parts, static analysis may be performed based on the seismic coefficient. If their shapes and structure are the same as those of Class B or C equipment described in section "6.6.3 Class B and C equipment," the analysis procedure described in section 6.3.3 may also be used.

(5) Sloshing

As pointed out in section 6.5.2, the analysis methods of sloshing of liquid in tank or container include the method according to Housner's theory, the method according to velocity potential theory, and the finite element method. At present, from the viewpoint of simplicity, the method using Housner's theory is usually adopted as the analysis method. In addition, when the sloshing response is evaluated by time history, the method using velocity potential theory is used.

In the following, we will discuss the method using Housner's theory and the method using the velocity potential theory.

a. Housner's theory

This analysis method is described in detail in TID Report 7024 (USAEC) [6.5.4-9]. It is an analysis method of the vibration in a tank containing liquid having a free surface, and has been commonly used in many cases. Housner's theory can be used for a ground-contact liquid storage tank having a flat bottom and a rectangular or circular uniform cross section. When a horizontal earthquake acceleration acts on a tank containing liquid, the following two types of dynamic hydraulic pressure are generated in the contained liquid and act on the tank:

- Impulsive pressure caused by horizontal inertial force of liquid,
- Convective pressure caused by liquid surface sloshing.

These two types are analyzed separately to determine the design input to the tank. In this analysis method, the tank body is assumed to be a rigid body, without considering the effect of interaction between liquid and tank due to the elastic deformation of the tank. The aforementioned impulsive pressure and convective pressure are as follows:

(i) Impulsive pressure (due to inertial force of the contained liquid

When the tank containing liquid is subjected to a horizontal acceleration, a portion of the liquid is taken as a fixed mass which is rigidly connected to the tank wall. The inertial force caused by this fixed mass is taken as the impulsive force. The impulsive force per unit area is the aforementioned impulsive pressure (see Figure 6.5.2-42).

(ii) Convective pressure (due to liquid sloshing)

When the tank containing liquid is subjected to a horizontal acceleration, vibration occurs in the liquid, with a dynamic hydraulic pressure acting on the side wall and the bottom. In this case, a portion of the liquid is taken as a free mass which is flexibly connected to the side wall. In addition, since the tank body is assumed to be a rigid body, the maximum vertical displacement of the liquid surface (sloshing wave height) and the horizontal force acting on the tank wall can be determined from the maximum relative vibration amplitude of the free mass. This horizontal force is the vibration force, and the vibration force per unit area is the aforementioned convective pressure (see Figure 6.5.2-42).

b. Velocity potential theory

The velocity potential theory is a vibration analysis method of liquid having a free surface. It has been used recently for the aseismic design of conventional industrial facilities. According to the method using the velocity potential theory, the tank is assumed to be a rigid body in calculating the dynamic response of the liquid contained in it; the pressure distribution on the side surface and bottom of the tank and liquid surface displacement profile are calculated, and the design seismic force of the tank is determined.

6.6 Stress/strength analysis

6.6.1 Load/stress combination

(1) Loads to be combined with seismic force

When the equipment/piping system is acted upon by a seismic force as described in section "6.4 Design seismic force," it is necessary to evaluate the combination of the seismic load generated by the seismic force and the loads which are generated in the operating state of the plant and should be combined with the earthquake.

For the loads which are not caused by earthquake but should be taken into consideration in combination with the seismic load, the operation state is described in section "6.3.2 Load Combination." Since details are described in "JEAG 4601-Supplement-1984," they will not be repeated here. Table 6.6.1-1 lists the other types of loads that should be combined with the seismic load.

For the aforementioned loads, the loads that have significant effects on the facility are evaluated.

Among the loads that take place in a loss of coolant accident, the following loads which act in a short period are not combined with the seismic load.

{1} Jet force
{2} Jet reaction force
{3} Pipe whip load
{4} Dynamic hydraulic load of BWR suppression pool water

(2) Summary of calculation of seismic stress

In this section, we will discuss the general items for the stress/strength evaluation performed on the base of the seismic load obtained in the seismic response analysis described in section "6.5 Seismic response analysis."

In strength evaluation in the aseismic design of equipment systems, usually, stress calculation is performed and the result is compared with the allowable stress. In addition, in some cases, evaluation may be performed in terms of load; in other cases, it is necessary to evaluate strain, deformation limit, and maintenance of function of equipment.

Stress/strength analysis is performed using an appropriate method suitable for the equipment under evaluation. The basic flow chart is shown in Figure 6.6.1-1. Generally speaking, there are the following two methods of stress evaluation:

Table 6.6.1-1. Loads that should be combined with seismic load.

Type	BWR	PWR
Type 1	Load due to the highest pressure generated in case of turbine trip or total loss of feedwater	Load due to the highest pressure generated in case of loss of overburden load or loss of off-site power
	• Reaction load when safety valve is activated • Bolt fastening force • Scram reaction force, etc.	• Reaction load when safety valve is activated • Bolt fastening force, etc.
	Pressure load and mechanical load generated after a loss of coolant accident excluding the period just after the accident	Same as left
Type 2	Highest pressure load for phenomenon which should be superposed with the earthquake	Highest pressure load for phenomenon which should be superposed with the earthquake
	Load due to air bubble vibration generated when safety relief valve	None
	Pressure load and mechanical load generated after a loss of coolant accident excluding the period just after the accident	Same as left
Type 3, 4, 5	Highest operation pressure	
	Design mechanical load	
Core support structure	Highest pressure difference for phenomenon which should be combined with the earthquake	
	Mechanical load for phenomenon which should be combined with the earthquake	
	Pressure load and mechanical load generated after a loss of coolant accident excluding the period just after the accident	
Other pump support structures, valves, core, internals	Highest operation pressure	
	Design mechanical load	

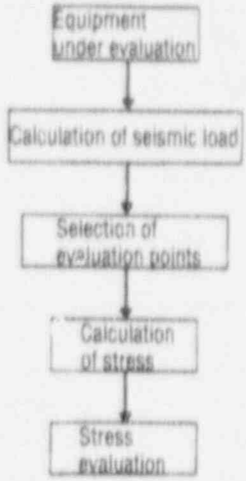


Figure 6.6.1-1. Basic flow chart of stress/strength analysis.

Evaluation by stress intensity: It applies for Type 1 vessels and piping, and Type 2 vessels; a detailed stress analysis is performed for this evaluation.

Evaluation by maximum stress: It applies for general equipment and support structures; a relatively simple stress calculation is performed for this evaluation.

For the stress calculation, depending on the importance of the equipment under evaluation and the complexity of the shape, different schemes, ranging from precise schemes to simpler schemes, can be used. In some cases, calculation is performed on a large-size computer using the finite element method, shell structural analysis, beam analysis, frame structure analysis, etc. In other cases, calculation can be performed using the basic formulas of strength of materials for simple-shaped parts. In order to derive the local stress of vessels, Bijlaard's method or the finite element method can be used. As a strength evaluation method other than stress evaluation, evaluation may be performed in terms of load. For this purpose, the standard load or limit load can be calculated beforehand, or, the allowable load can be assessed by tests.

6.6.2 Class As and A equipment

(1) Basic items of stress analysis

a. Stress intensity

In the seismic design of Type 1 vessels, stress analysis is performed and the result is combined with the stress due to internal pressure load or other load for evaluation. Details of this stress analysis are described in "Notification No. 501." According to "Notification No. 501," the basic scheme for the design of pressure vessels is that the "maximum shear stress theory" is adopted as the failure criterion, and evaluation is performed of "stress intensity." According to the maximum shear stress theory, when the principal stresses in the part are σ_1, σ_2, σ_3 ($\sigma_1 > \sigma_2 > \sigma_3$), yield takes place when the maximum shear stress $\frac{1}{2}(\sigma_1 - \sigma_3)$ becomes equal to the shear stress at the yield point in the tensile test. In the uniaxial tensile test, as σ_1 = Sy, $\sigma_2 = \sigma_3 = 0$, the yield condition becomes $\frac{1}{2}(\sigma_1 - \sigma_3) = \frac{1}{2}$Sy. In the design evaluation, by defining the stress as twice the maximum shear stress $\frac{1}{2}(\sigma_1 - \sigma_3) \times 2 = \sigma_1 - \sigma_3$ (similar for $\sigma_2 - \sigma_1$, $\sigma_3 - \sigma_2$) as "stress intensity," it is possible to make a direct comparison with the strength derived in the material strength test. Also, the evaluation method using the stress intensity may also be used for Type 1 piping and Type 2 vessels.

b. Stress classification

In the stress analysis, stresses are defined as follows:

Primary stress: The term "primary stress" refers to the normal stress or shear stress that meets the rule of simple equilibrium with respect to internal forces, external forces and moments.

The primary stress is the stress generated in an equipment to balance the inner pressure and external loads. That is, the primary stress is characterized by the fact that it is not self-restrictive. In other words, this stress is generated by external loads and if it is distributed over the entire thickness beyond the yield point, the material can no longer resist and failure takes place.

The primary stress can be further divided into general membrane stress, flexural stress, and local membrane stress. For each of these stresses, there is a specific limit of stress intensity.

Secondary stress: The term "secondary stress" refers to the normal stress or shear stress generated due to restraint by the adjacent portion and self restraint. That is, this type of stress is characterized by the fact that it is self-restrictive. In other words, as the secondary stress takes place, even if the part yields and further generates a little strain, there is no abnormal increase in the stress as a saturated state is reached for the stress. Hence, no failure takes place from the secondary stress only. Of course, this does not mean that the strain generated by the secondary stress can be increased without limit. Instead, a limit of the stress intensity should be determined in consideration of the shakedown characteristics.

Typical examples of the secondary stress include thermal stress and discontinuous stress. The thermal stress is generated due to temperature difference in the part. Due to this stress, deformation is generated, or, as the stress increases, a plastic flow initiates and the stress distribution becomes homogeneous over the entire body.

Discontinuous stress is a stress which takes place due to discontinuous deformation at places when the thickness of the part changes discontinuously, such as at a nozzle portion. Although this stress depends on the internal pressure and external loads, it is in a very limited portion as viewed from the overall vessel. Unlike the primary stress, which always maintains the stress state, in this case, as the stress increases, local plastic flow takes place and the distribution of the stress become homogenized.

Peak stress: The "peak stress" is a stress caused by local discontinuity, stress concentration or local thermal stress and it is additional to the primary stress and secondary stress. It is characterized by the feature that although no great deformation is caused by it, when it acts repeatedly, fatigue damage may take place.

In aseismic design, for the primary stress evaluation only, the seismic load is combined with the other loads; for the primary + secondary stress evaluation and primary + secondary + peak stress evaluation, however, only the seismic load is taken into consideration. In the fatigue analysis, evaluation is performed by determining the fatigue usage factor from the primary + secondary + peak stress. In this case, several methods may be used, such as the method using the number of equivalent number of cycles of seismic motion and the method in which the frequency of the stress range is directly derived from the time history evaluation of the equipment. In the case when the number of equivalent number of cycles of seismic motion is calculated, it is possible to use either the peak stress method (see Figure 6.6.2-1) or the energy conversion method (see Figure 6.6.2-2). In this case, the "fatigue usage factor" is obtained by adding up the ratio of the actual number of cycles in each stress cycle to the allowable number of cycles corresponding to the repetitive peak stress intensity for all of the stress cycles.

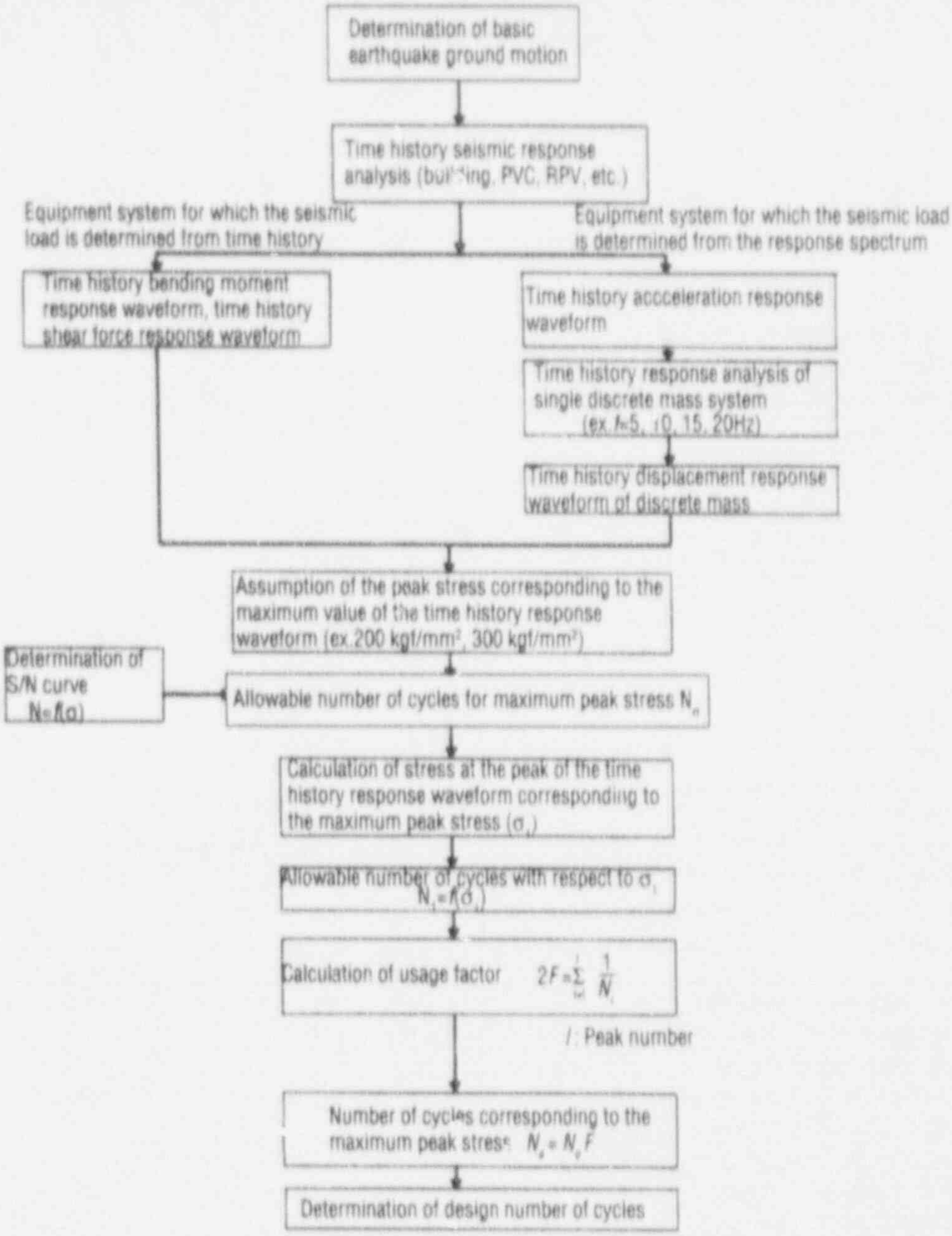


Figure 6.6.2-1. Determination of design number of cycles (peak stress method).

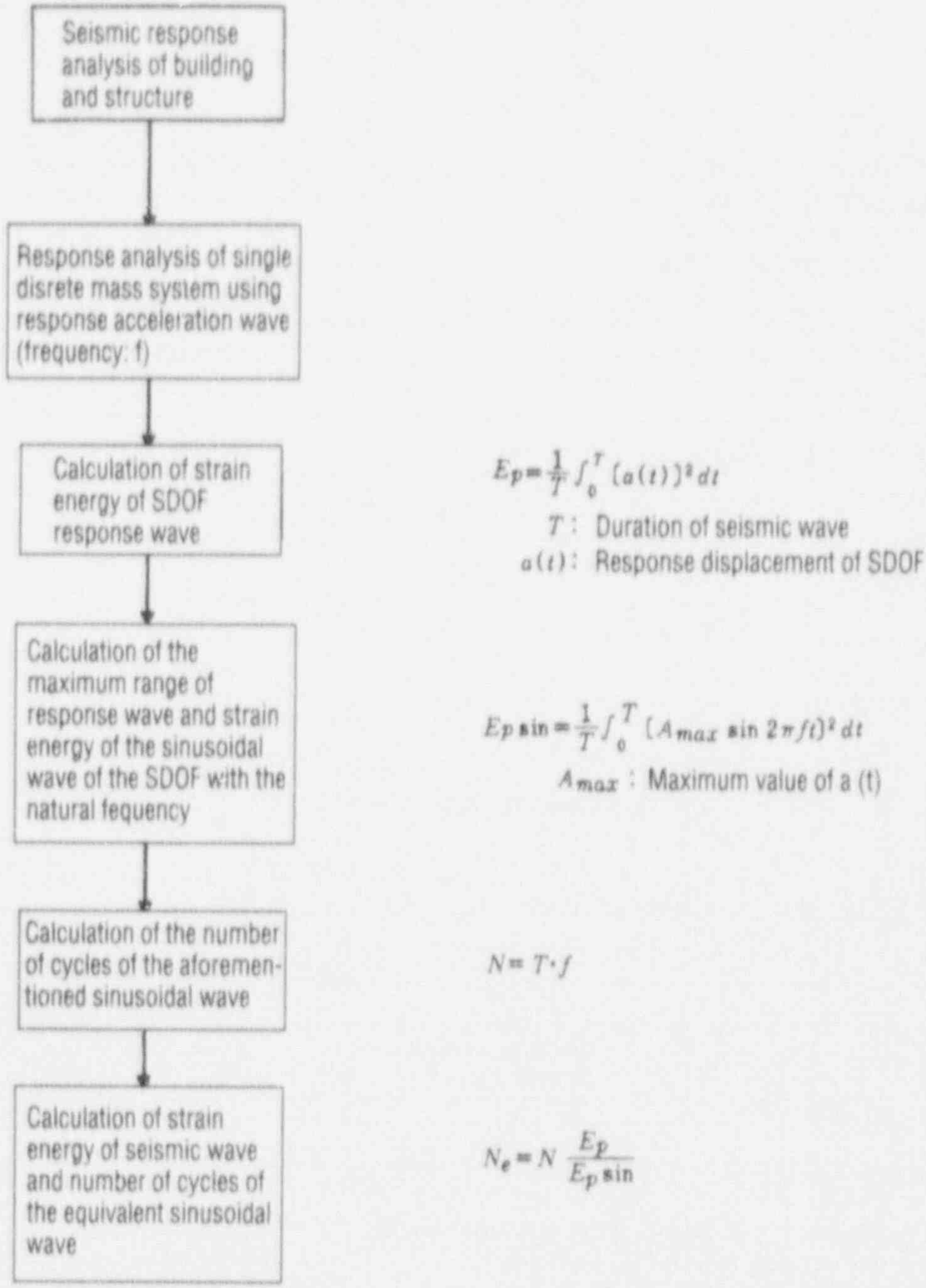


Figure 6.6.2-3. Determination of design number of cycles (energy conversion method).

(2) Type 1 equipment

a. Vessel

(a) Stress analysis flow chart

The structural design against the seismic load for Type 1 vessels involves not only evaluation of seismic stress, but also internal pressure, heat, self weight, etc. It is a portion of the overall stress evaluation scheme corresponding to the operation state of the plant. Figure 6.6.2-3 shows the procedure of stress analysis of a Type 1 vessel.

In the stress evaluation of the vessel, the loading conditions of self weight, internal pressure, heat, mechanical external force, etc., are considered, and the following portions are selected as the stress evaluation places:

- The portion with discontinuous cross section or shape.
- The portion where significant stress is generated due to attached pipe force.
- The portion where a concentration of stress is predicted.
- The portion with concentrated external force from support structure.

Figure 6.6.2-4 (PWR) and Figure 6.6.2-5 (BWR) show the examples of the stress evaluation portions for nuclear reactor vessels selected according to the aforementioned rules. Among these portions for evaluation, the places where significant stress takes place due to the external seismic force include the nozzle portion and the support structure mounting portion.

(b) Stress analysis methods

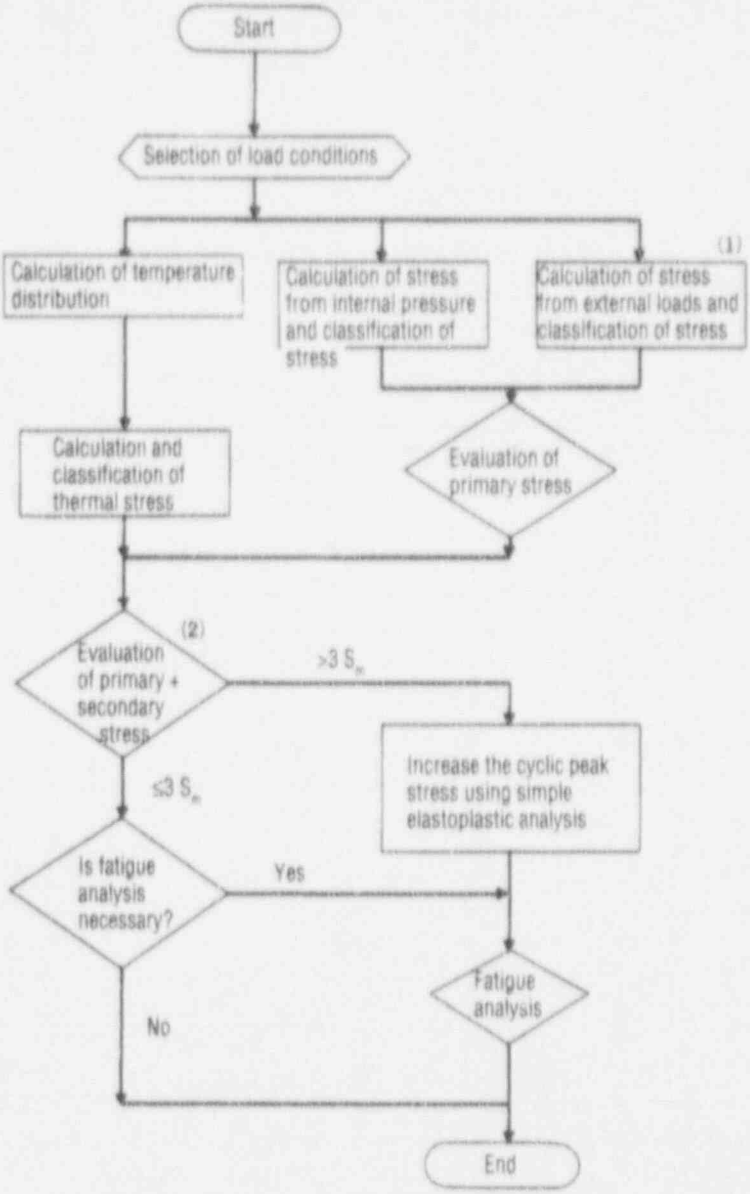
Depending on the shape and load type of the analysis portion, several schemes can be used for stress analysis of the vessel. For example, for the nozzle portion, in addition to the statically indeterminate method for the internal pressure load and thermal load, and Bijlaard's method for the seismic load from the piping system and other external loads; the finite element method may also be used for both cases. In the case of a 6-component load such as an external load, it may be taken directly as the design load in some cases. In some other cases, however, evaluation is made by combining the load components on the safe side. Table 6.6.2-1 illustrates an example of the primary general membrane stress of a Type 1 vessel. In the following, the stress analysis methods will be discussed with reference to the nozzle portion.

(i) Statically indeterminate method

As shown in Figure 6.6.2-6, the nozzle and other portions of the vessel are made of elements having simple shapes. In order to determine the stress in each element, the deformation of each element is determined independently using shell theory. Afterwards, from the condition of continuity of the displacement between different elements, the force acting between elements, i.e., the statically indeterminate force is calculated; by adding the stress generated by the internal pressure, etc., on the element independently and the stress due to the statically indeterminate force, the stress in each element can be calculated.

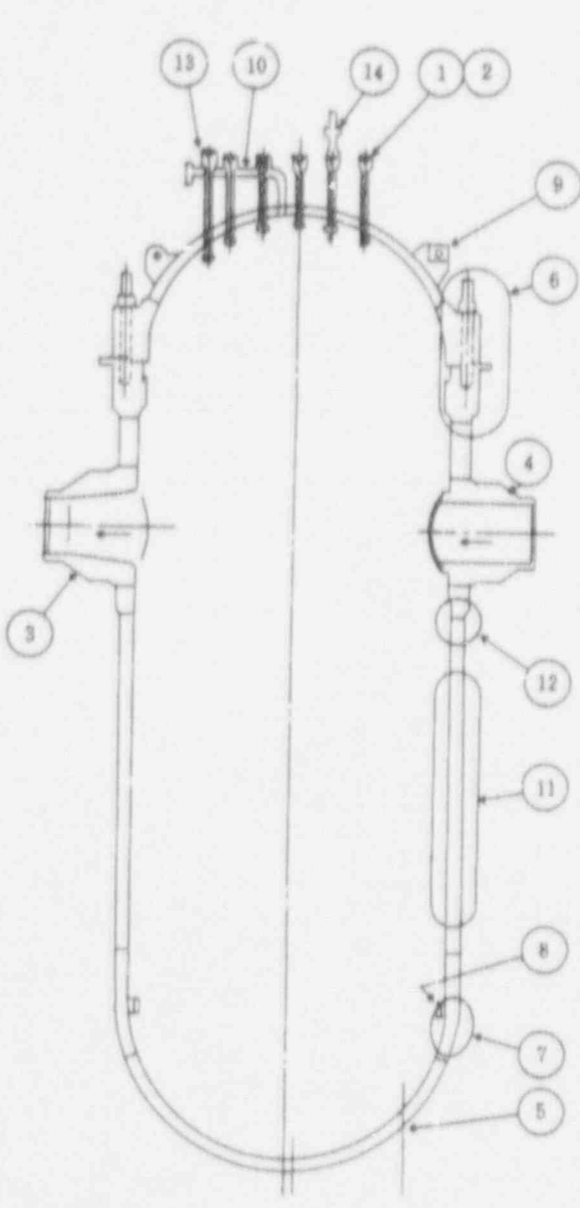
(ii) Stress due to external load

The stress in the nozzle, etc., mounted on the vessel due to piping external force and other external load can be determined in the following procedure. As the loads used in the analysis, the forces and bending moments are assumed at the tip of the nozzle as shown in Figure 6.6.2-7. The loads generated on the various cross sections are shown in Figure 6.6.2-8.



Note (1) The seismic load is included in the external loads.
Note (2) Primary + secondary stress evaluation for earthquake is performed for the stress range caused by seismic motion only.

Figure 6.6.2-3. Flow chart of stress analysis of Type 1 containment.



1. Control and driving unit housing
2. In-core thermocouple housing
3. Coolant inlet nozzle
4. Coolant outlet nozzle
5. Instrumentation nozzle
6. Closure head dome, closure head flange, upper shell flange connecting bolt/nuts
7. Lower shell, bottom head connecting portion
8. Core support fixture mounting portion
9. Rector vessel head lifting rig
10. Air evacuating pipe
11. Core region of shell
12. Upper shell/intermediate shell connecting portion
13. Adapter for control rod drive mechanism
14. Flange for in-core thermocouple (clamp portion)

Figure 6.6.2-4. Locations of stress analysis for nuclear reactor vessel (PWR, as a.n example).

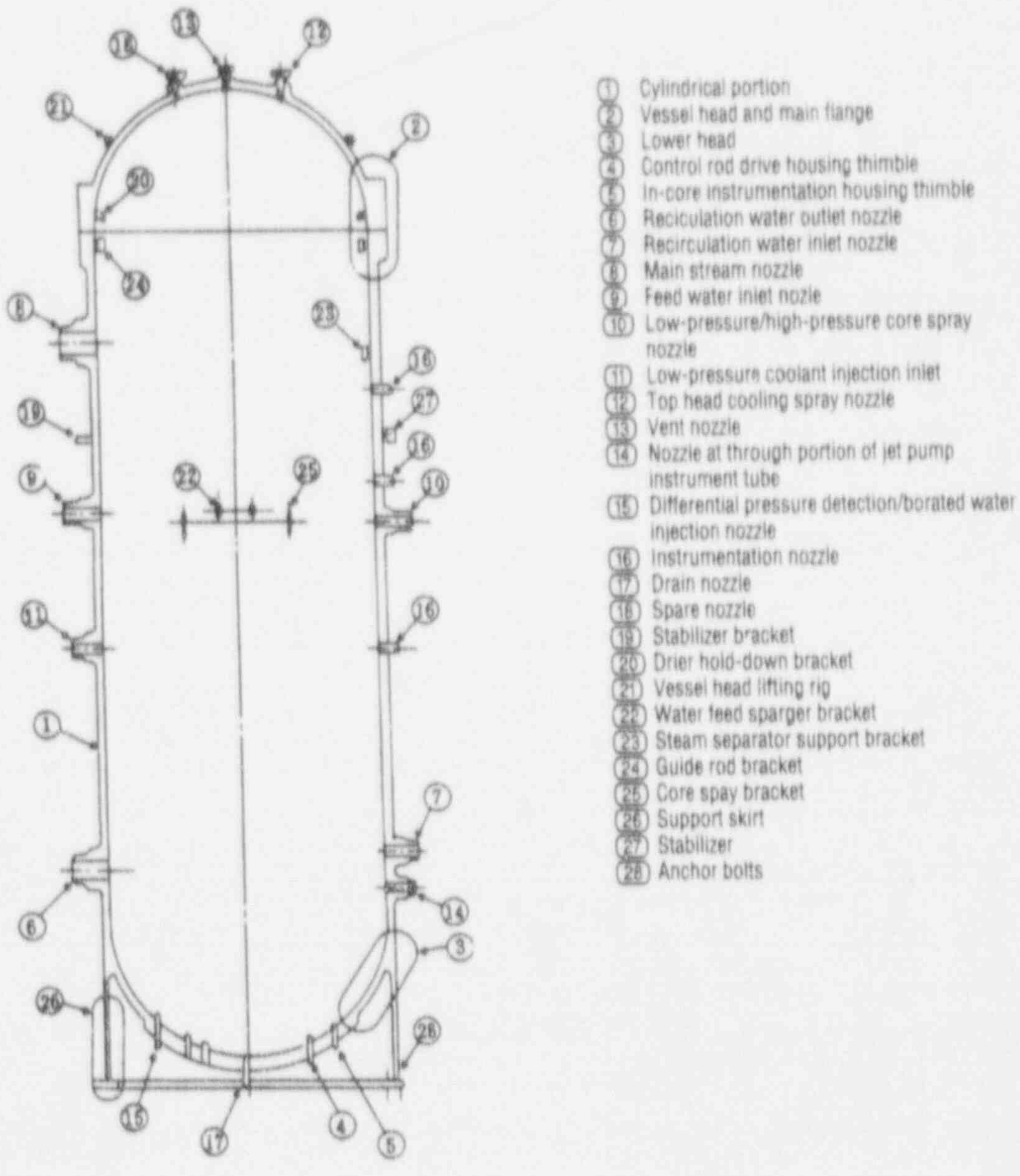
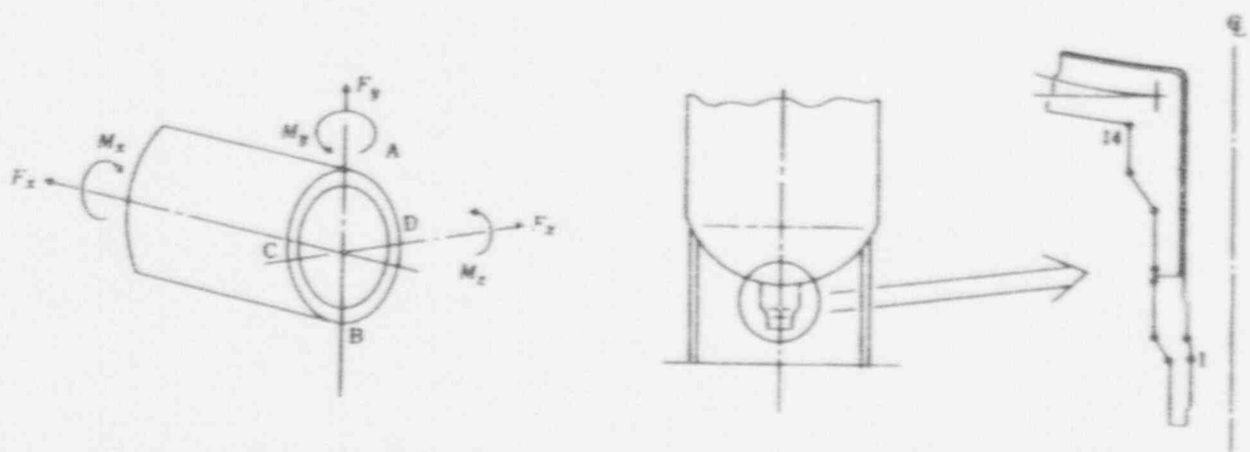


Figure 6.6.2-5. Locations of stress analysis for nuclear reactor vessels (BWR, as an example).

Table 6.6.[illegible] 1. Examples of primary membrane stress of nozzle of Type 1 vessel (Units: kgf/mm²).

Analysis location	Position on periphery	Stress due to pressure			Stress due to external load			Principal stress			Stress intensity			Allowable stress
		Axial direction	Circumferential direction	Radial direction	Axial direction	Circumferential direction	Radial direction	σ_1	σ_2	σ_3	$\sigma_1-\sigma_2$	$\sigma_2-\sigma_3$	$\sigma_3-\sigma_1$	
1	A	3.75	8.32	−0.92	0.17	0.0	0.16	3.92	8.33	−0.92	−4.41	9.25	−4.84	$1.2\ S_m$ = 13.8 kgf/mm²
	B	3.75	8.32	−0.92	0.17	0.0	[illegible]	3.91	8.34	−0.92	−4.43	9.26	−4.83	
	C	3.75	8.32	−0.92	0.17	0.0	[illegible]	3.90	8.34	−0.92	−4.44	9.26	−4.82	
	D	3.75	8.32	−0.92	0.17	0.0	0[illegible]	3.92	8.32	−0.92	−4.40	9.24	−4.84	
14	A	1.38	3.49	−0.92	0.06	0.0	0.[illegible]	[illegible]4	3.49	−0.92	−2.06	4.41	−2.36	min (S_y, ⅔ S_u) = 28.8 kgf/mm²
	B	1.38	3.49	−0.92	0.06	0.0	0.0[illegible]	1.43	3.49	−0.[illegible]	−2.06	4.41	−2.35	
	C	1.38	3.49	−0.92	0.06	0.0	0.11	1.43	3.50	−0.92	−2.06	4.42	−2.35	
	D	1.38	3.49	−0.92	0.06	0.0	0.04	1.44	3.49	−0.92	−2.05	4.41	−2.36	

(During S_1 earthquake)



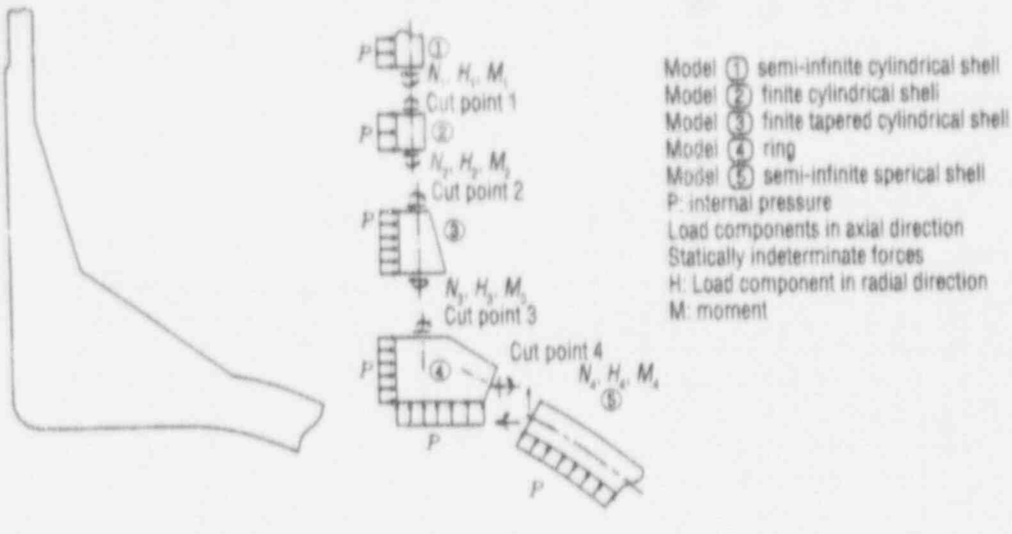


Figure 6.6.2-6. Model of statically indeterminate method.

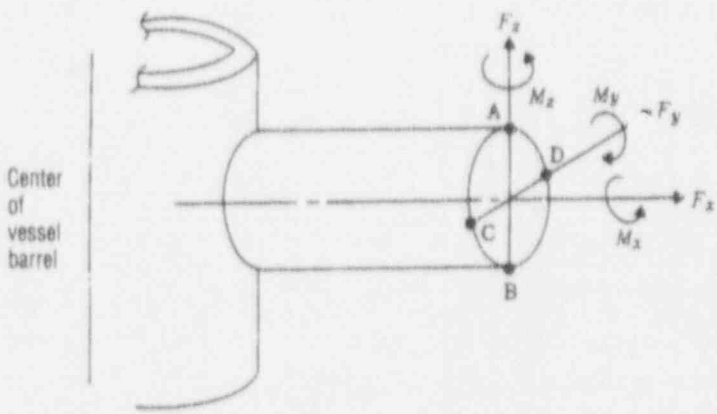


Figure 6.6.2-7. Loads acting on nozzle.

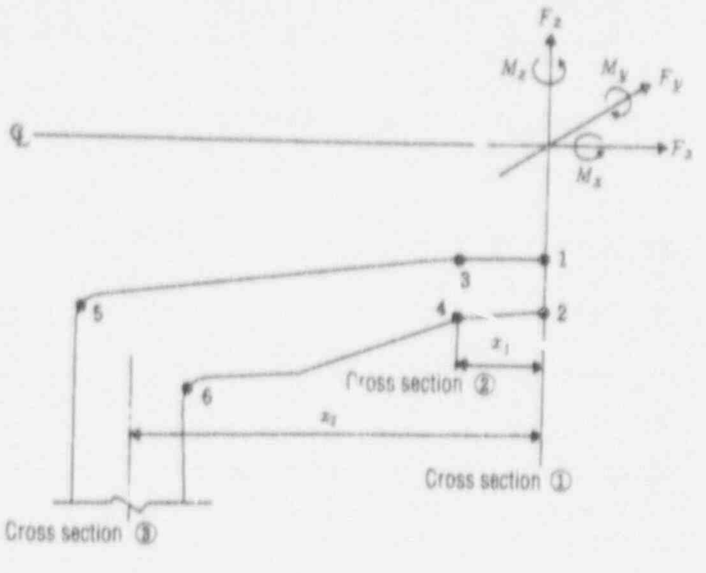


Figure 6.6.2-8. Loads on various cross sections of the nozzle.

Cross section {1}

$$F_x' = F_x \qquad F_y' = F_y \qquad F_z' = F_z$$
$$M_x' = M_x \qquad M_y' = M_y \qquad M_z' = M_z$$

Cross section {2}

$$F_x' = F_x \qquad F_y' = F_y \qquad F_z' = F_z$$
$$M_x' = M_x \qquad M_y' = M_y - x_1 F_z \qquad M_z' = M_z - x_1 F_y$$

Cross section {3}

$$F_x' = F_x \qquad F_y' = F_y \qquad F_z' = F_z$$
$$M_x' = M_x \qquad M_y' = M_y - x_2 F_z \qquad M_z' = M_z - x_2 F_y$$

Stresses on the cross sections of the nozzle are calculated using the following stress calculation formulas.

Cross sections {1} and {2}

$$\text{Cross-sectional area:} \quad A = \pi\left(r_0^2 - r_i^2\right) \tag{6.6.2-1}$$

$$\text{Moment of inertia:} \quad I = \pi\left(r_0^4 - r_i^4\right)/4 \tag{6.6.2-2}$$

Stress: σ_x

In barrel's axial direction (points A, B): $$\sigma_x = \frac{F_x}{A} \pm \frac{M_y C}{I} \tag{6.6.2-3}$$

In barrel's circumferential direction (points C, D): $$\sigma_x = \frac{F_x}{A} \pm \frac{M_z C}{I} \tag{6.6.2-4}$$

$$(\text{Points } A, B): \quad \tau = \frac{M_x C}{2I} \mp \frac{F_y}{A} \tag{6.6.2-5}$$

$$(\text{Points } C, D): \quad \tau = \frac{M_x C}{2I} \mp \frac{F_z}{A} \tag{6.6.2-6}$$

where, r_0: outer radius, r_i: inner radius, C: radius of gyration

The stress at the joint portion between nozzle and vessel at cross section {3} can be obtained by using Bijlaard's method. According to Bijlaard's method, the stresses are calculated using the following formulas from the internal forces N_x, N_ϕ, M_x, M_ϕ as dimensionless variables generated in the cylindrical shell or spherical shell by the 6 external force components F_x, F_y, F_z, M_x, M_y, M_z.

$$\sigma_x = \frac{N_x}{t} \pm \frac{6M_x}{t^2} \tag{6.6.2-7}$$

$$\sigma_\theta = \frac{N_\theta}{t} \pm \frac{6M_\theta}{t^2} \tag{6.6.2-8}$$

The double signs indicate the stresses on the inner and outer surfaces of the mounting portion.

In the barrel's axial direction (points A, B)

$$\sigma_x = (\sigma_x)F_x + (\sigma_x)M_y \qquad (6.6.2\text{-}9)$$

$$\sigma_\theta = (\sigma_\theta)F_x + (\sigma_\theta)M_y \qquad (6.6.2\text{-}10)$$

In the barrel's circumferential direction (points C, D)

$$\sigma_x = (\sigma_x)F_x + (\sigma_x)M_z \qquad (6.6.2\text{-}11)$$

$$\sigma_\theta = (\sigma_\theta)F_x + (\sigma_\theta)M_z \qquad (6.6.2\text{-}12)$$

Shear stress τ

$$\text{(Points } A,B\text{):} \quad \tau = \frac{M_x}{2\pi r_0^2 t} \mp \frac{F_y}{\pi r_0 t} \qquad (6.6.2\text{-}13)$$

$$\text{(Points } C,D\text{):} \quad \tau = \frac{M_x}{2\pi r_0^2 t} \mp \frac{F_z}{\pi r_0 t} \qquad (6.6.2\text{-}14)$$

The following is an example of Bijlaard's method used in the case of a cylindrical shell.

Bijlaard's method [6.6.3-3]

Bijlaard's method is a method of calculating the stress generated on the shell side when an external force acts on a rigid attachment mounted on a cylindrical shell or a spherical shell by using computing diagrams and tables. The details of this method are described in the reference.

In the following, on the base of the reference, we will present an abstract of the calculation method in the case when a circular attachment is mounted on a cylindrical shell, a case encountered frequently.

(a) Cylindrical shell parameters

(1) Shell parameter: γ

$$\gamma = R_m / T$$

where R_m: central radius of cylindrical shell; T: wall thickness of cylindrical shell

(2) Attachment parameter: β

$$\beta = \frac{0.875 r_0}{R_m}$$

where r_0: outer radius of cylindrical attachment

(b) Symbols

- By considering the deformation of shell caused by various loads, it is possible to determine the stress symbols [(+)...tension, (−)...compression]. (See Diagram 6.6.2-1.)

- In case 1, the force acting in the radial direction toward the center is similar to a local external pressure; hence, the stresses generated by it can be classified as follows:

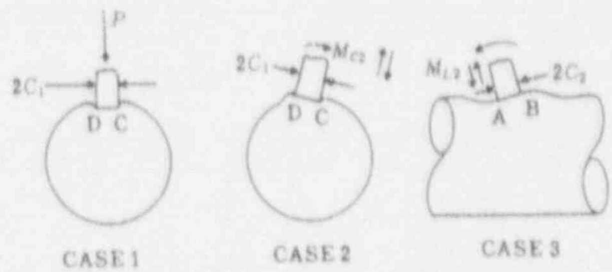
Shell: compressive membrane stress
Outer surface at points C, D: compressive flexural stresses
Inner surface at points C, D: tensile flexural stresses

- In cases 2 and 3, the bending moment can be taken as a pair of forces with the same magnitude and acting in opposite directions, and the stresses can be classified as follows:

Shell at points B, D: tensile membrane stress
Shell at points A, C: compressive membrane stress
Inner surface at points A, C: tensile flexural stress
Outer surface at points B, D: tensile flexural stress
Outer surface at points A, C: compressive flexural stress
Inner surface at points B, D: compressive flexural stress

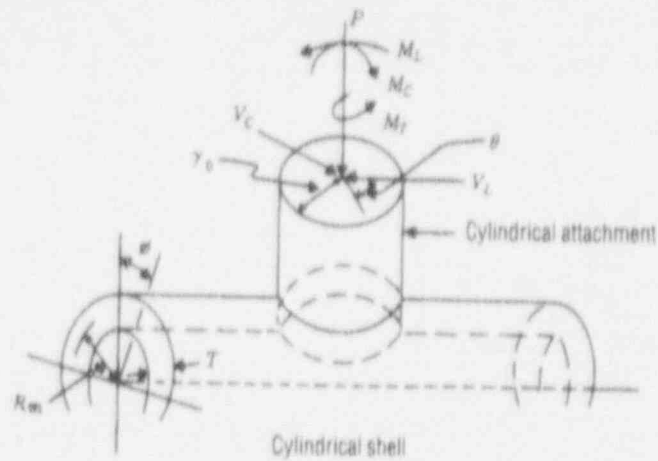
(c) Stress calculation

When external forces act as shown in Diagram 6.6.2-2, the stresses are calculated as follows. In the explanation, "Figure 3C" refers to the figure in reference [6.6.3-3], with examples illustrated in Appendix Figures 1-3. As an example, the stress due to radial force (P) can be determined using the following procedure.



P: Force in radial direction
M_C: Bending moment in circumferential direction
M_L: Bending moment in axial direction

Diagram 6.6.2-1.



P Force in radial direction
M_L Bending moment in axial direction
M_C Bending moment in circumferential direction
M_T Torsional moment
V_L Axial shear force
V_C Circumferential shear force
r_0 Radius of circular attachment

Diagram 6.6.2-2.

(1) Circumferential stress (σ_ϕ)

Step 1: According to "Figure 3C" (Appendix Figure 1), β and γ are used to determine dimensionless membrane force $N_\phi/(P/R_m)$.

Step 2: Similarly, according to "Figure 1C" (Appendix Figure 2), dimensionless bending moment (M_ϕ/P) is determined.

Step 3: From the known values of P, R_m, and T, membrane stress (N_ϕ/T) can be calculated:

$$\frac{N_\phi}{T} = \frac{N_\phi}{P/R_m} \cdot \frac{P}{R_m T}$$

Step 4: Similarly, flexural stress ($6M_\phi/T^2$) can be calculated:

$$\frac{6M_\phi}{T^2} = \frac{M_\phi}{P} \cdot \frac{6P}{T^2}$$

Step 5: Hence, with appropriate symbols, the circumferential stress can be expressed as follows:

$$\sigma_\phi = K_n \frac{N_\phi}{T} \pm K_b \frac{6M_\phi}{T^2}$$

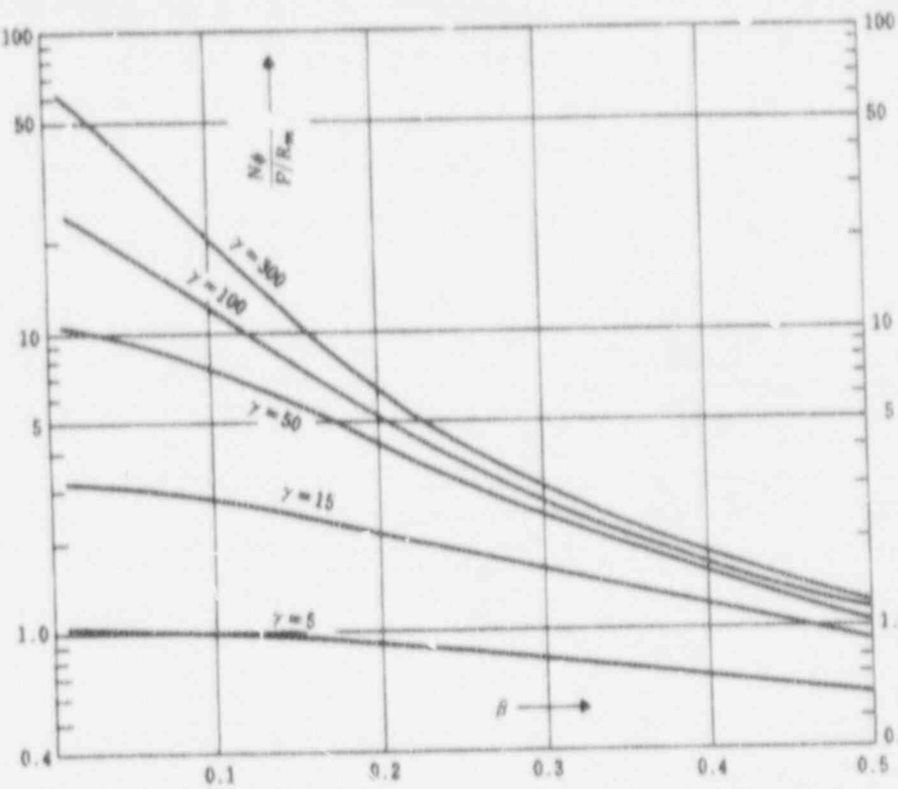


Fig. 1C - Membrane force $N_\phi/(P/R_m)$ due to an external radial load P on a circular cylinder

Appendix Figure 1. Stresses in shells.

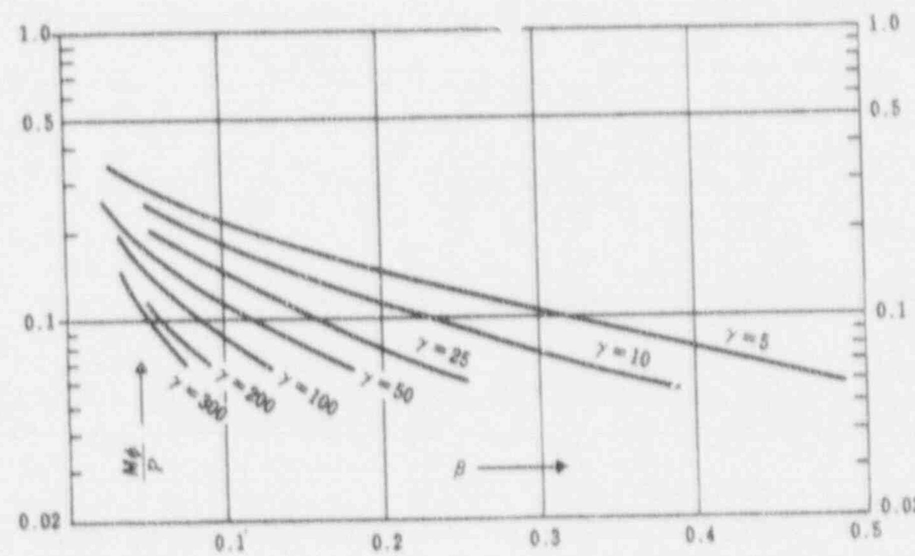


Fig. 1C - Bending moment M_ϕ/P due to an external radial load P on a circular cylinder (transverse axis)

Appendix Figure 2. Stresses in shells.

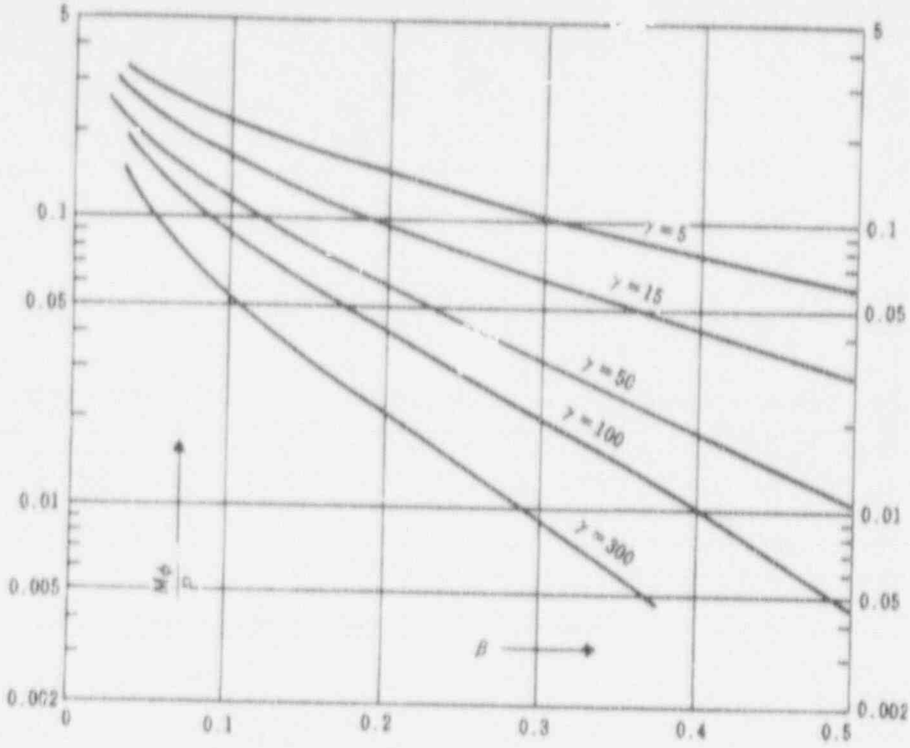


Fig. 1C-1 - Bending moment M_ϕ/P due to an external radial load P on a circular cylinder (longitudinal axis)

Appendix Figure 3. Stresses in shells.

(2) Axial stress (σ_x)

Similarly, $N_x/P/R_m$ and M_x/P can be determined from "Figure 3C" and "Figure 1C-1," and stresses are calculated as follows:

$$\frac{N_x}{T} = \frac{N_x}{P/R_m} \cdot \frac{P}{R_m T}$$

$$\frac{6M_x}{T^2} = \frac{M_x}{P} \cdot \frac{6P}{T^2}$$

Hence,

$$\sigma_x = K_n \frac{N_x}{T} \pm K_b \frac{6M_x}{T^2}$$

Also, the stress due to M_L and M_C can be calculated in the same procedure as above.

(iii) Finite element method (FEM)

According to finite element method, the analysis portion is divided into a finite number of elements, and the structural body is analyzed as an assembly of these elements.

According to this analysis method, for the nodal points determined to be compatible with the strain states in elements and the boundary conditions, the relations between nodal forces and displacements are derived and the results are summed up for all of the elements. That is, assume the nodal load of the overall structure is [R] and the unknown nodal displacement is [D], the equilibrium equation of the structure acted by known loads can be represented by the following stiffness equation:

$$[R] = [K]\,[D]$$

where [K] is a stiffness matrix of the overall system. By determining the displacement relation to ensure the continuity of displacements at the boundary of each element, the stiffness matrix for each element can be formed. As it is summed up for all of the elements, the stiffness equation for the overall structure can be derived.

As far as the scheme of representation of load is concerned, pressure and mechanical loads are taken as the equivalent nodal forces on the nodal points of elements; for the thermal load, temperature is directly input to the nodal points.

Figure 6.6.2-9 shows the procedure of stress analysis using the finite element method. Figure 6.6.2-10 shows an example of the analysis model.

Since the stress calculated by the finite element method is primary + secondary + peak stress, in order to perform stress evaluation, it is necessary to classify it into primary stress and primary + secondary stress.

(c) Classification of stresses

Classification of the stress determined using the statically indeterminate method is performed according to the location of the stress and the type of the load. (See Table 2.3 "Stress classification" in Chapter 1 of "Notification No. 501").

The stress calculated using the finite element method is a primary + secondary + peak stress. In order to perform stress evaluation, this stress must be classified into primary stress and primary + secondary stress according to the definition in section "6.6.2(1)b. Stress classification." The basic scheme of the classification can be explained with reference to the example shown in Figure 6.6.2-11.

{1} Primary stress

The primary stress is the average stress on the cross section. Therefore, it is obtained by averaging the primary + secondary + peak stress over the cross section. In addition, as the general primary membrane stress is a membrane stress which is not affected by the discontinuity of structure and stress concentration, it may be obtained using the conventional equations in some cases.

{2} Primary + secondary stress

For primary + secondary stress, first, the flexural stress, which is a secondary stress component, is calculated; then, it is added to the primary stress. The flexural stress is calculated at any position of the cross section by first determining the bending moment and then integrating it, followed by dividing by the section modulus.

{3} Primary + secondary + peak stress

When the elements are divided very finely in the finite element method, since the sum of the stress includes the peak stress, the results can be used directly. In the case when the peak stress component is not reflected, an appropriate stress concentration factor is multiplied by the primary + secondary stress.

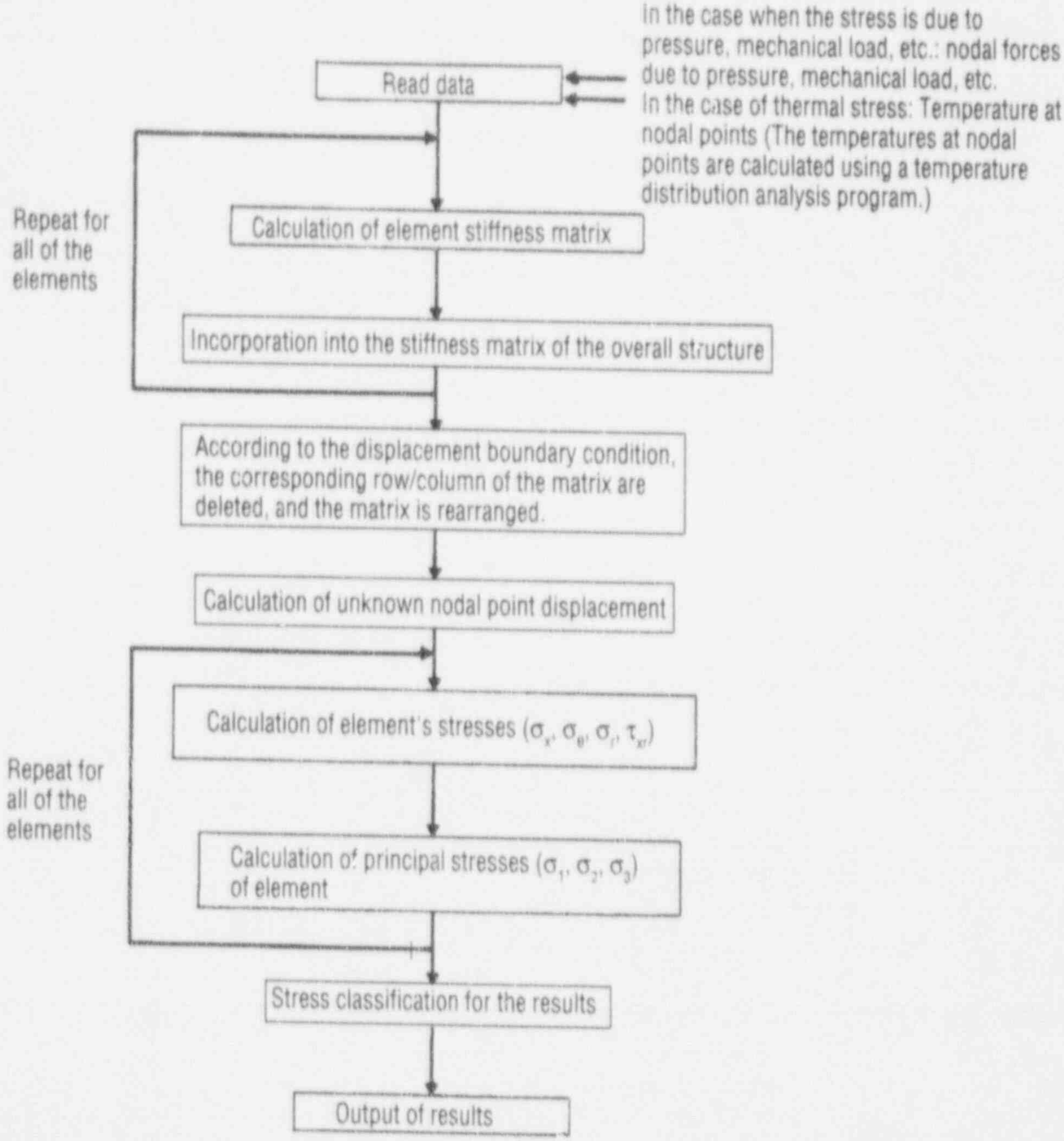


Figure 6.6.2-9. Stress analysis procedure using finite element method (in the case of two-dimensional axisymmetric body).

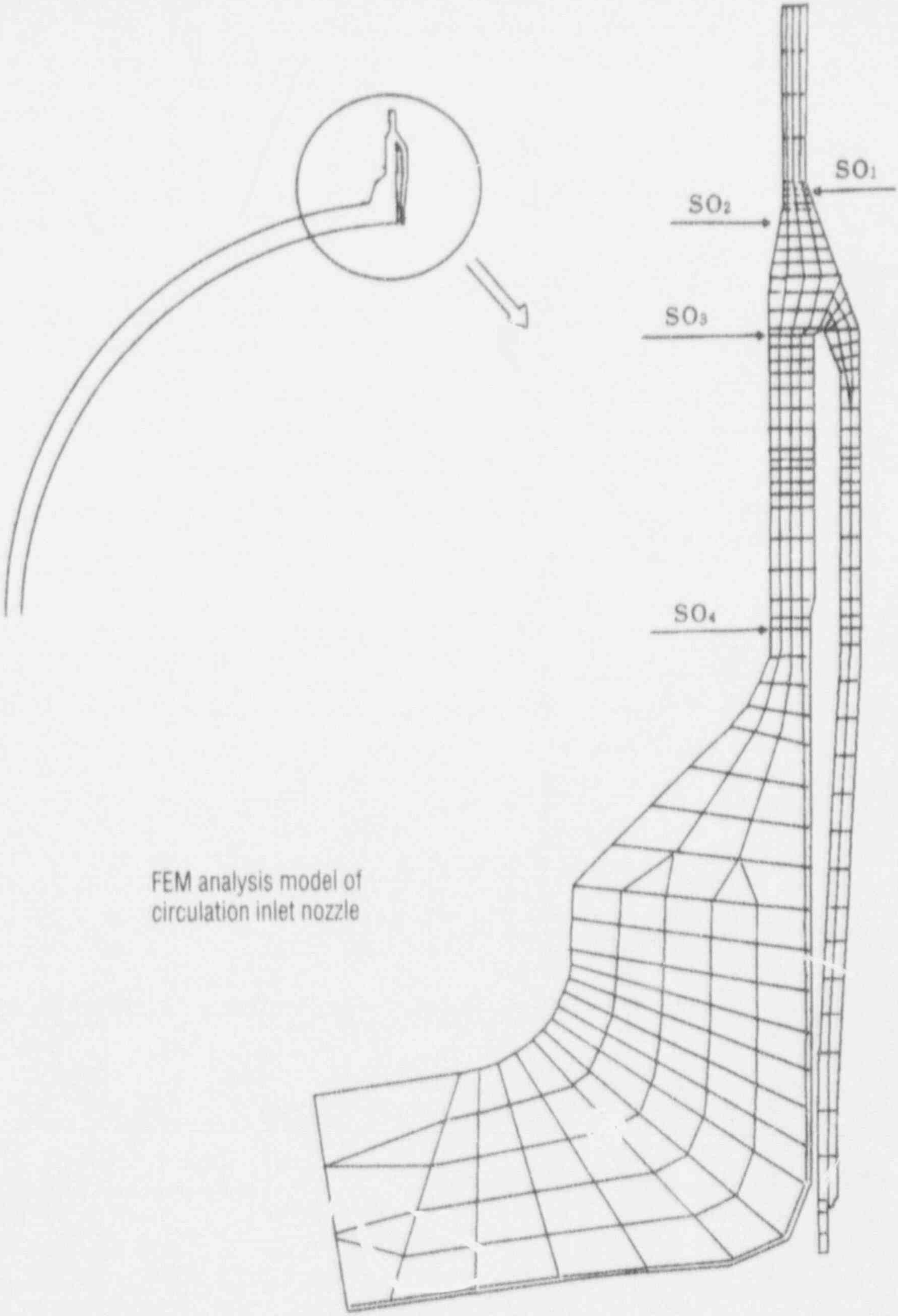


Figure 6.6.2-10. Example of analysis using finite element method.

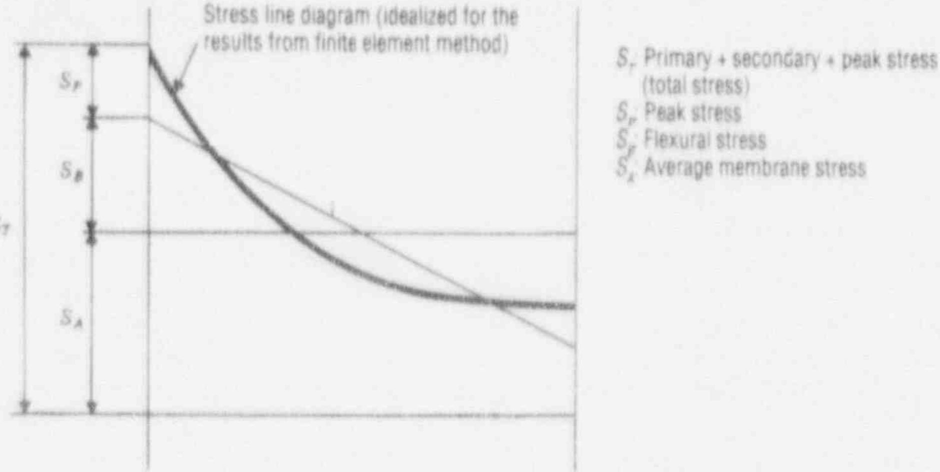


Figure 6.6.2-11. Stress classification (example in the case of calculation using finite element method).

(d) Stress intensity

The stresses obtained as results of analysis are classified and are evaluated by determining the principal stress and stress intensity according to the following procedure.

Calculation of principal stress

The calculated and classified stresses are summed up for each stress component.

The summed stress usually has 6 components, i.e., σ_x, σ_θ, σ_r, $\tau_{x\theta}$, $\tau_{\theta r}$, τ_{xr}. The principal stresses σ_1, σ_2, σ_3 are calculated as the 3 roots that satisfy the following equation:

$$\begin{aligned}&\sigma^3-(\sigma_x+\sigma_\theta+\sigma_r)\sigma^2+(\sigma_\theta\sigma_r+\sigma_r\sigma_x+\sigma_x\sigma_\theta-\tau_{x\theta}^2-\tau_{xr}^2-\tau_{\theta r}^2)\sigma\\&-\sigma_x\sigma_\theta\sigma_r+\sigma_x\tau_{\theta r}^2+\sigma_\theta\tau_{xr}^2+\sigma_r\tau_{x\theta}^2-2\tau_{x\theta}\tau_{xr}\tau_{\theta r}=0\end{aligned} \tag{6.6.2-15}$$

When there are only components σ_x, σ_θ, σ_r, $\tau_{x\theta}$, the principal stresses σ_1, σ_2, σ_3 can be derived from

$$\sigma_1,\sigma_2=\frac{\sigma_x+\sigma_\theta}{2}\pm\sqrt{\frac{(\sigma_x-\sigma_\theta)^2}{2}+\tau_{x\theta}^2} \tag{6.6.2-16}$$

$$\sigma_3=\sigma_r \tag{6.6.2-17}$$

Stress intensity

Among the following three principal stress differences, the largest absolute value is known as stress intensity.

$$S_{12} = \sigma_1 - \sigma_2, \quad S_{23} = \sigma_2 - \sigma_3, \quad S_{13} = \sigma_1 - \sigma_3 \tag{6.6.2-18}$$

b. Piping

Typical examples of the Type 1 piping include BWR-PLR piping, PWR-primary coolant piping, etc. The analysis models with respect to the seismic response analysis are described in section "6.5.2 Dynamic analysis method" and section "6.5.4 Seismic response analysis method." In the response analysis of piping, the seismic load obtained as a result of the seismic response analysis is regarded as a mechanical load, and evaluation is performed with other loads also taken into consideration. The procedure of the stress analysis is illustrated in Figure 6.6.2-12. The stress calculation formula for the Type 1 piping with respect to the seismic load are as follows. Evaluation is performed in terms of the stress intensity based on "Notification No. 501."

(a) In allowable stress states III_AS and IV_AS, the primary stress is calculated using the following formulas.

(i) For nozzle and butt weld type tee

$$S = \frac{B_1 P D_0}{200t} + \frac{B_{2b} M_{bp}}{Z_b} + \frac{B_{2r} M_{rp}}{Z_r} \tag{6.6.2-19}$$

where
- S: Primary stress (kgf/mm^2)
- P: Pressure in operation state which should be combined with earthquake (kgf/cm^2)
- D_0: Outer diameter of pipe (mm)
- t: Thickness of pipe wall (mm)
- M_{bp}: Bending moment due to mechanical load (including inertial force due to earthquake) of branch pipe connected to the nozzle or tee (kgf-mm)
- M_{rp}: Bending moment due to mechanical load (including inertial force due to earthquake) of principal pipe connected to the nozzle or tee (kgf-mm)
- Z_b: Sectional modulus of branch pipe connected to nozzle or tee (mm^3)
- Z_r: Sectional modulus of principal pipe connected to nozzle or tee (mm^3)
- B_1, B_{2b}, B_{2r}: Stress factors

(ii) For pipes other than those in (i)

$$S = \frac{B_1 P D_0}{200t} + \frac{B_2 M_{ip}}{Z_i} \tag{6.6.2-20}$$

where
- M_{ip}: Bending moment due to mechanical load (including inertial force due to earthquake) of pipe
- Z_i: Section modulus of pipe (mm^3)
- B_2: Stress factor
- S, B_1, P, D_0: Same as those defined in (i)

(b) In allowable stress states III_AS and IV_AS, the primary + secondary stress is calculated using the following formulas.

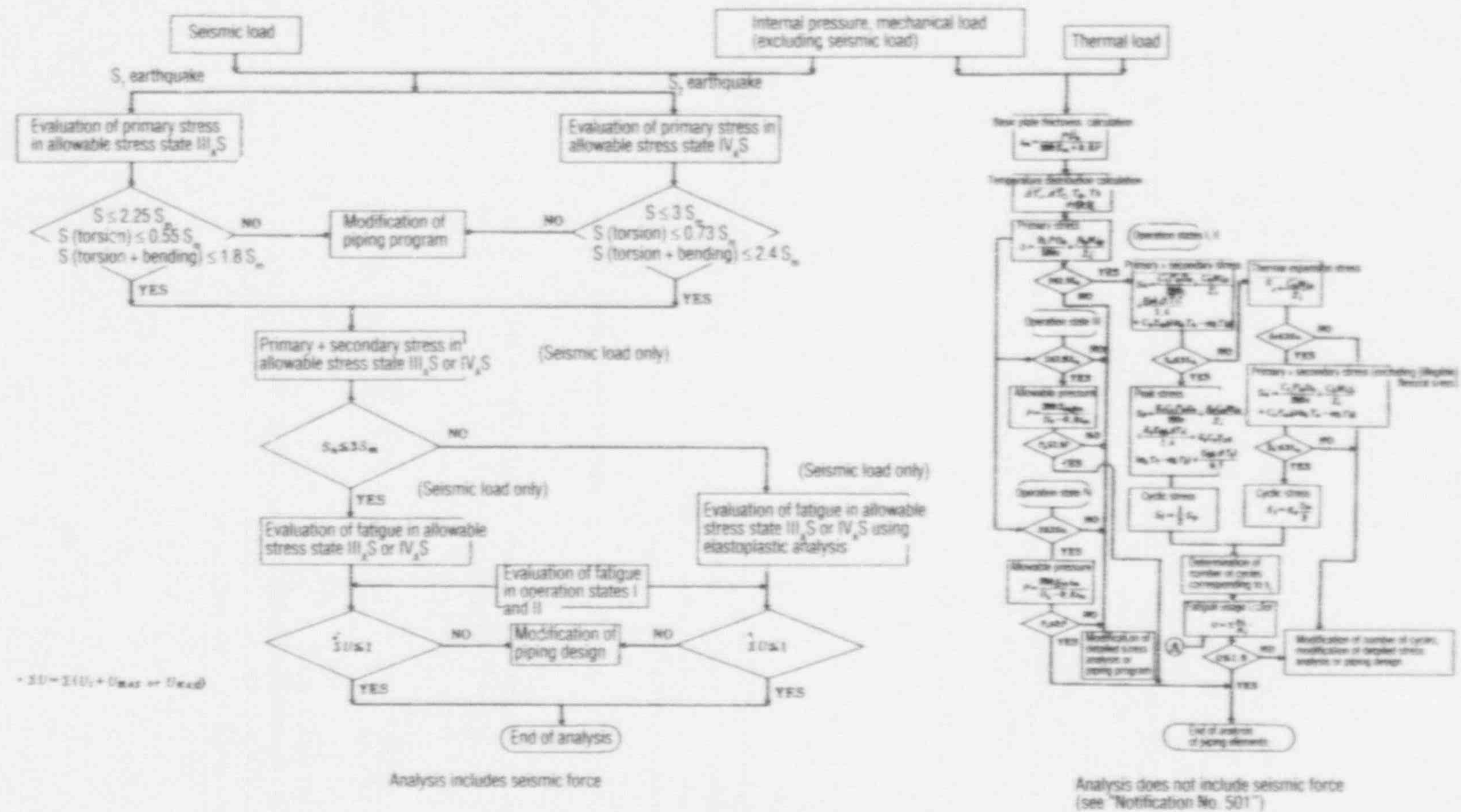


Figure 6.6.2-12. Stress analysis procedure of Type 1 piping.

(i) Nozzle and butt weld type tee

$$S_n = \frac{C_{2b} M_{bs}}{Z_b} + \frac{C_{2r} M_{rs}}{Z_r} \qquad (6.6.2\text{-}21)$$

where S_n: Stress obtained by adding primary stress and secondary stress (kgf/mm^2)

M_{bs}: Total amplitude range of bending moment generated by the inertial force and differential displacement due only to seismic motion S_1 or S_2 of the branched pipe connected to nozzle or tee (kgf·mm)

M_{rs}: Total amplitude range of bending moment generated by the inertial force and differential displacement due only to seismic motion S_1 or S_2 of the principal pipe connected to nozzle or tee (kgf·mm)

Z_b: Sectional modulus of branch pipe connected to nozzle or tee (mm^3)

Z_r: Sectional modulus of principal pipe connected to nozzle or tee (mm^3)

C_{2b}, C_{2r}: Stress factors in Item 48 of "Notification No. 501"

(ii) Pipes other than those in (i)

$$S_n = \frac{C_2 M_{is}}{Z_i} \qquad (6.6.2\text{-}22)$$

where M_{is}: Total amplitude range of bending moment generated by the inertial force and differential displacement due to seismic motion S_1 or S_2 only (kgf·mm)

C_2: Stress factor in Item 48 of "Notification No. 501"

Z_i: Section modulus of pipe (mm^3)

When S_n becomes larger than 3 S_m, elastoplastic analysis defined in "Notification 501" should be performed.

(c) The cyclic peak stress intensity used in the fatigue analysis of allowable stress states III_AS and IV_AS is calculated using the following formulas:

$$S_l = \frac{S_p}{2} \qquad (6.6.2\text{-}23)$$

where S_l: stress (kgf/mm^2)

S_p: value calculated using the following formulas

(i) For nozzle and butt connecting type tee

$$S_p = \frac{K_{2b} C_{2b} M_{bs}}{Z_b} + \frac{K_{2r} C_{2r} M_{rs}}{Z_r} \qquad (6.6.2\text{-}24)$$

(ii) For pipes other than those in (i)

$$S_p = \frac{K_2 C_2 M_{is}}{Z_i} \qquad (6.6.2\text{-}25)$$

K_{2b}, K_{2r}, K_2, C_{2b}, C_{2r}, C_2 are stress coefficients defined in Item 48 of "Notification No. 501." M_{bs}, M_{rs}, M_{is}, Z_b, Z_r, Z_i are the same as defined in section (b) above. As examples of the seismic load and stress generated in the piping, Tables 6.6.2-2 and 6.6.2-3 list the seismic loads and primary stresses at representative points in the PWR primary cooling equipment shown in Figure 6.5.2-13.

c. Pump

For the Type 1 pump, the stress analysis is performed in the same way as Type 1 vessels, with the operation state of the plant taken into consideration. Figure 6.6.2-13 shows the stress analysis procedure. In the stress analysis, the seismic load is taken as one of the external loads. The seismic load applied to the pump is due to the seismic inertial force generated in the pump body, the reaction force from piping and the support reaction force. As the pump stress analysis method, the two-dimensional finite element method is used for the casing subjected to the internal pressure load, thermal load and bolt fastening load. In addition, Bijlaard's method may be used for evaluation of the nozzle portion and casing mounting portion subjected to the seismic load. These methods are described in the vessel section. Please refer to that section.

d. Valve

Valves are designed against pressure and thermal loads according to "Notification No. 501." For the pipes connected to the valves, aseismic design is performed on the base of "Notification No. 501" as well as according to Item b above for the seismic load. The stiffness of the pressure portion of the valve body is much higher than the stiffness of the pipe connected to it. The strength against the seismic load is also higher at the pressure portion of the valve body than the piping side. As a result, even when no evaluation is made for the valve body with respect to the seismic load, it is acceptable if the design is performed according to "Notification No. 501." For purpose of reference, Figure 6.6.2-14 illustrates the procedure of stress analysis with respect to the internal pressure load and temperature load of the valve that is given in Article 81 of "Notification No. 501." For the valves connected to pipes with outer diameter smaller than 115 mm, Article 81 of "Notification No. 501" points out that no evaluation need be performed. The reasons are as follows:

(1) The small-diameter valves are manufactured by casting or forging. The actual thickness is much larger than the minimum necessary thickness; hence, there is a sufficient margin in the strength.
(2) The small-diameter valves are used for general purpose, and the safety of the products is proven in many actual cases.
(3) The small-diameter valves are for general use and have a large quantity. Hence, it is difficult to require to perform stress analysis for all of them.
(4) According to ASME, it is also defined that no stress analysis is needed for valves connected to pipes with nominal diameter of 4" (outer diameter of 115 mm) or smaller.

However, for valves which are predicted to have excessive high stresses during earthquake, "MITI Notification No. 501" Article 81 Item No. 1 points out that strength integrity should be confirmed according to stress evaluation.

However, the above provision is not applied for the case when appropriate measures to avoid generation of excessive high stress (for instance, fabrication of energy absorbing device) are used.

Table 6.6.2-2. Examples of seismic loads of PWR primary coolant piping.

Evaluation location	Type of load	Axial force (tf)			Bending moment (tf·m)		
		F_x	F_y	F_z	M_x	M_y	M_z
107	Self weight	−0.4	−0.1	17.8	0.1	−42.7	—
	S_1-X Earthquake	131.9	3.6	24.3	0.9	47.8	8.4
	S_1-Y Earthquake	61.3	7.4	10.0	4.9	21.1	17.5
	S_2-X Earthquake	225.8	6.2	39.7	1.5	76.7	14.3
	S_2-Y Earthquake	104.3	12.8	13.8	9.0	28.2	30.2
109	Self weight	−0.4	−0.1	10.5	0.1	−8.2	0.1
	S_1-X Earthquake	131.1	2.8	20.7	0.9	4.3	1.4
	S_1-Y Earthquake	60.7	5.9	7.2	4.9	3.3	3.3
	S_2-X Earthquake	224.4	4.8	34.7	1.5	6.1	2.4
	S_2-Y Earthquake	103.4	10.1	10.3	9.0	4.4	5.7
111	Self weight	−0.4	−0.1	10.5	0.1	12.2	0.2
	S_1-X Earthquake	131.1	2.8	20.7	0.9	39.6	4.3
	S_1-Y Earthquake	60.7	5.9	7.2	4.9	12.5	8.4
	S_2-X Earthquake	224.4	4.8	34.7	1.5	67.8	7.3
	S_2-Y Earthquake	103.4	10.1	10.3	9.0	18.6	14.4
112	Self weight	−0.4	−0.1	10.5	0.1	13.5	0.2
	S_1-X Earthquake	131.1	2.8	20.7	0.9	42.3	4.6
	S_1-Y Earthquake	60.7	5.9	7.2	4.9	13.4	9.2
	S_2-X Earthquake	224.4	4.8	34.7	1.5	72.2	8.0
	S_2-Y Earthquake	103.4	10.1	10.3	9.0	19.9	15.7

Table 6.6.2-3. Examples of primary stresses of PWR primary coolant piping (Units: kgf/mm²).

	Evaluation location	Stress due to pressure	Stress due to external load (self weight + earthquake)	Sum of stresses	Allowable stress
For S_1 earthquake	107	5.56	2.63	8.19	S_m = 26.5
	109	5.56	0.38	5.94	
	111	5.56	1.50	7.06	
	112	5.56	3.60	9.16	
For S_2 earthquake	107	5.56	3.48	9.04	3 S_m = 35.4
	109	5.56	0.48	6.04	
	111	5.56	2.32	7.88	
	112	5.56	5.54	11.10	

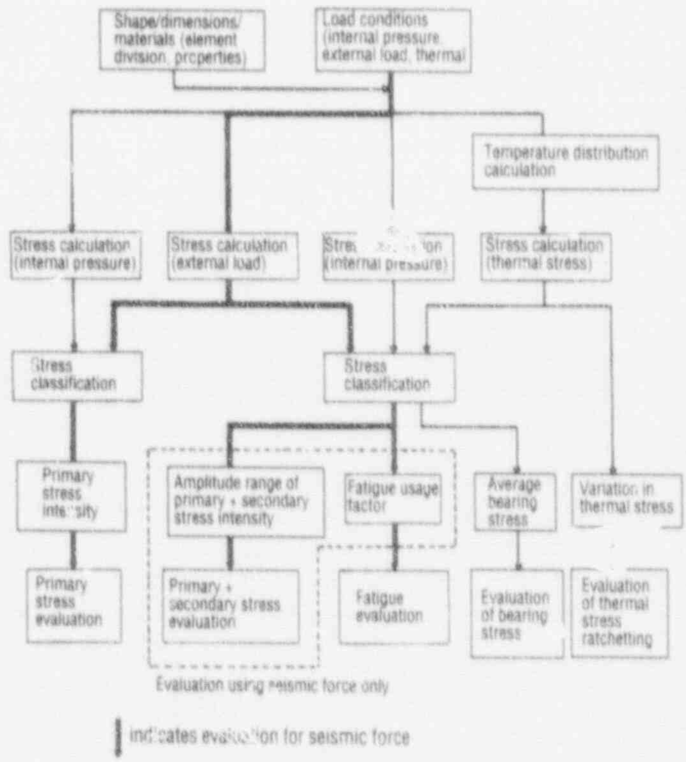
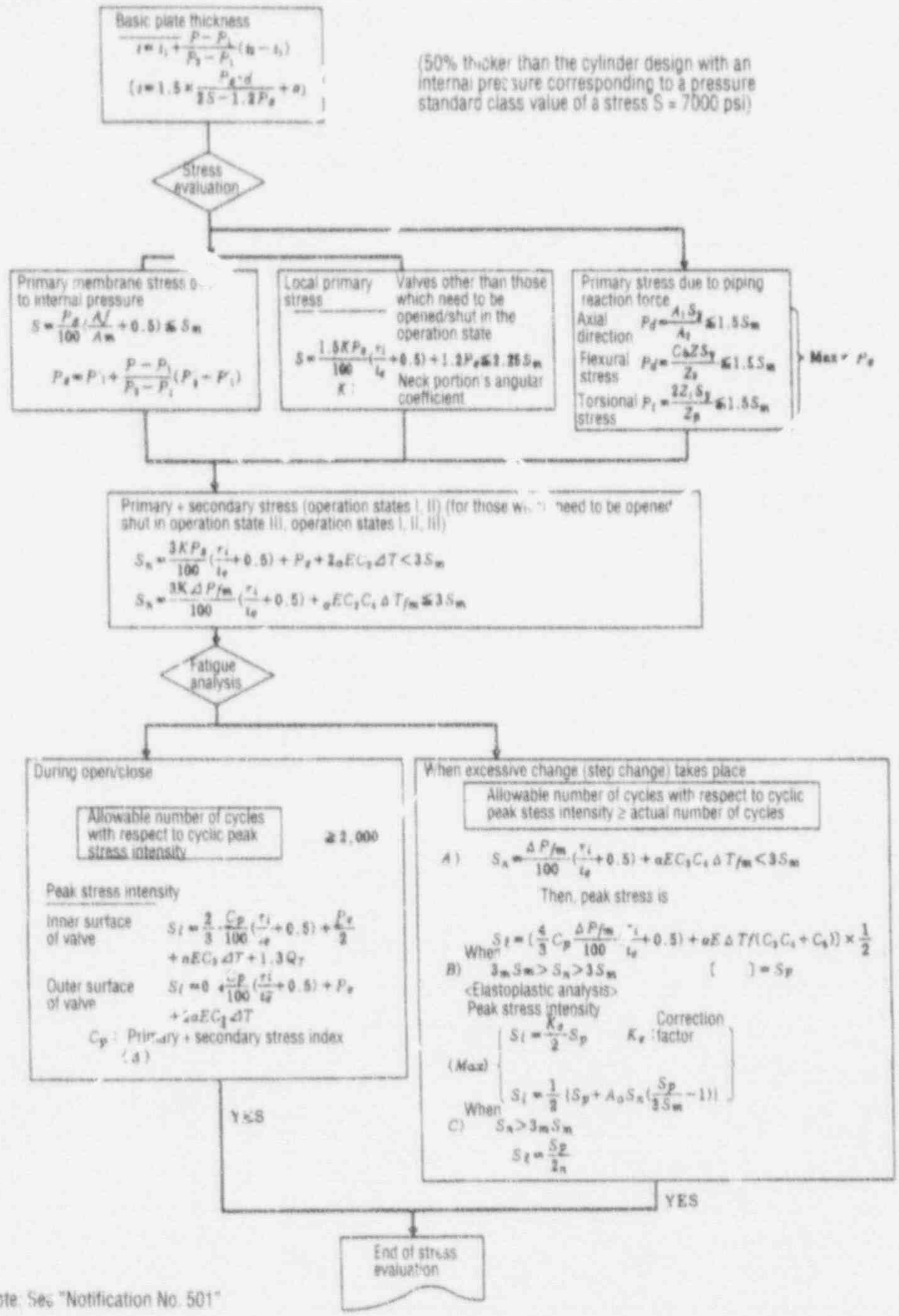


Figure 6.6.2-13. Stress analysis procedure of Type 1 pump.



Note: See "Notification No. 501"

Figure 6.6.2-14. Procedure of stress analysis of valve.

(3) Type 2 vessels

a. Type 2 vessel main body

For the main body of the type 2 vessels (nuclear containment vessel), an MDOF beam model is used as part of the soil-structure interaction model to perform the seismic response analysis to determine the shear force and bending moment. The procedure of the stress evaluation is shown in Figure 6.6.2-15 (PWR example) and Figure 6.6.2-16 (BWR example).

The stresses due to the seismic load are calculated using the following formulas, and they are evaluated in terms of the stress intensity.

$$\sigma_x = \pm\frac{M_i}{Z_i} + \left(\pm\frac{W_i}{A_i}C_V\right) \tag{6.6.2-26}$$

$$\sigma_y = \sigma_z = 0 \tag{6.6.2-27}$$

$$\tau = \frac{2Q_i}{A_i} \tag{6.6.2-28}$$

where M_i: Bending moment due to seismic load at the calculation point
Z_i: Section modulus of containment vessel at the calculation point
W_i: Self weight acting at the calculation point
A_i: Cross-sectional area of containment vessel at the calculation point = πDt
D: Average diameter of containment vessel at the calculation point
t: Wall thickness of containment vessel at the calculation point
Q_i: Shear force due to seismic load at the calculation point
C_V: Vertical seismic coefficient at the calculation point
σ_x: Stress in axial direction
σ_y: Stress in circumferential direction
σ_z: Stress in wall thickness direction
τ: Shear stress

Also, for the stresses due to the seismic load, the evaluation points are selected at high-stress locations. Figure 6.6.2-17 shows the evaluation points for a PWR. Figure 6.6.2-18 shows the evaluation points for a BWR. The aforementioned stresses due to the seismic load are combined appropriately with the stresses due to self weight, stresses due to crane wheel load or shear lug local load, and stresses due to pressure in accident for stress evaluation. Table 6.6.2-4 lists the examples of the primary general membrane stress of the PWR nuclear containment vessel. As pointed out in section "6.5.2(2) Vessels," for the PWR nuclear reactor containment vessel, in addition to the response obtained by the multiple discrete mass beam model, there are also the ovalization type vibration. In order to evaluate the effects of these vibration characteristics, an axisymmetric shell model is used to determine the ovalization response. However, it is safer to evaluate the bending moment at the fixed end, which is important in the strength evaluation of the nuclear reactor containment vessel, by using the beam-discrete mass system model.

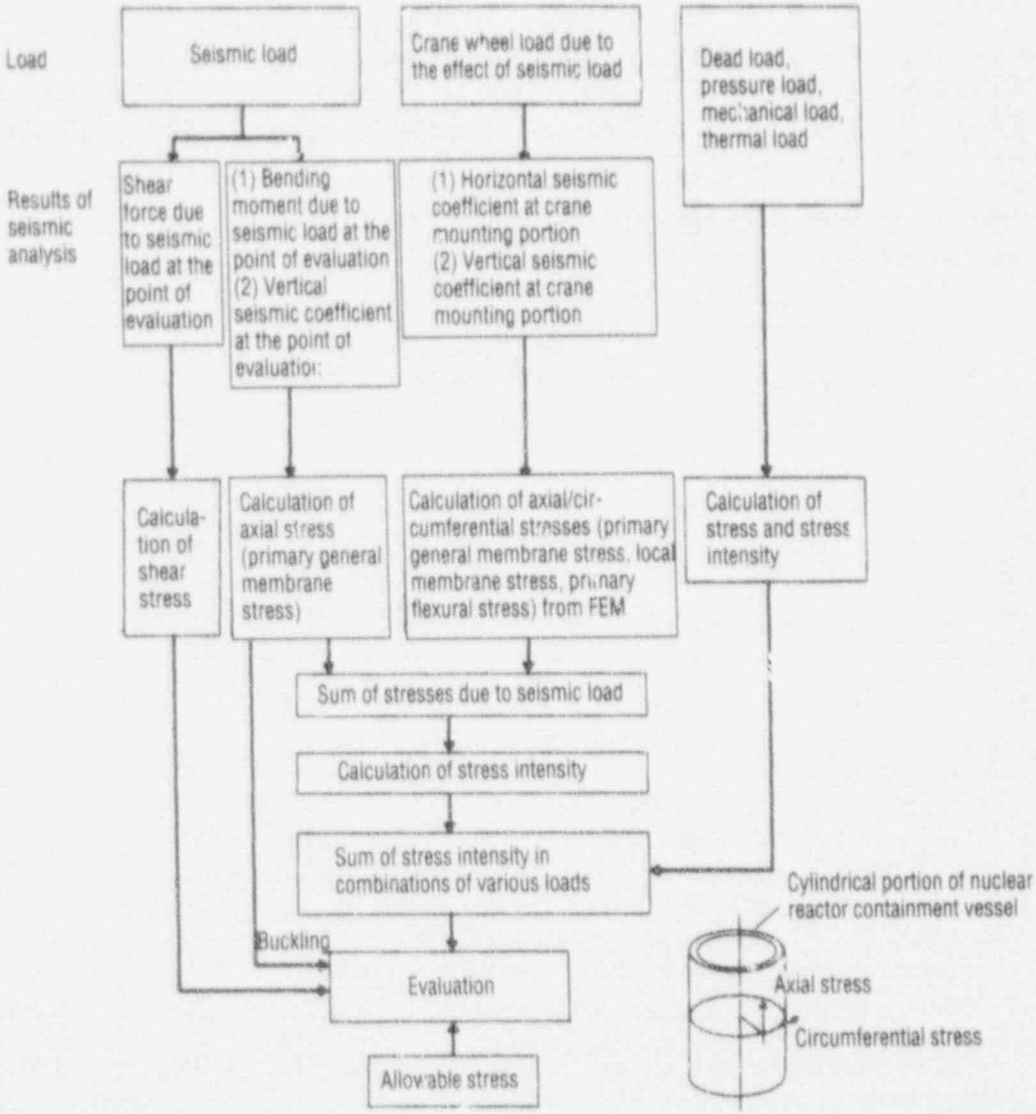


Figure 6.6.2-15. Procedure of stress evaluation of Type 2 vessel (PWR, as an example).

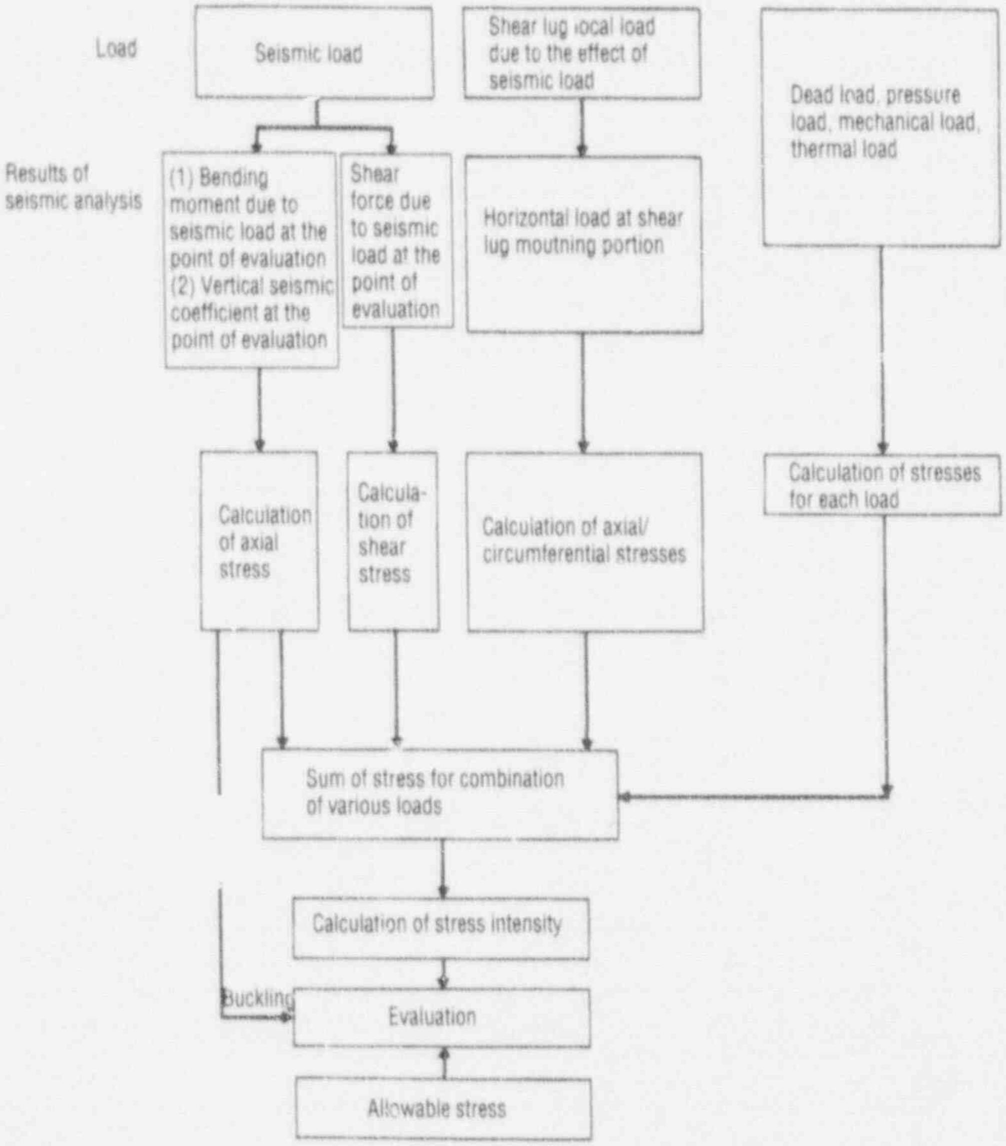
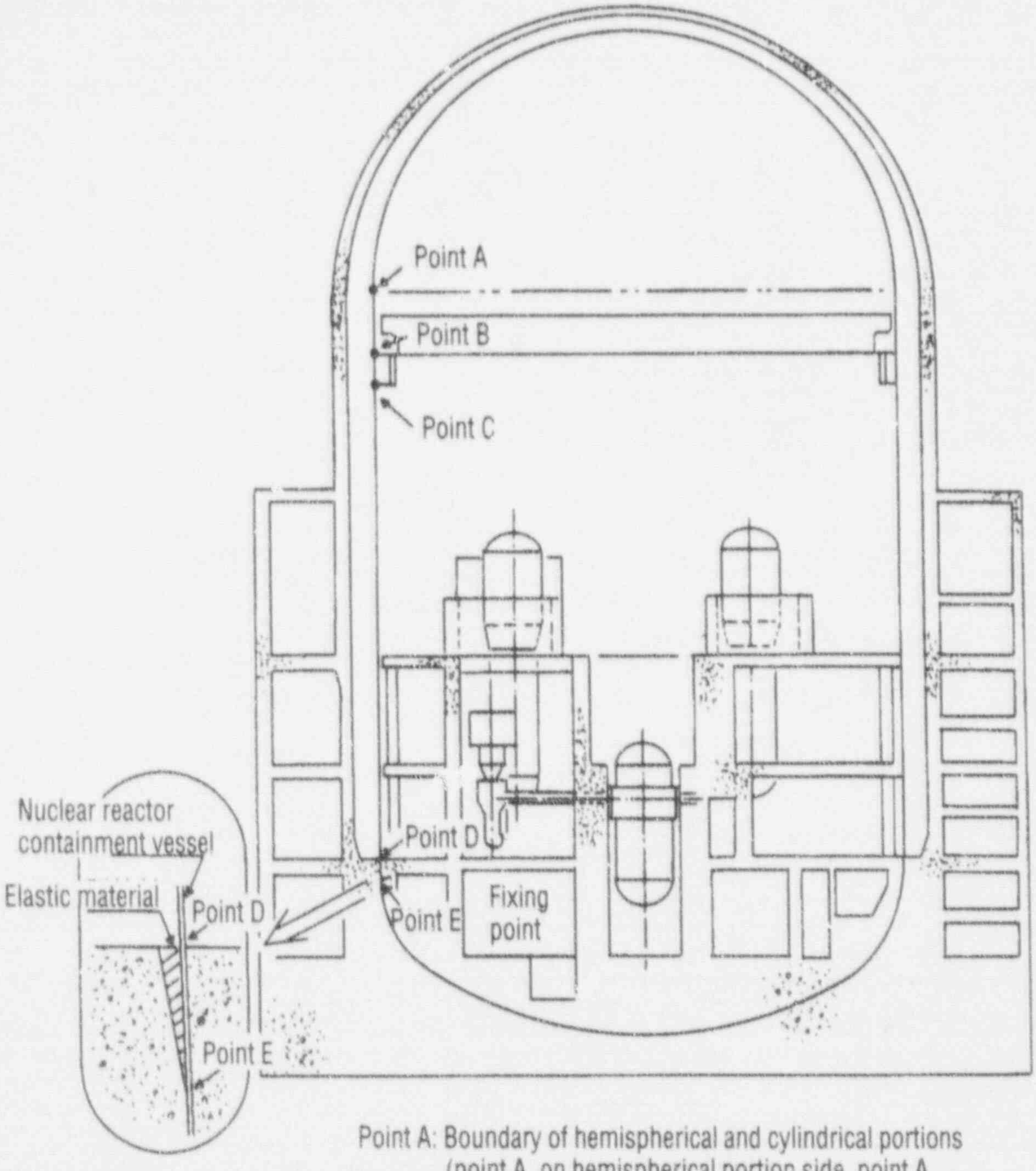


Figure 6.6.2-16. Procedure of stress evaluation of Type 2 vessel (BWR, as an example).



Point A: Boundary of hemispherical and cylindrical portions (point A_1 on hemispherical portion side, point A_2 on cylindrical portion side)
Point B: Upper end of ring girder
Point C: Lower end of ring girder
Point D: Upper end of elastic material
Point E: Lower end of elastic material (fixed point)

Figure 6.6.2-17. Points of stress evaluation of nuclear reactor containment vessel (PWR, as an example).

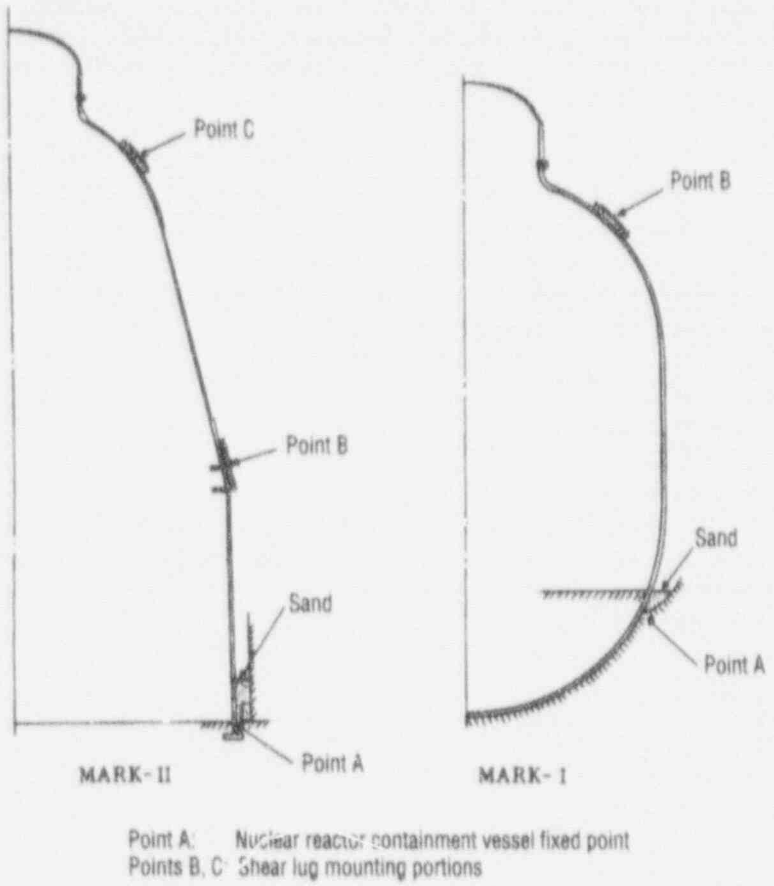


Figure 6.6.2-18. Stress evaluation points of nuclear reactor containment vessel (BWR, as example).

Table 6.6.2-4. Primary general membrane stress of nuclear reactor containment vessel (PWR, as example).

Evaluation point	Normal operation			LOCA			LOCA + S_1 seismic motion			Normal operation + S_2 seismic motion		
	$\sigma_x-\sigma_y$	$\sigma_y-\sigma_z$	$\sigma_z-\sigma_x$	$\sigma_x-\sigma_y$	$\sigma_y-\sigma_z$	$\sigma_z-\sigma_x$	$\sigma_x-\sigma_y$	$\sigma_y-\sigma_z$	$\sigma_z-\sigma_x$	$\sigma_x-\sigma_y$	$\sigma_y-\sigma_z$	$\sigma_z-\sigma_x$
Point E	−0.66	0	0.66	−7.24	13.20	−5.96	−4.40	13.20	−8.80	3.49	0	−3.49
							−10.08		−3.12	−4.81		4.81
Allowable stress	—			S_y = 24.0 kgf/mm^2			S_y = 24.0 kgf/mm^2			S_y = 24.0 kgf/mm^2		

At the fixed points of the nuclear reactor containment vessel, with respect to the axial compressive force and bending moment during earthquake, the following formula is used for evaluating buckling. The fixed portion is shown in Figure 6.6.2-17 as an example, with an elastic material. The buckling evaluation, however, is usually performed at a fixed point (point E) for conservative estimate.

$$\frac{\alpha(P/A)}{f_c} + \frac{\alpha(M/Z)}{f_b} \leq 1 \qquad (6.6.2\text{-}29)$$

This formula is applicable with l/R less than 5. When l/R is less than 0.5 due to a stiffening ring, etc., the effect may be analyzed by performing a separate evaluation.

P: Axial compressive load (kgf)
A: Cross-sectional area (mm^2)
M: Bending moment (kgf·mm)
Z: Section modulus (mm^3)
f_c: Buckling stress with respect to the axial compressive load; it is calculated as below (kgf/mm^2)
f_b: Buckling stress with respect to the bending moment; it is calculated as below (kgf/mm^2)
α: Safety factor; it is calculated as below (kgf/mm^2)
l: Barrel length (mm)
R: Average radius of the cylinder (mm)

Buckling stress with respect to compressive load (f_c)

$$f_c = \begin{cases} F & (\eta \leq \eta_1) \\ F \times \left\{1 - \dfrac{1}{6800}\left(F - \psi_c\{\eta_2\}\right)(\eta - \eta_1)\right\} & (\eta_1 < \eta < \eta_2) \\ \phi_c(\eta) & (\eta_2 \leq \eta \leq 800) \end{cases} \qquad (6.6.2\text{-}30)$$

$$\phi_c(\eta) = 0.6\frac{E}{\eta}\left(1 - 0.901\left\{1 - \exp\left(-\frac{1}{16}\sqrt{\eta}\right)\right\}\right) \qquad (6.6.2\text{-}31)$$

Buckling stress with respect to bending moment (f_b)

$$f_b = \begin{cases} F & (\eta \leq \eta_1) \\ F \times \left\{1 - \dfrac{1}{8400}\left(F - \phi_b\{\eta_3\}\right)(\eta - \eta_1)\right\} & (\eta_1 < \eta < \eta_3) \\ \phi_b(\eta) & (\eta_3 \leq \eta \leq 800) \end{cases} \qquad (6.6.2\text{-}32)$$

$$\phi_b(\eta) = 0.6\frac{E}{\eta}\left(1-0.731\left\{1-\exp\left(-\frac{1}{16}\sqrt{\eta}\right)\right\}\right) \tag{6.6.2-33}$$

Safety factor (α)

$$\alpha = \begin{cases} 1.0 & (\eta \le \eta_1) \\ 1.0+\dfrac{F}{13600}(\eta-\eta_1) & (\eta_1<\eta<\eta_2) \\ 1.5 & (\eta \ge \eta_2) \end{cases} \tag{6.6.2-34}$$

where F: Values of F defined in Article 88-3-1(A) of "Notification No. 501" (kgf/mm^2)
E: Longitudinal elastic modulus of the material (kgf/mm^2)
t: Thickness of cylinder wall (mm)
η: R/t
η_1: 1200/F
η_2: 8000/F
η_3: 9600/F

b. Penetrated portion

(a) Strength evaluation of penetrated portion

Various types of pipes penetrate the Type 2 vessels (nuclear reactor containment vessel). At these penetrated portions, it is necessary to consider the seismic load from the piping. In the design of the penetration portion, as descried in the examples, the design loads are predetermined, and during design of the route and support of the piping, efforts are made to ensure that the seismic load applied to the nuclear containment vessel is within the design load range. This is because at the time of design of the nuclear reactor containment vessel, the detailed design of the individual pipes has not yet been finished, and it is impractical to calculate the seismic force for the multiple pipes one by one and take them as the seismic input to the nuclear reactor containment vessel.

The basic scheme for determining the design load is that the stress due to the loads other than those loads at the penetrated portion is subtracted from the allowable stress for the nuclear reactor containment vessel, and the result is taken as the allowable stress for the penetrated portion for calculation of the allowable load. In (b) and (c) of the next item, we will discuss the general schemes of the allowable load determination methods for BWR and PWR. At the penetrated portions of the nuclear reactor containment vessel with large-diameter pipes or high-temperature pipes with relatively large diameters, bellows are used in some cases. The bellows are designed to have appropriate shapes to account for the design conditions, i.e., the amount of displacement and number of cycles during normal operation, earthquakes, and accidents. When the allowable load region is to be calculated for the nuclear reactor containment vessel, the stresses at the containment vessel and the penetrated joint portion due to the piping's external force can be determined using Bijlaard's method. When Bijlaard's method is applied, calculation tables are usually used [6.6.3-3]. However, when calculation is to be made for the nuclear reactor equipment, the range may be outside the range of the parameters in the available tables. In this case, extrapolation is needed. When the extrapolation is too large, FEM analysis, etc., should be performed to assess the appropriateness.

(b) Method of determining allowable load region in BWR

Figure 6.6.2-19 shows the procedure for determining the BWR allowable load region.

(i) PCV allowable stress intensity

There are two types of the stresses generated in the PCV barrel due to the piping external force during earthquake: local membrane stress and local flexural stress. Since the local flexural stress, which is a type of secondary stress, is dominant, attention is paid to the primary + secondary stress. The allowable value of the stress range for the primary + secondary stress due to earthquake is 3 S. Hence, the allowable stress intensity for the single-side amplitude due to earthquake is 1.5 S.

(ii) Stress intensity (σ_S*) generated in PCV due to loads other than piping external force

The stress range of the primary + secondary stress generated in the PCV barrel during earthquake includes the stress range due to the seismic loads (vertical and horizontal) on the PCV body and the stress range generated by the piping external force on the PCV barrel. Hence, in the case when the allowable load is determined with respect to the piping external fore, the stress σ_S* due to the load other than the piping reaction force, i.e., due to the seismic load of the PCV barrel, should be taken into consideration. In this case, σ_S* includes the following stresses:

{1} Stress caused by horizontal seismic load of PCV barrel (½ range)
{2} Stress caused by vertical seismic load of PCV barrel (½ range)

(iii) Margin of stress intensity of PCV (σ_S)

Since the allowable stress range due to earthquake only is 3 S, i.e., the ½ range is 1.5 S, the allowable value of the primary + secondary stress strength is obtained by subtracting from 1.5 S the seismic stress of the PCV barrel. Hence, the margin of PCV stress intensity, i.e., the allowable stress (σ_S) with respect to the piping external force, is as follows:

$$|\sigma_S| = 1.5S - |\sigma_S^*| \qquad (6.6.2\text{-}35)$$

where, | | represents stress intensity.

(iv) Allowable load region from PCV

(1) Stress per unit load at the penetrated portion

The stresses at the joint portion between the containment vessel and penetrated portion due to the external force are obtained using Bijlaard's method. When the penetrated portion is modeled as shown in Figure 6.6.2-20, if the dimensions of the nuclear reactor containment vessel and the penetrated portion can be determined, it is possible to use Bijlaard's method to obtain the stresses generated at points A and B per unit load. The piping external forces can be classified as axial forces and bending moments for each component. The shear force is found to be small and can be neglected, and the stress is calculated from M_C, M_L and P.

(2) Allowable load region

By using the stresses caused by unit loads of P, M_C, M_L ($\sigma_{p.u}$, $\sigma_{MC.u}$, $\sigma_{ML.u}$), the allowable load region with respect to the piping external force can be represented as follows:

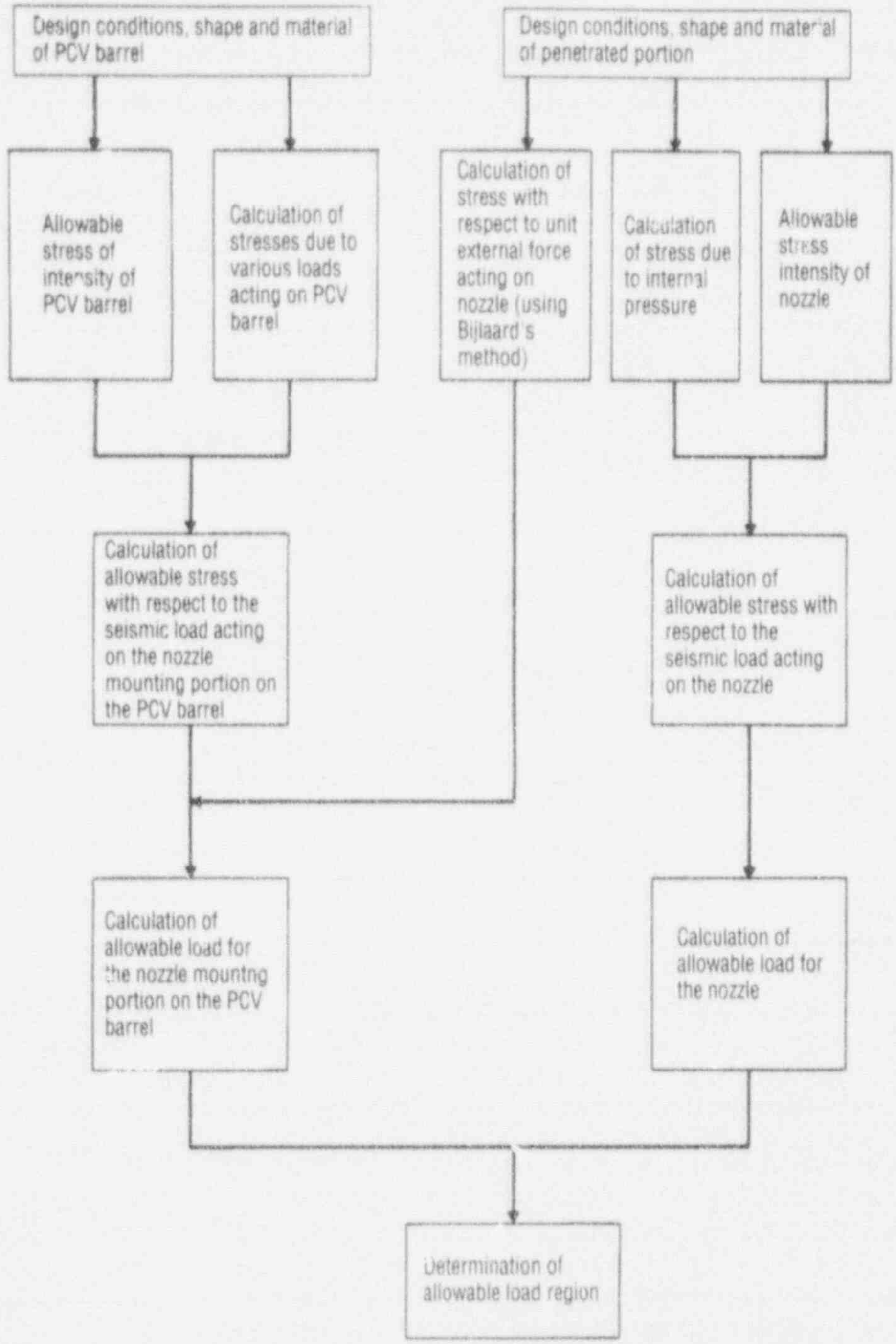


Figure 6.6.2-19. Procedure for determining allowable load region in the penetrated portion (BWR, as an example).

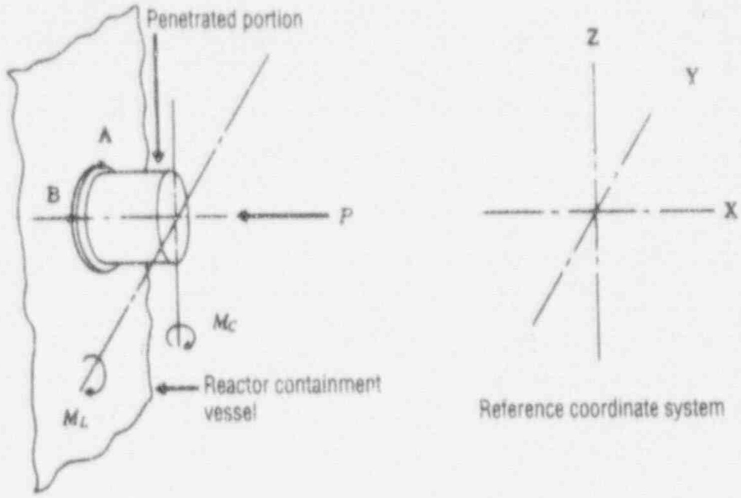


Figure 6.6.2-20.

$$|\sigma_S| = |\sigma_{p,s}|P + |\sigma_{MC,s}|M_C \tag{6.6.2-36}$$

$$|\sigma_S| = |\sigma_{p,s}|P + |\sigma_{ML,s}|M_L \tag{6.6.2-37}$$

where | | represents stress intensity.

In this case, since σ_S is known in (iii), the allowable load region can be derived with respect to P, M_C or P, M_L.

(v) Limit load region of nozzle

{1} Margin of stress intensity of nozzle

The stress other than the piping external force is axial stress. Evaluation is made of this axial stress.

The margins of stress intensity $|\sigma_{S1}|$, $|\sigma_{S2}|$ for the nozzle are defined as follows in consideration of the limitation of primary general membrane stress and primary + secondary stress.

<u>Primary stress</u>

For allowable stress state III_AS,

$$|\sigma_{S_1}| \le \min(S_y,\ 0.6S_u) - |\sigma_{S_1}^*| \tag{6.6.2-38}$$

For allowable stress state IV_AS,

$$|\sigma_{S_2}| \leq \min(S_y,\ 0.6S_u) - |\sigma_{S_2}^*| \qquad (6.6.2\text{-}39)$$

Here, $|\sigma_{S1}^*|$, $\sigma_{S2}^*|$ are stresses due to the internal pressure of the sleeve, etc.

Secondary stress

$$|\sigma_{S_1}| = \frac{3}{2}S, \quad |\sigma_{S_2}| = \frac{3}{2}S \qquad (6.6.2\text{-}40)$$

{2} Allowable load

The stress calculation is performed using P/A, M/Z. The margin of stress intensity in {1} can be used to calculate the allowable load.

$$|\sigma_{S_1}| = P/A + M/Z \qquad (6.6.2\text{-}41)$$

$$|\sigma_{S_2}| = P/A + M/Z \qquad (6.6.2\text{-}42)$$

(vi) Allowable load region of PCV barrel

This is the region which the allowable load with respect to the piping external force fully meets the requirement of the load regions shown in (iv) and (v).

(c) Method for setting design load in PWR

(i) C/V allowable stress intensity

The stresses generated by piping external force due to earthquake in C/V shell are mostly secondary stresses. Consequently, the design external force is set corresponding to the allowable value of the primary + secondary stress intensity of C/V. During an earthquake, the allowable value of the difference between the maximum value and minimum value of the primary + secondary force is 3 S. Consequently, the allowable stress intensity for the single amplitude in earthquake becomes 1.5 S.

(ii) Stress intensity generated in C/V due to loads other than piping external force

As far as the C/V allowable stress intensity is concerned, although the stress range generated by earthquake only is 3 S (single amplitude 1.5 S), the piping load determined here also contains the self weight of the piping; hence, the stress due to the C/V self weight is also included to the stress generated in the C/V body. Consequently, the following stresses are taken into consideration as the C/V body stresses in earthquake.

{1} Stress due to C/V self weight (σ_{L1})
{2} Stress due to horizontal seismic load of C/V body (single amplitude) ($\sigma_{\theta 2}$, σ_{L2})
{3} Stress due to vertical seismic load of C/V body (single amplitude) (σ_{L3})
σ_L: Stress in C/V axial direction
σ_θ: Stress in C/V circumferential direction

(iii) Margin of stress intensity of C/V

To determine the allowable value of the primary + secondary stress intensity, since the stress amplitude due to earthquake only is 3 S, i.e., the single amplitude is 1.5 S, the allowable value is obtained by subtracting from 1.5 S the seismic stress (single amplitude) of the C/V body. In addition, in this case, since the piping self weight is included in the load, the stress due to the self weight is also subtracted from 1.5 S. Assuming $\sigma_{a\theta}$, σ_{aL} are the margins of stress intensity of C/V, we have

$$|\sigma_{a\theta}| = 1.5S - |\sigma_{\theta 2}| \tag{6.6.2-43}$$

$$|\sigma_{aL}| = 1.5S - |\sigma_{L_1} + \sigma_{L_2} + \sigma_{L_3}| \tag{6.6.2-44}$$

where | | represents stress intensity.

(iv) Allowable load region derived fr m C/V

(1) Stress per unit load at penetrated portion

The stress generated by the pipin, external force at the joint portion between the containment vessel and penetrated portion can be obtained using Bijlaard's method. For the model of the penetrated portion shown in Figure 6.6.2-20, if the dimensions of the nuclear reactor containment vessel and the penetrated portion can be given, it is possible to determine the stress generated at points A and B per unit load.

The parameters needed for the calculation are as follows:

R_m: Average radius of containment vessel
r_0: Outer diameter of penetrated portion
t: Plate thickness of containment vessel
β: 0.875 r_0/Rm
γ: R_m/t

The relation between the load and stress generated point is listed in Table 6.6.2-5. The stress listed in the table is the membrane + flexural stress in the circumferential direction. In this case, we have $\sigma_{p1} < \sigma_{p2}$.

(2) Allowable load region

Among the stresses described in the above section, stresses caused by M_L and M_C occur at different locations, and the stress generated by M_C is higher than the stress generated by M_L. Consequently, the allowable

Table 6.6.2-5.

Load	Stress generated	
	Point A	Point B
P	σ_{p1}	σ_{p2}
M_L	σ_{ML}	—
M_C	—	σ_{MC}

stress is derived for point B. As a result, assuming the stresses generated by unit load in the above section are $\bar{\sigma}_{p1}$ and $\bar{\sigma}_{ML}$, the limit of load at point B has a range represented by the following formula. That is, the limit σ_a of the load is

$$\sigma_a = \bar{\sigma}_{pl} P + \sigma_{ML} M \tag{6.6.2-45}$$

with $M = M_L = M_C$.

In this way, the allowable load region can be derived for P, M.

(v) Characteristics of piping limit bending moment and piping external force

For small-diameter pipes (usually smaller than 4B), the limit bending moment defined from the allowable stress of the piping itself is less than the limit defined by the allowable load region of the C/V. Consequently, when the bending moment of the design external force is to be determined, the limit bending moment of the piping is taken as the limit. The characteristics of the external forces for each pipe should also be taken into consideration. For example, an excessively large reaction force should not take place in the flange stop and other special piping on the inner/outer surfaces of the C/V.

(vi) Determination of design external force

The design external force is determined from the allowable load region of the C/V in consideration of the characteristics of the piping's limit bending moment and the piping's external force, as well as experiences acquired in the past.

In addition, since the position of the sleeve end plate of the fixed penetrated portion is offset eccentrically from the plate center of the containment vessel (with an eccentric distance e), in addition to M_y and M_z, there are also bending moments generated by F_y, F_z in the nuclear reactor containment vessel. Consequently, the actual bending moments applied to the nuclear reactor containment vessel are as follows:

$$M_y = M_y' + eF_z \tag{6.6.2-46}$$

$$M_z = M_z' + eF_y \tag{6.6.2-47}$$

Hence, the allowable moments become

$$M_y' = M_y - eF_z \tag{6.6.2-48}$$

$$M_z' = M_z - eF_y \tag{6.6.2-49}$$

Although M_x has no influence on the stress evaluation of the nuclear reactor containment vessel, this moment is determined as similar to M_y and M_z.

(4) Type 3 equipment

The Type 3 equipment includes "Type 3 vessels," "Type 3 piping," "Type 3 pumps," and "Type 3 valves." According to "Notification No. 501," they refer to the following equipment:

{1} Equipment ne- or safe shutdown of the nuclear reactor, and equipment needed for guaranteeing safety in emergency. For this equipment, if failure/damage takes place, there is an indirect effect of causing radiation hazard for the public. (For the air conditioning duct belonging to the radioactive ray management equipment, it is limited to the range from the penetrated portion of the nuclear reactor containment vessel to the outer side isolation valve.)

{2} Equipment belonging to the circulation circuit of the fluid used mainly for driving the turbine and located in the range from the Type 1 equipment to the nearest stop valve.

{3} Equipment other than that defined in {1} and {2} and located in the range from the penetrated portion of the nuclear reactor containing vessel to the inner-side isolation valve or outer-side isolation valve.

These equipments are important equipments for safety. According to the aseismic importance classification, it may be classified as Class A or Class As and require dynam- alysis with respect to S_1 and S_2 earthquakes. Most of the vessels can be analyzed using the single discrete mass model, and the pumps are usually taken as rigid bodies. However, for vessels and pumps with complicated shapes, analysis is performed using a multiple discrete mass model. For the piping systems, in some cases, they are analyzed using a multiple discrete mass model; in other cases, they are supported with a predetermined support interval. As far as the valves are concerned, just as for the Type 1 valves, if the seismic strength on the piping side can be confirmed, there is no need to perform strength evaluation for the valve against the seismic load. For some Type 3 equipment, in addition to the strength evaluation in earthquake, it is also necessary to confirm the dynamic function. For this feature, a detailed description will be presented in section "6.7 Confirmation of functions of Class As and A equipment in earthquake."

a. Vessels

The vessels can be classified as several types according to their shapes and support forms. The typical forms are as follows:

- Two-leg-supported horizontal cylindrical shape
- Skirt-supported vertical cylindrical shape
- Four-leg-supported vertical cylindrical shape
- Flat-bottom cylindrical shape

Although most of the containers can be analyzed using the single discrete mass model, in some cases, such as a multistage heat exchanger, analysis may also be performed using a multiple discrete mass. Figure 6.6.2-21 shows the procedure of the aseismic design of the vessels. The stress evaluation is carried out for the barrel body, support portion and foundation bolts. The procedure of the stress evaluation of the barrel portion is shown in Figure 6.6.2-22. Buckling evaluation is also performed for the barrel portion and skirt portion acted upon by the seismic load. Usually, however, the stress due to the seismic load is small and it is almost always ignored in the fatigue analysis. In the aseismic design of a Type 3 vessel, from the calculation of the natural frequency to the stress evaluation, the formulas are almost completely available and shown in Figure 6.6.2-23.

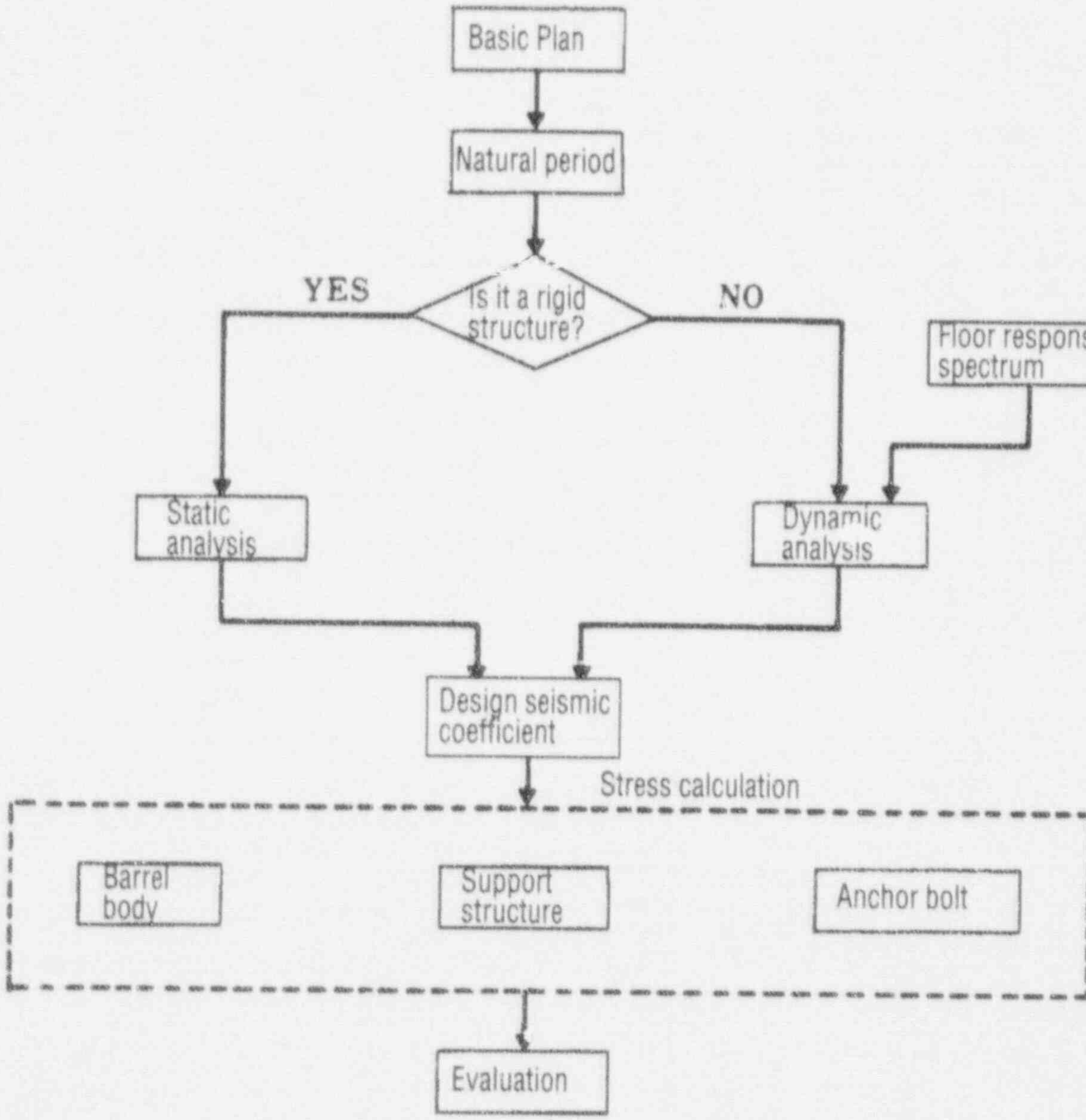
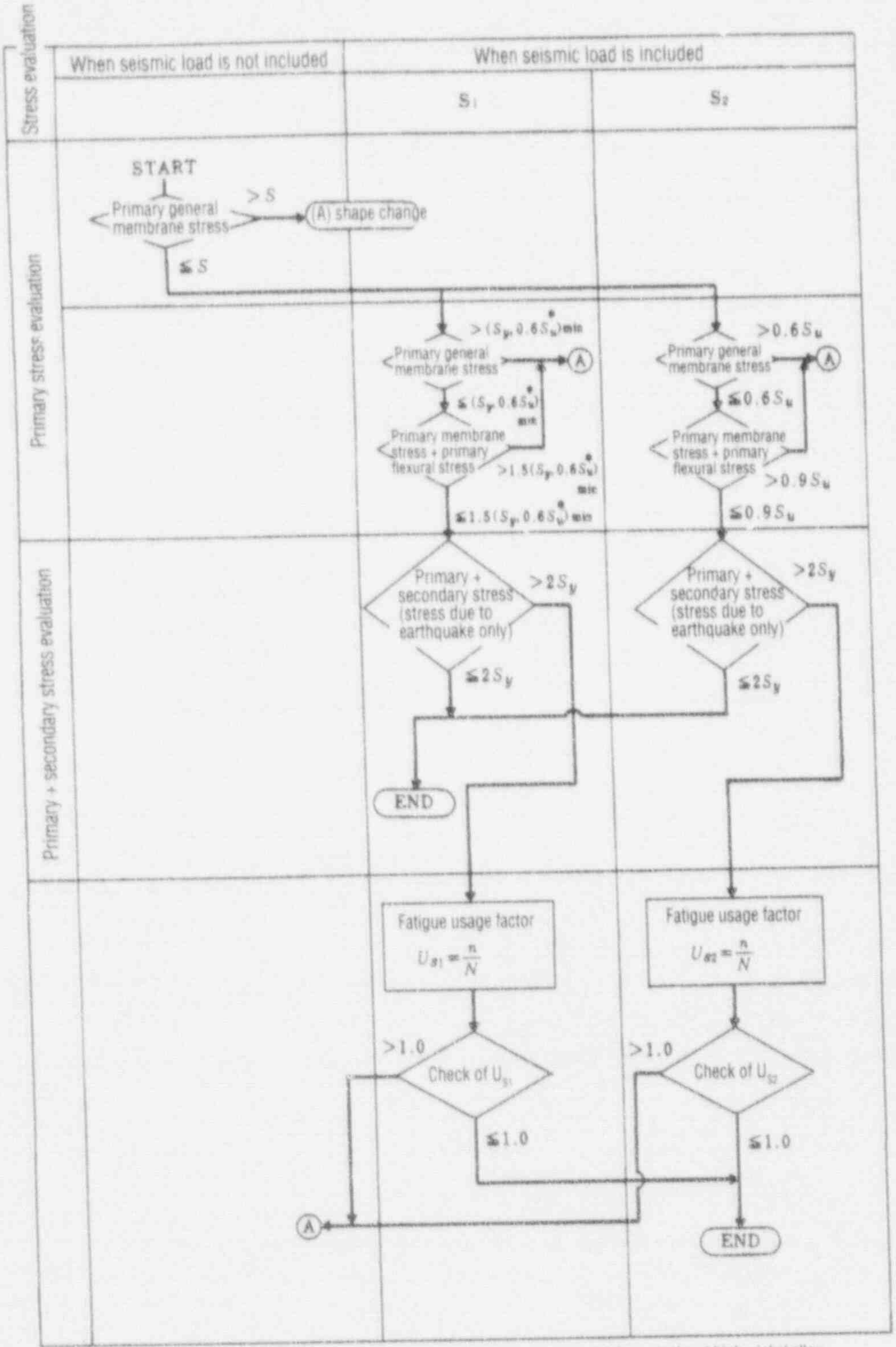


Figure 6.6.2-21. Procedure of aseismic design of Type 3 vessel.



Note: Asterisk indicates that it may be taken as 1.2 S for the austenite stainless steel and high-nickel alloy.

Figure 6.6.2-22. Procedure of stress evaluation for Type 3 vessel.

Skirt-supported vertical cylindrical container (heat exchanger)

Installation position (EL m)	Aseismic class	Highest pressure used (kgf/cm^2)	Highest temperature used (°C)	Weight (tf)	Height of center of gravity (mm)	Structural form	Computing model	Superposition of deformation modes	Natural period (s)	Horizontal seismic coefficient		Design seismic coefficient	
										Dynamic	Static	Horizontal	Vertical
	As	46	200	36.9	3,490	Skirt-supported vertical cylinder	Single discrete mass model	B + C	0.062	1.409	0.576	1.409	0.153

Part under evaluation	Stress due to internal pressure (kgf/mm^2)		Combined primary stress (kgf/mm^2) (σ_1)	Buckling stress (kgf/mm^2)	
	Longitudinal (σ_{X1})	Circumferential ($\sigma_{\theta 1}$)		Compressive stress (σ_c)	Flexural stress (σ_b)
Mirror plate	—	8.99	10.7	0.3	1.7

Part under evaluation	Primary stress (kgf/mm^2)		
	Compressive stress (σ_c)	Flexural stress (σ_b)	Shear stress (τ)
Skirt	0.4	3.5	0.9

Part under evaluation	Material	Allowable stress (kgf/mm^2)		Buckling evaluation (kgf/mm^2)		
		Shear stress (1.5 f_s)	Primary stress (σ_{1a})	Compressive stress (f_c)	Flexural stress (f_b)	$\frac{S_{ac}}{f_c} + \frac{S_{ab}}{f_b} \leq 1$
Mirror plate	SUS304	—	14.7	19.8	19.8	0.10
Skirt	SM41B	13.8	—	24.0	24.0	0.16

Part under evaluation	Material	Primary stress (kgf/mm^2)	Allowable stress (kgf/mm^2)
		Tensile stress (σ_1)	Tensile stress (1.5 f_t)
Anchor bolts	SNB7	6.6	46.2

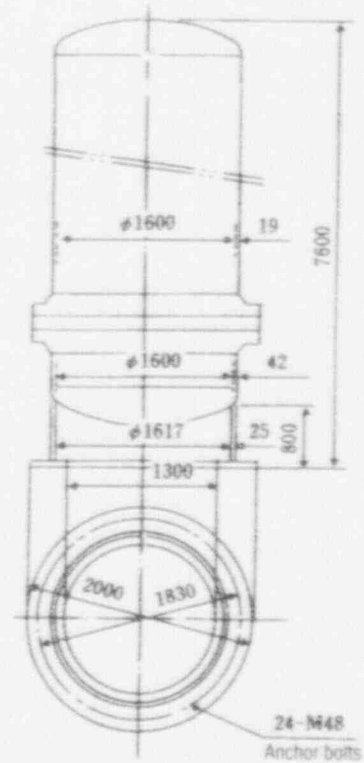


Figure 6.6.2-23. Aseismic calculation results of Type 3 vessel (example).

b. Piping

A nuclear power plant has a huge number of pipes, including Type 3 pipes. As far as the aseismic design of these pipes is concerned, for pipes having relatively large diameter and operating at relatively high temperature, a detailed multiple discrete mass model is used for each of them according to the corresponding piping route and support conditions. For the other pipes, standard support intervals are set. Tables 6.6.2-6 and 6.6.2-7 list the examples of classification of the design methods of PWR and BWR.

For the specific piping system, aseismic design is performed under the following basic guidelines:

{1} In principle, for the important piping system, the seismic support design is performed to ensure that the piping system is in the rigid region with respect to the dominant natural frequency of vibration.
{2} Except for Type 1 piping, bellows-shaped stretchable joints is used for the portions between support points with a large relative displacement in earthquake.
{3} For pipes with a high temperature of application, a hydraulic snubber or mechanical snubber can be used to provide both the seismic support effect and the effect of releasing the thermal expansion.

The stress/strength analysis of piping is described in detail in section "6.6.3(2) Piping."

As pointed out above, the aseismic design of piping is usually performed in combination with the design of the support structure. Design of the piping and support structure is performed according to the aseismic design procedure shown in Figure 6.6.2-24. The stress evaluation of Type 3 pipe is performed according to the procedure shown in Figure 6.6.2-25. Calculation of the stress including the earthquake is performed using the following formulas:

- Primary stress

$$S = \frac{PD_0}{400t} + \frac{0.75 i_1 (M_a + M_b)}{Z} \qquad (6.6.2\text{-}50)$$

where
S: Primary stress (kgf/mm^2)
P: Pressure in operation state which should be combined with the earthquake (kgf/cm^2)
D_0: Outer diameter of pipe (mm)
t: Wall thickness of pipe (mm)
i_1: Stress coefficient, which has the value defined in Item 57 of "Notification No. 501" or the value of 1.33, whichever is larger
M_a: Bending moment generated by the mechanical load of the pipe (limited to the self weight and other long-term loads) (kgf·mm)
Z: Sectional modulus of pipe (mm^3)
M_b: Bending moment generated by the mechanical load of the pipe (short-term loads, including earthquake) (kgf·mm)

- Variation value in primary stress + secondary stress

$$S_n = \frac{PD_0}{400t} + \frac{0.75 i_1 M_b^* + i_2 M_c}{Z} \qquad (6.6.2\text{-}51)$$

Table 6.6.2-6. Aseismic design methods of piping systems (PWR, as example).

Type of piping	Definition	Piping shape	Aseismic design method
High-temperature piping	- Class A (B) - Highest temperature used $>150°C$ - Aperture ≥ 4 B The aforementioned high-temperature piping and similar pipes	1. The piping and equipment interact as a coupled vibration system 2. Pipes with complicated shapes 3. Support mounting is performed irregularly. (Due to limitation imposed by the building shape) 4. Piping elements are analyzed in detail.	Multiple discrete mass beam model
Low-temperature piping	Others	1. Pipes with simple shapes. 2. Support mounting is performed in a regular way 3. Others	Simple model 1. Span simple support beam 2. Span simple support beam

Table 6.6.2-7. Aseismic design methods of piping (BWR, as example).

Aseismic design method		Piping under evaluation
Computer analysis using multiple discrete mass beam model	Dynamic analysis Static analysis	1. Class As, A, B 2. Pipes with complicated shapes 3. Piping elements are analyzed in detail 4. High-temperature piping 5. Others
Simple method using the simple model (Constant pitch span method)	Based on natural frequency of vibration	1. Class As, A, B 2. Pipes with simple shapes 3. Others
	Based on allowable stress	1. Class B, C 2. Pipes with simple shapes 3. Others

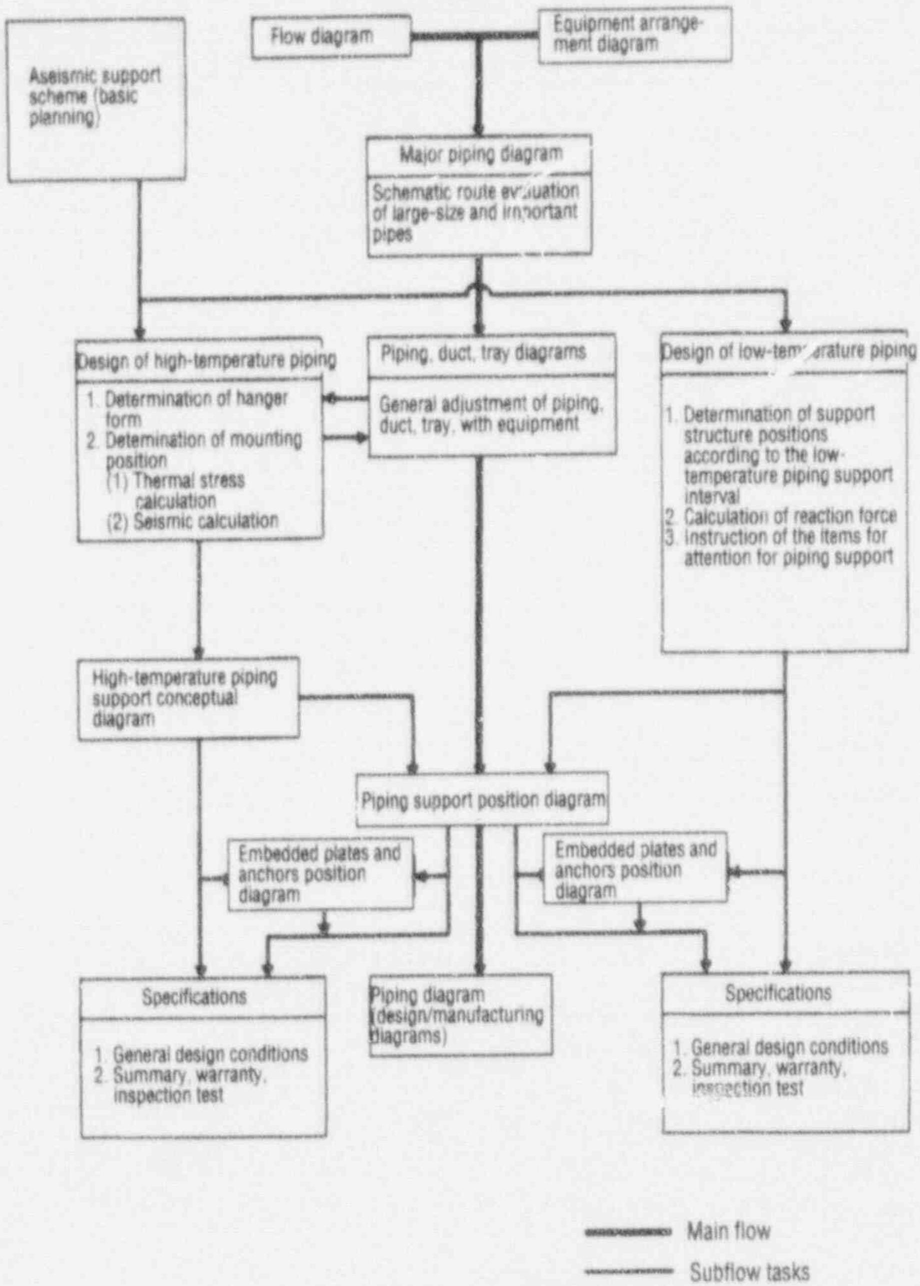
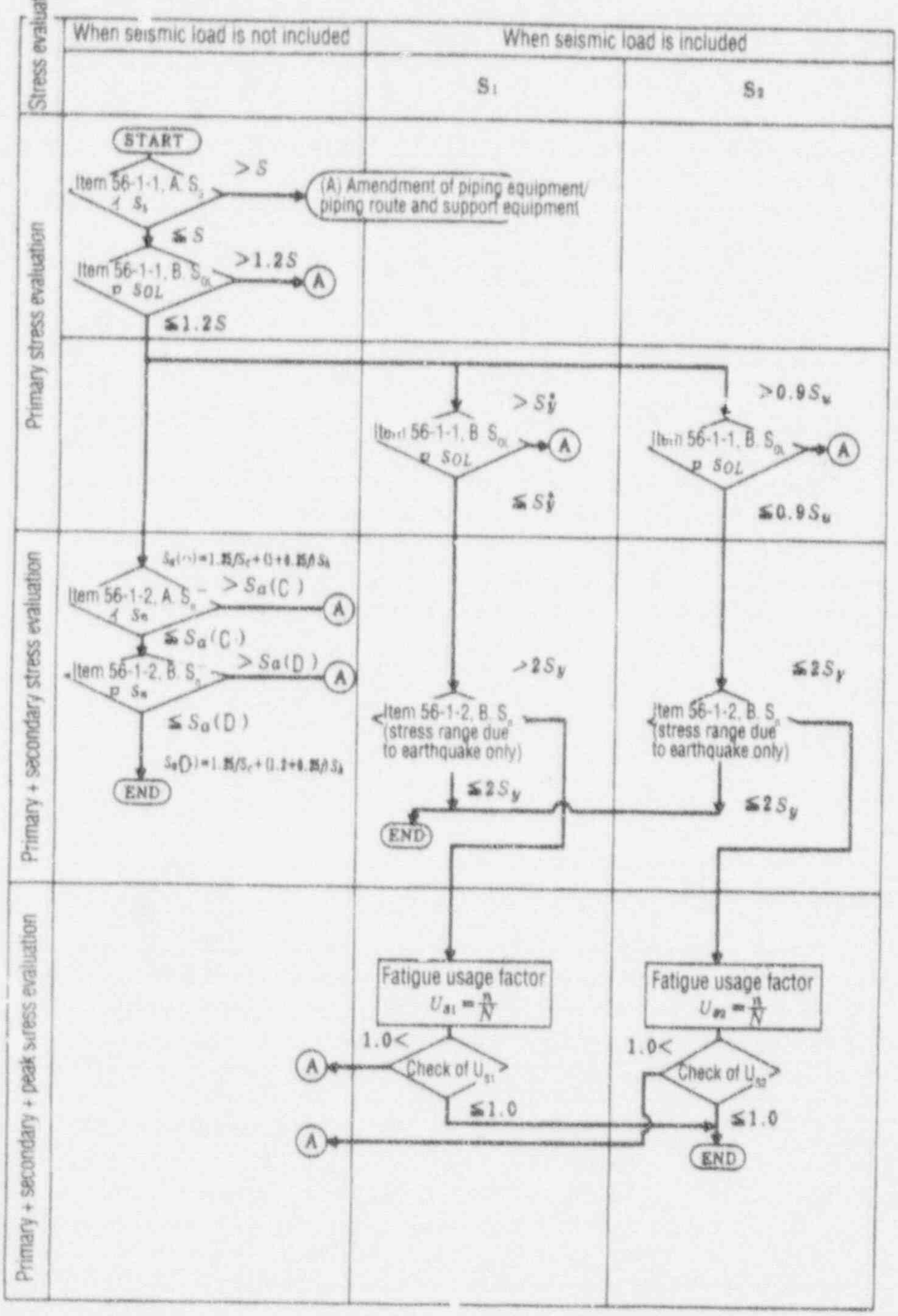


Figure 6.6.2-24. Procedure of aseismic design of piping and support structures.



Note: Asterisk indicates Sy or 1.2 S, whichever is larger, for austenitic stainless steel and high-nickel alloy.

Figure 6.6.2-25. Stress evaluation procedure of Type 3 piping.

where S_n: Stress derived by adding primary stress and secondary stress (kgf/mm^2)

i_2: Stress coefficient, which takes the value defined in Item 57 of "Notification No. 501" or the value of 1.0, whichever is larger

$M_b{}^*$: Total amplitude of moment generated by the inertial force of seismic motion S_1 or S_2 (kgf·mm)

M_c: Total amplitude of moment generated by the relative displacement of seismic motion S_1 or S_2 (kgf·mm)

P, D_0, t, i_1, Z: Same as those defined in the above explanation for the primary stress

Figure 6.6.2-26 and Table 6.6.2-8 show the example of the multiple discrete mass beam model and stress evaluation.

c. Pumps

Pumps can be divided, in terms of their aseismic structures, into vertical pumps which are analyzed using the multiple discrete mass model, and horizontal pumps which are taken as rigid bodies. The ideas for forming models for them are described in section "6.5.2(4) Other equipment." As far as stress evaluation is concerned, for the vertical pumps, it is performed for the column, support portion, and anchor bolts; for the horizontal pumps, it is performed for the mounting bolts and anchor bolts. As far as the reactive force from the piping is concerned, because the pump portion has a much higher strength than that of the piping, the stress evaluation is usually not implemented. Just as with the vessels, the aseismic design of pumps has also been fully established, with an example illustrated in Figure 6.6.2-27.

(5) Other equipment

In addition to the Type 1-3 equipment, there is also other equipment which can be classified as Class A or Class As according to its aseismic design importance. In the following, we will discuss the stress evaluation for such equipment.

a. Core support structure and internal structures

As pointed out in section "6.5.2(1) Basic guideline of formation of models for equipment/piping systems," for the core support structures, fuel assembly and internal structures of a BWR, an MDOF bending-shear beam model is used for performing time history analysis to derive the seismic load in consideration of the interaction with the nuclear reactor building and nuclear reactor containment vessel. The seismic load is combined with the load during normal operation for stress analysis to confirm that the stress generated is within the allowable range.

In the case of a PWR, the core support structures, fuel assembly and internal structures are analyzed by a multiple discrete mass beam model having a distributed mass with interaction with the nuclear reactor containment taken into consideration. In this case, the seismic load is derived using the spectral model method. Just as in the case of a BWR, the seismic load is combined with the other loads for stress evaluation. For the PWR fuel assembly, since its vibration is a nonlinear problem accompanying impact, in the aforementioned model, the time history response wave is input at the support point of the internal concrete nuclear reactor containment, and the time history response wave at the upper/lower core plates is determined. The obtained time history response wave is used for the analysis of the fuel assembly to simulate complex vibration behavior due to group collision, and nonlinear time history response analysis is performed to evaluate the grid impact force of the fuel assembly and the stress in the cladding. For the seismic performance of the control rod and fuel, analysis or insertion test which simulates the control rod and fuel assembly is performed to confirm the insertion ability of the control rod and the ability to maintain its shape for removal of the decay heat (see "JEAG 4601-Supplement-1984").

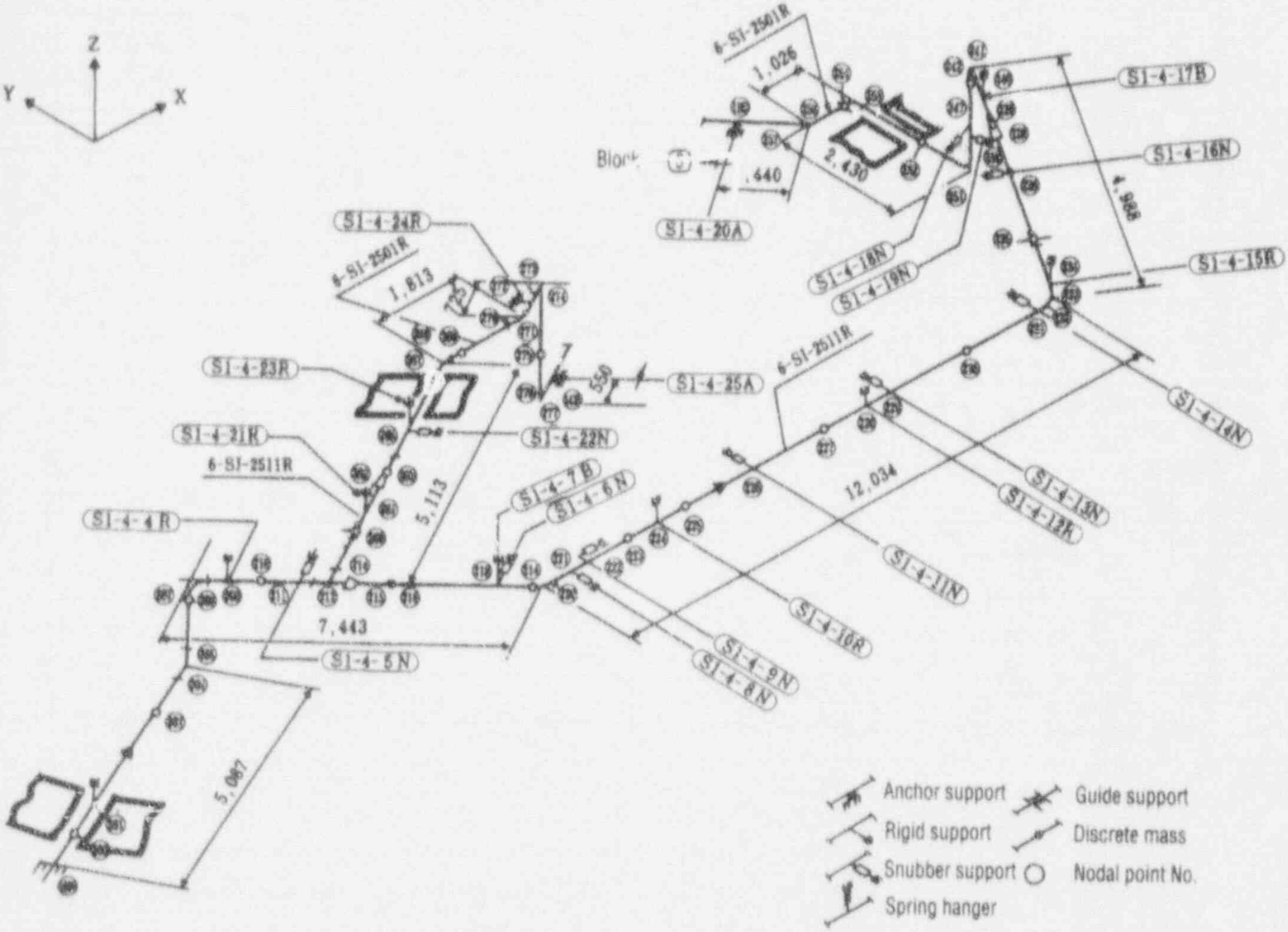


Figure 6.6.2-26. Example of multiple discrete mass analysis model of piping (example of PWR safety injection piping).

Table 6.6.2-8. Example of piping stress in earthquake (PWR safety injection pipe)
[S_1 earthquake; units: kgf/mm^2]

Nodal point no.	Primary stress			Secondary stress due to earthquake	Total stress of primary + secondary stress due to earthquake
	Primary stress except earthquake	Primary stress due to earthquake	Sum of stress		
201	4.41	0.70	5.11	2.32	3.8
204	4.33	0.39	4.72	2.11	2.9
205	4.15	0.33	4.48	2.08	2.8
207	4.15	0.27	4.41	1.68	2.3
211	4.23	0.53	4.76	1.77	2.9
212	4.38	1.29	5.67	2.50	5.1
213	4.16	0.59	4.75	1.94	3.3
215	4.38	1.20	5.58	5.82	8.3
216	5.24	2.10	7.34	1.61	5.9
218	4.60	1.00	5.60	1.74	3.[illegible]
219	4.24	1.05	5.29	0.69	2.8
220	4.20	1.00	5.20	0.53	2.6
260	4.19	0.74	4.93	1.25	2.8
261	5.04	1.51	6.55	0.78	3.8
262	4.29	1.30	5.58	0.54	3.2

Equipment name	Aseismic class
Horizontal pump	As

Installation building	Mounting position (EL. m)	Weight (kgf)	Height of center of gravity (m)	Horizontal seismic coefficient		Design seismic coefficient	
				Static	Dynamic*	Horizontal	Vertical
		17,000	1.050	0.576	0.624	0.624	0.153
		8,000	0.000				
		8,000	0.800				
		4,200	0.660				

*1.2 times the zero period acceleration of the floor

Evaluation portion	Material	Seismic stress (kgf/mm²)				Allowable stress (kgf/mm²)		
		Tensile	Compressive	Flexural	Shear	Tensile	Compressive	Shear
Anchor bolts	SS41	—	—	—	—	18.0	—	—
Pump mounting bolts	SS41	—	—	—	—	12.9	—	—
Pump support legs	SS41	0.2	0.3	—	0.1	24.0	23.9	13.0
Motor mounting bolts	SS41	0.2	—	—	—	18.0	—	—

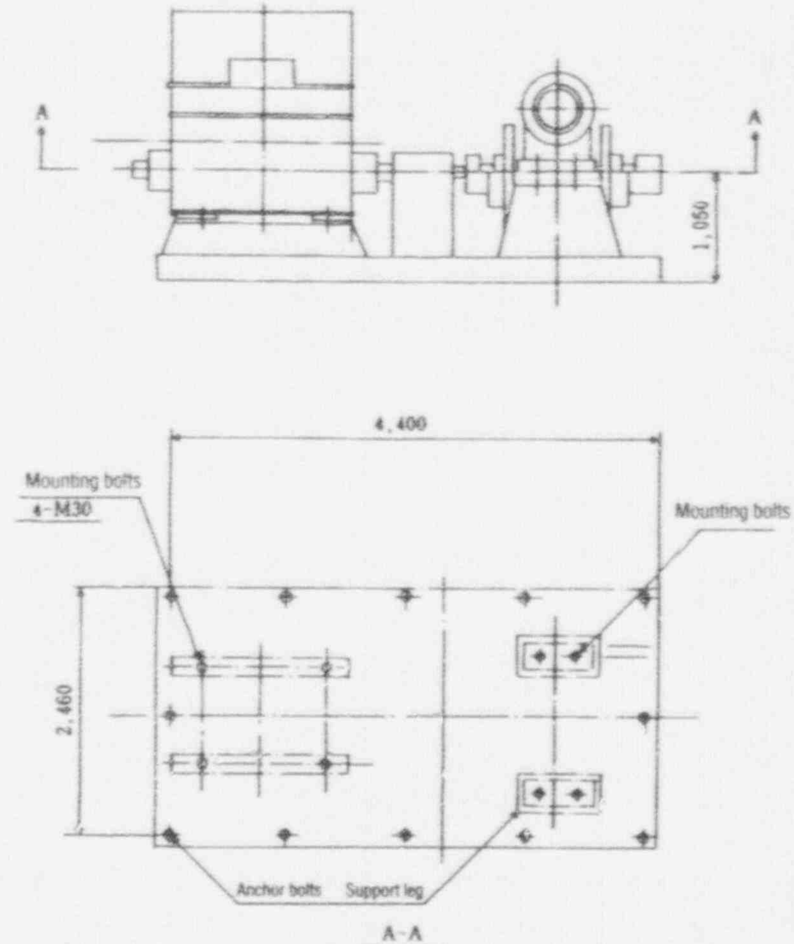


Figure 6.6.2-27. Example of seismic calculation results for Type 3 pump.

b. Seawater pump

It is a vertical pump. For its column portion extended into water, flexural stress and shear stress are determined from the flexural moment and shear force using multiple discrete mass analysis, and the obtained stresses are combined with the stress due to the highest pressure used for stress evaluation.

c. Spent fuel rack

The spent fuel rack is one of the facilities used for storing the spent fuel; it is classified as Class As in the aseismic importance classification. For a certain period, the spent fuel is cooled and stored in a spent fuel pit (pool). In this case, the used fuel rack is used to hold the fuel assembly at the prescribed position.

In the aseismic design of the PWR spent fuel rack, strength evaluation is performed for the cell which holds a single fuel assembly, and the support structure which connects cells in the horizontal direction and transmits the seismic force to the pit wall. Figure 6.6.2-28 shows an example of the analysis model for the single-cell body. This model is used to perform dynamic response analysis, to obtain the cell's response and support reaction forces for stress evaluation. The allowable stress of the support structure is used as the allowable stress.

In the aseismic calculation of the BWR spent fuel rack, strength evaluation is performed for the rack body, which assembles the cells that contains the fuel assembly, and the foundation bolts for fixing the rack on the pool floor. As shown in Figure 6.6.2-29, the analysis model of the rack is used for dynamic response analysis to determine the rack response and load of anchor bolts for the stress evaluation.

d. Electrical instrumentation control equipment

(a) Scheme of aseismic design of electrical instrumentation control equipment

In the aseismic design of a nuclear power plant, in order to be able to shut down the nuclear reactor safely in earthquake and to ensure the safety function later, it is necessary to confirm the ability to maintain the functions of the related electrical instrumentation control equipment. The functions of the electrical instrumentation control equipment include information detection, signal transmission, computation, various operational signals, as well as instructions, recording, alarm, etc., for electric power supply and information feeding. The confirmation of functions is described in section "6.7 Confirmation of functions of Class As and A equipment in earthquake." This section describes the outlines of the mechanical strength evaluation of the electrical instrumentation control equipment.

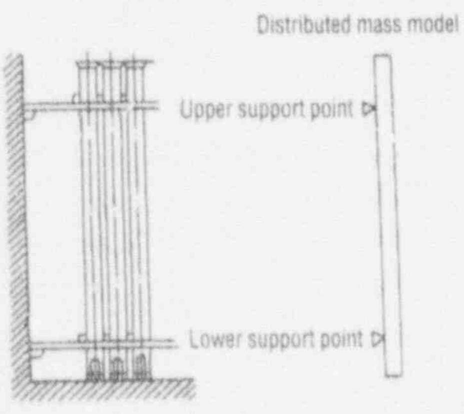


Figure 6.6.2-28. Analysis model of spent fuel rack (PWR, as example).

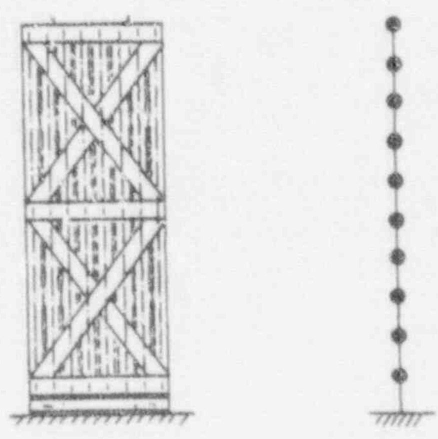


Figure 6.6.2-29. Analysis model of spent fuel rack (BWR, as example).

(b) Boards

For Class As and A boards, if an analysis is possible, the adopted analysis model should be checked experiments. If an analysis is not possible, experiment is performed to confirm the appropriateness of the mechanical strength for maintaining electrical functions. For equipment which already have certain test data, these data are used. For Class C equipment, it is necessary to check the appropriateness of the mechanical strength against the seismic force applied to Class As and A equipment only for such that might affect functions of other Class As, A equipment in case of damage.

(i) Vibration test method

In principle, the board used in the test has the same structure as that of the actual equipment, with internal units (or simulated parts) attached to it, and it is mounted on the vibration test table using a method as similar to the actual method as possible. In the test, first of all, a continuous sinusoidal wave scanning vibration test is performed to measure the natural frequency of vibration, etc. Then, forced vibration test is performed to derive the response acceleration at each portion, to confirm that there is no trouble in the mechanical strength of the board.

(ii) Analysis method

By performing a response analysis, in which the board is represented by a multiple discrete mass beam model and the design floor response spectrum of the floor on which it is mounted is input, or a static analysis, the mechanical strength of the major beams and support structures (anchor bolts, etc.) is confirmed. In addition, for the board for which vibration tests have been performed, its mechanical strength and appropriateness of function are evaluated by comparing the test data and the acceleration of the floor surface on which the panel is mounted. As an example of the analysis of electrical boards, the analysis model, eigenvalue analysis and stress evaluation of PWR nuclear reactor's board are shown in Figure 6.6.2-30, Table 6.6.2-9, and Table 6.6.2-10, respectively.

(c) Instrumentation

As far as the instrumentation (control switch, relay, breaker, detector, seismograph, etc.) is concerned, for those in Class As and A, the parts are selected to ensure that the safety of the plant is not harmed even when they operate erroneously. At the same time, for each part, vibration test is performed to ensure the mechanical strength and appropriate functions.

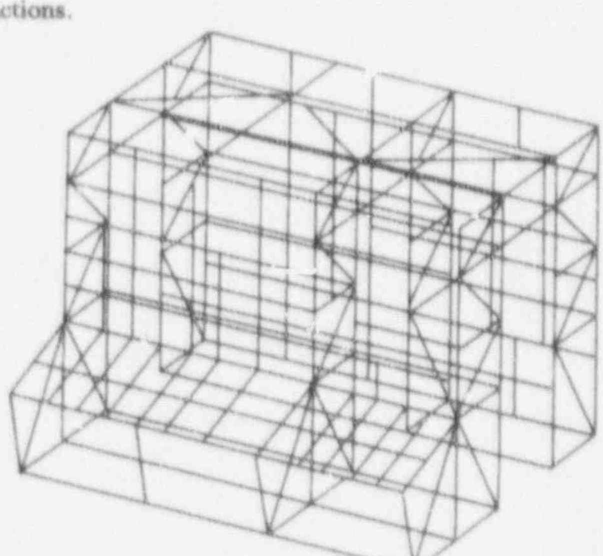


Figure 6.6.2-30. Analysis model of nuclear reactor's board (PWR, as example).

Table 6.6.2-9. Eigenvalue analysis of nuclear reactors board (PWR, as example).

Mode no.	Natural frequency of vibration (Hz)	Participation factor		
		X-direction	Y-direction	Z-direction
1	28.4	0.009	0.233	0.002
2	40.7	0.001	0.364	0.024
3	50.2	−0.080	−0.136	−0.087
4	51.4	0.748	0.188	−0.033
5	53.7	−0.366	0.708	0.007
6	58.0	−0.058	0.238	0.005

Table 6.6.2-10. Example of stress evaluation of nuclear reactor board (Units: kgf/mm^2).

Stress			In S_1 earthquake		In S_2 earthquake	
			Maximum stress	Allowable stress	Maximum stress	Allowable stress
Frame portion	Tensile stress		0.6	25.0	0.7	28.7
	Shear stress		3.2	14.4	3.7	16.5
	Compressive stress		0.6	22.0	0.7	24.7
	Flexural stress		5.2	25.0	6.1	28.7
	Combination of stresses	Tensile + flexural	0.21[1]	≤1.0	0.22[1]	≤1.0
		Compressive + flexural	0.21[2]	≤1.0	0.22[2]	≤1.0
Foundation welded portion	Sum of stresses		1.5	13.8	1.9	16.5

[1] Value calculated as $(\sigma_t + \sigma_b) / 1.5\ f_t$
[2] Value calculated as $(\sigma_c / 1.5\ f_c) + (\sigma_b / 1.5\ f_b)$

The part used in the test is the actual part which is in power-on state and is fixed on the vibration table using a method as similar as possible to the practical assembly method. For the parts of the same type and the same form, test should be made at least for one unit of equipment.

In the test, a sinusoidal wave or a sinusoidal beat wave is applied based on the natural frequency obtained in the scanning vibration test. In this case, vibration may be performed at the acceleration level at the installation location to check the mechanical strength and appropriateness of function. In another scheme, the erroneous operation limit acceleration is determined and compared with the acceleration at the installation location. In this way, the mechanical strength and appropriateness of functions can be assessed.

Class B instruments are those which form the boundary with the fluid. For these parts, it is only necessary to check the mechanical strength to ensure that they would not be ruptured and cause damage to the boundary.

(d) Cable trays

The aseismic design of the cable trays is basically the same as that for piping/ducts. That is, the system supported by the support structure is designed to ensure the cable support (strength) function without generating excessively high response under the seismic input condition applied to the system. Calculation of the support spans can be performed using either of the following methods: the support span is determined so that calculated stress may be not higher than the allowable stress by performing dynamic response analysis; or the natural period is predetermined to ensure that design is within the rigid region.

(e) Air conditioning equipment

The aseismic design of the air conditioning equipment is described schematically in the section about the strength evaluation of aseismic Class A air conditioning unit and duct. Confirmation of function of fan damper, etc., in earthquake will be described in section "6.7 Confirmation of functions of Class As and A equipment." The main body of the air conditioning unit from the viewpoint of strength is the frame structure, which usually is a rigid structure. If needed, just as in the case of analysis of the electrical board, analysis is performed using a multiple discrete mass beam model for stress evaluation of the beam part, welded portion, and anchor bolts. For the allowable stress of the beam part, the allowable stress of the support structure is adopted. The duct may have a circular or square cross-section. The possible structures include welding, spiral, folding, etc. Just as in the case of the low-temperature piping, the aseismic design of the ducts is performed by determining the support span. When the support span is calculated, the sectional stiffness evaluation and buckling evaluation are performed with the special features of the sheet structure taken into consideration. In these evaluation procedures, the formulas of thin-wall cylinder or thin-wall rectangular shell based on the beam theory are modified based on experiment.[1]

(f) Emergency power supply equipment

The emergency power supply equipment is important equipment classified as aseismic importance Class As. It includes emergency diesel generator, battery, etc. Since their main bodies have a sufficiently rigid structure, strength evaluation points are selected for anchor bolts in the aseismic design.

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[1] In the modification, the appropriate correction coefficients are determined according to experimental results and references.

6.6.3 Class B and C equipment

(1) Vessels

a. Basic procedure of aseismic design

The major types of class B and C vessels are as follows:

{1} Skirt-support vertical cylindrical vessel
{2} Flat-bottom cylindrical vessel
{3} Four-leg vertical cylindrical vessel
{4} Horizontal cylindrical vessel
{5} Lug-support vertical cylindrical vessel

The general procedure of the aseismic design of these containers is shown in Figure 6.6.3-1. That is, according to the results of calculation of natural period from the general shape of the container, the design seismic coefficient is calculated. The obtained design seismic coefficient is used to perform stress evaluation for the barrel body, support legs, anchor bolts, and other parts to be evaluated. Figure 6.6.3-2 shows the stress evaluation procedure. Figure 6.6.3-3 shows the example of a model for calculation of natural period. Usually, design of Class B and C equipment is performed using the static seismic coefficient. However, for some Class B equipment which might have resonance, evaluation should be performed using the dynamic seismic force.

For each type, examples of the calculation methods of standard natural period and stress [H-K-7] will be presented. All of these calculation methods can be applied to Class As and A vessels. However, when the stress calculation is performed, it is necessary to take the vertical seismic coefficient into consideration. In the calculation method to be described below, the uncertainty of the support condition is taken into consideration to provide conservative results.

b. Skirt-support vertical cylindrical container (see Figure 6.6.3-4)

Conditions assumed

{1} The weight of the container is concentrated at the center of gravity.
{2} The lower end portion of the skirt is fixed by multiple anchor bolts on the foundation and is thus taken as a fixed end.
{3} The seismic force is assumed as acting on the container in the horizontal direction. The design seismic coefficient in the vertical direction is not considered.
{4} In the case when a structure with restraint horizontal displacement is set on the top portion of the container, this portion is taken as pin-supported.

Analysis conditions

{1} For the deformation modes, the flexural/shear deformations, when the container/skirt are considered as beams, are taken into consideration.
{2} For the skirt portion, if a manhole is arranged without reinforcement, the effect of the opening is taken into consideration.

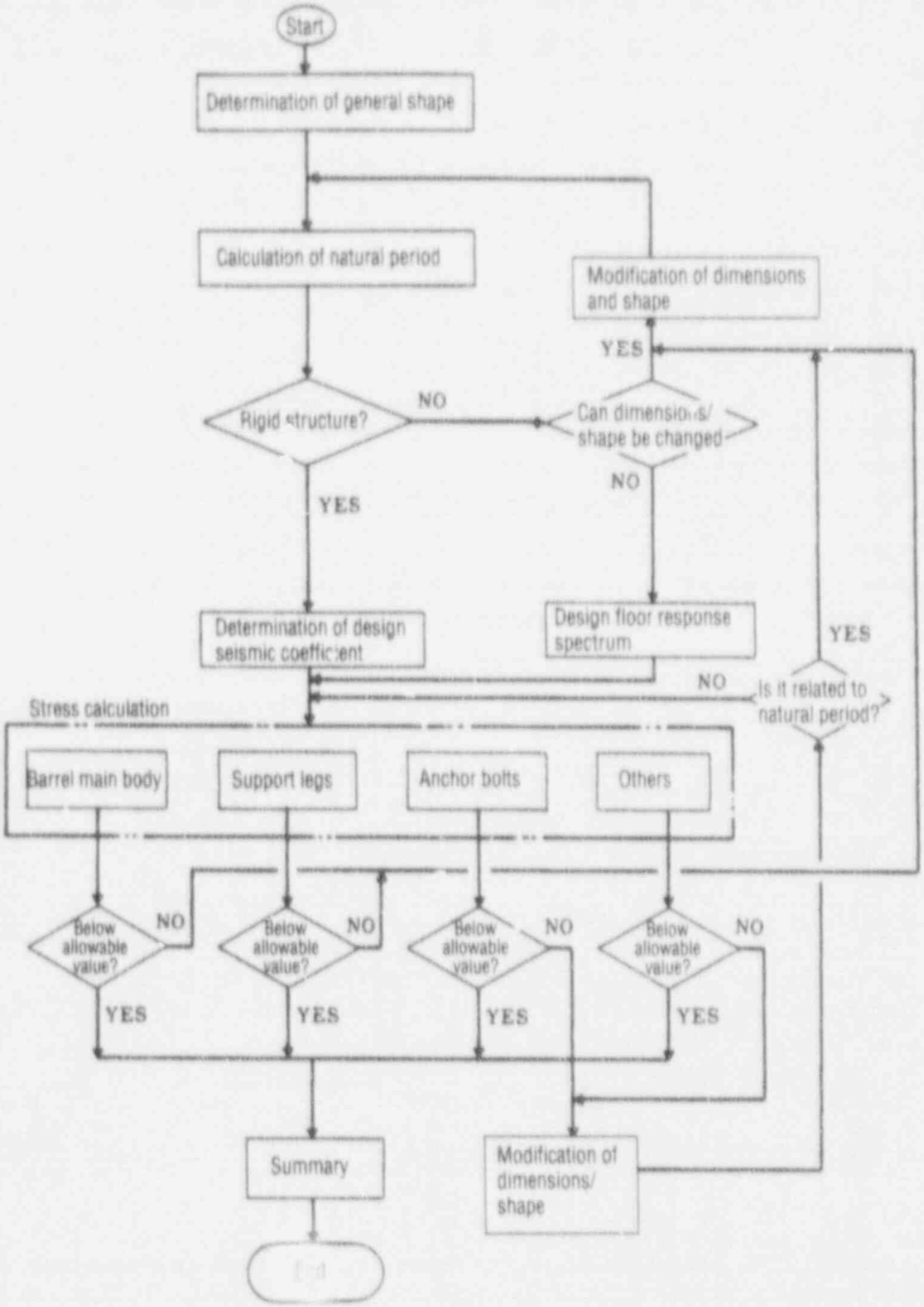


Figure 6.6.3-1. Aseismic design procedure.

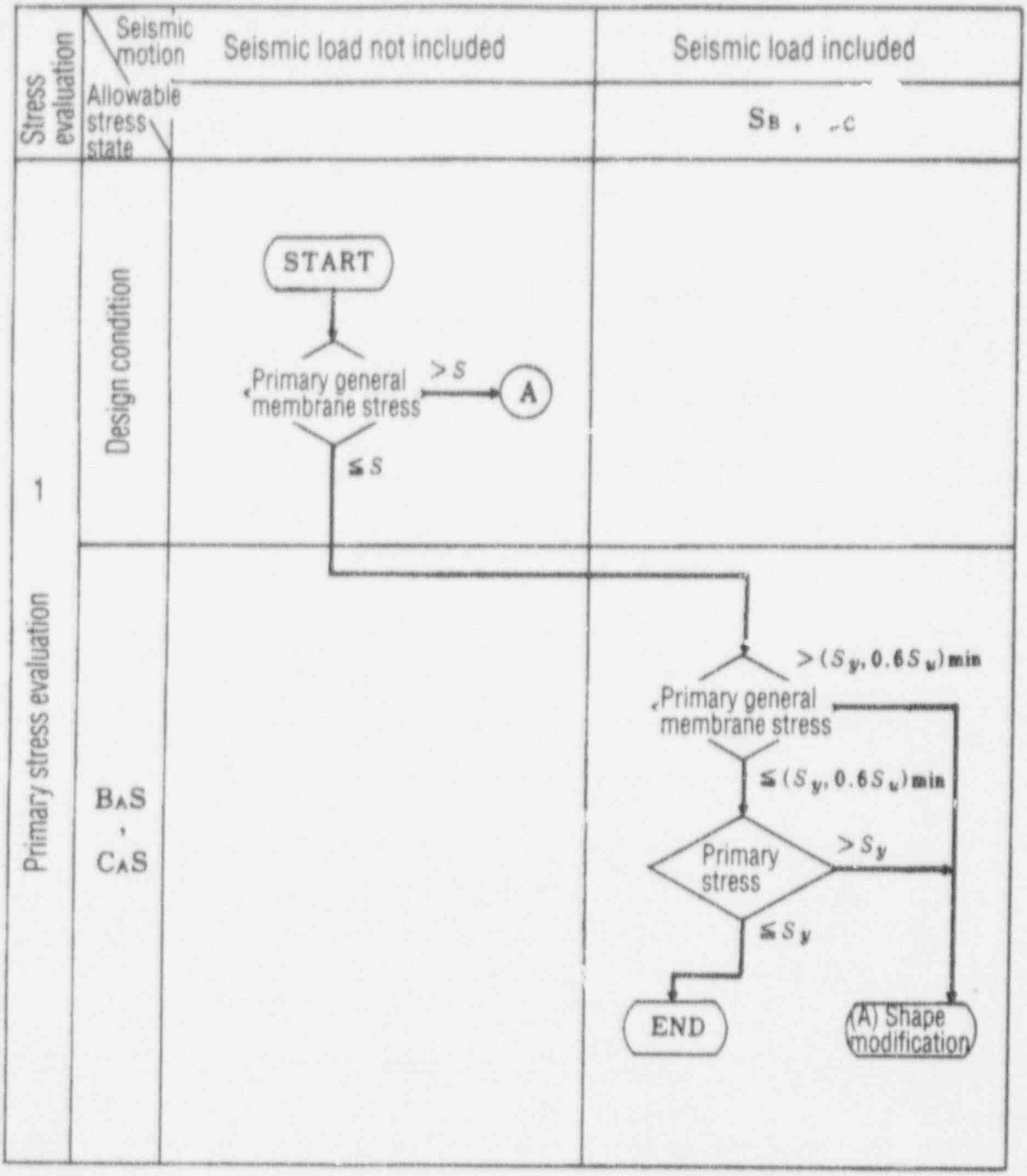


Figure 6.6.3-2. Procedure of stress evaluation of Class B, C containers.

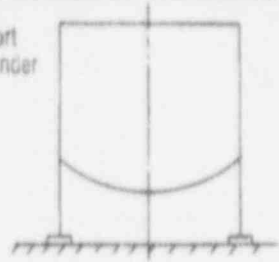
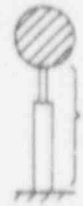
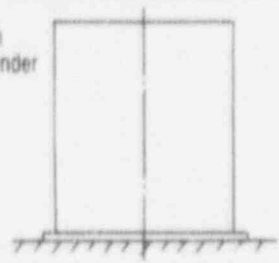
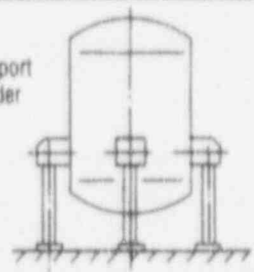
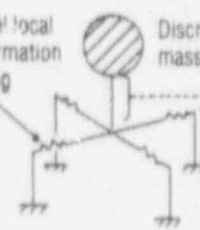
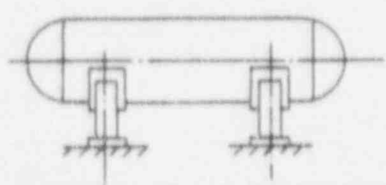
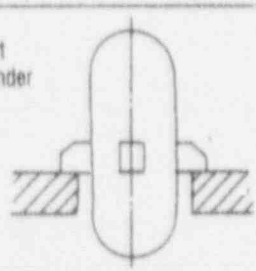
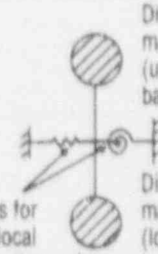
Container/tank shape	Model	Evaluation
Skirt-support vertical cylinder 		• Natural period • Barrel stress • Skirt stress • Anchor bolts stress
Flat-bottom vertical cylinder 		• Natural period • Barrel stress • Anchor bolts stress
Four-leg-support vertical cylinder 		• Natural period • Barrel stress • Legs stress • Anchor bolts stress
Horizontal cylinder 		• Natural period (longitudinal, transverse) • Barrel stress • Legs stress • Anchor bolts stress
Lug-support vertical cylinder 		• Natural period • Barrel stress • Lugs stress • Mounting bolts stress

Figure 6.6.3-3. Examples of natural period calculation models and evaluation methods.

Definitions of symbols (symbols commonly used in the calculation formulas of Class B and C equipment)

Symbol	Definition of symbol	Units
A_s	Barrel effective shear cross-sectional area	mm^2
A_b	Axial cross-sectional area of anchor bolts	mm^2
C_H	Design seismic coefficient in horizontal direction	—
D_i	Inner diameter of barrel	mm
E	Longitudinal modulus of elasticity of barrel	kgf/mm^2
E_s	Longitudinal modulus of elasticity of legs	kgf/mm^2
F	Value defined in Item 88-3-1-A(A) in "Notification No. 501"	kgf/mm^2
F_b	Tensile force acting on anchor bolts	kgf
f_{t0}	Allowable tensile stress of anchor bolts acted upon by tensile force only	kgf/mm^2
f_{ts}	Allowable tensile stress of anchor bolts acted upon by both tensile force and shear force simultaneously	kgf/mm^2
f_{sb}	Allowable shear stress of anchor bolts acted upon by shear force only	kgf/mm^2
G	Shear modulus of elasticity of barrel	kgf/mm^2
g	Acceleration of gravity (=9800)	mm/s^2
G_s	Shear modulus of elasticity of legs or skirt	kgf/mm^2
H	Water head	mm
I	Moment of inertia of barrel	mm^4
K	Spring constant	kgf/mm^2
P_r	Highest pressure used	kgf/cm
s	Ratio of longitudinal modulus of elasticity of anchor bolt to foundation	—
S_a	Allowable stress of barrel	kgf/mm^2
S_u	Value determined in Appendix Table 10 in "Notification 501"	kgf/mm^2
S_y	Value determined in Appendix Table 9 in "Notification No. 501"	kgf/mm^2
T	Natural period	s
t	Barrel plate thickness	mm
W_0	Effective operational weight of container	kgf
β, β_1, β_2	Attachment parameter according to Reference [6.6.3-2]	—
γ	Shell parameter according to Reference [6.6.3-2]	—
ρ	Specific gravity of liquid	—
ρ'	Specific weight of liquid ($= \rho \times 10^{-6}$ kgf/mm^3)	kgf/mm^2
σ_0	Maximum value of primary general membrane stress or combined stress of barrel	kgf/mm^2
σ_1	Maximum value of primary stress in barrel	kgf/mm^2
σ_b	Maximum value of tensile stress generated in anchor bolts	kgf/mm^2
σ_s	Maximum value of the combined stress of legs, skirt or lugs	kgf/mm^2
τ_b	Maximum value of shear stress generated in anchor bolts	kgf/mm^2

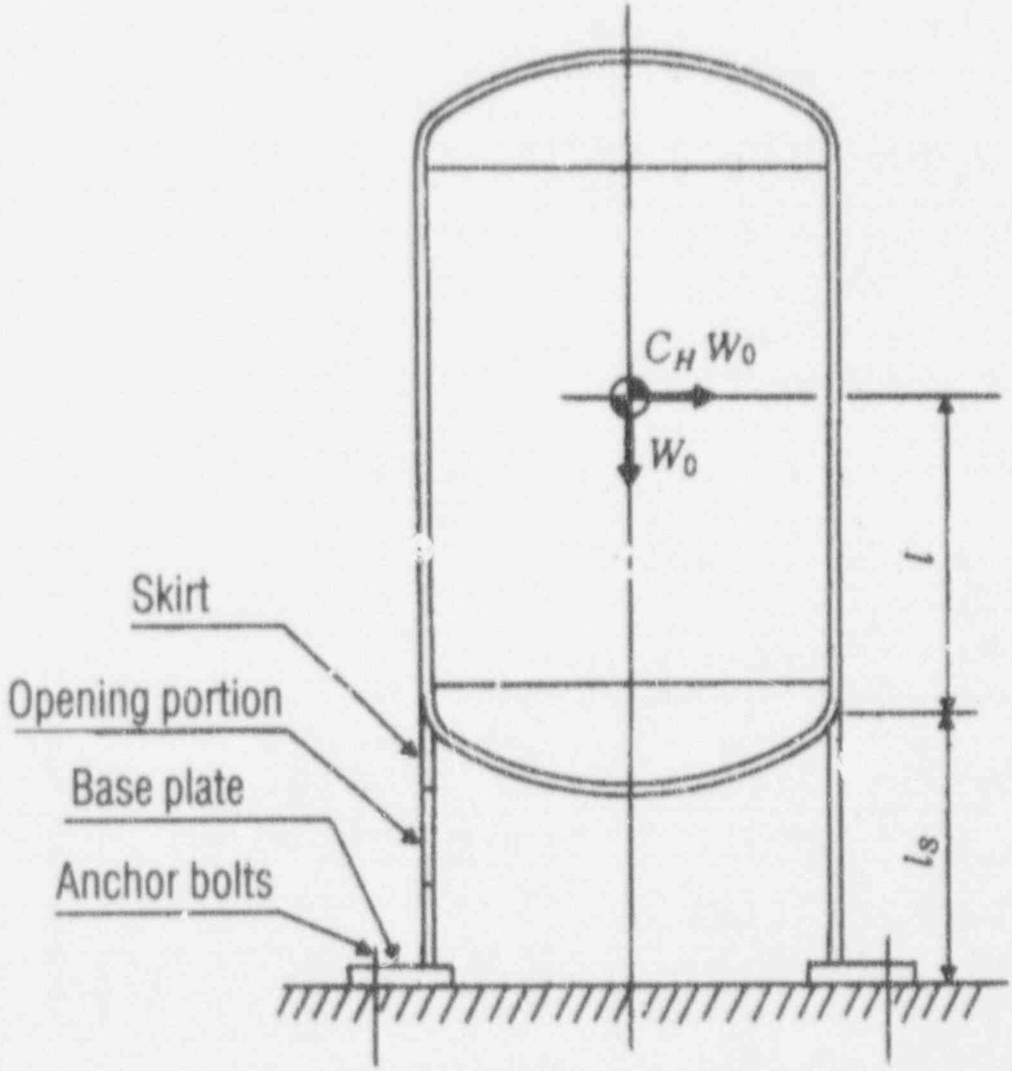


Figure 6.6.3-4. Schematic structural diagram.

Definitions of symbols (symbols used in the calculation formulas of skirt-support vertical cylindrical container).

Symbol	Definition of symbol	Units
A_{se}	Effective shear cross-sectional area of skirt	mm^2
D_c	Pitch circle diameter of anchor bolts	mm
t_1	Plate width corresponding to the area of anchor bolts	mm
t_2	Width corresponding to foundation on compression side	mm
C_c	Coefficient in anchor bolt calculation	—
C_t	Coefficient in anchor bolt calculation	—
D_{bo}	Outer diameter of base plate	mm
D_{bi}	Inner diameter of base plate	mm
D_j	Diameter of opening on skirt (j = 1,2,3...)	mm
D_s	Inner diameter of skirt	mm
F_c	Compressive force acting on the foundation	kg
f_b	Allowable buckling stress with respect to flexural moment	kgf/mm^2
f_c	Allowable buckling stress with respect to axial compression load	kgf/mm^2
f_t	Allowable tensile stress of skirt	kgf/mm^2
I_s	Moment of inertia of skirt	mm^4
e	Coefficient used in calculation of anchor bolts	—
k	Load coefficient for neutral axis in calculation of anchor bolts	—
l	Distance from joint point of barrel and skirt to center of gravity	mm
l_1, l_2	Distance from neutral axis to load acting point in calculation of anchor bolts	mm
l_r	Distance from center of gravity of container to upper end support portion	mm
l_s	Length of skirt	mm
M_s	Overturning moment acting on skirt	kgf·mm
M_{s1}	Overturning moment acting on upper end portion of skirt	kgf·mm
M_{s2}	Overturning moment acting on lower end portion of skirt	kgf·mm
n	Number of anchor bolts	—
t_s	Thickness of skirt	mm
W_e	Empty weight of the upper portion of container above the skirt joint portion	kgf
Q	Arbitrary horizontal force acting on the center of gravity	kgf
Q'	Reactive force acting on upper end support portion due to Q	kgf
Y	Maximum circumference on the horizontal cross section of skirt opening portion	mm
z	Coefficient used in calculation of anchor bolts	—
α	Angle determined for the neutral axis in calculation of anchor bolts	rad
η	Safety factor with respect to buckling stress	—
σ_{0c}	Combined compressive stress of barrel	kgf/mm^2

Definitions of symbols (symbols used in the calculation formulas of skirt-support vertical cylindrical container). *(Cont'd)*

Symbol	Definition of symbol	Units
σ_{0t}	Combined tensile stress of barrel	kgf/mm^2
σ_c	Compressive stress generated on foundation	kgf/mm^2
σ_{s1}	Axial stress due to weight of skirt during operation	kgf/mm^2
σ_{s2}	Axial stress due to bending moment of skirt	kgf/mm^2
σ_{x1}, $\sigma_{\phi 1}$	Axial/circumferential stresses generated in barrel due to static water head or internal pressure	kgf/mm^2
σ_{x2}	Axial tensile stress due to weight of barrel in operation	kgf/mm^2
σ_{x3}	Axial compressive strength due to empty weight of barrel	kgf/mm^2
σ_{x4}	Axial stress generated in the barrel by earthquake	kgf/mm^2
σ_{ϕ}	Sum of circumferential stresses of barrel	kgf/mm^2
τ_s	Shear stress generated in skirt by earthquake	kgf/mm^2
τ	Shear stress generated in barrel by earthquake	kgf/mm^2
σ_{xc}	Sum of axial stresses in barrel (compressive side)	kgf/mm^2
σ_{xt}	Sum of axial stresses in barrel (tensile side)	kgf/mm^2
$\phi_1(x)$	Function of allowable buckling stress with respect to compressive load	kgf/mm^2
$\phi_2(x)$	Function of allowable buckling stress with respect to flexural moment	kgf/mm^2
δ	Displacement amount of upper end of container due to load Q	mm
δ'	Displacement amount of upper end of container due to load Q'	mm
δ_0	Displacement amount at center of gravity of container due to loads Q, Q'	mm

(a) Calculation method of natural period (see Figure 6.6.3-5)

(i) Calculation model

Based on the aforementioned conditions, the container is taken as either a single discrete mass vibration model with a fixed lower end or a single discrete mass vibration model with a fixed lower end and a supported upper end.

(ii) Natural period

(1) When the lower end is fixed

The spring constant K due to flexural and shear deformations can be expressed as follows:

$$K = \frac{1}{\dfrac{l^3}{3EI} + \dfrac{1}{3E_s I_s}\left(3l^2 l_s + 3l l_s^2 + l_s^3\right) + \dfrac{l}{GA_e} + \dfrac{l_s}{G_s A_{se}}} \qquad (6.6.3\text{-}1)$$

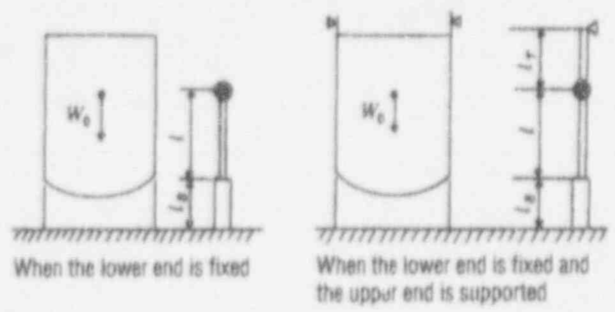


Figure 6.6.3-5. Calculation model of natural period.

In this case, with the effects of manhole, etc., on the skirt cross section taken into consideration, the sectional properties of the barrel and skirt an be represented as follows:

$$I_s = \frac{\pi}{8}(D_s+t_s)^3 t_s - \frac{1}{4}(D_s+t_s)^2 t_s Y \tag{6.6.3-2}$$

$$Y = \sum_j (D_s+t_s)\sin^{-1}\left(\frac{D_j}{D_s+t_s}\right) \tag{6.6.3-3}$$

$$A_{se} = \frac{2}{3}\{\pi(D_s+t_s)-Y\}t_s \tag{6.6.3-4}$$

$$I = \frac{\pi}{8}(D_i+t)^3 t \tag{6.6.3-5}$$

$$A_s = \frac{2}{3}\pi(D_i+t)t \tag{6.6.3-6}$$

Hence, the natural period can be derived using the formula:

$$T = 2\pi\sqrt{\frac{W_0}{Kg}} \tag{6.6.3-7}$$

(2) When the lower end is fixed and the upper end is supported

As shown in Figure 6.6.3-6, the reactive force Q' generated at the upper end support portion when a load Q acts in the horizonal direction at the position of the center of gravity can be derived by making the displacements of the upper end caused by different loads equal to each other.

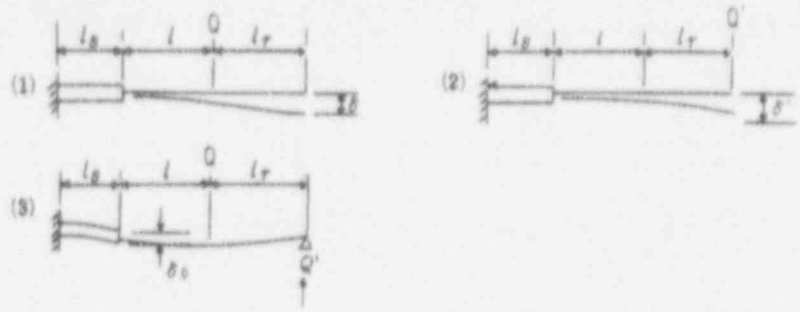


Figure 6.6.3-6. Deformation model in the case with fixed lower end and supported upper end.

In the case of Figure 6.6.3-6(1),

$$I_s = \frac{\pi}{8}(D_s+t_s)^3 t_s - \frac{1}{4}(D_s+t_s)^2 t_s Y \tag{6.6.3-2}$$

$$Y = \sum (D_s+t_s)\sin^{-1}\left(\frac{D_i}{D_s+t_s}\right) \tag{6.6.3-3}$$

$$A_{sw} = \frac{2}{3}\{\pi(D_s+t_s)-Y\}t_s \tag{6.6.3-4}$$

$$I = \frac{\pi}{8}(D_i+t)^3 t \tag{6.6.3-5}$$

$$A_t = \frac{2}{3}\pi(D_i+t)t \tag{6.6.3-6}$$

$$T = 2\pi\sqrt{\frac{W_0}{Kg}} \tag{6.6.3-7}$$

$$\begin{aligned} \delta = {} & \frac{Ql^2}{6EI}(2l+3l_r) \\ & + \frac{Q}{6E_sI_s}\{2l_s^3+3l_s^2 l_r+6l_s l(l_s+l+l_r)\} \\ & + \frac{Ql}{GA_t}+\frac{Ql_s}{G_sA_{sw}} \end{aligned} \tag{6.6.3-8}$$

In the case of Figure 6.6.3-6(2),

$$\delta' = \frac{Q'(l+l_r)^3}{3EI} + \frac{Q'}{3E_sI_s}\{3(l+l_r)^2 l_s + 3(l+l_r)l_s^2 + l_s^3\} + \frac{Q'(l+l_r)}{GA_e} + \frac{Q' l_s}{G_sA_{se}} \tag{6.6.3-9}$$

By setting formula (6.6. .) equal to formula (6.6.3-9), we have,

$$Q' = Q\left\{\frac{\dfrac{l^2(2l+3l_r)}{6EI} + \dfrac{2l_s^3 + 3l_s^2 l_r + 6l_s l(l_s + l + l_r)}{6E_sI_s} + \dfrac{l}{GA_e} + \dfrac{l_s}{G_sA_{se}}}{\dfrac{(l+l_r)^3}{3EI} + \dfrac{3(l+l_r)^2 l_s + 3(l+l_r)l_s^2 + l_s^3}{3E_sI_s} + \dfrac{l+l_r}{GA_e} + \dfrac{l_s}{G_sA_{se}}}\right\} \tag{6.6.3-10}$$

It is possible to determine displacement δ_0 at the position of the center of gravity as shown in Figure 6.6.3-6(3), and the spring constant K can be represented by the following formula:

$$K = \frac{Q}{\delta_0} = 1 \Bigg/ \left\{\frac{l^3}{3EI} + \frac{3l^2 l_s + 3l l_s^2 + l_s^3}{3E_sI_s} + \left(1 - \frac{Q'}{Q}\right)\left(\frac{l}{GA_e} + \frac{l_s}{G_sA_{se}}\right) - \frac{Q'}{Q}\left(\frac{2l^3 + 3l^2 l_r}{6EI} + \frac{3l_s^2 l + l_s^3 + 3l_s l^2 + 3l_s l l_r + \frac{3}{2}l_s^2 l_r}{3E_sI_s}\right)\right\} \tag{6.6.3-11}$$

The natural period is determined using formula (6.6.3-7).

(b) Calculation methods of stresses

(i) Barrel stresses

(1) Stress due to static water head or internal pressure

In the case of static water head,

$$\sigma_{\phi 1} = \frac{\rho' H D_i}{2t} \tag{6.6.3-12}$$

$$\sigma_{x1} = 0 \tag{6.6.3-13}$$

In the case of internal pressure

$$\sigma_{\phi 1} = \frac{P_r(D_i + 1.2t)}{200t} \tag{6.6.3-14}$$

$$\sigma_{x1} = \frac{P_r(D_i + 1.2t)}{400t} \tag{6.6.3-15}$$

(2) Stress due to weight during operation

With the joint point between barrel and skirt taken as the boundary, in the upper portion, a compressive stress due to the self weight of the barrel is generated; in the lower portion, a tensile stress due to the self weight of the lower barrel portion and the weight of the content is generated.

For the upper barrel (compressive stress)

$$\sigma_{x3} = \frac{W_e}{\pi (D_i + t)t} \tag{6.6.3-16}$$

For the lower barrel

$$\sigma_{x2} = \frac{W_0 - W_e}{\pi (D_i + t)t} \tag{6.6.3-17}$$

(3) Stress due to horizontal seismic force

The maximum bending moment due to the horizontal seismic force occurs at the joint portion between barrel and skirt. The axial stress due to this bending moment and the shear stress due to the seismic force are determined as follows.

(a) When the lower end is fixed

$$\sigma_{x4} = \frac{4C_H W_0 l}{\pi (D_i + t)^2 t} \tag{6.6.3-18}$$

$$\tau = \frac{2C_H W_0}{\pi (D_i + t)t} \tag{6.6.3-19}$$

(b) When the lower end is fixed and the upper end is supported

$$\sigma_{x4} = \frac{4C_H W_0 \left| l - \frac{Q'}{Q}(l + l_r) \right|}{\pi (D_i + t)^2 t} \tag{6.6.3-20}$$

$$\tau = \frac{2C_H W_0 \left(1 - \frac{Q'}{Q} \right)}{\pi (D_i + t) t} \tag{6.6.3-21}$$

(4) Stress combinations

The barrel stresses calculated in (1)-(3) can be combined as follows.

<u>Primary general membrane stress</u>

(a) Combined tensile stress

$$\sigma_\phi = \sigma_{\phi 1} \tag{6.6.3-22}$$

$$\sigma_{xt} = \sigma_{x1} + \sigma_{x2} + \sigma_{x4} \tag{6.6.3-23}$$

$$\sigma_{0t} = \frac{1}{2}\left\{ \sigma_\phi + \sigma_{xt} + \sqrt{(\sigma_\phi - \sigma_{xt})^2 + 4\tau^2} \right\} \tag{6.6.3-24}$$

(b) Combined compressive stress

$$\sigma_\phi = -\sigma_{\phi 1} \tag{6.6.3-25}$$

$$\sigma_{xc} = -\sigma_{x1} + \sigma_{x3} + \sigma_{x4} \tag{6.6.3-26}$$

When σ_{xc} has a positive value (compression side), the following combined compressive stress is obtained.

$$\sigma_{0c} = \frac{1}{2}\left\{ \sigma_\phi + \sigma_{xc} + \sqrt{(\sigma_\phi - \sigma_{xc})^2 + 4\tau^2} \right\} \tag{6.6.3-27}$$

Since the primary stress is the same as the primary general membrane stress, its formulas can be omitted here.

(ii) Skirt stresses

(1) Stress due to weight during operation

The compressive stress generated at the skirt bottom portion due to weight during operation is calculated by the following formula;

$$\sigma_{s1} = \frac{W_0}{\{\pi(D_s + t_s) - Y\}t_s} \tag{6.6.3-28}$$

(2) Stress due to horizontal seismic force

A bending moment acts on the skirt due to the horizontal seismic force. The axial stress due to this bending moment and the shear stress due to the seismic force can be calculated as follows:

(a) When the lower end is fixed

$$\sigma_{s2} = \frac{M_s}{(D_s + t_s)t_s\left\{\frac{\pi}{4}(D_s + t_s) - \frac{Y}{2}\right\}} \tag{6.6.3-29}$$

$$\tau_s = \frac{2C_H W_0}{\{\pi(D_s + t_s) - Y\}t_s} \tag{6.6.3-30}$$

where,

$$M_s = C_H W_0 (l_s + l) \tag{6.6.3-31}$$

(b) When the lower end is fixed and the upper end is supported

The axial stress is represented by formula (6.6.3-29). Bending moment M_S is taken as following M_{S1} or M_{S2}, whichever is greater.

$$M_{s1} = C_H W_0 \left| l - \frac{Q'}{Q}(l + l_r) \right| \tag{6.6.3-32}$$

$$M_{s2} = C_H W_0 \left| l_s + l - \frac{Q'}{Q}(l_s + l + l_r) \right| \tag{6.6.3-33}$$

$$\tau_s = \frac{2C_H W_0\left(1 - \frac{Q'}{Q}\right)}{\{\pi(D_s + t_s) - Y\}t_s} \tag{6.6.3-34}$$

(3) Combination of stresses

The combination of stresses is represented as follows:

$$\sigma_s = \sqrt{(\sigma_{s1} + \sigma_{s2})^2 + 3\tau_s^2} \tag{6.6.3-35}$$

(iii) Anchor bolt stresses

(1) Shear stress

(a) When the lower end is fixed

$$\tau_b = \frac{C_H W_0}{nA_b} \tag{6.6.3-36}$$

(b) When the lower end is fixed and the upper end is supported

$$\tau_b = \frac{C_H W_0\left(1 - \frac{Q'}{Q}\right)}{nA_b} \tag{6.6.3-37}$$

(2) Tensile stress

When the lower end is fixed, overturning moment M_s acting on the foundation is calculated using formula (6.6.3-31); when the lower end is fixed and the upper end is supported, it is calculated using formula (6.6.3-33). In the case when the overturning moment acts, the tensile load of the anchor bolts and the compressive load of the foundation portion can be derived in consideration of the equilibrium between the load and displacement (see Figure 6.6.3-7), with the following procedure:

$$t_1 = \frac{nA_b}{\pi D_c} \tag{6.6.3-38}$$

$$t_2 = \frac{1}{2}(D_{bo} - D_{bi}) - t_1 \tag{6.6.3-39}$$

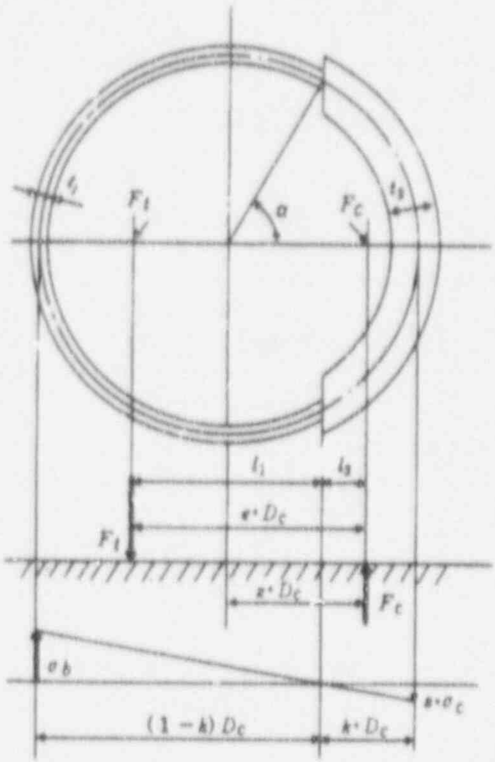


Figure 6.6.3-7. Diagram illustrating loads on foundation.

{a} Coefficient k is derived after σ_b, σ_c are assumed:

$$k = \frac{1}{1 + \dfrac{\sigma_b}{s\sigma_c}} \tag{6.6.3-40}$$

{b} α is determined.

$$\alpha = \cos^{-1}(1 - 2k) \tag{6.6.3-41}$$

{c} Constants e, z, C_t, C_c can be calculated as follows:

$$e = \frac{1}{2}\left\{\frac{(\pi-\alpha)\cos^2\alpha + \frac{1}{2}(\pi-\alpha) + \frac{3}{2}\sin\alpha\cos\alpha}{(\pi-\alpha)\cos\alpha + \sin\alpha} + \frac{\frac{1}{2}\alpha - \frac{3}{2}\sin\alpha\cos\alpha + \alpha\cos^2\alpha}{\sin\alpha - \alpha\cos\alpha}\right\} \tag{6.6.3-42}$$

$$z = \frac{1}{2}\left\{\cos\alpha + \left(\frac{\frac{1}{2}\alpha - \frac{3}{2}\sin\alpha\cos\alpha + \alpha\cos^2\alpha}{\sin\alpha - \alpha\cos\alpha}\right)\right\} \tag{6.6.3-43}$$

$$C_t = \frac{2\{(\pi - \alpha)\cos\alpha + \sin\alpha\}}{1 + \cos\alpha} \tag{6.6.3-44}$$

$$C_c = \frac{2(\sin\alpha - \alpha\cos\alpha)}{1 - \cos\alpha} \tag{6.6.3-45}$$

{d} F_t and F_c are calculated using these constants:

$$F_t = \frac{M_s - W_0 z D_c}{e D_c} \tag{6.6.3-46}$$

$$F_c = F_t + W_0 \tag{6.6.3-47}$$

When no tensile force acts on the foundation bolts, α is equal to π. Consequently, the values of formulas (6.6.3-42) and (6.6.3-43) when α approaches π, i.e., $e = 0.75$, $z = 0.25$, are substituted into formula (6.6.3-46). From the obtained value of F_t, the presence/absence of tensile force is judged as follows:

If $F_t \leq 0$, no tensile force exists.
If $F_t > 0$, tensile force exists and the following calculation is performed.

{e} σ_b and σ_c are calculated:

$$\sigma_b = \frac{2F_t}{t_1 D_c C_t} \tag{6.6.3-48}$$

$$\sigma_c = \frac{2F_c}{(t_2 + s t_1) D_c C_c} \tag{6.6.3-49}$$

It is then checked to see if σ_b and σ_c are very similar to the values assumed in {a}. In this case, σ_b and σ_c are considered as the stresses between the anchor bolts and foundation.

(c) Evaluation method

(i) Evaluation of natural period

Based on the natural period derived in (a), (ii), the design horizontal seismic coefficient is confirmed.

Table 6.6.3-1

Stress type	Allowable stress S_a
Primary general membrane stress	Design yield strength S_y or 0.6 times design fracture strength S_u, whichever is smaller. For austenitic stainless steel and high-nickel allow, however, it may also be taken as 1.2 times the value defined in Appendix Table 6 in "Notification No. 501."

(ii) Evaluation of stresses

{1} Evaluation of stress in barrel

{a} The combination of stresses determined in "(b)(i){4} Combination of stresses" should be lower than allowable stress S_a at the highest temperature of application for the barrel (see Table 6.6.3-1). The evaluation scheme of the primary stress is omitted here since the calculated stress is equal to the primary general membrane stress.

{2} Skirt stress evaluation

{a} The combination of stresses derived in "(b)(ii){3} Combination of stresses" should be less than allowable stress f_t.

$$f_t = \left(\frac{F}{1.5}\right)1.5 \qquad (6.6.3\text{-}50)$$

{b} The compressive membrane stress should meet the following formula:

$$\frac{\eta\sigma_{s1}}{f_c} + \frac{\eta\sigma_{s2}}{f_b} \leq 1 \qquad (6.6.3\text{-}51)$$

where, f_c is defined as follows

$$f_c = F, \quad \text{when} \quad \frac{D_s + 2t_s}{2t_s} \leq \frac{1200}{F} \qquad (6.6.3\text{-}52)$$

$$f_c = F\left[1 - \frac{1}{6800}\left\{F - \Phi_1\left(\frac{8000}{F}\right)\right\}\left(\frac{D_s + 2t_s}{2t_s} - \frac{1200}{F}\right)\right],$$
$$\text{when} \quad \frac{1200}{F} < \frac{D_s + 2t_s}{2t_s} < \frac{8000}{F} \qquad (6.6.3\text{-}53)$$

$$f_c = \Phi_1\left(\frac{D_s+2t_s}{2t_s}\right), \quad \text{when} \quad \frac{8000}{F} \le \frac{D_s+2t_s}{2t_s} \le 800 \tag{6.6.3-54}$$

where $\Phi_1(x)$ is a function defined as follows:

$$\Phi_1(x) = 0.6\frac{E_s}{x}\left[1-0.901\left\{1-\exp\left(-\frac{1}{16}\sqrt{x}\right)\right\}\right] \tag{6.6.3-55}$$

f_b is defined as follows:

$$f_b = F, \quad \text{when} \quad \frac{D_s+2t_s}{2t_s} \le \frac{1200}{F} \tag{6.6.3-56}$$

$$f_b = F\left[1-\frac{1}{8400}\left\{F-\Phi_2\left(\frac{9600}{F}\right)\right\}\left(\frac{D_s+2t_s}{2t_s}-\frac{1200}{F}\right)\right], \quad \text{when} \quad \frac{1200}{F} < \frac{D_s+2t_s}{2t_s} < \frac{9600}{F} \tag{6.6.3-57}$$

$$f_b = \Phi_2\left(\frac{D_s+2t_s}{2t_s}\right), \quad \text{when} \quad \frac{9600}{F} \le \frac{D_s+2t_s}{2t_s} \le 800 \tag{6.6.3-58}$$

where $\Phi_2(x)$ is a function defined as follows:

$$\Phi_2(x) = 0.6\frac{E_s}{x}\left[1-0.731\left\{1-\exp\left(-\frac{1}{16}\sqrt{x}\right)\right\}\right] \tag{6.6.3-59}$$

η is the safety factor and is defined as follows:

$$\eta = 1, \quad \text{when} \quad \frac{D_s+2t_s}{2t_s} \le \frac{1200}{F} \tag{6.6.3-60}$$

$$\eta = 1+\frac{0.5F}{6800}\left(\frac{D_s+2t_s}{2t_s}-\frac{1200}{F}\right), \quad \text{when} \quad \frac{1200}{F} < \frac{D_s+2t_s}{2t_s} < \frac{8000}{F} \tag{6.6.3-61}$$

$$\eta = 1.5, \quad \text{when} \quad \frac{8000}{F} \le \frac{D_s+2t_s}{2t_s} \tag{6.6.3-62}$$

Table 6.6.3-2

	Allowable tensile stress f_{t0}	Allowable shear stress f_{sb}
Formulas of calculation	$\left(\frac{F}{2}\right) 1.5$	$\left(\frac{F}{1.5\sqrt{3}}\right) 1.5$

{3} Anchor bolt stress evaluation

The tensile stress σ_b of the anchor bolts derived in (b)(iii) should be less than the allowable tensile stress f_{ts} derived using the following two formulas. Shear stress τ_b should be less than allowable shear stress f_{sb} of the bolts acted upon by shear force only.

$$
\begin{aligned}
f_{ts} &= 1.4 f_{t0} - 1.6\tau_b \\
f_{ts} &\le f_{t0}
\end{aligned}
\qquad (6.6.3\text{-}63)
$$

where f_{t0}, f_{sb} are defined as in Table 6.6.3-2.

c. Flat-bottom vertical cylindrical container (see Figure 6.6.3-8)

Assumed conditions

{1} The weight of the container is assumed to be concentrated at the center of gravity.
{2} The container has its lower end plate (bottom plate) fixed on the foundation by multiple anchor bolts; hence, the lower portion of the barrel is taken as fixed.
{3} The seismic force is taken as acting on the container in the horizontal direction. The design seismic coefficient in the vertical direction is not considered.

Analysis condition

{1} As the deformation mode, the flexural and shear deformations, when the entire container is considered as a beam, are considered.

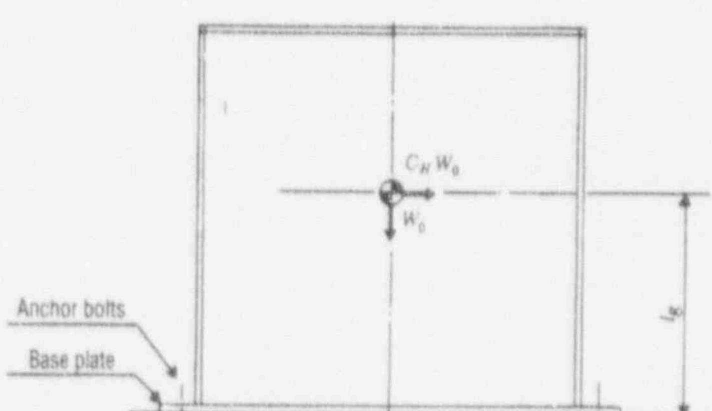


Figure 6.6.3-8. Schematic structural diagram.

Definitions of symbols (symbols used in the calculation formulas for flat-bottom cylindrical container)

Symbol	Definition of symbol	Units
D_{bo}	Effective outer diameter of base plate	mm
D_{bi}	Effective inner diameter of base plate	mm
D_c	Pitch circle diameter of anchor bolts	mm
t_1	Plate width corresponding to anchor bolt area	mm
t_2	Effective width for compressive-side foundation	mm
C_c	Coefficient in anchor bolt calculation	—
C_t	Coefficient in anchor bolt calculation	—
F_c	Compressive force acting on the foundation	kgf
e	Coefficient in anchor bolt calculation	—
k	Load coefficient for neutral axis in anchor bolt calculation	—
W_e	Empty weight of container	kgf
z	Coefficient in anchor bolt calculation	—
α	Angle for determining the neutral axis in anchor bolt calculation	rad
l_g	Distance from foundation to container center of gravity	mm
l_1, l_2	Distance from the neutral axis to load acting point in anchor bolt calculation	mm
n	Number of foundation bolts	—
σ_{0c}	Barrel's combined primary general membrane stress (compressive side)	kgf/mm^2
σ_{0t}	Barrel's combined primary general membrane stress (tensile side)	kgf/mm^2
σ_c	Compressive stress generated in the foundation	kgf/mm^2
σ_{x1}, $\sigma_{\phi 1}$	Axial and circumferential stresses generated in barrel by static water head or internal pressure	kgf/mm^2
σ_{x2}	Axial compression stress due to empty weight of barrel	kgf/mm^2
σ_{x3}	Axial stress due to horizontal seismic force acting on the barrel	kgf/mm^2
σ_{ϕ}	Sum of primary general membrane stresses acting in circumferential direction of barrel	kgf/mm^2
τ	Shear stress due to horizontal seismic force acting on the barrel	kgf/mm^2
σ_{xc}	Sum of primary general membrane stresses acting in the axial direction of the barrel (compressive side)	kgf/mm^2
σ_{xt}	Sum of primary general membrane stresses acting in the axial direction of the barrel (tensile side)	kgf/mm^2

(a) Calculation method of natural period

(i) Calculation model

Under the aforementioned conditions, the container can be represented by the single discrete mass oscillation model with fixed lower end shown in Figure 6.6.3-9.

(ii) Natural period

The spring constant K due to flexural and shear deformations can be calculated by the following formula:

$$K = \frac{1}{\dfrac{l_g^3}{3EI} + \dfrac{l_g}{GA_e}} \qquad (6.6.3\text{-}64)$$

where the cross-sectional parameters of the barrel can be represented as follows:

$$I = \frac{\pi}{8}(D_i + t)^3 t \qquad (6.6.3\text{-}65)$$

$$A_e = \frac{2}{3}\pi(D_i + t)t \qquad (6.6.3\text{-}66)$$

Hence, the natural period can be derived as follows:

$$T = 2\pi\sqrt{\frac{W_0}{K \cdot g}} \qquad (6.6.3\text{-}67)$$

(b) Calculation method of stresses

(i) Stresses in barrel

{1} Stress due to static water head or internal pressure
In the case of static water head,

$$\sigma_{\phi 1} = \frac{\rho' H D_i}{2t} \qquad (6.6.3\text{-}68)$$

$$\sigma_{x1} = 0 \qquad (6.6.3\text{-}69)$$

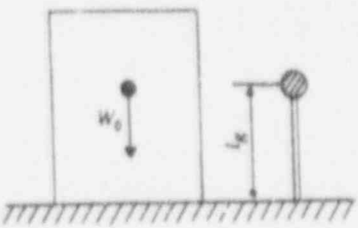


Figure 6.6.3-9. Calculation model of natural period.

In the case of internal pressure,

$$\sigma_{\phi 1} = \frac{P_r(D_i + 1.2t)}{200t} \tag{6.6.3-70}$$

$$\sigma_{x1} = \frac{P_r(D_i + 1.2t)}{400t} \tag{6.6.3-71}$$

{2} Stress due to weight in operation

At the point where the barrel and the baseplate are joined, a compressive stress due to the self weight of the barrel is generated:

$$\sigma_{x2} = \frac{W_e}{\pi(D_i + t)t} \tag{6.6.3-72}$$

{3} Stress due to horizontal seismic force

Due to the horizontal seismic force, the maximum bending moment takes place at the portion joining the base plate. The axial stress due to the bending moment and the shear stress due to the seismic force can be calculated as follows:

$$\sigma_{x3} = \frac{4C_H W_0 l_g}{\pi(D_i + t)^2 t} \tag{6.6.3-73}$$

$$\tau = \frac{2C_H W_0}{\pi(D_i + t)t} \tag{6.6.3-74}$$

{4} Combination of stresses

The stresses calculated in {1}-{3} are comb[illegible] as follows:

Primary general membrane stress

{a} Combined tensile stress

$$\sigma_{\phi} = \sigma_{\phi 1} \tag{6.6.3-75}$$

$$\sigma_{xt} = \sigma_{x1} - \sigma_{x2} + \sigma_{x3} \tag{6.6.3-76}$$

$$\sigma_{0t} = \frac{1}{2}\left\{\sigma_{\phi} + \sigma_{xt} + \sqrt{\left(\sigma_{\phi} - \sigma_{xt}\right)^2 + 4\tau^2}\right\} \tag{6.6.3-77}$$

{b} Combined compressive stress

$$\sigma_{\phi} = -\sigma_{\phi 1} \tag{6.6.3-78}$$

$$\sigma_{xc} = -\sigma_{x1} + \sigma_{x2} + \sigma_{x3} \tag{6.6.3-79}$$

When σ_{xc} has a positive value (compressive side), the following combined compressive stress can be determined:

$$\sigma_{0c} = \frac{1}{2}\left\{\sigma_{\phi} + \sigma_{xc} + \sqrt{\left(\sigma_{\phi} - \sigma_{xc}\right)^2 + 4\tau^2}\right\} \tag{6.6.3-80}$$

Description of [illegible] primary stress is omitted here because it is the same as the primary general membrane stress.

(ii) Stress of anchor bolts

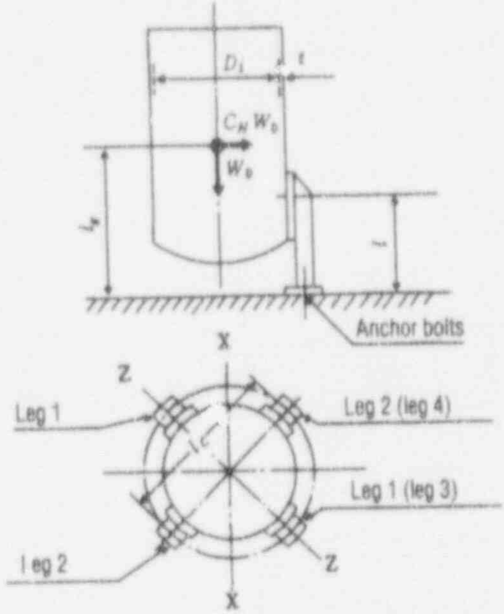
The stress calculation of the anchor bolts is performed in the same way as in (b)(iii) and with fixed lower end in section "b. Skirt-support vertical cylindrical container."

(c) Evaluation method

(i) Based on the natural period derived in (a)(ii), the horizontal design seismic coefficient is checked.

(ii) Stress evaluation

In (c)(ii) in section "6.6.3(1)b. Skirt-support vertical cylindrical container," "{1} Barrel stress evaluation" and "{3} Anchor bolt stress evaluation" are used here for stress evaluation.



For the leg part, the radial direction of the barrel is taken as the r-axis, and the direction perpendicular to it is taken as the t-axis.

Figure 6.6.3-10. Schematic structural diagram.

d. Four-leg-support cylindrical container (See Figure 6.6.3-10)

Assumed conditions

{1} The weight of the container is considered to be concentrated at the center of gravity.
{2} The seismic force acting on the container is assumed in the horizontal direction, while the design seismic coefficient in the vertical direction is not taken into consideration.
{3} In the case that anchor bolts are arranged in a row (viewed in the direction perpendicular to horizontal force (F_0)), supporting condition at the lower end of the leg is assumed as simply supported.

Analysis conditions

{1} At the portion where legs are mounted on the barrel plate, the local deformation of the barrel is taken into consideration.
{2} As the deformation modes, the flexural and shear deformation of the leg are considered.

(a) Calculation method of natural period

(i) Calculation models

There are the following four types of calculation models for the container as the support conditions of the lower ends of legs are combined differently.

- Both legs 1 and 2 are fixed
- Both legs 1 and 2 are simply supported
- Leg 1 is fixed, leg 2 is simply supported
- Leg 1 is simply supported, leg 2 is fixed

Figures 6.6.3-8-6.6.3-11 illustrate the deformation modes in these cases.

Definitions of symbols (symbols used in the calculation formulas of 4-leg-support vertical cylindrical container)

Symbol	Definition of symbol	Units
A_f	Cross-sectional area of compressive flange of legs	mm^2
A_{sf}	Cross-sectional area of T-shaped cross section composed of compressive flange of leg and 1/6 of the web	mm^2
a	Width of leg bottom plate in radial direction	mm
A_s	Cross-sectional area of leg	mm^2
A_{sr}	Effective shear sectional area of leg with respect to the radial axis	mm^2
A_{st}	Effective shear sectional area of leg with respect to the circumferential axis	mm^2
A_{s1}	Shear sectional area of leg with respect to radial axis	mm^2
A_{s2}	Shear sectional area of leg with respect to circumferential axis	mm^2
b	Width of leg bottom plate in circumferential direction	mm
C	Correction coefficient of leg with respect to buckling bending moment	—
C_1	Half of the width of attachment at the root portion where the leg is attached to the barrel (circumferential direction of barrel)	mm
C_2	Half of the width of the attachment at the root portion where the leg is attached to the barrel (axial direction of barrel)	mm
C_c	Value obtained from Reference [6.6.3-2]	—
C_l	Value obtained from Reference [6.6.3-2]	—
d_1	Distance in radial direction from leg bottom plate end surface to anchor bolt center	mm
d_2	Distance in circumferential direction from leg bottom plate end surface to anchor bolt center	mm
d_b	Outer diameter of anchor bolts	mm
e	Distance from leg center to eccentric load acting point	mm
F_0	Horizontal force in vibration model system	kgf
f_c	Leg's allowable compressive stress	kgf/mm^2
f_{br}	Leg's allowable flexural stress around radial axis	kgf/mm^2
f_{bt}	Leg's allowable flexural stress arouond the axis perpendicular to the radial direction	kgf/mm^2
f_t	Leg's allowable tensile stress	kgf/mm^2
h	Height of leg cross section	mm
i	Leg radius of gyration with respect to the weak axis	mm
i_f	Radius of gyration with respect to the web axis of T-shaped cross section made of compressive flange of leg and 1/6 the web	mm
I_{sr}	Moment of inertia of leg with respect to radial axis	mm^4
I_{st}	Moment of inertia of leg with respect to circumferential axis	mm^4

Definitions of symbols (symbols used in the calculation formulas of 4-leg-support vertical cylindrical container) *(Cont'd)*

Symbol	Definition of symbol	Units
I_{sf}	Moment of inertia around the web axis of the T-shaped cross section composed of the leg's compressive flange and 1/6 the web	mm^4
I_t	Leg's torsional moment coefficient	mm^4
K_1, K_2	Constants according to Reference [6.6.3-2]	—
K_c	Local spring constant with respect to circumferential bending moment in the root portion where the leg is attached to the barrel (value obtained from Reference [6.6.3-4])	—
K_l	Local spring constant with respect to longitudinal bending moment in the root portion where the leg is attached to the barrel (value obtained from Reference [6.6.3-4])	—
K_r	Local spring constant with respect to the radial load at the root portion where the leg is attached to the barrel (value obtained from Reference [6.6.3-4]	—
	Length of leg	mm
l_c	Distance between central axes of legs	mm
l_g	Distance from foundation to center of gravity of the upper portion of container	mm
l_k	Effective buckling length of leg	mm
M_1	Vertical moment at the root portion where the leg is attached to the barrel due to seismic force in the Z-direction	kgf·mm
M	Torsional moment at the root portion where the leg is attached to the barrel due to seismic force in the Z-direction	kgf·mm
M	Circumferential moment at the root portion where the leg is attached to the barrel due to seismic force in the Z-direction	kgf·mm
M_l	Vertical moment at the root portion where the leg is attached to the barrel due to weight in operation	kgf·mm
M_{s1}, M_{s2}	Bending moment acting on the upper/lower ends of leg	kgf·mm
M_{x1}	Combined moment acting on the bottom portion of legs 1 and 4 due to earthquake in the X-direction	kgf·mm
M_{x2}	Combined moment acting on the bottom portion of legs 2 and 3 due to earthquake in the X-direction	kgf·mm
M_{z1}	Combined moment acting on the bottom portion of leg 1 due to earthquake in the Z-direction	kgf·mm
M_{z2}	Combined moment acting on the bottom portion of legs 2 and 4 due to earthquake in the Z-direction	kgf·mm
M_{z3}	Combined moment acting on the bottom portion of leg 3 due to earthquake in the Z-direction	kgf·mm
N_x	Membrane force in axial direction generated in the barrel	kgf/mm
N_ϕ	Membrane force in circumferential direction generated in the barrel	kgf/mm

Definitions of symbols (symbols used in the calculation formulas of 4-leg-support vertical cylindrical container)

(Cont'd)

Symbol	Definition of symbol	Units
n	Number of anchor bolts for each leg	—
n_1, n_2	Number of anchor bolts acted upon by tensile force	—
P	Radial load of root portion where the leg is attached to the barrel due to weight in operation	kgf
P_1	Radial load of root portion where the leg is attached to the barrel due to earthquake in the Z-direction	kgf
Q	Circumferential load of root portion where the leg is attached to the barrel due to earthquake in the Z-direction	kgf
R	Axial force of leg due to weight in operation	kgf
R_1	Axial force acting on the leg due to earthquake in the Z-direction	kgf
R_{x1}	Axial force acting on legs 1 and 4 due to earthquake in the X-direction	kgf
R_{x2}	Axial force acting on legs 2 and 3 due to earthquake in the Z-direction	kgf
R_{z1}	Axial force acting on leg 1 due to earthquake in the Z-direction	kgf
R_{z2}	Axial force acting on legs 2 and 4 due to earthquake in the Z-direction	kgf
R_{z3}	Axial force acting on leg 3 due to earthquake in the Z-direction	kgf
r_m	Average radius of barrel	mm
u	Distance from central axis of leg to center of barrel wall	mm
X_n	Width of foundation which receives compressive force	mm
Z_{sp}	Torsional section modulus of leg	mm^3
Z_{sr}	Leg section modulus with respect to radial axis	mm^3
Z_{st}	Leg section modulus with respect to circumferential axis	mm^3
δ	Displacement of center of gravity due to horizontal force F_0	mm
Δ_r	Local displacement in radial direction of barrel due to weight in operation	mm
Δ_{r1}	Local displacement in radial direction of barrel due to horizontal force F_0	mm
Δ_{x1}	Displacement in horizontal direction of upper end of leg 1 due to horizontal force F_0	mm
Δ_{x3}	Displacement in horizontal direction of upper end of leg 2 due to horizontal force F_0	mm
Δ_{y1}	Displacement in vertical direction of leg 1 due to horizontal force F_0	mm
θ	Local angle of inclination at the root portion where the leg is attached to the barrel due to weight during operation	rad
θ_1	Angle of inclination of leg 1 due to horizontal force F_0	rad
θ_1'	Local angle of inclination at the root portion where leg 1 is attached to the barrel due to horizontal force F_0	rad
θ_3	Angle of inclination of leg 2 due to horizontal force F_0	rad
θ_0	Angle of inclination of central axis of barrel due to horizontal force F_0	rad

Definitions of symbols (symbols used in the calculation formulas of 4-leg-support vertical cylindrical container)
(Cont'd)

Symbol	Definition of symbol	Units
$\sigma_{11} \sim \sigma_{14}$	Combined primary stress in barrel when seismic force acts in the Z-direction	kgf/mm^2
$\sigma_{15} \sim \sigma_{16}$	Combined primary stress in barrel when seismic force acts in the X-direction	kgf/mm^2
$\sigma_{b1} \sim \sigma_{b3}$	Tensile stress generated in anchor bolts due to earthquake in the Z-direction	kgf/mm^2
σ_{b4}, σ_{b5}	Tensile stress generated in anchor bolts due to earthquake in the X-direction	kgf/mm^2
σ_{s1}, σ_{s2}	Compressive/flexural stress of leg due to weight in operation	kgf/mm^2
$\sigma_{s3} \sim \sigma_{s5}$	Compressive/flexural stress of leg due to earthquake in the Z-direction	kgf/mm^2
$\sigma_{s6} \sim \sigma_{s8}$	Compressive/flexural stress of leg due to earthquake in the X-direction	kgf/mm^2
σ_{sc}	Sum of compressive stresses in leg	kgf/mm^2
σ_{sr}	Sum of compressive-side flexural stresses around radial axis of the leg	kgf/mm^2
σ_{st}	Sum of compressive-side flexural stresses around axis perpendicular to the radial direction of the leg	kgf/mm^2
σ_{sx}	Combined stress of leg in the case when seismic force acts in the X-direction	kgf/mm^2
σ_{sz1}, σ_{sz2}	Combined stress of leg in the case when seismic force acts in the Z-direction	kgf/mm^2
$\sigma_{0\phi}$	Primary general membrane stress in circumferential direction of barrel	kgf/mm^2
σ_{0x}	Primary general membrane stress in axial direction of barrel	kgf/mm^2
$\sigma_{\phi 1}$, σ_{x1}	Stresses in circumferential direction and axial direction of barrel due to internal pressure or static water head	kgf/mm^2
σ_{x2}	Axial stress of barrel due to weight in operation	kgf/mm^2
$\sigma_{\phi 3}$, σ_{x3}	Circumferential and axial stress in barrel due to vertical moment generated by weight in operation	kgf/mm^2
$\sigma_{\phi 4}$, σ_{x4}	Circumferential and axial stresses in barrel due to radial load generated by weight in operation	kgf/mm^2
σ_{x5}	Axial stress in barrel due to tipping moment when seismic force acts	kgf/mm^2
$\sigma_{\phi 6}$, σ_{x6}	Circumferential and axial stresses due to radial load when seismic force acts in the Z-direction	kgf/mm^2
$\sigma_{\phi 7}$, σ_{x7}	Circumferential and axial stresses of barrel due to vertical moment when seismic force acts in the Z-direction	kgf/mm^2
$\sigma_{\phi 8}$, σ_{x8}	Circumferential and axial stresses due to circumferential moment when seismic force acts in the Z-direction	kgf/mm^2
$\sigma_{\phi 9}$, σ_{x9}	Circumferential and axial stresses due to radial load when seismic force acts in the X-direction	kgf/mm^2
$\sigma_{\phi 10}$, σ_{x10}	Circumferential and axial stress due to vertical moment when seismic force acts in the X-direction	kgf/mm^2
$\sigma_{\phi 11}$, σ_{x11}	Circumferential and axial stresses due to circumferential moment when seismc force acts in the X-direction	kgf/mm^2
σ_{xx1}, σ_{xx2}	Sum of axial primary stresses in barrel when seismic force acts in the X-direction	kgf/mm^2

Definitions of symbols (symbols used in the calculation formulas of 4-leg-support vertical cylindrical container)

(Cont'd)

Symbol	Definition of symbol	Units
$\sigma_{xz1} \sim \sigma_{xz4}$	Sum of axial primary stresses in barrel when seismic force acts in the Z-direction	kgf/mm^2
$\sigma_{\phi x1}$, $\sigma_{\phi x2}$	Sum of circumferential primary stresses in barrel when seismic force acts in the X-direction	kgf/mm^2
$\sigma_{\phi z1}$, $\sigma_{\phi z1}$	Sum of circumferential primary stresses in barrel when seismic force acts in the Z-direction	kgf/mm^2
τ_3	Shear stress due to torsional moment generated at the root portion where the leg is attached to the barrel due to seismic force in the Z-direction	kgf/mm^2
τ_6	Shear stress due to torsional moment generated at the root portion where the leg is attached to the barrel due to earthquake in the X-direction	kgf/mm^2
$\tau_{b1} \sim \tau_{b3}$	Shear stress generated in anchor bolts due to earthquake in the Z-direction	kgf/mm^2
τ_{b4}, τ_{b5}	Shear stress generated in anchor bolts due to earthquake in the X-direction	kgf/mm^2
τ_{c1}	Circumferential shear stress generated at the root portion where the leg is attached to the barrel due to earthquake in the Z-direction	kgf/mm^2
τ_{c4}	Circumferential shear stress generated at the root portion where the leg is attached to the barrel due to earthquake in the X-direction	kgf/mm^2
τ_{l1}	Axial shear stress generated at the root portion where the leg is attached to the barrel due to the weight in operation	kgf/mm^2
τ_{l2}	Axial shear stress generated at the root portion where the leg is attached to the barrel due to earthquake in the Z-direction	kgf/mm^2
τ_{l5}	Axial shear stress generated at the root portion where the leg is attached to the barrel due to earthquake in the X-direction	kgf/mm^2
τ_{s1}	Shear stress of leg due to earthquake in the [illegible]-direction	kgf/mm^2
τ_{s2}, τ_{s3}	Shear stress of leg due to earthquake in the Z-direction	kgf/mm^2
τ_{s4}	Shear stress of leg due to earthquake in the X-direction	kgf/mm^2
λ	Effective slenderness ratio of leg	—
Λ	Limit slenderness ratio of leg	—
ν	Safety factor with respect to buckling	—

(ii) Natural period

For each leg and barrel, by forming the equations of equilibrium conditions for loads, moments and deformations, the natural period is derived as follows.

{1} When both legs 1 and 2 are fixed (see Figure 6.6.3-11)
From the balance of horizontal forces

$$2P_1 + 2Q = F_0 \tag{6.6.3-81}$$

From the balance of overturning moment

$$2M_1 - 2M_3 + 2R_1 r_m = F_0(l_g - l) \tag{6.6.3-82}$$

where

$$r_m = (D_i + t)/2 \tag{6.6.3-83}$$

For leg 1, horizontal displacement, angle of inclination, and vertical displacement are related to each other as follows.

$$\Delta_{xl} = \frac{P_1 l^3}{3E_s I_{sc}} + \frac{P_1 l}{G_s A_{sr}} + \frac{(M_1 - R_1 u) l^2}{2E_s I_{sr}} \tag{6.6.3-84}$$

where,

$$u = \frac{l_c}{2} - r_m \tag{6.6.3-85}$$

$$\theta_1 = \frac{(M_1 - R_1 u) l}{E_s I_{sr}} + \frac{P_1 l^2}{2E_s I_{sr}} \tag{6.6.3-86}$$

$$\Delta_{yl} = \frac{R_1 l}{A_s E_s} \tag{6.6.3-87}$$

The radial local displacement and local angle of inclination of the barrel are as follows:

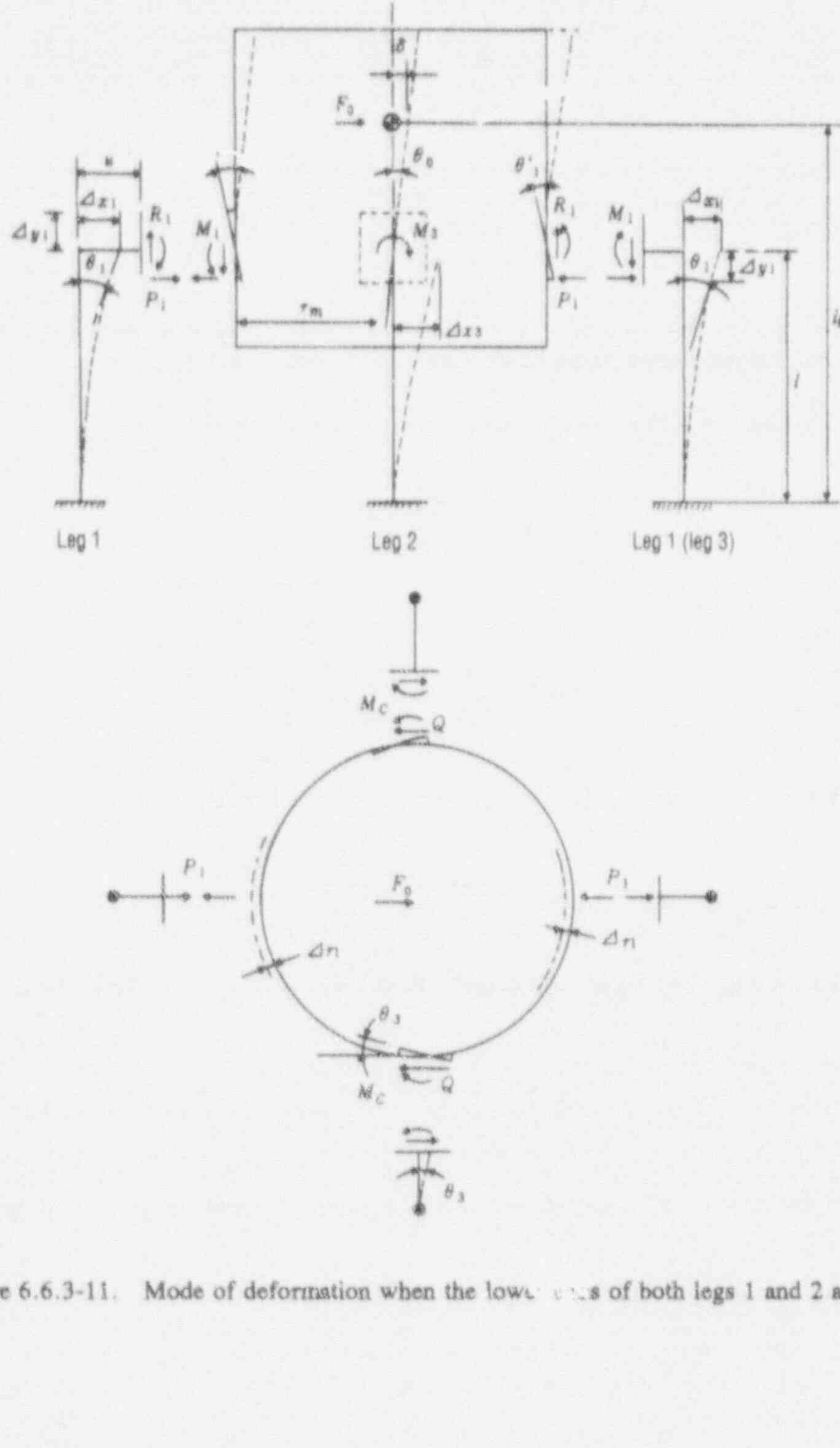


Figure 6.6.3-11. Mode of deformation when the lowe[illegible]s of both legs 1 and 2 are fixed.

$$\Delta_{rl} = \frac{K_r P_1}{r_m E} \qquad (6.6.3\text{-}88)$$

$$\theta_1' = \frac{k_l M_1}{r_m^3 \beta^2 E} \qquad (6.6.3\text{-}89)$$

where K_r and K_l are constants for the local displacement due to radial load of the barrel and the local angle of inclination due to longitudinal flexural moment according to Reference [6.6.3-4].

For leg 2, the angle of inclination and horizontal displacement are as follows:

$$\theta_0 = -\frac{M_3 l}{E_g I_{gr}} + \frac{Q l^2}{2E_g I_{gr}} \qquad (6.6.3\text{-}90)$$

$$\Delta_{x3} = \frac{Q l^3}{3E_g I_{gr}} + \frac{Q l}{G_g A_m} - \frac{M_3 l^2}{2E_g I_{gr}} \qquad (6.6.3\text{-}91)$$

From the balance of the angles of inclination of leg 1 and the barrel,

$$\theta_1 + \theta_1' - \theta_0 = 0 \qquad (6.6.3\text{-}92)$$

Since the torsional angle of leg 2 is equal to the local angle of inclination of the barrel, we have

$$\theta_3 = \frac{(Qu - M_c) l}{G_g J_s} = \frac{K_c M_c}{r_m^3 \beta^2 E} \qquad (6.6.3\text{-}93)$$

where K_c is a constant for the load angle of inclination due to the circumferential flexural moment based on Reference [6.6.3-4].

From the balance of horizontal displacement of leg and barrel,

$$\Delta_{xl} + \Delta_{rl} = \Delta_{x3} + u\,\theta_3 \qquad (6.6.3\text{-}94)$$

From the balance in the vertical direction,

$$\Delta_{yl} - u\theta_1 - r_m\theta_0 = 0 \qquad (6.6.3\text{-}95)$$

By substituting formulas (6.6.3-86), (6.6.3-87), and (6.6.3-90) into equation (6.6.3-95), we have

$$\frac{R_1 l}{A_t E_s} - \frac{u(M_1 - R_1 u) l}{E_t J_m} - \frac{u P_1 l^2}{2E_t J_m} + \frac{r_m M_3 l}{E_t J_{sr}} - \frac{r_m Q l^2}{2E_3 J_{sr}} = 0 \tag{6.6.3-96}$$

By substituting formulas (6.6.3-86), (6.6.3-89), and (6.6.3-90) into equation (6.6.3-92), we have

$$\frac{M_3 l}{E_t J_{sr}} - \frac{Q l^2}{2E_t J_{sr}} + \frac{(M_1 - R_1 u) l}{E_t J_m} + \frac{P_1 l^2}{2E_t J_m} + \frac{K_r M_1}{r_m^3 \beta^2 E} = 0 \tag{6.6.3-97}$$

Equation (6.6.3-93) can be rearranged to

$$\frac{u Q l}{G_t J_s} - \frac{M_c l}{G_t J_s} - \frac{K_c M_c}{r_m^3 \beta^2 E} = 0 \tag{6.6.3-98}$$

By substituting formulas (6.6.3-84), (6.6.3-88), (6.6.3-91) and (6.6.3-93) into equation (6.6.3-94), we have

$$\begin{aligned} &\frac{P_1 l^3}{3E_t J_m} + \frac{P_1 l}{G_t A_{sr}} + \frac{(M_1 - R_1 u) l^2}{2E_t J_m} + \frac{K_r P_1}{r_m E} \\ &- \frac{Q l^3}{3E_t J_{sr}} - \frac{Q l}{G_t A_m} + \frac{M_3 l^2}{2E_t J_{sr}} - \frac{u K_c M_c}{r_m^3 \beta^2 E} = 0 \end{aligned} \tag{6.6.3-99}$$

Hence, for the 6 variables P_1, Q, R_1, M_1, M_3 and M_C, there is a group of equations (6.6.3-81), (6.6.3-82), (6.6.3-96) -(6.6.3-99).

Displacement δ of the center of gravity of the barrel and natural period T can be represented by the following equations:

$$\delta = \Delta_{xl} + \Delta_{rl} + (l_g - l)\theta_0 + \frac{(lg - l)^3}{3EI} F_0 + \frac{(lg - l)}{GA_s} F_0 \tag{6.6.3-100}$$

$$K = \frac{F_0}{\delta} \tag{6.6.3-101}$$

$$T = 2\pi \sqrt{\frac{W_0}{K g}} \tag{6.6.3-102}$$

Here, the sectional properties of the barrel can be expressed as follows:

$$I = \frac{\pi}{8}(D_i + t)^3 t \tag{6.6.3-103}$$

$$A_s = \frac{2}{3}\pi(D_i + t)t \tag{6.6.3-104}$$

{2} When both legs 1 and 2 are simply supported (see Figure 6.6.3-12)
Just as in the case of {1}, the following equations can be obtained:

$$2P_1 + 2Q = F_0 \tag{6.6.3-105}$$

$$2M_1 - 2M_3 + 2R_1 r_m = F_0(l_g - l) \tag{6.6.3-106}$$

$$\Delta_{xl} = \frac{P_1 l^3}{3E_f J_{sl}} + \frac{P_1 l}{G_f A_{sl}} + l\theta_1 \tag{6.6.3-107}$$

$$P_1 l + M_1 = R_1 u \tag{6.6.3-108}$$

$$\Delta_{yl} = \frac{R_1 l}{A_s E_s} \tag{6.6.3-109}$$

$$\Delta_{rl} = \frac{K_r P_1}{r_m E} \tag{6.6.3-110}$$

$$\theta_1' = \frac{K_t M_1}{r_m^3 \beta^2 E} \tag{6.6.3-111}$$

$$M_3 = Ql \tag{6.6.3-112}$$

$$\Delta_{x3} = \frac{Q_1 l^3}{3E_f J_{sl}} + \frac{Ql}{G_f A_{sl}} + l\theta_0 \tag{6.6.3-113}$$

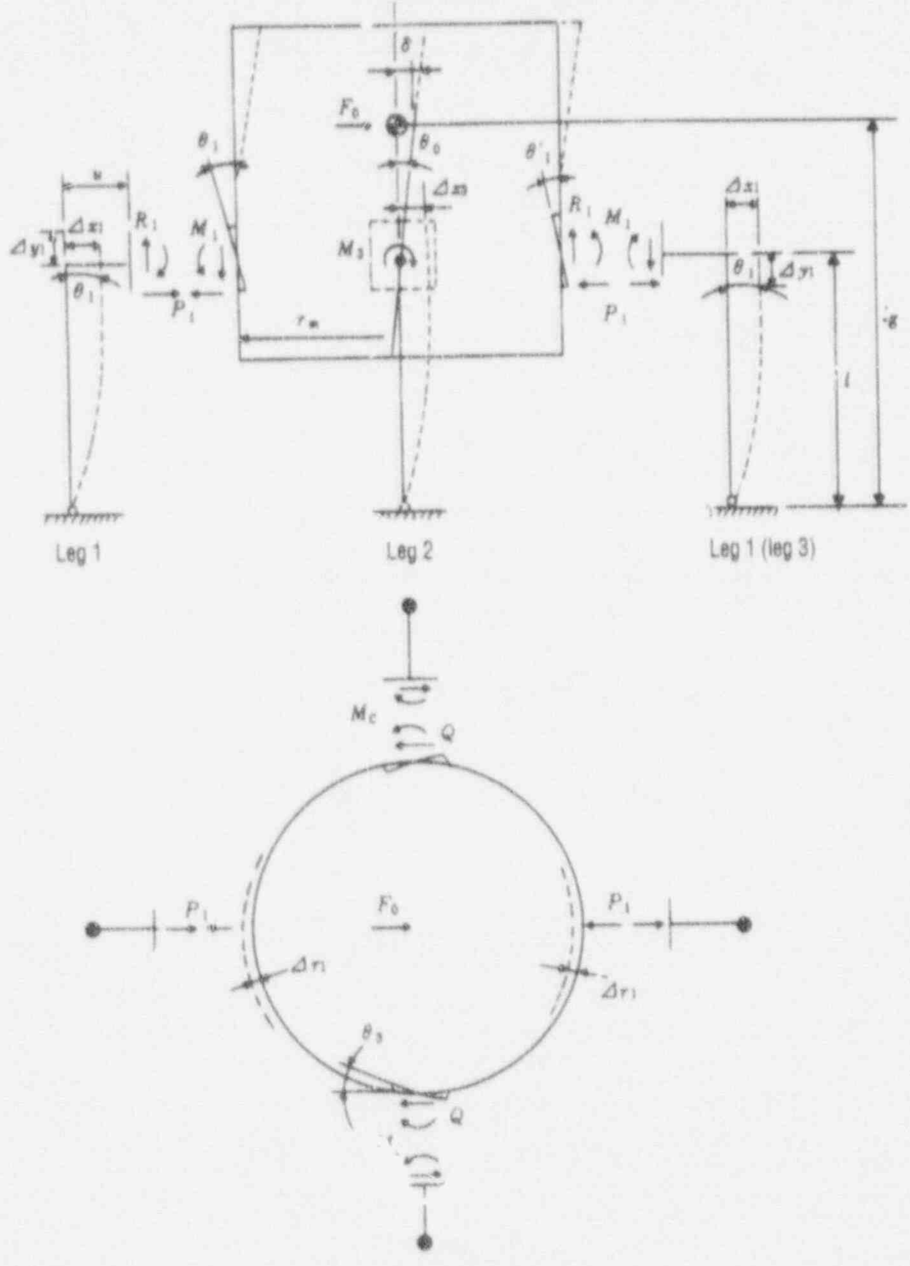


Figure 6.6.3-12. Deformation mode when the lower ends of both legs 1 and 2 are simply supported.

$$\theta_1 + \theta_1' - \theta_0 = 0 \tag{6.6.3-114}$$

$$\theta_3 = \frac{(Qu - M_c)l}{G_f J_s} = \frac{K_t M_c}{r_m^3 \beta^2 E} \tag{6.6.3-115}$$

$$\Delta_{xl} + \Delta_{rl} = \Delta_{x3} + u\theta_3 \tag{6.6.3-116}$$

$$\Delta_{yl} - u\theta_1 - r_m\theta_0 = 0 \tag{6.6.3-117}$$

Then, the natural period can be derived in the same way as in {1}.

{3} When leg 1 is fixed and leg 2 is simply supported (see Figure 6.6.3-13)
In this case, we have the following formulas:

$$2P_1 + 2Q = F_0 \tag{6.6.3-118}$$

$$2M_1 - 2M_3 + 2R_1 r_m = F_0(l_g - l) \tag{6.6.3-119}$$

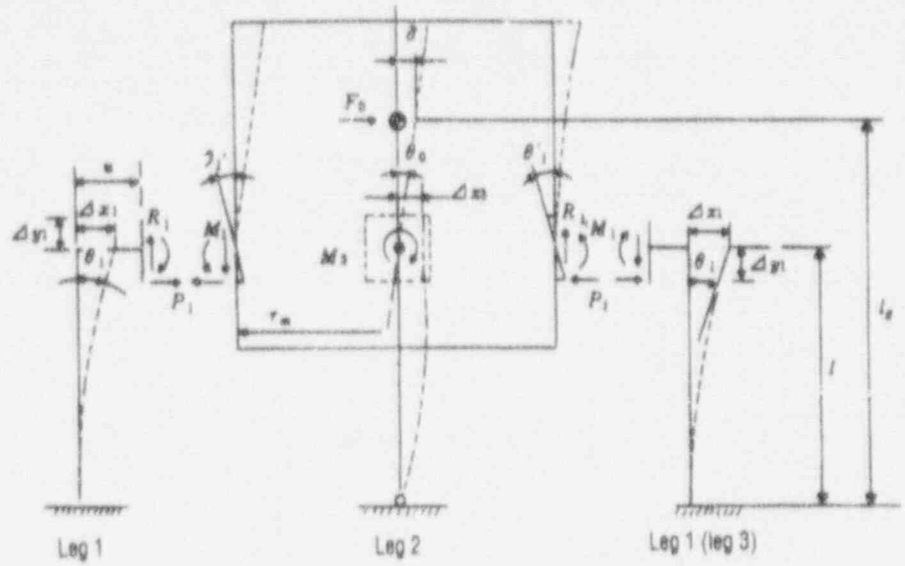
$$\Delta_{xl} = \frac{P_1 l^3}{3E_f I_{sx}} + \frac{P_1 l}{G_f A_w} + \frac{(M_1 - R_1 u) l^2}{2E_f I_{sx}} \tag{6.6.3-120}$$

$$\theta_1 = \frac{(M_1 - R_1 u) l}{E_f I_{sx}} + \frac{P_1 l^2}{2E_f I_x} \tag{6.6.3-121}$$

$$\Delta_{yl} = \frac{R_1 l}{A_s E_s} \tag{6.6.3-122}$$

$$\Delta_{rl} = \frac{K_r P_1}{r_m E} \tag{6.6.3-123}$$

$$\theta_1' = \frac{K_t M_1}{r_m^3 \beta^2 E} \tag{6.6.3-124}$$



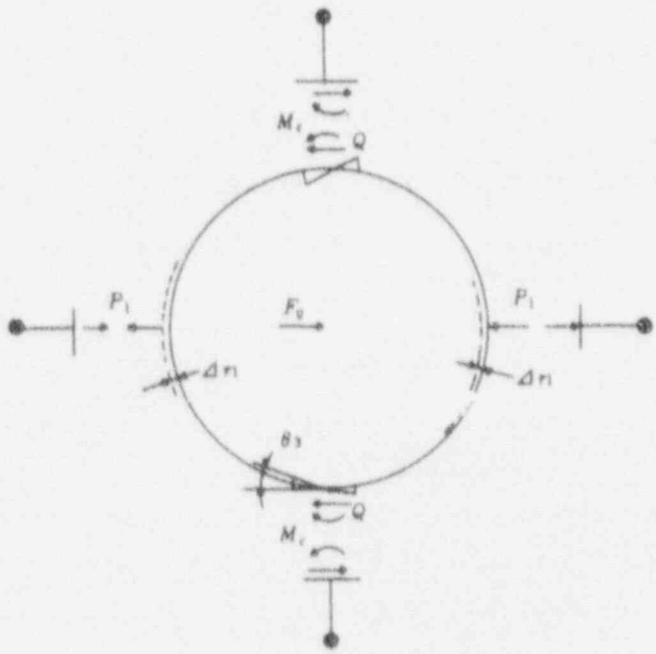


Figure 6.6.3-13. Deformation mode when the lower end of leg 1 is fixed and the lower end of leg 2 is simply supported.

$$M_3 = Q\ell \tag{6.6.3-125}$$

$$\Delta_{x3} = \frac{Q\ell^3}{3E_sJ_{sr}} + \frac{Q\ell}{G_sA_{sr}} + l\theta_0 \tag{6.6.3-126}$$

$$\theta_1 + \theta_1' - \theta_0 = 0 \tag{6.6.3-127}$$

$$\theta_3 = \frac{(Qu - M_c)l}{G_sJ_s} = \frac{K_cM_c}{r_m^3\beta^2 E} \tag{6.6.3-128}$$

$$\Delta_{xl} + \Delta_{rl} = \Delta_{x3} + u\theta_3 \tag{6.6.3-129}$$

$$\Delta_{yl} - u\theta_1 - r_m\theta_0 = 0 \tag{6.6.3-130}$$

Then, the natural period can be derived in the same way as in {1}.

{4} When leg 1 is simply supported and leg 2 is fixed (see Figure 6.6.3-14)
In this case, we have the following formulas:

$$2P_1 + 2Q = F_0 \tag{6.6.3-131}$$

$$2M_1 - 2M_3 + 2R_1r_m = F_0(l_g - l) \tag{6.6.3-132}$$

$$\Delta_{xl} = \frac{P_1l^3}{3E_sJ_{sl}} + \frac{P_1l}{G_sA_{sr}} + l\theta_1 \tag{6.6.3-133}$$

$$P_1l + M_1 = R_1u \tag{6.6.3-134}$$

$$\Delta_{yl} = \frac{R_1l}{A_sE_s} \tag{6.6.3-135}$$

$$\Delta_{rl} = \frac{K_rP_1}{r_mE} \tag{6.6.3-136}$$

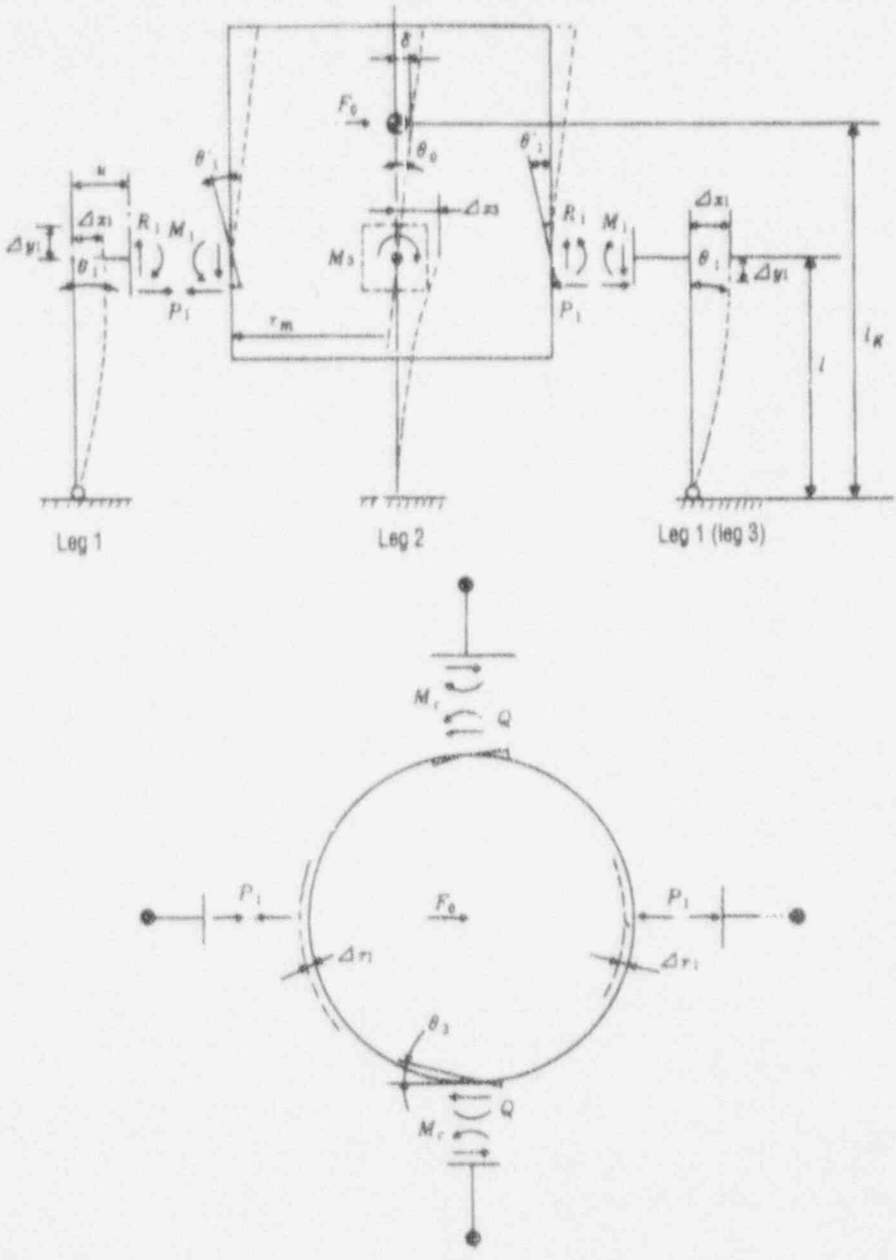


Figure 6.6.3-14. Deformation mode when the lower end of leg 1 is simply supported and the lower end of leg 2 is fixed.

$$\theta_1' = \frac{K_l M_1}{r_m^3 \beta^2 E} \tag{6.6.3-137}$$

$$\theta_0 = -\frac{M_3 l}{E_s I_{sr}} + \frac{Q \cdot l^2}{2E_s I_{sr}} \tag{6.6.3-138}$$

$$\Delta_{x3} = \frac{Q \cdot l^3}{3E_s I_{sr}} + \frac{Q \cdot l}{G_s A_{st}} - \frac{M_3 l^2}{2E_s I_{sr}} \tag{6.6.3-139}$$

$$\theta_1 + \theta_1' - \theta_0 = 0 \tag{6.6.3-140}$$

$$\theta_3 = \frac{(Qu - M_c) l}{G_s J_s} = \frac{K_c M_c}{r_m^3 \beta^2 E} \tag{6.6.3-141}$$

$$\Delta_{xl} + \Delta_{rl} = \Delta_{x3} + u\theta_3 \tag{6.6.3-142}$$

$$\Delta_{yl} - u\theta_1 - r_m \theta_0 = 0 \tag{6.6.3-143}$$

Then, the natural period can be derived in the same way as in {1}.

(b) Calculation method of stress

(i) Stresses of barrel

{1} Stresses due to static water head or internal pressure
In the case of static water head,

$$\sigma_{\phi 1} = \frac{\rho' H D_i}{2t} \tag{6.6.3-144}$$

$$\sigma_{xl} = 0 \tag{6.6.3-145}$$

In the case of internal pressure,

$$\sigma_{\phi 1} = \frac{P_r(D_i + 1.2t)}{200t} \tag{6.6.3-146}$$

$$\sigma_{x1} = \frac{P_r(D_i + 1.2t)}{400t} \tag{6.6.3-147}$$

(2) Stress due to weight during operation

$$\sigma_{x2} = \frac{W_0}{\pi (D_i + t)t} \tag{6.6.3-148}$$

(3) Stress at root portion where the leg is attached to the barrel due to weight in operation
When the leg lower end is fixed (see Figure 6.6.3-15)

$$R = \frac{W_0}{4} \tag{6.6.3-149}$$

Since the displacement of the leg in the radial direction is equal to the local displacement of the barrel in the radial direction,

$$\Delta_r = \frac{-P \cdot l^3}{3E_s I_{st}} + \frac{-P \cdot l}{G_s A_{sr}} + \frac{(Ru - M_l)l^2}{2E_s I_{st}} = \frac{K_r P}{r_m E} \tag{6.6.3-150}$$

In addition, the angle of inclination of the upper end of the leg is equal to the local angle of inclination of the barrel,

$$\theta = \frac{(Ru - M_l)l}{E_s I_{st}} - \frac{P \cdot l^2}{2E_s I_{st}} = \frac{K_l M_l}{r_m^3 \beta^2 E} \tag{6.6.3-151}$$

From the set of equations (6.6.3-149) through (6.6.3-151), we have

$$M_l = \frac{\left(\frac{l^3}{12E_s I_{st}} + \frac{l}{G_s A_{sr}} + \frac{K_r}{r_m E}\right)\frac{W_0 \mu l}{4E_s I_{st}}}{\left\{\left(\frac{l^3}{3E_s I_{st}} + \frac{l}{G_s A_{sr}} + \frac{K_r}{r_m E}\right)\left(\frac{l}{E_s I_{st}} + \frac{K_l}{r_m^3 \beta^2 E}\right) - \left(\frac{l^2}{2E_s I_{st}}\right)^2\right\}} \tag{6.6.3-152}$$

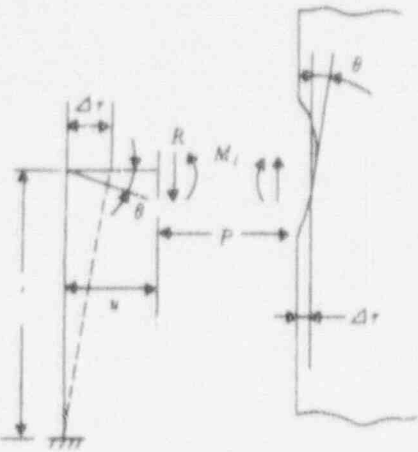


Figure 6.6.3-15. Deformation of leg and barrel due to weight in operation in the case when the lower end of the leg is fixed.

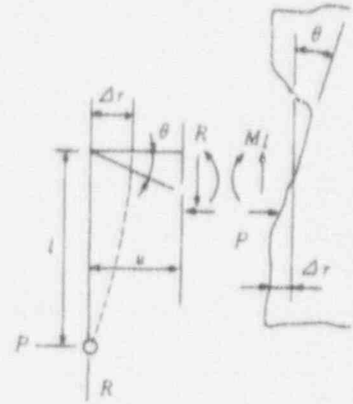


Figure 6.6.3-16. Deformations of leg and barrel due to weight in operation when the lower end of leg is simply supported.

$$P = \frac{\dfrac{(W_0/4)u - M_l}{2E_s I_{st}} l^2}{\dfrac{l^3}{3E_s I_{st}} + \dfrac{l}{G_s A_{sr}} + \dfrac{K_r}{r_m E}} \tag{6.6.3-153}$$

When the lower end of the leg is simply supported (see Figure 6.6.3-16), instead of equations (6.6.3-150) and (6.6.3-151), the following equations are obtained:

$$\Delta_r = \frac{-P \cdot l^3}{3E_s I_{st}} + \frac{-P \cdot l}{G_s A_{sr}} + \theta \cdot l = \frac{K_r P}{r_m E} \tag{6.6.3-154}$$

$$P \cdot l + M_l = Ru \tag{6.6.3-155}$$

$$\theta = \frac{K_l M_l}{r_m^3 \beta^2 E} \tag{6.6.3-156}$$

For the above set of equations, we have

$$P = \frac{\dfrac{W_0}{4} ul}{l^2 + \dfrac{r_m^3 \beta^2 E}{K_l}\left(\dfrac{l^3}{3E_s I_{st}} + \dfrac{l}{G_s A_{sr}} + \dfrac{K_r}{r_m E}\right)} \tag{6.6.3-157}$$

$$M_l = \frac{W_0}{4} \cdot u - P \cdot l \tag{6.6.3-158}$$

For the local stresses in the barrel generated by vertical bending moment M_l, shell parameter γ and attachment parameters β are used and the values are derived from the table in Reference [6.6.3-2] (marked with asterisk in the following equations):

$$\sigma_{\phi 3} = \left(\frac{N_\phi}{M_l/(r_m^2\beta)}\right)^* \cdot \left(\frac{M_l}{r_m^2 t\beta}\right) C_l^* \tag{6.6.3-159}$$

$$\sigma_{x3} = \left(\frac{N_x}{M_l/(r_m^2\beta)}\right)^* \cdot \left(\frac{M_l}{r_m^2 t\beta}\right) C_l^* \tag{6.6.3-160}$$

$$r_m = (D_i + t)/2 \tag{6.6.3-161}$$

$$\gamma = r_m/t \tag{6.6.3-162}$$

$$\beta_1 = C_1/r_m \tag{6.6.3-163}$$

$$\beta_2 = C_2/r_m \tag{6.6.3-164}$$

$$\beta = \sqrt[3]{\beta_1 \beta_2^2} \tag{6.6.3-165}$$

The local stresses of barrel generated by radial load P are as follows:

$$\sigma_{\phi 4} = \left(\frac{N_\phi}{P/r_m}\right)^* \cdot \left(\frac{P}{r_m t}\right) \tag{6.6.3-166}$$

$$\sigma_{x4} = \left(\frac{N_x}{P/r_m}\right)^* \cdot \left(\frac{P}{r_m t}\right) \tag{6.6.3-167}$$

β is defined as follows:

When $\beta_1/\beta_2 \geq 1$

$$\beta = \left(1 - \frac{1}{3}(\beta_1/\beta_2 - 1)(1 - K_1^*)\right)\sqrt{\beta_1\beta_2} \qquad (6.6.3\text{-}168)$$

When $\beta_1/\beta_2 < 1$

$$\beta = \left(1 - \frac{4}{3}(1 - \beta_1/\beta_2)(1 - K_2^*)\right)\sqrt{\beta_1\beta_2} \qquad (6.6.3\text{-}169)$$

The shear stress due to reaction force R is as follows:

$$\tau_{sl} = \frac{R}{4C_2 t} \qquad (6.6.3\text{-}170)$$

{4} Flexural stress of barrel due to horizontal earthquake

$$\sigma_{x5} = \frac{C_H W_0 (l_g - l)(D_i + 2t)}{2I} \qquad (6.6.3\text{-}171)$$

{5} Stress at the root portion where the leg is attached to the barrel caused by earthquake in the Z-direction

The value obtained by replacing unit load F_0 by $C_H W_0$ in calculation of the characteristic period in (a) is used.

Just as in {3}, the local stresses generated in the barrel due to radial load P_1 are as follows:

$$\sigma_{\phi 6} = \left(\frac{N_\phi}{P_1/r_m}\right)^* \cdot \left(\frac{P_1}{r_m t}\right) \qquad (6.6.3\text{-}172)$$

$$\sigma_{x6} = \left(\frac{N_x}{P_1/r_m}\right)^* \cdot \left(\frac{P_1}{r_m t}\right) \qquad (6.6.3\text{-}173)$$

Just as in {3}, the local stresses generated in the barrel due to vertical bending moment M_1 are as follows:

$$\sigma_{\phi 7} = \left(\frac{N_\phi}{M_1/(r_m^2\beta)}\right)^* \cdot \left(\frac{M_1}{r_m^2 t\beta}\right) C_l^* \qquad (6.6.3\text{-}174)$$

$$\sigma_{x7} = \left(\frac{N_x}{M_1/(r_m^2\beta)}\right)^* \cdot \left(\frac{M_1}{r_m^2 t\beta}\right) C_l^* \qquad (6.6.3\text{-}175)$$

The local stresses generated by circumferential moment M_c are as follows:

$$\sigma_{\phi 8} = \left(\frac{N_\phi}{M_c/(r_m^2\beta)}\right)^* \cdot \left(\frac{M_c}{r_m^2 t\beta}\right) C_c^* \qquad (6.6.3\text{-}176)$$

$$\sigma_{x8} = \left(\frac{N_x}{M_c/(r_m^2\beta)}\right)^* \cdot \left(\frac{M_c}{r_m^2 t\beta}\right) C_c^* \qquad (6.6.3\text{-}177)$$

where β is defined as follows:

$$\beta = \sqrt[3]{\beta_1^2\beta_2} \qquad (6.6.3\text{-}178)$$

The shear stress due to circumferential shear force Q is as follows:

$$\tau_{c1} = \frac{Q}{4C_1 t} \qquad (6.6.3\text{-}179)$$

The shear stress due to vertical shear force R_1 is as follows:

$$\tau_{l2} = \frac{R_1}{4C_2 t} \qquad (6.6.3\text{-}180)$$

The local shear stress generated in the barred due to torsional moment M_3 is as follows:

$$\tau_3 = \frac{M_3}{2\pi C_1^2 t} \qquad (6.6.3\text{-}181)$$

In this formula, when $C_1 > C_2$, C_1 is replaced by C_2.

{6} Stress at the root portion where the leg is attached to the barrel due to earthquake in the X-direction
The values obtained by multiplying the right-hand sides of equations (6.6.3-172) through (6.6.3-177) by $1/\sqrt{2}$ are used. They are $\sigma_{\phi 9}$, σ_{x9} in the case of radial load, $\sigma_{\phi 10}$, σ_{x10} in the case of vertical bending moment, and $\sigma_{\phi 11}$, σ_{x11} in the case of circumferential flexural moment.

Also, the values obtained by multiplying the right-hand side of equations (6.6.3-179) through (6.6.3-181) by $1/\sqrt{2}$ are used. It is τ_{c4} in the case of circumferential shear force, τ_{l5} in the case of vertical shear force, and τ_6 in the case of torsional moment.

{7} Combination of stresses
The stresses generated at the root portion where the leg is attached to the barrel as calculated from {1}–{6} are combined as follows:

{a} Primary general membrane stress

$$\sigma_0 = \max(\sigma_{0\phi}, \sigma_{0x}) \qquad (6.6.3\text{-}182)$$

where,

$$\sigma_{0\phi} = \sigma_{\phi 1} \qquad (6.6.3\text{-}183)$$

$$\sigma_{0x} = \sigma_{x1} + \sigma_{x2} + \sigma_{x5} \qquad (6.6.3\text{-}184)$$

{b} Combination when earthquake acts in the Z-direction

<u>Primary stress</u>

A. Root portion of leg 1 (see Figure 6.6.3-17)

For the first point of evaluation, we have

$$\sigma_{\phi z1} = \sigma_{\phi 1} + \sigma_{\phi 3} + \sigma_{\phi 4} + \sigma_{\phi 6} + \sigma_{\phi 7} \qquad (6.6.3\text{-}185)$$

$$\sigma_{xz1} = \sigma_{x1} + \sigma_{x2} + \sigma_{x3} + \sigma_{x4} + \sigma_{x5} + \sigma_{x6} + \sigma_{x7} \qquad (6.6.3\text{-}186)$$

$$\sigma_{11} = \frac{1}{2}\left\{\sigma_{\phi z1} + \sigma_{xz1} + \sqrt{(\sigma_{\phi z1} - \sigma_{xz1})^2}\right\} \qquad (6.6.3\text{-}187)$$

For the second point of evaluation, we have

$$\sigma_{\phi z2} = \sigma_{\phi 1} + \sigma_{\phi 4} + \sigma_{\phi 6} \qquad (6.6.3\text{-}188)$$

$$\sigma_{xz2} = \sigma_{x1} + \sigma_{x2} + \sigma_{x4} + \sigma_{x5} + \sigma_{x6} \qquad (6.6.3\text{-}189)$$

$$\sigma_{12} = \frac{1}{2}\left\{\sigma_{\phi z2} + \sigma_{xz2} + \sqrt{(\sigma_{\phi z2} - \sigma_{xz2})^2 + 4(\tau_{l1} + \tau_{l2})^2}\right\} \qquad (6.6.3\text{-}190)$$

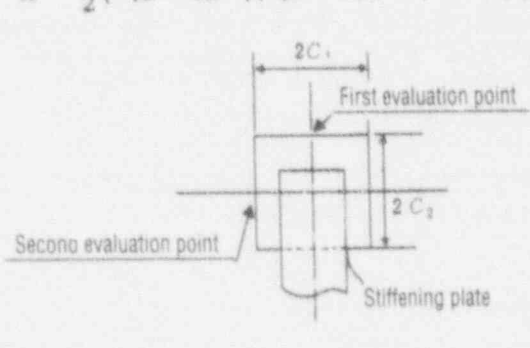


Figure 6.6.3-17. Evaluation points on barrel.

B. Root portion of leg 2

For the first point of evaluation, we have

$$\sigma_{\phi x3} = \sigma_{\phi 1} + \sigma_{\phi 3} + \sigma_{\phi 4} \qquad (6.6.3\text{-}191)$$

$$\sigma_{xx3} = \sigma_{x1} + \sigma_{x2} + \sigma_{x3} + \sigma_{x4} \qquad (6.6.3\text{-}192)$$

$$\sigma_{13} = \frac{1}{2}\left\{\sigma_{\phi x3} + \sigma_{xx3} + \sqrt{\left(\sigma_{\phi x3} - \sigma_{xx3}\right)^2 + 4\left(\tau_{c1} + \tau_3\right)^2}\right\} \qquad (6.6.3\text{-}193)$$

For the second point o[illegible] eval atic[illegible] we have

$$\ldots + \sigma_{\phi 4} + \sigma_{\phi 8} \qquad (6.6.3\text{-}[illegible])$$

$$\sigma_{xx4} \ldots \sigma_{x2} + \sigma_{x4} + \sigma_{x8} \qquad (6.6.3\text{-}195)$$

$$\sigma_{14} = \frac{1}{2}\left\{\sigma_{\phi x4} + \sigma_{xx4} + \sqrt{\left(\sigma_{\phi x4} - \sigma_{xx1}\right)^2 + 4\left(\tau_{l1} + \tau_3\right)^2}\right\} \qquad (6.6.3\text{-}196)$$

{c} Combined primary stress when seismic force acts in the X-directio[illegible]
For the first point of evaluation, we have

$$\sigma_{\phi x1} = \sigma_{\phi 1} + \sigma_{\phi 3} + \sigma_{\phi 4} + \sigma_{\phi 9} + \sigma_{\phi 10} \qquad (6.6.3\text{-}197)$$

$$\sigma_{xx1} = \sigma_{x1} + \sigma_{x2} + \sigma_{x3} + \sigma_{x4} + \sigma_{x5} + \sigma_{x9} + \sigma_{x10} \qquad (6.6.3\text{-}198)$$

$$\sigma_{15} = \frac{1}{2}\left\{\sigma_{\phi x1} + \sigma_{xx1} + \sqrt{\left(\sigma_{\phi x1} - \sigma_{xx1}\right)^2 + 4\left(\tau_{c4} + \tau_6\right)^2}\right\} \qquad (6.6.3\text{-}199)$$

For the second point of evi uation,

$$\sigma_{\phi x2} = \sigma_{\phi 1} + \sigma_{\phi 4} + \sigma_{\phi 9} + \sigma_{\phi 11} \qquad (6.6.3\text{-}200)$$

$$\sigma_{xx2} = \sigma_{x1} + \sigma_{x2} + \sigma_{x4} + \sigma_{x5} + \sigma_{x9} + \sigma_{x11} \qquad (6.6.3\text{-}201)$$

$$\sigma_{16} = \frac{1}{2}\left\{\sigma_{\phi x2} + \sigma_{xx2} + \sqrt{\left(\sigma_{\phi x2} - \sigma_{xx2}\right)^2 + 4\left(\tau_{l1} + \tau_{15} + \tau_6\right)^2}\right\} \qquad (6.6.3\text{-}202)$$

(ii) Stresses in legs

The calculation is performed for the leg with larger load.

{1} Stresses due to weight during operation

$$\sigma_{s1} = \frac{R}{A_s} \tag{6.6.3-203}$$

$$\sigma_{s2} = \frac{\max\left(\left|R u - M_l - P \cdot l\right|, \left|R u - M_l\right|\right)}{Z_{st}} \tag{6.6.3-204}$$

When the lower end of the leg is simply supported, we have

$$\sigma_{s2} = \frac{P \cdot l}{Z_{st}} \tag{6.6.3-205}$$

$$\tau_{s1} = \frac{P}{A_{s1}} \tag{6.6.3-206}$$

{2} Stresses due to earthquake in the Z-direction

$$\sigma_{s3} = \frac{R_1}{A_s} \tag{6.6.3-207}$$

$$\sigma_{s4} = \frac{\max\left(\left|R_1 u - M_1 - P_1 l\right|, \left|R_1 u - M_1\right|\right)}{Z_{st}} \tag{6.6.3-208}$$

When the lower end of the leg is simply supported, we have

$$\sigma_{s4} = \frac{P_1 l}{Z_{st}} \tag{6.6.3-209}$$

$$\tau_{s2} = \frac{P_1}{A_{s1}} \tag{6.6.3-210}$$

1

IMAGE EVALUATION
TEST TARGET (MT-3)

1.0 1.1 1.25 1.4 1.6 1.8 2.0 2.2 2.5 2.8 3.2

150mm

6"

1

IMAGE EVALUATION TEST TARGET (MT-3)

1.0 1.1 1.25 1.4 1.6 1.8 2.0 2.2 2.5 2.8 3.2 3.6 4.0

150mm

6"

1

IMAGE EVALUATION TEST TARGET (MT-3)

150mm

6"

1

IMAGE EVALUATION TEST TARGET (MT-3)

150mm

6"

For leg 2, we have

$$\sigma_{s5} = \frac{\max\left(\left|Q\cdot l - M_3\right|, \left|M_3\right|\right)}{Z_{sr}} \tag{6.6.3-211}$$

When the lower end of the leg is simply supported, we have

$$\sigma_{s5} = \frac{Q\cdot l}{Z_{sr}} \tag{6.6.3-212}$$

$$\tau_{s3} = \frac{Q}{A_{s2}} + \frac{Q\cdot u - M_c}{Z_{sp}} \tag{6.6.3-213}$$

{3} Stresses due to earthquake in the X-direction

$$\sigma_{s6} = \frac{R_1}{\sqrt{2}A_s} \tag{6.6.3-214}$$

$$\sigma_{s7} = \frac{\max\left(\left|R_1 u - M_1 - P_1 l\right|, \left|R_1 u - M_1\right|\right)}{\sqrt{2}Z_{st}} \tag{6.6.3-215}$$

$$\sigma_{s8} = \frac{\max\left(\left|Q\cdot l - M_3\right|, \left|M_3\right|\right)}{\sqrt{2}Z_{sr}} \tag{6.6.3-216}$$

When the lower end of the leg is simply supported, σ_{s7}, σ_{s8} are as follows:

$$\sigma_{s7} = \frac{P_1 l}{\sqrt{2}Z_{st}} \tag{6.6.3-217}$$

$$\sigma_{s8} = \frac{Q\cdot l}{\sqrt{2}Z_{sr}} \tag{6.6.3-218}$$

$$\tau_{s4} = \frac{P_1}{\sqrt{2}A_{s1}} + \frac{Q}{\sqrt{2}A_{s2}} + \frac{Q\cdot u - M_c}{\sqrt{2}Z_{sp}} \tag{6.6.3-219}$$

{4} Combination of stresses

The maximum stress in the leg is as follows. When the seismic force acts in the Z-direction,

For leg 1:

$$\sigma_{s1} = \sqrt{(\sigma_{s1} + \sigma_{s2} + \sigma_{s3} + \sigma_{s4})^2 + 3(\tau_{s1} + \tau_{s2})^2} \tag{6.6.3-220}$$

For leg 2:

$$\sigma_{s2} = \sqrt{(\sigma_{s1} + \sigma_{s2} + \sigma_{s5})^2 + 3(\tau_{s1} + \tau_{s3})^2} \tag{6.6.3-221}$$

When the seismic force acts in the X-direction,

$$\sigma_{s3} = \sqrt{(\sigma_{s1} + \sigma_{s2} + \sigma_{s6} + \sigma_{s7} + \sigma_{s8})^2 + 3(\tau_{s1} + \tau_{s4})^2} \tag{6.6.3-222}$$

(iii) Stresses in anchor bolts

Vertical load, horizontal shear force, torsional moment around vertical axis and overturning moment act on the foundation (see Figure 6.6.3-18).

{1} When seismic force acts in the Z-direction

{a} Shear stresses

For the anchor bolts of leg 1

$$\tau_{b1} = \frac{P_1 - P}{nA_b} \tag{6.6.3-223}$$

For the anchor bolts of legs 2 and 4,

$$\tau_{b2} = \frac{\sqrt{Q^2 + P^2}}{nA_b} + \frac{Qu - M_c}{nA_b\sqrt{\left(\frac{a - 2d_1}{2}\right)^2 + \left(\frac{b - 2d_2}{2}\right)^2}} \tag{6.6.3-224}$$

When n = 2 and the bolts are arranged in a row perpendicular to the radial direction, we have

$$\tau_{b2} = \frac{\sqrt{Q^2 + P^2}}{2A_b} + \frac{Qu - M_c}{A_b(b - 2d_2)} \tag{6.6.3-225}$$

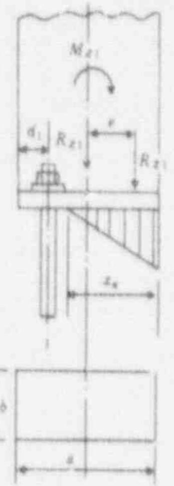


Figure 6.6.3-18. Loads generated by external loads acting on the foundation portion.

When the bolts are arranged in a row in the radial direction, we have

$$\tau_{b2} = \frac{\sqrt{Q^2 + P^2}}{2A_b} + \frac{Qu - M_c}{A_b(a - 2d_1)} \qquad (6.6.3\text{-}226)$$

When n = 1,

$$\tau_{b2} = \frac{\sqrt{Q^2 + P^2}}{A_b} + \frac{16(Qu - M_c)}{\pi d_b^3} \qquad (6.6.3\text{-}227)$$

For the anchor bolts of leg 3

$$\tau_{b3} = \frac{P_1 + P}{nA_b} \qquad (6.6.3\text{-}228)$$

{b} Tensile stress

For leg 1 with fixed lower end, the moment and vertical load acting on the bottom portion of the leg are as follows:

$$M_{zl} = |P_1 l + M_1 - R_1 u - (P l + M_l - R u)| \qquad (6.6.3\text{-}229)$$

$$R_{zl} = R - R_1 \qquad (6.6.3\text{-}230)$$

Suppose the ratio of moment to compressive load is

$$e = M_{z1}/R_{z1} \tag{6.6.3-231}$$

When R_{z1} is negative or when

$$e > \frac{a}{6} + \frac{d_1}{3} \tag{6.6.3-232}$$

tensile form F_b is generated in the anchor bolt. This tensile force can be derived as follows.

As position X_n of the neutral axis is derived from

$$X_n^3 + 3\left(e - \frac{a}{2}\right)X_n^2 - \frac{6sA_b n_1}{b}\left(e + \frac{a}{2} - d_1\right)(a - d_1 - X_n) = 0 \tag{6.6.3-233}$$

the tensile force generated in the anchor bolt becomes

$$F_b = \frac{R_{z1}\left(e - \frac{a}{2} + \frac{X_n}{3}\right)}{a - d_1 - \frac{X_n}{3}} \tag{6.6.3-234}$$

Hence, the tensile force generated in the anchor bolt can be represented by the following formula:

$$\sigma_{b1} = \frac{F_b}{n_1 A_b} \tag{6.6.3-235}$$

When the lower end of the leg is simply supported in the radial direction, no moment is generated. Hence, when vertical load R_{z1} is negative, a tensile stress is generated in the anchor bolt.

$$F_b = -R_{z1} \tag{6.6.3-236}$$

$$\sigma_{b1} = \frac{F_b}{nA_b} \tag{6.6.3-237}$$

For the anchor bolt at leg 2, when the lower end of the leg is fixed,

$$M_{z2} = \sqrt{(Q \cdot l - M_3)^2 + (P \cdot l + M_l - Ru)^2} \tag{6.6.3-238}$$

$$R_{z2} = R \tag{6.6.3-239}$$

are used to replace M_{z1} and R_{z1}, respectively, and the obtained stress of the anchor bolt is taken as σ_{b2}.

However, when the tensile stress of anchor bolt obtained by replacing a, b, d_1, and n_1 with b, a, d_2, and n_2, respectively, is greater than σ_{b2}, the value is taken as σ_{b2}. When the lower end of the leg is simply supported in the direction perpendicular to the radial direction,

$$M_{z2} = |P \cdot l + M_l - R \cdot u| \qquad (6.6.3\text{-}240)$$

$$R_{z2} = R \qquad (6.6.3\text{-}241)$$

are used to replace M_{z1} and R_{z1}, respectively, and the stress of the anchor bolt obtained using equations (6.6.3-231) through (6.6.3-235) is taken as σ_{b2}. When the lower end of the leg is simply supported in the radial direction,

$$M_{z2} = |Q \cdot l - M_3| \qquad (6.6.3\text{-}242)$$

$$R_{z2} = R \qquad (6.6.3\text{-}243)$$

are used to replace M_{z1} and R_{z1}, respectively, and the stress of the anchor bolt obtained by using equations (6.6.3-231) through (6.6.3-235) is taken as σ_{b2}. In addition, a, b, d_1, and n_1 are replaced by b, a, d_2, and n_2, respectively. When the lower end of the leg is simply supported in the radial direction and the direction perpendicular to the radial direction, only compressive load takes place; hence, no tensile stress is generated in the anchor bolt. For the anchor bolt on leg 3, when the lower end of the leg is fixed,

$$M_{z3} = |P_1 \cdot l + M_1 - R_1 u + (P \cdot l + M_l - R \cdot u)| \qquad (6.6.3\text{-}244)$$

$$R_{z3} = R + R_1 \qquad (6.6.3\text{-}245)$$

are used to replace M_{z1} and R_{z1}, respectively, and the stresses of the anchor bolt obtained using equations (6.6.3-231) through (6.6.3-235) is taken as σ_{b3}. In the case when the lower end of the leg is simply supported in the radial direction, there is only compressive load; hence, no tensile stress is generated in the anchor bolt.

(2) When seismic force acts in the X-direction (see Figure 6.6.3-19)

(a) Shear stress

For the anchor bolts on legs 1 and 4, we have

$$\tau_{b4} = \frac{\sqrt{\left(\frac{P_1}{\sqrt{2}} - P\right)^2 + \left(\frac{Q}{\sqrt{2}}\right)^2}}{nA_b} + \frac{Qu - M_c}{\sqrt{2}\, nA_b \sqrt{\left(\frac{a-2d_1}{2}\right)^2 + \left(\frac{b-2d_2}{2}\right)^2}} \qquad (6.6.3\text{-}246)$$

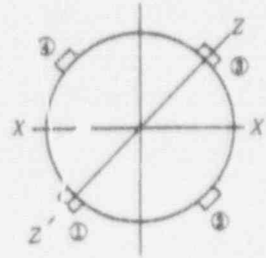


Figure 6.6.3-19. Directions of seismic forces.

When n = 2 and the bolts are set in a row in the radial direction,

$$\tau_{b4} = \frac{\sqrt{\left(\frac{P_1}{\sqrt{2}} - P\right)^2 + \left(\frac{Q}{\sqrt{2}}\right)^2}}{2A_b} + \frac{Qu - M_c}{\sqrt{2}A_b(a - 2d_1)} \tag{6.6.3-247}$$

When the bolts are set in a row perpendicular to the radial direction,

$$\tau_{b4} = \frac{\sqrt{\left(\frac{P_1}{\sqrt{2}} - P\right)^2 + \left(\frac{Q}{\sqrt{2}}\right)^2}}{2A_b} + \frac{Qu - M_c}{\sqrt{2}A_b(b - 2d_2)} \tag{6.6.3-248}$$

When n = 1,

$$\tau_{b4} = \frac{\sqrt{\left(\frac{P_1}{\sqrt{2}} - P\right)^2 + \left(\frac{Q}{\sqrt{2}}\right)^2}}{A_b} + \frac{16(Qu - M_c)}{\sqrt{2}\pi d_b^3} \tag{6.6.3-249}$$

For the anchor bolts on legs 2 and 3,

$$\tau_{b5} = \frac{\sqrt{\left(\frac{P_1}{\sqrt{2}} + P\right)^2 + \left(\frac{Q}{\sqrt{2}}\right)^2}}{nA_b} + \frac{Qu - M_c}{\sqrt{2}nA_b\sqrt{\left(\frac{a-2d_1}{2}\right)^2 + \left(\frac{b-2d_2}{2}\right)^2}} \tag{6.6.3-250}$$

When n = 2 and the bolts are set in a row in the radial direction,

$$\tau_{b5} = \frac{\sqrt{\left(\frac{P_1}{\sqrt{2}}+P\right)^2+\left(\frac{Q}{\sqrt{2}}\right)^2}}{2A_b} + \frac{Qu-M_c}{\sqrt{2}A_b(a-2d_1)} \tag{6.6.3-251}$$

When the bolts are set in a row perpendicular to the radial direction,

$$\tau_{b5} = \frac{\sqrt{\left(\frac{P_1}{\sqrt{2}}+P\right)^2+\left(\frac{Q}{\sqrt{2}}\right)^2}}{2A_b} + \frac{Qu-M_c}{\sqrt{2}A_b(b-2d_2)} \tag{6.6.3-252}$$

When n = 1,

$$\tau_{b5} = \frac{\sqrt{\left(\frac{P_1}{\sqrt{2}}+P\right)^2+\left(\frac{Q}{\sqrt{2}}\right)^2}}{A_b} + \frac{16(Qu-M_c)}{\sqrt{2}\pi d_b^3} \tag{6.6.3-253}$$

{b} Tensile stress

With respect to legs 1 and 4, when the lower ends of the legs are fixed, the moment and vertical load acting on the bottom portion of the legs are as follows:

$$M_{xl} = \left(\left\{\frac{1}{\sqrt{2}}(P_1 l+M_1-R_1 u)-(P\cdot l+M_l-R u)\right\}^2 + \left\{\frac{1}{\sqrt{2}}(Q\cdot l-M_3)\right\}^2\right)^{1/2} \tag{6.6.3-254}$$

$$R_{xl} = R - \frac{R_1}{\sqrt{2}} \tag{6.6.3-255}$$

Similar to {1}, the stress of the anchor bolts derived form equations (6.6.3-231) through (6.6.3-235) is taken as σ_{b4}. However, if the tensile stress derived by replacing a, b, d_1, and n_1 with b, a, d_2, and n_2, respectively, is greater than σ_{b4}, this larger value is taken as σ_{b4}.

When the lower end of the leg is simply supported in the direction perpendicular to the radial direction,

$$M_{x1} = \left| \frac{1}{\sqrt{2}}(P_1 l + M_1 - R_1 u) - (P \cdot l + M_l - R u) \right| \qquad (6.6.3\text{-}256)$$

R_{x1} is expressed by formula (6.6.3-255). Just as in {1}, the stress of the anchor bolt derived using the set of equations (6.6.3-231) through (6.6.3-235) is taken as σ_{b4}.

When the lower end of the leg is simply supported in the radial direction,

$$M_{x1} = \frac{1}{\sqrt{2}} |Q \cdot l - M_3| \qquad (6.6.3\text{-}257)$$

R_{x1} is expressed by formula (6.6.3-255). Just as in {1}, the stress of the anchor bolt derived using the set of equations (6.6.3-231) through (6.6.3-235) is taken as σ_{b4}. In this case, a, b, d_1, and n_1 are replaced by b, a, d_2, and n_2, respectively.

In the case when the lower end of the leg is simply supported in the radial direction and the direction perpendicular to the radial direction, no moment is generated; hence, when vertical load R_{x1} is negative, a tensile stress j is generated in the anchor bolt.

$$F_b = -R_{x1} \qquad (6.6.3\text{-}258)$$

$$\sigma_{b4} = \frac{F_b}{n A_b} \qquad (6.6.3\text{-}259)$$

For the anchor bolts on legs 2 and 3, when the lower ends of the legs are fixed, we have

$$M_{x2} = \left(\left\{ \frac{1}{\sqrt{2}}(P_1 l + M_1 - R_1 u) + (P \cdot l + M_l - R u) \right\}^2 + \left\{ \frac{1}{\sqrt{2}}(Q \cdot l - M_3) \right\}^2 \right)^{1/2} \qquad (6.6.3\text{-}260)$$

$$R_{x2} = R + \frac{R_1}{\sqrt{2}} \qquad (6.6.3\text{-}261)$$

Just as in {1}, the stress in the anchor bolt derived using the set of equations (6.6.3-231) through (6.6.3-235) is taken as σ_{b5}. However, if the tensile stress in the anchor bolt derived when a, b, d_1, and n_1 are replaced by b, a, d_2, and n_2, respectively is larger than σ_{b5}, the larger value is taken as σ_{b5}.

When the lower end of the leg is simply supported in the direction perpendicular to the radial direction,

$$M_{x2} = \left|\frac{1}{\sqrt{2}}(P_1 l + M_1 - R_1 u) + (P l + M_l - R u)\right| \qquad (6.6.3\text{-}262)$$

R_{x2} is expressed by formula (6.6.3-261). Just as in {1}, the stress in the anchor bolt derived using the set of equations (6.6.3-231) through (6.6.3-235) is taken as σ_{b5}.

When the lower end of the leg is simply supported in the radial direction, we have

$$M_{x2} = \frac{1}{\sqrt{2}}\left|Q l - M_3\right| \qquad (6.6.3\text{-}263)$$

R_{x1} is expressed by formula (6.6.3-261). Just as in {1}, the stress in the anchor bolt derived using the set of equations (6.6.3-231) through (6.6.3-235) is taken as σ_{b5}. In this case, a, b, d_1, and n_1 are replaced by b, a, d_2, and n_2, respectively.

When the lower end of the leg is simply supported in the radial direction and the direction perpendicular to the radial direction, there is only compressive load, and no tensile stress is generated in the anchor bolt.

(c) Evaluation methods

(i) Evaluation of natural period

Based on the natural period derived in section (a), the design horizontal seismic coefficient is confirmed.

(ii) Evaluation of stress

{1} Evaluation of stress in barrel

The evaluation is performed according to section "6.6.3(1)b. Skirt-support vertical cylindrical container."

{2} Evaluation of stress in leg

{a} The combined stress derived in (b)(ii){4} should be less than allowable tensile stress f_t.

$$f_t = \left(\frac{F}{1.5}\right)1.5 \qquad (6.6.3\text{-}264)$$

{b} The combination of compressive stress and bending-caused stress on the compressive side should satisfy the following relation:

$$\frac{\sigma_{sr}}{f_{br}} + \frac{\sigma_{st}}{f_{bt}} + \frac{\sigma_{sc}}{f_c} \le 1 \qquad (6.6.3\text{-}265)$$

A. f_c is defined as follows:

When $\lambda \leq \Lambda$

$$f_c = 1.5\left\{1 - 0.4\left(\frac{\lambda}{\Lambda}\right)^2\right\}\frac{F}{\nu} \tag{6.6.3-266}$$

When $\lambda > \Lambda$

$$f_c = 1.5(0.277F)\left(\frac{\Lambda}{\lambda}\right)^2 \tag{6.6.3-267}$$

where,

$$\lambda = l_k / i \tag{6.6.3-268}$$

$$\Lambda = \sqrt{\frac{\pi^2 E_s}{0.6F}} \tag{6.6.3-269}$$

$$\nu = 1.5 + \frac{2}{3}\left(\frac{\lambda}{\Lambda}\right)^2 \tag{6.6.3-270}$$

$$i = \sqrt{\frac{\min(I_{sr}, I_{st})}{A_s}} \tag{6.6.3-271}$$

l_k is the effective buckling length and is se' as 1.2 l when the lower end of the leg is fixed and as 2.1 l when the lower end of the leg is simply supported.

B. f_{br} and f_{bt} are defined as follows:

(A) When the leg is made of steep pipe,

$$f_{br} = f_{bt} = f_t \tag{6.6.3-272}$$

(B) When the leg is made of rolled steel with weaker axis in the radial direction,

$$f_{br} = f_t \tag{6.6.3-273}$$

f_{bt} is taken as either the value calculated using the following two formulas, whichever is larger, or f_t, whichever is smaller.

$$f_{bt} = \left(1 - 0.4\frac{l^2}{C\Lambda^2 i_f^2}\right) f_t \qquad (6.6.3\text{-}274)$$

$$f_{bt} = \left(\frac{0.433 E_s A_f}{l h}\right) 1.5 \qquad (6.6.3\text{-}275)$$

where i_f is the radius of gyration of the area around the web axis of the T-shaped cross section made of the compression flange of the leg and 1/6 the web. It is defined as follows:

$$i_f = \sqrt{\frac{I_{sf}}{A_{sf}}} \qquad (6.6.3\text{-}276)$$

C is the value calculated using the following formula or 2.3, whichever is smaller; M_{S2} and M_{S1} are the bending moments around the stronger axis at the two ends of the leg, respectively. In this case, the ratio of M_{S2} to M_{S1} is taken as less than one. It is positive in the case of single curvature and negative in the case of double curvature.

$$C = 1.75 - 1.05\left(\frac{M_{s2}}{M_{s1}}\right) + 0.3\left(\frac{M_{s2}}{M_{s1}}\right)^2 \qquad (6.6.3\text{-}277)$$

(C) When the leg is made of rolled steel and the stronger axis is in the radial direction

After derivation in the same way as in (B), f_{br} is replaced by f_{bt}, and f_{bt} is replaced by f_{br}.

C. Classification of the stresses is as follows.

(A) For leg 1 when the seismic force acts in the Z-direction

$$\sigma_{sc} = \sigma_{s1} + \sigma_{s3} \qquad (6.6.3\text{-}278)$$

$$\sigma_{st} = \sigma_{s2} + \sigma_{p4} \qquad (6.6.3\text{-}279)$$

$$\sigma_{sr} = 0 \qquad (6.6.3\text{-}280)$$

When the stronger axis of the leg is in the direction perpendicular to the radial direction and the lower end of the leg is fixed around the axis, bending moments M_{S1} and M_{S2} around the stronger axis are calculated using the following two formulas. When the absolute value of M_{S2} is larger than the absolute value of M_{S1}, M_{S1} and M_{S2} are exchanged.

(Same in the following)

$$M_{sl} = -(M_l + M_1) + (R + R_1)\mu \tag{6.6.3-281}$$

$$M_{s2} = -(P + P_1)l - (M_l + M_1) + (R + R_1)\mu \tag{6.6.3-282}$$

When the stronger axis of the leg is in the direction perpendicular to the radial direction and the lower end of the leg is simply supported around the axis, C is taken as 1.75.

(B) For leg 2 in the case when the seismic force is in the Z-direction

$$\sigma_{sc} = \sigma_{sl} \tag{6.6.3-283}$$

$$\sigma_{st} = \sigma_{s2} \tag{6.6.3-284}$$

$$\sigma_{sr} = \sigma_{s3} \tag{6.6.3-285}$$

When the stronger axis of the leg is in the radial direction and the lower end of the leg is fixed around the axis, the bending moments around the stronger axis become

$$M_{sl} = M_3 \tag{6.6.3-286}$$

$$M_{s2} = M_3 - Ql \tag{6.6.3-287}$$

When the stronger axis of the leg is perpendicular to the radial axis and the lower end of the leg is fixed around the axis, the bending moments around the stronger axis are as follows

$$M_{sl} = -M_l + Ru \tag{6.6.3-288}$$

$$M_{s2} = -Pl - M_l + Ru \tag{6.6.3-289}$$

When the lower end of the leg is simply supported around the stronger axis, C is set as 1.75.

(C) When the seismic force acts in the X-direction

$$\sigma_{sc} = \sigma_{sl} + \sigma_{s6} \tag{6.6.3-290}$$

$$\sigma_{st} = \sigma_{s2} + \sigma_{s7} \tag{6.6.3-291}$$

$$\sigma_{sr} = \sigma_{s8} \tag{6.6.3-292}$$

When the stronger axis of the log is perpendicular to the radial direction and the lower end of the leg is fixed around the axis, the bending moments around the stronger axis are as follows:

$$M_{s1} = -M_l + Ru + \frac{1}{\sqrt{2}}(-M_1 + R_1 u) \tag{6.6.3-293}$$

$$M_{s2} = -P \cdot l - M_l + Ru + \frac{1}{\sqrt{2}}(-P_1 l - M_1 + R_1 u) \tag{6.6.3-294}$$

When the stronger axis of the leg is in the radial direction and the lower end of the leg is fixed around the axis, the bending moments around the stronger axis are as follows:

$$M_{s1} = \frac{1}{\sqrt{2}} M_1 \tag{6.6.3-295}$$

$$M_{s2} = \frac{1}{\sqrt{2}}(M_1 - Q \cdot l) \tag{6.6.3-296}$$

When the lower end of the leg is simply supported around the stronger axis, C is taken as 1.75.

{3} Stress evaluation of foundation bolts

Evaluation is performed according to section "6.6.3(1)b. Skirt-support vertical cylindrical container."

e. Horizontal cylindrical container (see Figure 6.6.3-20)

Conditions assumed

{1} The weight of the container and content is concentrated on the central axis of the barrel.

{2} If the anchor bolts used to fix the leg on the foundation are arranged in a row for each leg as viewed from the direction perpendicular to the deformation direction of the leg, the lower end [of the leg] is taken as simply supported. Otherwise, it is taken as fixed.

{3} The container has its barrel supported by two legs, which are mounted on the foundation by anchor bolts. Of these two legs, one leg can slide with respect to the foundation in the longitudinal direction of the leg.

{4} The seismic force is taken as acting in the horizontal direction on the container. The design seismic coefficient in the vertical direction is not taken into consideration.

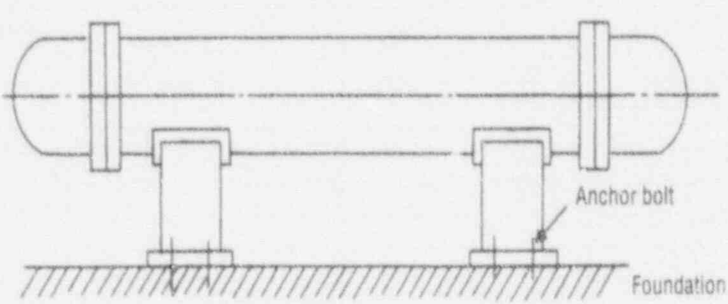


Figure 6.6.3-20. Schematic diagram of horizontal cylindrical container.

Definitions of symbols (symbols used in the calculation formulas of horizontal cylindrical containers)

Symbol	Definition of symbol	Units
A_s	Leg sectional area	mm^2
A_{s1}	Effective shear sectional area with respect to the longitudinal direction of the leg	mm^2
A_{s2}	Effective shear sectional area with respect to the transverse direction of the leg	mm^2
A_{s3}	Shear sectional area with respect to the longitudinal direction of the leg	mm^2
A_{s4}	Shear sectional area with respect to the transverse direction of the leg	mm^2
a	Longitudinal width of leg bottom plate	mm
b	Transverse width of leg bottom plate	mm
C_c	Value obtained from Reference [6.6.3-2]	—
C_l	Value obtained from Reference [6.6.3-2]	—
C_1	1/2 the width of attachment at the root portion where the leg is attached to the barrel (transverse direction of barrel)	mm
C_2	1/2 the width of attachment at the root portion where the leg is attached to the barrel (longitudinal direction of barrel)	mm
d_1	Longitudinal distance between sides of leg bottom plate and center of anchor bolt	mm
d_2	Transverse distance between sides of leg bottom plate and center of anchor bolt	mm
e	Distance from center of leg to eccentric load acting point	mm
f_t	Allowable tensile stress of leg	kgf/mm^2
h_1	Height from foundation to the root portion where the leg is attached to the barrel	mm
h_2	Height from foundation to the center of barrel	mm
I_x	Moment of inertia with respect to the longitudinal axis of the leg	mm^4
I_y	Moment of inertia with respect to the transverse axis of the leg	mm^4
j_1	Number of static loads divided as distribution of load	—
j_2	Number of static loads acting from leg 1 in direction opposite to leg 2 (loads on leg 1 not included)	—
j_3	Number of static loads acting from leg 2 in direction opposite to leg 1 (loads on leg 2 not included)	—
K_c	Leg spring constant (when a horizontal force is acting in the transverse direction of barrel)	kgf/mm
K_l	Leg spring constant (when a horizontal force is acting in the longitudinal direction of barrel)	kgf/mm
K_1, K_2	Constant defined in Reference [6.6.3-2]	—
l_i	Distances from leg 1 to various loads (the distance on the leg-2 side is positive, the distance on the opposite side is negative)	mm
l_0	Distance between centers of legs	mm
M	Moment acting on leg bottom plate	kgf·mm

<u>Definitions of symbols</u> (symbols used in the calculation formulas of horizontal cylindrical containers)
(Cont'd)

Symbol	Definition of symbol	Units
M_c	Moment at root portion where the leg is attached to the barrel due to transverse seismic force	kgf·mm
M_{c1}	Moment acting on leg bottom surface due to transverse seismic force	kgf·mm
M_l	Moment at root portion where the leg is attached to the barrel due to longitudinal seismic force	kgf·mm
M_{l1}	Moment acting on leg bottom surface due to longitudinal seismic force	kgf·mm
M_1, M_2	Moment at legs 1 and 2 due to operation weight of the barrel	kgf·mm
N_x	Axial membrane force generated in the barrel	kgf/mm
N_ϕ	Circumferential membrane force generated in the barrel	kgf/mm
n	Number of foundation bolts for each leg	—
n_1, n_2	Number of foundation bolts acted upon by tensile force	—
P	Reactive force acting on the root portion where the leg is attached to the barrel	kgf
P_l	Vertical load acting on the root portion where the leg is attached to the barrel due to longitudinal seismic force	kgf
P_s	Vertical load acting on the leg bottom portion due to longitudinal seismic force	kgf
P_{s1}	Vertical load acting on the leg bottom portion due to transverse seismic force	kgf
R_1, R_2	Weights loaded on legs 1 and 2, respectively	kgf
r_m	Average radius of barrel at the root portion where the leg is attached	mm
r_0	Outer radius of barrel at the root portion where the leg is attached	mm
t_e	Effective plate thickness of the barrel at the root portion where the leg is attached	mm
W_i	Static load	kgf
W_s	Weight of leg	kgf
X_n	Width of foundation acted upon by compressive force	mm
Z	Sectional modulus of barrel according to Reference [6.6.3-3]	mm^3
Z_{sx}	Sectional modulus with respect to the longitudinal axis of the leg	mm^3
Z_{sy}	Sectional modulus with respect to the transverse axis of the leg	mm^3
ϕ	Half of the angle of the effective range of the barrel according to Reference [6.6.3-3]	rad
ϕ_0	Angle from barrel leg end portion to vertical axis	rad
σ_{0c}	Combined primary general membrane stress of barrel when a transverse seismic force acts	kgf/mm^2
σ_{0cx}	Sum of axial primary general membrane stresses in the barrel when seismic force acts in the transverse direction	kgf/mm^2
$\sigma_{0c\phi}$	Sum of circumferential primary general membrane stresses in the barrel when seismic force acts in the transverse direction	kgf/mm^2

<u>Definitions of symbols</u> (symbols used in the calculation formulas of horizontal cylindrical containers)
(Cont'd)

Symbol	Definition of symbol	Units
σ_{0l}	Combined primary general membrane stress in barrel when seismic force acts in the longitudinal direction	kgf/mm^2
σ_{0lx}	Sum of axial primary general membrane stresses in barrel when seismic force acts in the longitudinal direction	kgf/mm^2
$\sigma_{0l\phi}$	Sum of circumferential primary general membrane stresses in the case when the seismic force acts in the longitudinal direction	kgf/mm^2
σ_{1c}	Combined primary stress in barrel when the seismic force acts in the transverse direction	kgf/mm^2
σ_{1cx}	Sum of axial primary stresses when transverse seismic force acts	kgf/mm^2
$\sigma_{1c\phi}$	Sum of circumferential primary stresses in barrel when transverse seismic force acts	kgf/mm^2
σ_{1l}	Combined primary stress when longitudinal seismic force acts	kgf/mm^2
σ_{1lx}	Sum of axial primary stresses in barrel when longitudinal seismic force acts	kgf/mm^2
$\sigma_{1l\phi}$	Sum of circumferential primary stresses in barrel when longitudinal seismic force acts	kgf/mm^2
σ_{b1}	Tensile stress generated in the anchor bolts due to the longitudinal seismic force	kgf/mm^2
σ_{b2}	Tensile stress generated in the anchor bolt due to transverse seismic force	kgf/mm^2
σ_{sc}	Combined stress in leg when transverse seismic force acts	kgf/mm^2
σ_{sl}	Combined stress in leg when longitudinal seismic force acts	kgf/mm^2
σ_{s1}	Compressive stress in leg due to weight in operation	kgf/mm^2
σ_{s2}	Sum of compressive and flexural stresses generated in the leg due to longitudinal seismic force	kgf/mm^2
σ_{s3}	Flexural stress generated in leg due to transverse seismic force	kgf/mm^2
$\sigma_{\phi 1}$, σ_{x1}	Circumferential and axial stresses generated in barrel due to internal pressure or static water head	kgf/mm^2
σ_{x2}	Axial stress generated in barrel due to longitudinal bending moment of barrel	kgf/mm^2
$\sigma_{\phi 3}$, σ_{x3}	Circumferential and axial stresses generated in the root portion where the leg is attached to the barrel due to weight in operation	kgf/mm^2
$\sigma_{\phi 4}$, σ_{x4}	Sum of circumferential and axial stresses generated at the root portion where the leg is attached to the barrel due to longitudinal seismic force	kgf/mm^2
$\sigma_{\phi 41}$, σ_{x41}	Circumferential and axial stresses generated due to moment at the root portion where the leg is attached to the barrel due to longitudinal seismic force	kgf/mm^2
$\sigma_{\phi 42}$, σ_{x42}	Circumferential and axial stresses generated by vertical load at the root portion where the leg is attached to the barrel due to longitudinal seismic force	kgf/mm^2
σ_{x43}	Stress in barrel due to horizontal load generated by longitudinal seismic force	kgf/mm^2
$\sigma_{\phi 5}$, σ_{x5}	Circumferential and axial stresses generated by movement at root portion where the leg is attached to the barrel due to transverse seismic force	kgf/mm^2

Definitions of symbols (symbols used in the calculation formulas of horizontal cylindrical containers) *(Cont'd)*

Symbol	Definition of symbol	Units
τ_{b1}	Shear stress generated in anchor bolt due to longitudinal seismic force	kgf/mm^2
τ_{b2}	Shear stress generated in anchor bolt due to transverse seismic force	kgf/mm^2
τ_c	Shear stress generated at the root portion where the leg is attached to the barrel due to transverse seismic force	kgf/mm^2
τ_l	Shear stress generated at the root portion where the leg is attached to the barrel due to longitudinal seismic force	kgf/mm^2
τ_{s2}	Shear stress generated in the leg due to longitudinal seismic force	kgf/mm^2
τ_{s3}	Shear stress generated in the leg due to transverse seismic force	kgf/mm^2

Analysis conditions

{1} The barrel of the container is taken as rigid, while the flexural and shear deformations of the leg are taken into consideration.

{2} Since leg 2 can slide in the longitudinal direction, all of the forces in this direction act on leg 1.

(a) Calculation method of natural period

(i) Calculation model

Figures 6.6.3-21-6.6.3-24 show the load state of the container and the moment generated in the barrel. Under the aforementioned conditions, the container is taken as a single discrete mass model as shown in Figure 6.6.3-25 and 6.6.3-26.

(ii) Natural period in longitudinal direction

The spring constant in Figure 6.6.3-25 is

$$K_l = \frac{1}{\dfrac{h_1^3}{12E_sI_y} + \dfrac{h_1}{G_sA_{sl}}} \tag{6.6.3-297}$$

When the anchor bolts of leg 1 are set in a row as viewed from the transverse direction, the coefficient of "12" in equation (6.6.3-297) should be replaced by "3."

The natural period is

$$T_1 = 2\pi\sqrt{\frac{W_0}{K_l g}} \tag{6.6.3-298}$$

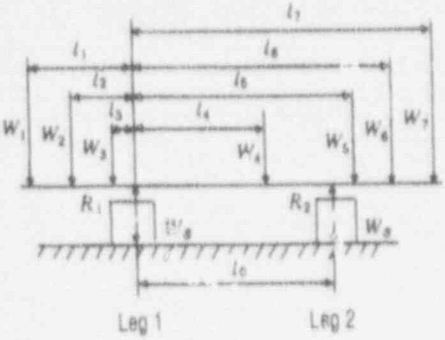


Figure 6.6.3-21. Load state.

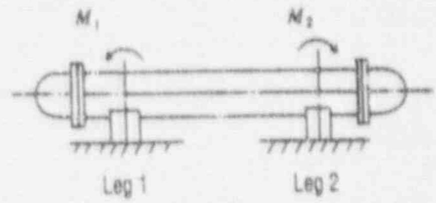


Figure 6.6.3-22. Bending moments at leg positions.

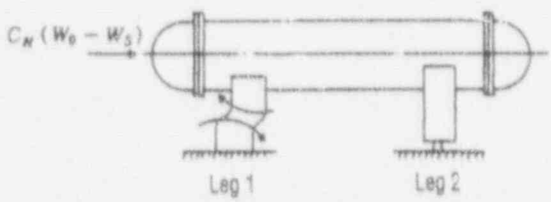


Figure 6.6.3-23. Local moment acting on barrel due to longitudinal load.

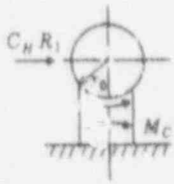


Figure 6.6.3-24. Local moment acting on barrel due to transverse load.

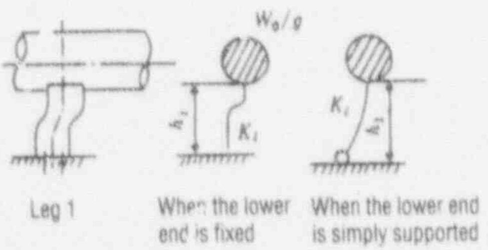


Figure 6.6.3-25. Calculation model of natural period in longitudinal direction.

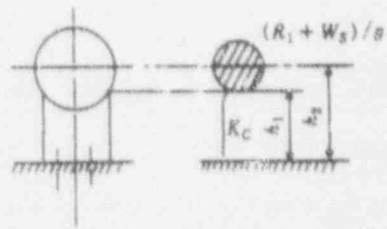


Figure 6.6.3-26. Calculation model of natural period in transverse direction.

(iii) Natural period in the transverse direction

The spring constant in Figure 6.6.3-26 is as follows:

$$K_c = \frac{1}{\dfrac{h_1^2(3h_2-h_1)}{6E_sI_x} + \dfrac{(h_2-h_1)h_1(h_2-h_1/2)}{E_sI_x} + \dfrac{h_1}{G_sA_{s2}}} \tag{6.6.3-299}$$

The natural period is

$$T_2 = 2\pi\sqrt{\frac{R_1+W_s}{K_c g}} \tag{6.6.3-300}$$

When $R_2 > R_1$ for the weight acting on the legs, R_1 is replaced by R_2.

(b) Calculation methods of stresses

(i) Stresses in barrel

{1} Weights acting on legs

The weight acting on the leg can be derived from the balance of moments. In Figure 6.6.3-23, from the balance of moments around leg 1, the following equation can be obtained:

$$\sum_{i=1}^{ll} W_i l_i - R_2 l_0 = 0 \tag{6.6.3-301}$$

Hence, the weights acting on the legs can be represented as follows:

$$R_2 = \sum_{i=1}^{ll} W_i l_i / l_0 \tag{6.6.3-302}$$

$$R_1 = \sum_{i=1}^{ll} W_i - R_2 \tag{6.6.3-303}$$

{2} Bending moments

As shown in Figure 6.6.3-21, the barrel is taken as a beam acted upon by concentrated load.

Bending moments M_1 and M_2 at the root portion where the leg is attached as shown in Figure 6.6.3-22 can be expressed as follows:

$$M_1 = \sum_{i=1}^{j3} W_i |l_i| \qquad (6.6.3\text{-}304)$$

$$M_2 = \sum_{i=j1-j3+1}^{j1} W_i |l_i - s_0| \qquad (6.6.3\text{-}305)$$

{3} Stresses due to static water head or internal pressure

In the case of static water head, we have

$$\sigma_{\phi 1} = \frac{\rho' H D_i}{2t} \qquad (6.6.3\text{-}306)$$

$$\sigma_{x1} = \frac{\rho' H D_i}{4t} \qquad (6.6.3\text{-}307)$$

In the case of internal pressure, we have

$$\sigma_{\phi 1} = \frac{P_r (D_i + 1.2t)}{200t} \qquad (6.6.3\text{-}308)$$

$$\sigma_{x1} = \frac{P_r (D_i + 1.2t)}{400t} \qquad (6.6.3\text{-}309)$$

{4} Stress (at the root portion of attachment of leg 1) due to the longitudinal bending moment generated by the weight in operation

The stress generated at the ro portion where the leg is attached to the barrel due to the bending moment derived in {2} can be derived as follows.

According to Reference (6.6.3-3), this bending moment does not act uniformly with respect to the cross section of the barrel; at the leg attachment portion, it is replaced by the bending moment in the circumferential direction, and produces a local deformation of the barrel.

Now, suppose the range of influence of the stress in the barrel due to the longitudinal bending moment is up to the point $\theta_0/6$ above the leg, the effective sectional area of the barrel with respect to the longitudinal bending moment becomes 2θ of the cylindrical shell, as illustrated in Figure 6.6.3-27. Hence, the stress can be represented by the following formula:

$$\sigma_{x2} = \frac{M_1}{Z} \qquad (6.6.3\text{-}310)$$

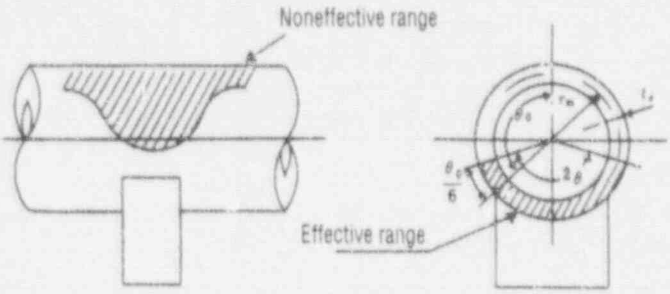


Figure 6.6.3-27. Effective range of leg-attachment root portion.

where,

$$r_m = \frac{D_i + t_e}{2} \tag{6.6.3-311}$$

$$Z = r_m^2 t_e \left(\frac{\theta + \sin\theta\cos\theta - 2\sin^2\theta/\theta}{\sin\theta/\theta - \cos\theta} \right) \tag{6.6.3-312}$$

{5} Stresses at leg-attachment root portion due to weight in operation

Local stresses are generated due to the leg's reaction force at the leg-attachment root portion of the barrel.

The reactive force acting on the barrel attachment root portion of leg 1 can be represented by the following formula:

$$P = R_1 \tag{6.6.3-313}$$

According to Reference [6.6.3-3], the local stress of the barrel generated by this reaction force P can be derived as follows:

$$\gamma = r_m / t_e \tag{6.6.3-314}$$

$$\beta_1 = C_1 / r_m \tag{6.6.3-315}$$

$$\beta_2 = C_2 / r_m \tag{6.6.3-316}$$

$\beta_1/\beta_2 \geq 1$,

$$\beta = \left(1 - \frac{1}{3}(\beta_1/\beta_2 - 1)(1 - K_1^*)\right)\sqrt{\beta_1\beta_2} \tag{6.6.3-317}$$

$\beta_1/\beta_2 < 1$,

$$\beta = \left(1 - \frac{4}{3}(1 - \beta_1/\beta_2)(1 - K_2^*)\right)\sqrt{\beta_1\beta_2} \tag{6.6.3-318}$$

From shell parameter γ and attachment parameter β_1 values are used to obtain constants from the table in Reference [6.6.3-2] (marked by *); then the stresses are obtained as follows:

$$\sigma_{\phi 3} = \left(\frac{N_\phi}{P/r_m}\right)^* \cdot \left(\frac{P}{r_m t_e}\right) \tag{6.6.3-319}$$

$$\sigma_{x3} = \left(\frac{N_x}{P/r_m}\right)^* \cdot \left(\frac{P}{r_m t_e}\right) \tag{6.6.3-320}$$

{6} Stresses at leg-attachment root portion due to longitudinal seismic force

Since leg 2 can slide freely in the longitudinal direction, leg 1 deforms as shown in Figure 6.6.3-23, and the bending moment and vertical load (force couple) acting on the leg-attachment root portion are as follows (see Figure 6.6.3-29):

$$M_l = \frac{1}{2} C_H (W_0 - W_s) h_1 \tag{6.6.3-321}$$

$$P_l = C_H (W_0 - W_s) \frac{h_2 - \frac{1}{2} h_1}{l_0} \tag{6.6.3-322}$$

When the lower end of the leg is simply supported, the coefficient of 1/2 in formula (6.6.3-321) should be replaced by 1, and the coefficient of 1/2 in formula (6.6.3-322) should be replaced by 0.

Just as in {5}, the local stresses in the barrel generated by bending moment M_l and vertical load P_l can be derived from Reference [6.6.3-2].

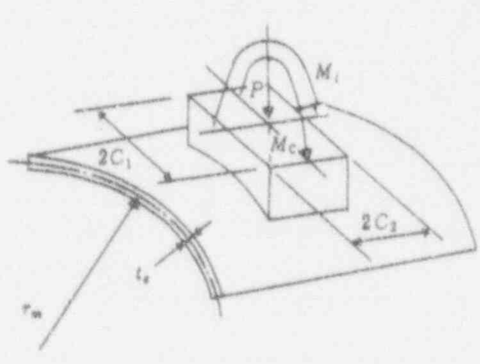


Figure 6.6.3-29. Forces acting by leg on barrel.

The stresses generated by bending moment M_l are as follows:

$$\sigma_{\phi 41} = \left(\frac{N_\phi}{M_l/(r_m^2\beta)}\right)^* \cdot \left(\frac{M_l}{r_m^2 t_e \beta}\right) C_l^* \tag{6.6.3-323}$$

$$\sigma_{x41} = \left(\frac{N_x}{M_l/(r_m^2\beta)}\right)^* \cdot \left(\frac{M_l}{r_m^2 t_e \beta}\right) C_l^* \tag{6.6.3-324}$$

where attachment parameter β is defined as follows:

$$\beta = \sqrt[3]{\beta_1 \beta_2^2} \tag{6.6.3-325}$$

The stresses generated by vertical load P_l are as follows:

$$\sigma_{\phi 42} = \left(\frac{N_\phi}{P_l/r_m}\right)^* \cdot \left(\frac{P_l}{r_m t_e}\right) \tag{6.6.3-326}$$

$$\sigma_{x42} = \left(\frac{N_x}{P_l/r_m}\right)^* \cdot \left(\frac{P_l}{r_m t_e}\right) \tag{6.6.3-327}$$

In addition, due to the horizontal load, the following tensile stress is generated in the barrel:

$$\sigma_{x43} = \frac{C_H(W_0 - W_s)}{\pi(D_i + t)t} \tag{6.6.3-328}$$

Hence, the stresses generated in the barrel due to bending moment M_l, vertical force P_l and horizontal load are as follows:

$$\sigma_{\phi 4} = \sigma_{\phi 41} + \sigma_{\phi 42} \tag{6.6.3-329}$$

$$\sigma_{x4} = \sigma_{x41} + \sigma_{x42} + \sigma_{x43} \tag{6.6.3-330}$$

When the seismic force acts in the longitudinal direction, the shear stress generated at the root portion where leg 1 is attached can be calculated as follows:

$$\tau_l + \frac{C_H(W_0 - W_s)}{4C_2 t} \qquad (6.6.3\text{-}331)$$

{7} Stresses at leg-attachment root portion due to transverse seismic force

When a transverse seismic force acts, bending moment M_C generated at the root portion where leg 1 is attached is as follows:

$$M_c = C_H R_1 r_o \qquad (6.6.3\text{-}332)$$

$$r_0 = \frac{D_i}{2} + t_s \qquad (6.6.3\text{-}333)$$

The local stress generated in the barrel due to this bending moment M_C can be derived from Reference [6.6.3-2] using the same method as in {5} and {6}.

Shell parameter γ is the same as in {5}, while attachment parameter β is defined as follows:

$$\beta = \sqrt[3]{\beta_1^2 \beta_2} \qquad (6.6.3\text{-}334)$$

Hence, the stresses can be expressed as follows:

$$\sigma_{\phi 5} = \left(\frac{N_\phi}{M_c/(r_m^2\beta)}\right)^* \cdot \left(\frac{M_c}{r_m^2\beta t_s}\right) C_c^* \qquad (6.6.3\text{-}335)$$

$$\sigma_{x5} = \left(\frac{N_x}{M_c/(r_m^2\beta)}\right)^* \cdot \left(\frac{M_c}{r_m^2\beta t_s}\right) C_c^* \qquad (6.6.3\text{-}336)$$

In addition, when a transverse seismic force acts, the shear stress generated at the root portion where leg 1 attached can be expressed as follows:

$$\tau_c = \frac{C_H R_1}{4C_1 t} \qquad (6.6.3\text{-}337)$$

{8} Combinations of stresses

The stresses generated in the root portion of the barrel where leg 1 is attached calculated in {3}-{7} are combined as follows.

{a} Primary general membrane stress

A. When longitudinal seismic force acts

$$\sigma_{0l} = \max\left(\sigma_{0l\phi}, \sigma_{0lx}\right) \tag{6.6.3-338}$$

where,

$$\sigma_{0l\phi} = \sigma_{\phi 1} \tag{6.6.3-339}$$

$$\sigma_{0lx} = \sigma_{x1} + \sigma_{x2} + \sigma_{x43} \tag{6.6.3-340}$$

B. When transverse seismic force acts

$$\sigma_{0c} = \max\left(\sigma_{0c\phi}, \sigma_{0cx}\right) \tag{6.6.3-341}$$

where,

$$\sigma_{0c\phi} = \sigma_{\phi 1} \tag{6.6.3-342}$$

$$\sigma_{0cx} = \sigma_{x1} + \sigma_{x2} \tag{6.6.3-343}$$

Hence, the maximum value of the primary general membrane stress generated in the barrel can be represented as follows:

$$\sigma_0 = \max\left(\sigma_{0l},\ \sigma_{0c}\right) \tag{6.6.3-344}$$

{b} Primary stresses

A. When longitudinal seismic force acts

$$\sigma_{1l} = \frac{1}{2}\left\{\left(\sigma_{1l\phi} + \sigma_{1lx}\right) + \sqrt{\left(\sigma_{1l\phi} - \sigma_{1lx}\right)^2 + 4\tau_l^2}\right\} \tag{6.6.3-345}$$

where

$$\sigma_{1l\phi} = \sigma_{\phi 1} + \sigma_{\phi 3} + \sigma_{\phi 4} \tag{6.6.3-346}$$

$$\sigma_{1lx} = \sigma_{x1} + \sigma_{x2} + \sigma_{x3} + \sigma_{x4} \tag{6.6.3-347}$$

B. When transverse seismic force acts

$$\sigma_{1c} = \frac{1}{2}\left\{\left(\sigma_{1c\phi} + \sigma_{1cx}\right) + \sqrt{\left(\sigma_{1c\phi} - \sigma_{1cx}\right)^2 + 4\tau_c^2}\right\} \tag{6.6.3-348}$$

where,

$$\sigma_{1c\phi} = \sigma_{\phi 1} + \sigma_{\phi 3} + \sigma_{\phi 5} \tag{6.6.3-349}$$

$$\sigma_{1cx} = \sigma_{xl} + \sigma_{x2} + \sigma_{x3} + \sigma_{x5} \tag{6.6.3-350}$$

Hence, the maximum value of the primary stress generated in the barrel can be represented as follows:

$$\sigma_1 = \max\left(\sigma_{1l}, \sigma_{1c}\right) \tag{6.6.3-351}$$

(ii) Stresses in leg

Calculation is performed for the leg subjected to the largest weight.

{1} Compressive stress due to weight during operation

$$\sigma_{sl} = \frac{R_1 + W_s}{A_s} \tag{6.6.3-352}$$

{2} Stresses due to longitudinal seismic force

$$\sigma_{s2} = \frac{M_{l1}}{Z_{sy}} + \frac{P_l}{A_s} \tag{6.6.3-353}$$

where,

$$M_{l1} = \frac{1}{2} C_H W_0 h_l \tag{6.6.3-354}$$

When the lower end of the leg is simply supported, the coefficient of 1/2 is replaced by 1.

The shear stress is

$$\tau_{s2} = \frac{C_H W_0}{A_{s3}} \tag{6.6.3-355}$$

{3} Stresses due to transverse seismic force

The flexural stress is

$$\sigma_{s3} = \frac{C_H(R_1 + W_s)h_2}{Z_{sx}} \tag{6.6.3-356}$$

The shear stress is

$$\tau_{s3} = \frac{C_H(R_1 + W_s)}{A_{s4}} \tag{6.6.3-357}$$

{4} Combination of stresses

In the case of longitudinal seismic force,

$$\sigma_{sl} = \sqrt{(\sigma_{s1} + \sigma_{s2})^2 + 3\tau_{s2}^2} \tag{6.6.3-358}$$

In the case of transverse seismic force,

$$\sigma_{sc} = \sqrt{(\sigma_{s1} + \sigma_{s3})^2 + 3\tau_{s3}^2} \tag{6.6.3-359}$$

Hence, the maximum stress generated in the log can be represented as follows:

$$\sigma_s = \max(\sigma_{sl},\ \sigma_{sc}) \tag{6.6.3-360}$$

(iii) Stresses in anchor bolts

(1) When longitudinal seismic force acts

{a} Shear stress

$$\tau_{bl} = \frac{C_H W_0}{n A_b} \tag{6.6.3-361}$$

{b} Tensile stress

When a longitudinal seismic force acts, the moment acting on the leg bottom surface is

$$M = M_{II} \tag{6.6.3-362}$$

and the vertical load is

$$P_s = R_1 + W_s - P_l \tag{6.6.3-363}$$

Now, suppose the ratio of moment to compressive load is

$$e = M/P_s \tag{6.6.3-364}$$

When e is negative and when the following relation is satisfied

$$e > \frac{a}{6} + \frac{d_1}{3} \tag{6.6.3-365}$$

a tensile force is generated in the anchor bolt. This tensile force can be derived as follows (see Figure 6.6.3-30).

The position X_n of the neutral axis can be derived from the following equation:

$$X_n^3 + 3\left(e - \frac{a}{2}\right)X_n^2 - \frac{6sA_b n_1}{b}\left(e + \frac{a}{2} - d_1\right)(a - d_1 - X_n) = 0 \tag{6.6.3-366}$$

the tensile force generated in the anchor bolt becomes:

$$F_b = \frac{P_s\left(e - \frac{a}{2} + \frac{X_n}{3}\right)}{a - d_1 - \frac{X_n}{3}} \tag{6.6.3-367}$$

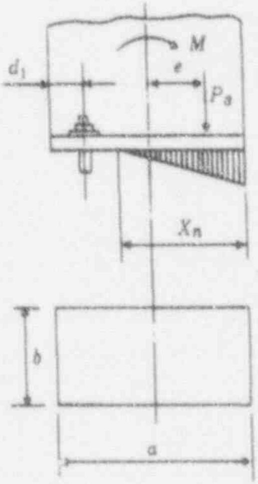


Figure 6.6.3-30. Load generated in foundation due to external loads acting on it.

Hence, the tensile stress generated in the anchor bolt is as follows:

$$\sigma_{bl} = \frac{F_b}{n_f A_b} \tag{6.6.3-368}$$

When the lower end of the leg is simply supported, no moment is generated on the leg's bottom surface; hence, when the vertical load P_S is negative, a tensile stress is generated in the anchor bolt.

$$F_b = -P_s \tag{6.6.3-369}$$

$$\sigma_{bl} = \frac{F_b}{nA_b} \tag{6.6.3-370}$$

{2} When transverse seismic force acts

When weight R_2 acting on the leg is larger than R_1, R_1 is replaced by R_2 in the calculation.

{a} Shear stress

$$\tau_{b2} = \frac{C_H(R_1 + W_s)}{nA_b} \tag{6.6.3-371}$$

{b} Tensile stress

When a transverse seismic force acts, the moment acting on the bottom surface of the log is

$$M_{cl} = C_H(R_1 + W_s)h_2 \tag{6.6.3-372}$$

The vertical force is

$$P_{sl} = R_1 + W_s \tag{6.6.3-373}$$

The tensile stress is derived in the same way as in {1}, except that M_{C1} is replaced by M, P_{S1} by P_S, d_2 by d_1, a by b, b by a, and n_2 by n_1. The obtained stress in the anchor bolt is taken as σ_{b2}.

(c) Evaluation method

(i) Evaluation of natural period

Based on the natural period derived in (a), the horizontal design seismic coefficient is confirmed.

(ii) Evaluation of stress

{1} Stress evaluation of barrel
It is performed according to section "6.6.3(1)b. Skirt-support vertical cylindrical container."

{2} Stress evaluation of leg
It is performed according to section "6.6.3(1)b. Skirt-support vertical cylindrical container." Evaluation of buckling is excluded.

{3} Stress evaluation of anchor bolt
In both the longitudinal direction and transverse direction, evaluation is performed according to section "6.6.3(1)b. Skirt-support vertical cylindrical container."

f. Lug-support vertical cylindrical container (see Figure 6.6.3-31)

Assumed conditions

{1} With reference to the center of lug attachment, the weight of the container is divided into the upper side portion and the lower side portion. For each portion, the weight is assumed to be concentrated at the center of gravity of that portion.

{2} Mounting of lugs on the foundation is done by mounting bolts. At the mounting portion of lugs on foundation, stretching of the mounting bolts is taken into consideration.

{3} The lugs can slide in the radical direction, they do not resist load in the radial direction.

{4} With respect to the circumferential load, the lugs are considered to be a pin structure, and the center between the mounting bolts as the axis of rotation.

However, in the case when the lugs have a structure that prevents rotation, it is possible to ignore the rotation of the lugs.

{5} The seismic force is assumed to act in the horizontal direction on the container. The design seismic coefficient in the vertical direction is not taken into consideration.

Analysis conditions

{1} The flexural and shear deformations of the barrel are taken into consideration.

{2} The local deformation is taken into consideration at the mounting portion where the leg is attached to the barrel.

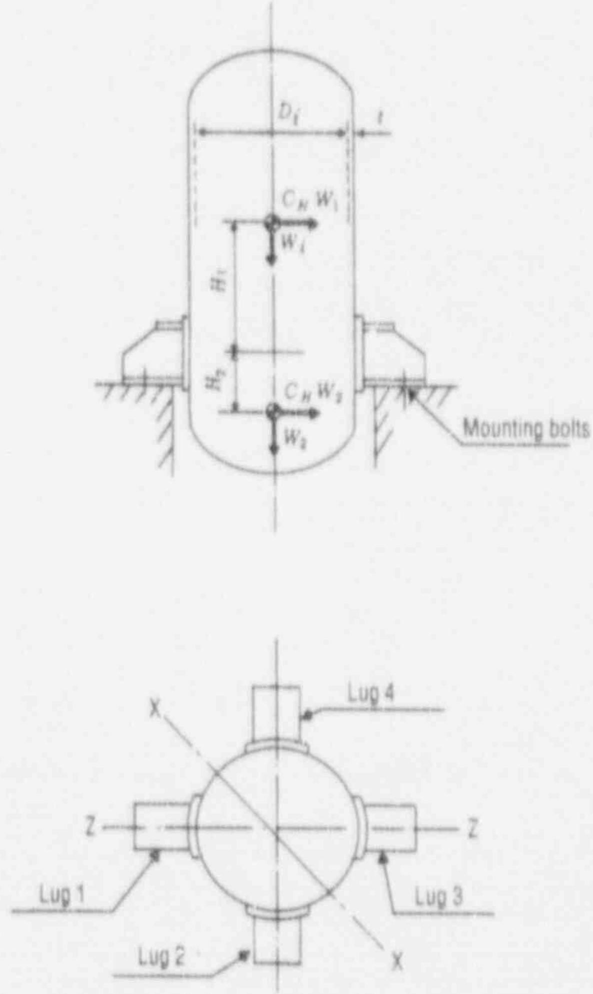


Figure 6.6.3-31. Schematic structural diagram.

Definitions of symbols (symbols used in the calculation formulas of lug-support vertical cylindrical container)

Symbol	Definition of symbol	Units
A_{be}	Effective sectional area of mounting bolts	mm^2
A_{s1}	Shear sectional area of lug with respect to vertical load	mm^2
A_{s2}	Shear sectional area of lug with respect to circumferential load	mm^2
a	Distance from lug end surface in radial direction to barrel wall center	mm
b	Distance from lug end surface in radial direction to center of mounting bolts	mm
c	Distance from end surface of foundation platform to center of mounting bolts	mm
C_1	Half of the attachment width at the root portion where the lug is mounted on the barrel (circumferential direction of barrel)	mm
C_2	Half of the attachment width at the root portion where the lug is mounted on the barrel (axial direction of barrel)	mm
C_c	Value defined in Reference [6.6.3-2]	—
C_l	Value defined in Reference [6.6.3-2]	—
d	Distance between mounting bolt centers	mm
e	Half the width of lug bottom plate	mm
E_b	Longitudinal elastic modulus of mounting bolt	kgf/mm^2
F_0	Horizontal force in vibration model system	kgf
F_1	Horizontal force at the center of gravity of the upper portion of the vibration model system	kgf
F_2	Horizontal force at the center of gravity of the lower portion of the vibration model system	kgf
F_{01}	Vertical reaction force acting on mounting bolt due to weight in operation	kgf
F_{02}	Vertical reaction force acting on end surface of foundation platform due to weight in operation	kgf
F_{11}	Vertical reaction force acting on mounting bolts of lug 1 due to horizontal forces F_1 and F_2	kgf
F_{12}	Vertical reaction force acting on end surface of lug 1 in radial direction due to horizontal forces F_1 and F_2	kgf
F_{21}	Vertical reaction force acting on mounting bolts of lug 3 due to horizontal forces F_1 and F_2	kgf
F_{22}	Vertical reaction force acting on end surface of lug 3 on foundation platform due to horizontal forces F_1 and F_2	kgf
F_{31}, F_{32}	Vertical reaction forces acting on mounting bolts of lugs 2 and 4 due to horizontal forces F_1 and F_2	kgf
f_t	Allowable tensile stress of lug	kgf/mm^2
H_1	Distance between lug attachment center and center of gravity of the upper portion	mm
H_2	Distance between lug attachment center and center of gravity of the lower portion	mm

<u>Definitions of symbols</u> (symbols used in the calculation formulas of lug support vertical cylindrical container)
(Cont'd)

Symbol	Definition of symbol	Units
K_c	Local spring constant at the lug attachment root portion of barrel with respect to the circumferential bending moment (value defined in Reference [6.6.3-4])	—
K_l	Local spring constant at the lug attachment root portion of barrel with respect to the longitudinal bending moment (value defined in Reference [6.6.3-4])	—
k_c	Value defined in Reference [6.6.3-4]	—
k_l	Value defined in Reference [6.6.3-4]	—
k_1	Spring constant with respect to inclination of the central axis of the barrel	kgf·mm/rad
k_2	Spring constant with respect to horizontal movement of the central axis of the barrel	kgf/mm
k_3	Deformation spring constant due to flexural and shear [stresses] for the upper portion of the barrel	kgf/mm
k_4	Deformation spring constant due to flexural and shear [stresses] for the lower portion of the barrel	kgf/mm
L_b	Effective length of mounting bolt	mm
M_1, M_2	Vertical moments at the root portion where the lug is attached to the barrel due to horizontal forces F_1 and F_2	kgf·mm
M_3	Torsional moment at the root portion where the lug is attached to the barrel due to horizontal forces F_1 and F_2	kgf·mm
M_c	Circumferential moment at the root portion where the lug is attached to the barrel due to horizontal forces F_1 and F_2	kgf·mm
M_l	Vertical moment at the root portion where the lug is attached to the barrel due to the weight in operation	kgf·mm
N_x	Membrane force generated in axial direction in barrel	kgf/mm
N_ϕ	Membrane force generated in circumferential direction in barrel	kgf/mm
n	Number of mounting bolts for each lug	—
ω	Angular velocity of vibration system	rad/s
Q	Circumferential load at the root portion where the lug is attached to the barrel due to horizontal forces F_1 and F_2	kgf
R	Vertical reaction force at the root portion where the lug is attached to the barrel due to weight in operation	kgf
R_1	Vertical reaction force at the root portion where the lug is attached to the barrel due to horizontal forces F_1 and F_2	kgf
θ	Angle of inclination of the central axis of the barrel due to horizontal forces F_1 and F_2	rad
θ_0	Local angle of inclination of the root portion where the lug is attached to the barrel due to weight in operation	rad

Definitions of symbols (symbols used in the calculation formulas of lug-support vertical cylindrical container) *(Cont'd)*

Symbol	Definition of symbol	Units
θ_1	Local angle of inclination of the root portion where lug 1 is attached to the barrel due to horizontal forces F_1 and F_2	rad
θ_2	Local angle of inclination of the root portion where lug 3 is attached to the barrel due to horizontal forces F_1 and F_2	rad
θ_3	Circumferential torsional angle of lugs 2 and 4 due to horizontal forces F_1 and F_2	rad
θ_{s0}	Angle of inclination with respect to lug foundation due to weight in operation	rad
θ_{s1}	Angle of inclination of lug 1 with respect to the foundation due to horizontal forces F_1 and F_2	rad
θ_{s2}	Angle of inclination of lug 3 with respect to foundation due to horizontal forces F_1 and F_2	rad
r_m	Average radius of barrel	mm
W_1	Effective operational weight of the upper portion above the attachment center of lug	kgf
W_2	Effective operational weight of the lower portion below the attachment center of lug	kgf
Z_{sp}	Lug torsional sectional modulus	mm^3
Z_{sl}	Sectional modulus of lug with respect to the radial axis	mm^3
Z_{st}	Sectional modulus of lug with respect to the circumferential axis	mm^3
Δ_{x1}	Horizontal displacement of the central axis of barrel due to horizontal forces F_1 and F_2	mm
Δ_{x2}	Horizontal displacement due to the flexural and shear deformation of the upper portion of barrel caused by horizontal force F_1	mm
Δ_{x3}	Horizontal displacement due to flexural and shear deformation of the lower portion of barrel caused by horizontal force F_2	mm
δ_{11}	Horizontal displacement of the center of gravity of the upper portion when a unit horizontal force is applied to the center of gravity of the upper portion	mm
δ_{12}	Horizontal displacement of the center of gravity of the upper portion in the case when a unit horizontal force is applied to the center of gravity of the lower portion	mm
δ_{21}	Horizontal displacement of the center of gravity of the lower portion when a unit horizontal force is applied to the center of gravity of the upper portion	mm
δ_{22}	Horizontal displacement of the center of gravity of the lower portion when a unit horizontal force is applied to the center of gravity of the lower portion	mm
ϵ	Restraint coefficient (when rotation of lug is restrained =1; when rotation of lug is not restrained =0)	—
$\sigma_{11} \sim \sigma_{16}$	Combined primary stress in barrel when seismic force acts in the Z-direction	kgf/mm^2
$\sigma_{17} \sim \sigma_{110}$	Combined primary stress in barrel when seismic force acts in the X-direction	kgf/mm^2

<u>Definitions of symbols</u> (symbols used in the calculation formulas of lug-support vertical cylindrical container) *(Cont'd)*

Symbol	Definition of symbol	Units
$\sigma_{b1} \sim \sigma_{b3}$	Tensile stress generated in the mounting bolt due to seismic force in the Z-direction	kgf/mm^2
σ_{b4}, σ_{b5}	Tensile stress generated in the mounting bolt due to seismic force in the X-direction	kgf/mm^2
σ_{s1}	Flexural stress in lug due to weight in operation	kgf/mm^2
$\sigma_{s2} \sim \sigma_{s4}$	Flexural stress in lug due to seismic force in the Z-direction	kgf/mm^2
σ_{s5}, σ_{s6}	Flexural stress in lug due to seismic force in the Z-direction	kgf/mm^2
$\sigma_{1s} \sim \sigma_{3s}$	Combined stress in lug due to seismic force in the Z-direction	kgf/mm^2
σ_{4s}, σ_{5s}	Combined stress in lug due to seismic force in the Z-direction	kgf/mm^2
$\sigma_{0\phi}$	Circumferential primary general membrane stress in barrel	kgf/mm^2
σ_{0x}	Axial primary general film stress in barrel	kgf/mm^2
$\sigma_{xx1} \sim \sigma_{xx6}$	Sum of axial primary stresses in barrel in the case when seismic force acts in the Z-direction	kgf/mm^2
$\sigma_{xz1} \sim \sigma_{xz4}$	Sum of axial primary stresses in barrel in the case when seismic force acts in the Z-direction	kgf/mm^2
$\sigma_{\phi x1} \sim \sigma_{\phi x6}$	Sum of circumferential primary stresses in barrel in the case when seismic force acts in the Z-direction	kgf/mm^2
$\sigma_{\phi z1} \sim \sigma_{\phi z4}$	Sum of circumferential primary stresses in barrel in the case when seismic force acts in the Z-direction	kgf/mm^2
$\sigma_{\phi 1}, \sigma_{x1}$	Circumferential and axial stresses in barrel due to internal pressure on static water head	kgf/mm^2
σ_{x2}	Axial stress in barrel due to weight in operation	kgf/mm^2
$\sigma_{\phi 3}, \sigma_{x3}$	Circumferential and axial stresses in barrel due to vertical moment generated by the weight in operation	kgf/mm^2
σ_{x4}	Axial stress in barrel due to tipping moment in the case when the seismic force acts in the horizontal direction	kgf/mm^2
$\sigma_{\phi 5}, \sigma_{x5}$	Circumferential and axial stresses at the root portion where lug 1 is attached to the barrel due to the vertical moment in the case when seismic force acts in the Z-direction	kgf/mm^2
$\sigma_{\phi 6}, \sigma_{x6}$	Circumferential and axial stresses at the root portion where lug 3 is attached to the barrel due to the vertical moment in the case when seismic force acts in the Z-direction	kgf/mm^2
$\sigma_{\phi 7}, \sigma_{x7}$	Circumferential and axial stresses at the root portion where lugs 2 and 4 are attached to the barrel due to the circumferential moment in the case when seismic force acts in the Z-direction	kgf/mm^2

Definitions of symbols (symbols used in the calculation formulas of lug-support vertical cylindrical container) *(Cont'd)*

Symbol	Definition of symbol	Units
$\sigma_{\phi 8}$, σ_{x8}	Circumferential and axial stresses at the root portion where lugs 1 and 4 are attached to the barrel due to the vertical moment in the case when seismic force acts in the X-direction	kgf/mm^2
$\sigma_{\phi 9}$, σ_{x9}	Circumferential and axial stresses at the root portion where lugs 2 and 3 are attached to the barrel due to the vertical moment when seismic force acts in the X-direction	kgf/mm^2
$\sigma_{\phi 10}$, σ_{x10}	Circumferential and axial stresses at the root poriton where the lug is attached to the barrel due to the circumferential moment in the case when seismic force acts in the X-direction	kgf/mm^2
τ_3	Shear stress due to torsional moment generated at the root portion where the lug is attached to the barrel due to seismic force acting in the Z-direction	kgf/mm^2
τ_6	Shear stress due to torsional moment generated at the root portion where the lug is attached to the barrel due to seismic force acting in the X-direction	kgf/mm^2
τ_{b2}	Shear stress generated in mounting bolt due to seismic force in the Z-direction	kgf/mm^2
τ_{b4}, τ_{b5}	Shear stress generated in mounting bolt due to seismic force in the X-direction	kgf/mm^2
τ_{c1}	Circumferential shear stress generated in the root portion where the lug is attached to the barrel due to seismic force in the Z-direction	kgf/mm^2
τ_{c4}	Circumferential shear stress generated in the root portion where the lug is attached to the barrel due to seismic force in the X-direction	kgf/mm^2
τ_{l1}	Axial shear stress generated in the root portion where the lug is attached on the barrel due to the weight in operation	kgf/mm^2
τ_{l2}	Axial shear stress generated in the root portion where the lug is attached to the barrel due to seismic force acting in the Z-direction	kgf/mm^2
τ_{l5}	Axial shear stress generated in the root portion where the lug is attached to the barrel due to seismic force acting in the X-direction	kgf/mm^2
τ_{s1}	Shear stress in lug due to weight in operation	kgf/mm^2
τ_{s2}, τ_{s3}, τ_{s4}	Shear stress in lug due to seismic force in the Z-direction	kgf/mm^2
τ_{s5}, τ_{s6}	Shear stress in lug due to seismic force in the X-direction	kgf/mm^2

(a) Calculation method of natural period

(i) Calculation model

Under the aforementioned conditions the container is taken as a two-discrete mass system of vibration model with the intermediate portion supported by springs as shown in Figure 6.6.3-32.

(ii) Natural period

The natural period of the two-discrete mass system can be calculated as follows:

$$T = \frac{2\pi}{\omega} \tag{6.6.3-374}$$

$$\frac{W_1 W_2}{g^2}(\delta_{11}\delta_{22} - \delta_{12}\delta_{21})\omega^4 - \frac{1}{g}(\delta_{11}W_1 + \delta_{22}W_2)\omega^2 + 1 = 0 \tag{6.6.3-375}$$

where δ_{11} and δ_{21} are the horizontal displacements of the upper and lower centers of gravity when a unit horizontal force acts on the position of the upper center of gravity of the container; δ_{12} and δ_{22} are the horizontal displacements of the upper and lower centers of gravity when a unit horizontal force acts on the position of the lower center of gravity of the container.

They can be expressed as follows:

$$\delta_{11} = \frac{H_1^2}{k_1} + \frac{1}{k_2} + \frac{1}{k_3} \tag{6.6.3-376}$$

$$\delta_{21} = \delta_{12} = \frac{1}{k_2} - \frac{H_1 H_2}{k_1} \tag{6.6.3-377}$$

$$\delta_{22} = \frac{H_2^2}{k_1} + \frac{1}{k_2} + \frac{1}{k_4} \tag{6.6.3-378}$$

Spring constants k_1, k_2, k_3, and k_4 can be derived as follows:

{1} Spring constant k_1 with respect to the inclination of central axis of barrel

Spring constant k_1 is expressed by the following formula:

$$k_1 = \frac{F_1 H_1 - F_2 H_2}{\theta} \tag{6.6.3-379}$$

where θ is derived as follows by forming the equilibrium equations of loads, moments and displacements with respect to lug, mounting bolts and barrel.

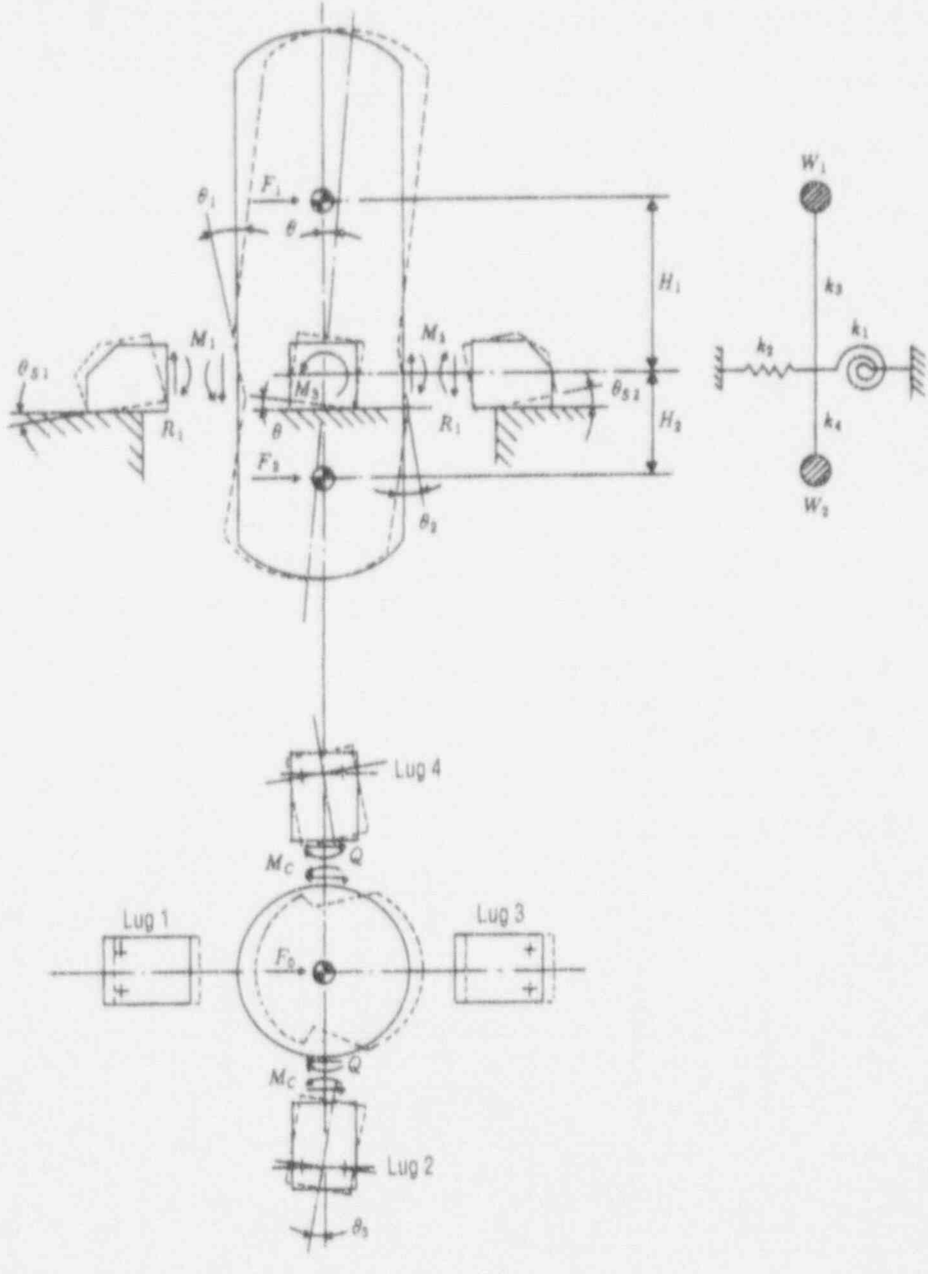


Figure 6.6.3-32. Deformatinon mode.

(a) For loads, moments and displacements of barrel

According to balance of horizontal forces,

$$F_0 = F_1 + F_2 = 2Q \tag{6.6.3-380}$$

According to balance of overturning moments,

$$F_1 H_1 - F_2 H_2 - M_1 - M_2 - 2M_3 - 2R_1 r_m = 0 \tag{6.6.3-381}$$

where r_m is defined as follows:

$$r_m = (D_i + t)/2 \tag{6.6.3-382}$$

The local inclination angles at the root portions where lugs 1 and 3 are attached to the barrel due to overturning moment can be derived as follows from the values (denoted by *) derived from the table in Reference [6.6.3-4] using the shell parameter γ and attachment parameter β.

$$\gamma = r_m / t \tag{6.6.3-383}$$

$$\beta_1 = C_1 / r_m \tag{6.6.3-384}$$

$$\beta_2 = C_2 / r_m \tag{6.6.3-385}$$

$$\beta = k_l^* \sqrt[3]{\beta_1 \beta_2^2} \tag{6.6.3-386}$$

The local angles of inclination can be derived using the following formulas:

$$\theta_1 = \frac{M_1 K_l^*}{r_m^3 \beta^2 E} \tag{6.6.3-387}$$

$$\theta_2 = \frac{M_2 K_l^*}{r_m^3 \beta^2 E} \tag{6.6.3-388}$$

(b) For lug 1, when the inclination is as shown in Figure 6.6.3-33, from the balance of moments, we have

$$F_{12} a - F_{11}(a - b) + M_1 = 0 \tag{6.6.3-389}$$

When $F_1 H_1 < F_2 H_2$, we have

$$F_{12}(a - b - c) - F_{11}(a - b) + M_1 = 0 \tag{6.6.3-390}$$

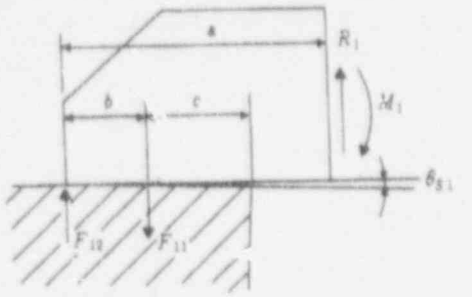


Figure 6.6.3-33. Moments and forces acting on lug 1.

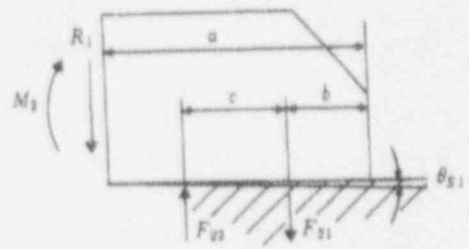


Figure 6.6.3-34. Moments and forces acting on lug 3.

From the balance condition for vertical forces, we have

$$F_{12} - F_{11} + R_1 = 0 \tag{6.6.3-391}$$

{c} For lug 3, when the inclination is as shown in Figure 6.6.3-34, from the balance of moments, we have

$$F_{21}(a-b) - F_{22}(a-b-c) + M_2 = 0 \tag{6.6.3-392}$$

When $F_1H_1 < F_2F_2$, we have

$$F_{21}(a-b) - F_{22}a + M_2 = 0 \tag{6.6.3-393}$$

From the balance condition for vertical forces, we have

$$F_{22} - F_{21} - R_1 = 0 \tag{6.6.3-394}$$

{d} For lugs 2 and 4, when the inclination is as shown in Figure 6.6.3-35, from the balance of moments, we have

$$-F_{31}\frac{d}{2} + F_{32}\frac{d}{2} - (F_{31} + F_{32})e + M_3 = 0 \tag{6.6.3-395}$$

From the relation between stretching forces of mounting bolts, we have

$$\frac{F_{31}}{e + \dfrac{d}{2}} = \frac{F_{32}}{e - \dfrac{d}{2}} \tag{6.6.3-396}$$

The inclination angle with respect to the foundation of the lug can be derived from the balance condition of elongation and force of the mounting bolt (see Figure 6.6.3-36).

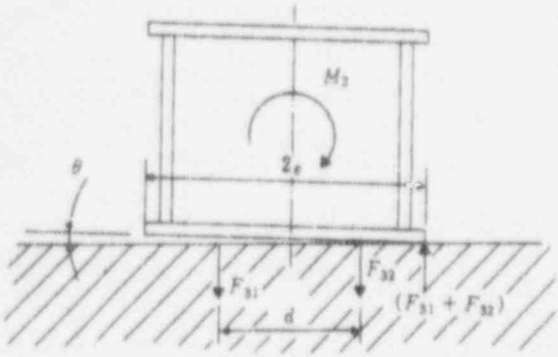


Figure 6.6.3-35. Moments and forces acting on lugs 2 and 4.

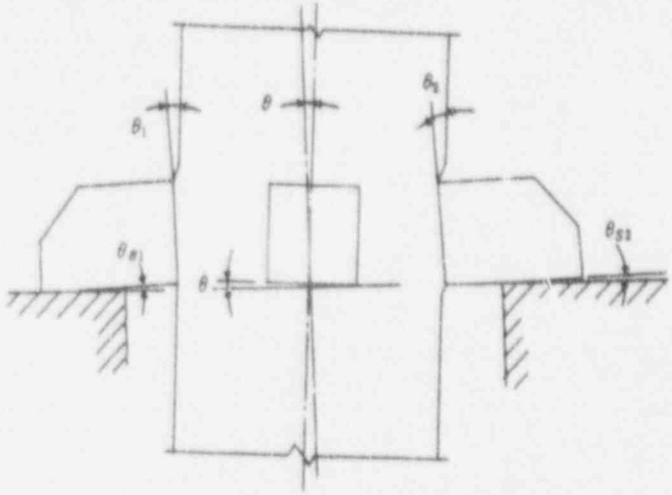


Figure 6.6.3-36. Inclination angles of barrel and lugs due to overturning moment.

For lug 1,

$$\theta_{s1} = \frac{F_{11}L_b}{nA_{be}E_b b} \tag{6.6.3-397}$$

For lug 3,

$$\theta_{s2} = \frac{F_{21}L_b}{nA_{be}E_b c} \tag{6.6.3-398}$$

For lugs 2 and 4,

$$\theta = \frac{F_{31}L_b}{A_{be}E_b\left(e + \frac{d}{2}\right)} \tag{6.6.3-399}$$

The following relations are established among inclination angle of the barrel's central axis, local inclination angles θ_1 and θ_2 at the root portions where lugs are attached to the barrel, and inclination angles θ_{S1} and θ_{S2} of lugs with respect to the foundation:

$$\theta_{s1} - \theta_1 + \theta = 0 \quad (6.6.3\text{-}400)$$

$$\theta_{s2} - \theta_2 + \theta = 0 \quad (6.6.3\text{-}401)$$

$$a\theta_{s1} - 2r_m\theta + (a-b-c)\theta_{s2} = 0 \quad (6.6.3\text{-}402)$$

However, when $F_1H_1 < F_2H_2$,

$$(a-b-c)\theta_{s1} - 2r_m\theta + a\theta_{s2} = 0 \quad (6.6.3\text{-}403)$$

The inclination angle θ of the central axis of the barrel can be derived by solving the aforementioned set of equations (see Figure 6.6.3-37).

{2} Spring constant k_2 with respect to horizontal movement of barrel

Spring constant k_2 can be expressed as follows:

$$k_2 = \frac{F_0}{\Delta_{x1}} \quad (6.6.3\text{-}404)$$

where Δ_{x1} is defined as

$$\Delta_{x1} = (a-b)\theta_3 \quad (6.6.3\text{-}405)$$

θ_3 is derived as follows.

From the balance of bending moments of lugs 2 and 4, we have

$$M_c = Q(a-b)(1-\epsilon) \quad (6.6.3\text{-}406)$$

The local inclination angles of the root portions where lugs 2 and 4 are attached to the barrel due to the horizontal forces can similarly be calculated using the following formula form the value (denoted by asterisk) derived from the table in Reference [6.6.3-4] using the shell parameter γ and attachment parameter β.

$$\theta_3 = \frac{M_c K_c^*}{r_m^3 \beta^2 E} \quad (6.6.3\text{-}407)$$

where β is defined as follows:

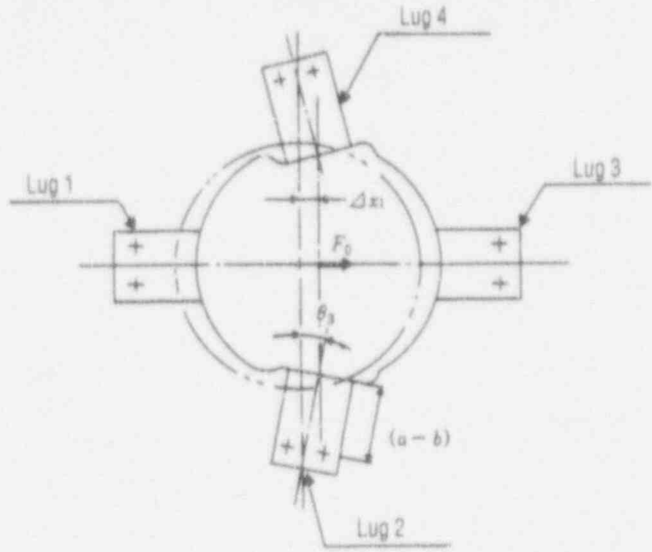


Figure 6.6.3-37. Displacement of central axis of barrel due to horizontal force.

$$\beta = k_c^* \sqrt[3]{\beta_1^2 \beta_2} \tag{6.6.3-408}$$

{3} The deformation spring constant k_3 due to the flexural and shear stresses in the upper portion of the barrel caused by horizontal force can be represented by the following formula:

$$k_3 = \frac{F_1}{\Delta_{x2}} \tag{6.6.3-409}$$

where, Δ_{x2} is defined as

$$\Delta_{x2} = \frac{F_1 H_1^3}{3EI} + \frac{F_1 H_1}{GA_s} \tag{6.6.3-410}$$

The sectional properties of the barrel are represented as follows:

$$I = \frac{\pi}{8}(D_i + t)^3 t \tag{6.6.3-411}$$

$$A_s = \frac{2}{3}\pi(D_i + t)t \tag{6.6.3-412}$$

{4} The deformation spring constant k_4 due to the flexural and shear stresses in the lower portion of the barrel caused by horizontal force can be represented as follows:

$$k_4 = \frac{F_2}{\Delta_{x3}} \tag{6.6.3-413}$$

where Δ_{x3} is defined as follows:

$$\Delta_{x3} = \frac{F_2 H_2^3}{3EI} + \frac{F_2 H_2}{GA_e} \tag{6.6.3-414}$$

(b) Calculation method of stresses

(i) Stresses in barrel

{1} Stress due to static water head or internal pressure

In the case of static water head

$$\sigma_{\phi 1} = \frac{\rho' H D_i}{2t} \tag{6.6.3-415}$$

$$\sigma_{x1} = 0 \tag{6.6.3-416}$$

In the case of internal pressure

$$\sigma_{\phi 1} = \frac{P_r(D_i + 1.2t)}{200t} \tag{6.6.3-417}$$

$$\sigma_{x1} = \frac{P_r(D_i + 1.2t)}{400t} \tag{6.6.3-418}$$

{2} Stress due to weight in operation

$$\sigma_{x2} = \frac{W_0}{\pi t(D_i + t)} \tag{6.6.3-419}$$

{3} Stress at the root portion where the lug is attached to the barrel due to weight in operation

According to the balance condition for forces in the vertical direction due to weight W_0 in operation, we have

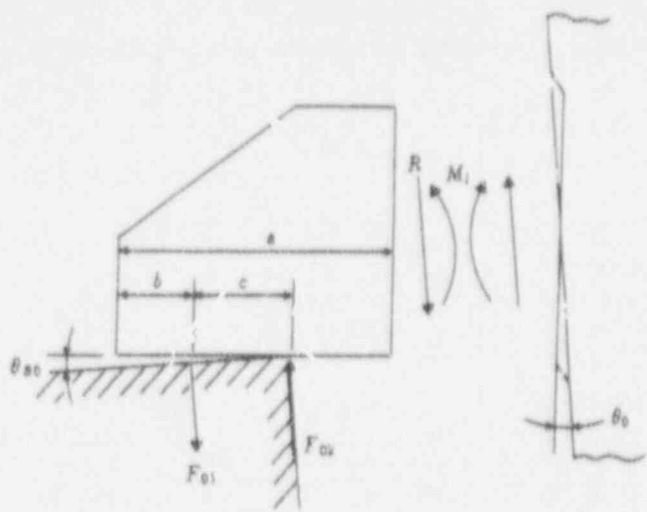


Figure 6.6.3-38. Moments and forces acting on barrel and lugs due to vertical load.

$$4R - W_0 = 0 \tag{6.6.3-420}$$

When the lug inclines as shown in Figure 6.6.3-38, from the balance conditions for moments and forces, we have

$$F_{02}(a-b-c) - F_{01}(a-b) - M_l = 0 \tag{6.6.3-421}$$

$$F_{02} - F_{01} - R = 0 \tag{6.6.3-422}$$

Just as in (a)(ii), the local angle of inclination of lug due to the weight in operation can be derived using the following formula:

$$\theta_0 = \frac{M_l K_l^*}{r_m^3 \beta^2 E} \tag{6.6.3-423}$$

Just as in (a)(ii), the angle of inclination of the lug with respect to the foundation due to the weight in operation can be derived as follows:

$$\theta_{s0} = \frac{F_{01} L_b}{n A_{bw} E_b C} \tag{6.6.3-424}$$

Local angle of inclination in θ_0 of the lug-attachment root portion is equal to lug's angle of inclination θ_{s0}:

$$\theta_0 = \theta_{s0} \tag{6.6.3-425}$$

The aforementioned set of equations can be solved to obtain R, M_l and F_{01} as follows:

$$R = \frac{W_0}{4} \tag{6.6.3-426}$$

$$M_l = \frac{R(a-b-c)}{1 + \dfrac{nA_{bw}E_bK_l^*c^2}{r_m^3\beta^2 EL_b}} \tag{6.6.3-427}$$

$$F_{01} = \frac{R(a-b-c) - M_l}{c} \tag{6.6.3-428}$$

The local stresses in the barrel generated by vertical flexural moment M_l can be calculated using the following formulas from the values in the table of Reference [6.6.3-2] according to shell parameter γ and attachment parameter β:

$$\sigma_{\phi 3} = \left(\frac{N_\phi}{M_l/(r_m^2\beta)}\right)^* \cdot \left(\frac{M_l}{r_m^2 t\beta}\right) C_l^* \tag{6.6.3-429}$$

$$\sigma_{x3} = \left(\frac{N_x}{M_l/(r_m^2\beta)}\right)^* \cdot \left(\frac{M_l}{r_m^2 t\beta}\right) C_l^* \tag{6.6.3-430}$$

where β is defined as follows:

$$\beta = \sqrt[3]{\beta_1\beta_2^2} \tag{6.6.3-431}$$

The shear stress due to reaction force R is

$$\tau_{ll} = \frac{R}{4C_2 t} \tag{6.6.3-432}$$

(4) The flexural stress in the barrel due to the horizontal seismic force is as follows:

$$\sigma_{x4} = \frac{C_H W_1 H_1 (D_i + 2t)}{2I} \tag{6.6.3-433}$$

However, if $W_1H_1 < W_2H_2$, W_1H_1 should be replaced by W_2H_2

(5) Stresses at the root portion where the lug is attached to the barrel due to seismic force in the Z-direction

In the formula for calculating the natural period in (a), horizontal force F_1 is replaced by $C_H W_1$, and F_2 is replaced by $C_H W_2$. The obtained value is used in this case.

Just as in (i)(3), the local stresses in the barrel generated by vertical bending moments M_1 and M_2 can be calculated as follows:

$$\sigma_{\phi 5} = \left(\frac{N_\phi}{M_1/(r_m^2\beta)}\right)^* \cdot \left(\frac{|M_1|}{r_m^2 t\beta}\right) C_l^* \qquad (6.6.3\text{-}434)$$

$$\sigma_{x5} = \left(\frac{N_x}{M_1/(r_m^2\beta)}\right)^* \cdot \left(\frac{|M_1|}{r_m^2 t\beta}\right) C_l^* \qquad (6.6.3\text{-}435)$$

$$\sigma_{\phi 6} = \left(\frac{N_\phi}{M_2/(r_m^2\beta)}\right)^* \cdot \left(\frac{|M_2|}{r_m^2 t\beta}\right) C_l^* \qquad (6.6.3\text{-}436)$$

$$\sigma_{x6} = \left(\frac{N_x}{M_2/(r_m^2\beta)}\right)^* \cdot \left(\frac{|M_2|}{r_m^2 t\beta}\right) C_l^* \qquad (6.6.3\text{-}437)$$

where β is defined as follows:

$$\beta = \sqrt[3]{\beta_1 \beta_2^2} \qquad (6.6.3\text{-}438)$$

The local stresses generated in the barrel due to circumferential bending moment M_C are as follows:

$$\sigma_{\phi 7} = \left(\frac{N_\phi}{M_c/(r_m^2\beta)}\right)^* \cdot \left(\frac{|M_c|}{r_m^2 t\beta}\right) C_c^* \qquad (6.6.3\text{-}439)$$

$$\sigma_{x7} = \left(\frac{N_x}{kl_c/(r_m^2\beta)}\right)^* \cdot \left(\frac{|M_c|}{r_m^2 t\beta}\right) C_c^* \qquad (6.6.3\text{-}440)$$

where β is defined as follows:

$$\beta = \sqrt[3]{\beta_1^2 \beta_2} \qquad (6.6.3\text{-}441)$$

The shear stress due to the circumferential shear force Q is

$$\tau_{cl} = \frac{|Q|}{4C_1 t} \tag{6.6.3-442}$$

The shear stress due to the vertical shear force R_1 is

$$\tau_{l2} = \frac{|R_1|}{4C_2 t} \tag{6.6.3-443}$$

The local shear stress generated in the barrel due to torsional moment M_3 is

$$\tau_3 = \frac{|M_3|}{2\pi C_1^2 t} \tag{6.6.3-444}$$

When $C_1 > C_2$, however, C_1 should be replaced by C_2.

{6} Stresses due to seismic force in the X-direction

The values obtained by multiplying the right-hand sides of equations (6.6.3-436) through (6.6.3-437) and (6.6.3-439) through (6.6.3-440) by $1/\sqrt{2}$ are used for $\sigma_{\phi 8}$ and σ_{x8} in the case of vertical bending moment M_1, $\sigma_{\phi 9}$ and σ_{x9} in the case of vertical bending moment M_2, and $\sigma_{\phi 10}$ and σ_{x10} in the case of circumferential bending moment. Also, the values obtained by multiplying the right-hand sides of equations (6.6.3.442) through (6.6.3-444) by $1/\sqrt{2}$ are used for τ_{C4} in the case of circumferential shear force, τ_{l5} in the case of vertical shear force, and τ_6 in the case of torsional moment.

{7} Combinations of stresses

The stresses generated at the root portion where the lug is attached to the barrel calculated in {1}–{6} are combined as follows.

{a} Primary general membrane stress

$$\sigma_0 = \max(\sigma_{0\phi}, \sigma_{0x}) \tag{6.6.3-445}$$

$$\sigma_{0\phi} = \sigma_{\phi 1} \tag{6.6.3-446}$$

$$\sigma_{0x} = \sigma_{x1} + \sigma_{x2} + \sigma_{x4} \tag{6.6.3-447}$$

{b} Primary stress when seismic force acts in the Z-direction

A. At the root portion where lug 1 is attached (see Figure 6.6.3-39)

For evaluation point No. 1, we have

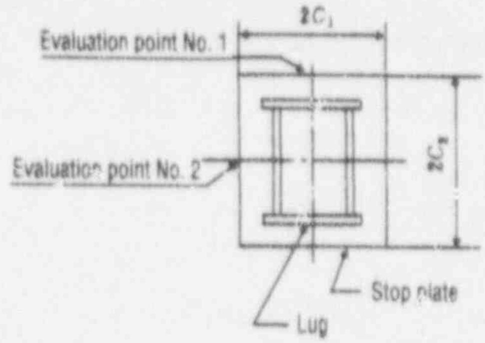


Figure 6.6.3-39. Evaluation points in barrel.

$$\sigma_{\phi c1} = \sigma_{\phi 1} + \sigma_{\phi 3} + \sigma_{\phi 5} \tag{6.6.3-448}$$

$$\sigma_{xc1} = \sigma_{x1} + \sigma_{x2} + \sigma_{x3} + \sigma_{x4} + \sigma_{x5} \tag{6.6.3-449}$$

$$\sigma_{11} = \frac{1}{2}\left\{\sigma_{\phi c1} + \sigma_{xc1} + \sqrt{\left(\sigma_{\phi c1} - \sigma_{xc1}\right)^2}\right\} \tag{6.6.3-450}$$

For evaluation point No. 2, we have

$$\sigma_{\phi c2} = \sigma_{\phi 1} \tag{6.6.3-451}$$

$$\sigma_{xc2} = \sigma_{x1} + \sigma_{x2} + \sigma_{x4} \tag{6.6.3-452}$$

$$\sigma_{12} = \frac{1}{2}\left\{\sigma_{\phi c2} + \sigma_{xc2} + \sqrt{\left(\sigma_{\phi c2} - \sigma_{xc2}\right)^2 + 4\left(\tau_{l1} + \tau_{l2}\right)^2}\right\} \tag{6.6.3-453}$$

B. At the root portions where lugs 2 and 4 are attached

For evaluation point No. 1, we have

$$\sigma_{\phi c3} = \sigma_{\phi 1} + \sigma_{\phi 3} \tag{6.6.3-454}$$

$$\sigma_{xc3} = \sigma_{x1} + \sigma_{x2} + \sigma_{x3} \tag{6.6.3-455}$$

$$\sigma_{13} = \frac{1}{2}\left\{\sigma_{\phi c3} + \sigma_{xc3} + \sqrt{\left(\sigma_{\phi c3} - \sigma_{xc3}\right)^2 + 4\left(\tau_{c1} + \tau_3\right)^2}\right\} \tag{6.6.3-456}$$

For evaluation point No. 2, we have

$$\sigma_{\phi c4} = \sigma_{\phi 1} + \sigma_{\phi 7} \tag{6.6.3-457}$$

$$\sigma_{xc4} = \sigma_{x1} + \sigma_{x2} + \sigma_{x7} \tag{6.6.3-458}$$

$$\sigma_{14} = \frac{1}{2}\left\{\sigma_{\phi c4} + \sigma_{xc4} + \sqrt{\left(\sigma_{\phi c4} - \sigma_{xc4}\right)^2 + 4\left(\tau_{l1} + \tau_3\right)^2}\right\} \tag{6.6.3-459}$$

C. At the root portion where lug 3 is attached

At evaluation point No. 1, we have

$$\sigma_{\phi x5} = \sigma_{\phi 1} + \sigma_{\phi 3} + \sigma_{\phi 6} \qquad (6.6.3\text{-}460)$$

$$\sigma_{xx5} = \sigma_{x1} + \sigma_{x2} + \sigma_{x3} + \sigma_{x4} + \sigma_{x6} \qquad (6.6.3\text{-}461)$$

$$\sigma_{15} = \frac{1}{2}\left\{\sigma_{\phi x5} + \sigma_{xx5} + \sqrt{\left(\sigma_{\phi x5} - \sigma_{xx5}\right)^2}\right\} \qquad (6.6.3\text{-}462)$$

At evaluation point No. 2, we have

$$\sigma_{\phi x6} = \sigma_{\phi 1} \qquad (6.6.3\text{-}463)$$

$$\sigma_{xx6} = \sigma_{x1} + \sigma_{x2} + \sigma_{x4} \qquad (6.6.3\text{-}464)$$

$$\sigma_{16} = \frac{1}{2}\left\{\sigma_{\phi x6} + \sigma_{xx6} + \sqrt{\left(\sigma_{\phi x6} - \sigma_{xx6}\right)^2 + 4\left(\tau_{l1} + \tau_{l2}\right)^2}\right\} \qquad (6.6.3\text{-}465)$$

(c) Combined primary stress when seismic force acts in the X-direction

A. At the root portions where lugs 1 and 4 are attached

At evaluation point No. 1, we have

$$\sigma_{\phi x1} = \sigma_{\phi 1} + \sigma_{\phi 3} + \sigma_{\phi 10} \qquad (6.6.3\text{-}466)$$

$$\sigma_{xx1} = \sigma_{x1} + \sigma_{x2} + \sigma_{x3} + \sigma_{x4} + \sigma_{x6} \qquad (6.6.3\text{-}467)$$

$$\sigma_{17} = \frac{1}{2}\left\{\sigma_{\phi x1} + \sigma_{xx1} + \sqrt{\left(\sigma_{\phi x1} - \sigma_{xx1}\right)^2 + 4\left(\tau_{c4} + \tau_{6}\right)^2}\right\} \qquad (6.6.3\text{-}468)$$

At evaluation point No. 2, we have

$$\sigma_{\phi x2} = \sigma_{\phi 1} + \sigma_{\phi 10} \qquad (6.6.3\text{-}469)$$

$$\sigma_{xx2} = \sigma_{x1} + \sigma_{x2} + \sigma_{x4} + \sigma_{x10} \qquad (6.6.3\text{-}470)$$

$$\sigma_{18} = \frac{1}{2}\left\{\sigma_{\phi x2} + \sigma_{xx2} + \sqrt{\left(\sigma_{\phi x2} - \sigma_{xx2}\right)^2 + 4\left(\tau_{l1} + \tau_{l3} + \tau_{6}\right)^2}\right\} \qquad (6.6.3\text{-}471)$$

B. At the root portions where lugs 2 and 3 are attached

At evaluation point No. 1, we have

$$\sigma_{\phi x3} = \sigma_{\phi 1} + \sigma_{\phi 3} + \sigma_{\phi 9} \tag{6.6.3-472}$$

$$\sigma_{xx3} = \sigma_{x1} + \sigma_{x2} + \sigma_{x3} + \sigma_{x4} + \sigma_{x9} \tag{6.6.3-473}$$

$$\sigma_{19} = \frac{1}{2}\left\{\sigma_{\phi x3} + \sigma_{xx3} + \sqrt{\left(\sigma_{\phi x3} - \sigma_{xx3}\right)^2 + 4\left(\tau_{c4} + \tau_6\right)^2}\right\} \tag{6.6.3-474}$$

At evaluation point No. 2, we have

$$\sigma_{\phi x4} = \sigma_{\phi 1} + \sigma_{\phi 10} \tag{6.6.3-475}$$

$$\sigma_{xx4} = \sigma_{x1} + \sigma_{x2} + \sigma_{x4} + \sigma_{x10} \tag{6.6.3-476}$$

$$\sigma_{110} = \frac{1}{2}\left\{\sigma_{\phi x4} + \sigma_{xx4} + \sqrt{\left(\sigma_{\phi x4} - \sigma_{xx4}\right)^2 + 4\left(\tau_{l1} + \tau_{l5} + \tau_6\right)^2}\right\} \tag{6.6.3-477}$$

(ii) Stresses in lugs

{1} Stresses due to weight during operation

$$\sigma_{s1} = \frac{M_l}{Z_{s}} \tag{6.6.3-478}$$

$$\tau_{s1} = \frac{R}{A_{s1}} \tag{6.6.3-479}$$

{2} Stresses due to seismic force in the Z-direction

For lug 1:

$$\sigma_{s2} = \frac{|M_1|}{Z_{s}} \tag{6.6.3-480}$$

$$\tau_{s2} = \frac{|R_1|}{A_{s1}} \tag{6.6.3-481}$$

For lugs 2 and 4,

$$\sigma_{s3} = \frac{|M_c|}{Z_{sl}} \tag{6.6.3-482}$$

$$\tau_{s3} = \frac{|M_3|}{Z_{sp}} + \frac{|Q|}{A_{s2}} \tag{6.6.3-483}$$

For lug 3,

$$\sigma_{g4} = \frac{|M_2|}{Z_{g2}} \tag{6.6.3-484}$$

$$\tau_4 = \frac{|R_1|}{A_{g1}} \tag{6.6.3-485}$$

{3} Stresses due to seismic force in the X-direction

For lugs 1 and 4,

$$\sigma_{g5} = \frac{|M_1|}{\sqrt{2}Z_{g1}} + \frac{|M_c|}{\sqrt{2}Z_{g1}} \tag{6.6.3-486}$$

$$\tau_{g5} = \frac{|R_1|}{\sqrt{2}A_{g1}} + \frac{|M_3|}{\sqrt{2}Z_{gp}} + \frac{|Q|}{\sqrt{2}A_{g2}} \tag{6.6.3-487}$$

For lugs 2 and 3,

$$\sigma_{g6} = \frac{|M_2|}{\sqrt{2}Z_{g2}} + \frac{|M_c|}{\sqrt{2}Z_{g1}} \tag{6.6.3-488}$$

$$\tau_{g6} = \frac{|R_1|}{\sqrt{2}A_{g1}} + \frac{|M_3|}{\sqrt{2}Z_{gp}} + \frac{|Q|}{\sqrt{2}A_{g2}} \tag{6.6.3-489}$$

{4} Combinations of stresses

The maximum stresses in the lugs are as follows. When the seismic force acts in the Z-direction,

For lug 1,

$$\sigma_{1s} = \sqrt{(\sigma_{g1} + \sigma_{g2})^2 + 3(\tau_{g1} + \tau_{g2})^2} \tag{6.6.3-490}$$

For lugs 2, 4,

$$\sigma_{2s} = \sqrt{(\sigma_{g1} + \sigma_{g3})^2 + 3(\tau_{g1} + \tau_{g3})^2} \tag{6.6.3-491}$$

For lug 3,

$$\sigma_{s} = \sqrt{(\sigma_{g1} + \sigma_{g4})^2 + 3(\tau_{g1} + \tau_{g4})^2} \tag{6.6.3-492}$$

When the seismic force acts in the X-direction,

For lugs 1 and 4,

$$\sigma_{4s} = \sqrt{(\sigma_{g1} + \sigma_{g5})^2 + 3(\tau_{g1} + \tau_{g5})^2} \tag{6.6.3-493}$$

For lugs 2 and 3,

$$\sigma_{5s} = \sqrt{(\sigma_{g1} + \sigma_{g6})^2 + 3(\tau_{g1} + \tau_{g6})^2} \tag{6.6.3-494}$$

(iii) Stresses in mounting bolts

The stresses in the mounting bolts can be calculated as follows.

{1} When the seismic force acts in the Z-direction

For lug 1,

$$\sigma_{b1} = \frac{|F_{11}| + F_{01}}{nA_b} \tag{6.6.3-495}$$

For lugs 2 and 4,

$$\sigma_{b2} = \frac{|F_{31}|}{A_b} + \frac{F_{01}}{nA_b} \tag{6.6.3-496}$$

$$\tau_{b2} = \frac{|Q|(1 - \epsilon)}{nA_b} \tag{6.6.3-497}$$

For lug 3,

$$\sigma_{b3} = \frac{|F_{21}| + F_{01}}{nA_b} \tag{6.6.3-498}$$

{2} When the seismic force acts in the X-direction

For lugs 1 and 4,

$$\sigma_{b4} = \frac{|F_{11}| + F_{01}}{\sqrt{2}nA_b} + \frac{|F_{31}|}{\sqrt{2}A_b} \tag{6.6.3-499}$$

$$\tau_{b4} = \frac{|Q|(1-\epsilon)}{\sqrt{2}nA_b} \tag{6.6.3-500}$$

For lugs 2 and 3,

$$\sigma_{b5} = \frac{|F_{21}| + F_{01}}{\sqrt{2}nA_b} + \frac{|F_{31}|}{\sqrt{2}A_b} \tag{6.6.3-501}$$

$$\tau_{b6} = \frac{|Q|(1-\epsilon)}{\sqrt{2}nA_b} \tag{6.6.3-502}$$

(c) Evaluation methods

(i) Evaluation of natural period

From the natural period derived in (a), the design seismic coefficient in the horizontal direction is confirmed.

(ii) Evaluation of stresses (see Table 6.6.3-3)

{1} Evaluation of stresses in barrel
It is performed according to section "6.6.3(1)b. Skirt-support vertical cylindrical container."

{2} Evaluation of stresses in lugs
Evaluation is made of the combined stress in lugs derived in (b)(ii)(4) according to section "6.6.3(1)b. Skirt-support vertical cylindrical container." Evaluation of buckling, however, is excluded.

{3} Evaluation of stresses in mounting bolts
It is performed according to section "6.6.3(1)b. Skirt-support vertical cylindrical container."

(2) Piping

a. Basic procedures of aseismic design

Piping is designed using appropriate design methods in consideration of size, temperature of application, etc. Table 6.6.3-4 lists the classes of piping and the appropriate standard design methods. The aforementioned methods are standard design methods. If needed, dynamic analysis should be performed. Figure 6.6.3-40 shows an example of a portion of the procedure used in aseismic design of piping.

Table 6.6.3-3. Example of stress evaluation results of four-leg vertical cylindrical container.

1. Design conditions

Equipment name	Importance class in aseismic design	Installation location and floor height (m)	Natural period (s)	Design seismic coefficient in horizontal direction	Design seismic coefficient in vertical direction	Highest pressure of application (kgf/cm^2)	Highest temperature of application (°C)	Specific gravity
Four-leg cylindrical tank	B	Nuclear reactor building E.L. 0.0*	0.046	C_H=0.29	—	Static water head	66	100

*Standard floor level

2. Main features of equipment

W_0 (kgf)	D_i (mm)	t (mm)	E (kgf/mm^2)	E_s (kgf/mm^2)	G (kgf/mm^2)	G_s (kgf/mm^2)	l (mm)	l_c (mm)	l_g (mm)	H (mm)	K_c (—)	K_l (—)	K_r (—)
5300	1750	6	19600	19600	7530	7530	645	1846	1120	1160	18090	4970	20360

C_1 (mm)	C_2 (mm)	I_{sf} (mm^4)	I_{sr} (mm^4)	I_{st} (mm^4)	A_f (mm^2)	A_s (mm^2)	A_{sf} (mm^2)	A_{sr} (mm^2)	A_{s1} (mm^2)	A_{t1} (mm^2)	A_{t2} (mm^2)	Z_{sp} (mm^3)	Z_{sr} (mm^3)
130	130	1.057×10^6	2.462×10^7	3.134×10^6	1.215×10^3	3.814×10^3	1.374×10^3	2.150×10^3	1.567×10^3	1.672×10^3	1.375×10^3	1.312×10^4	2.462×10^5

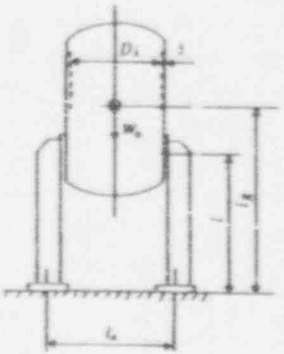
Z_{s1} (mm^3)	J_s (mm^4)	h (mm)	s (—)	n (—)	n_1 (—)	n_2 (—)	a (mm)	b (mm)	A_b (mm^2)	d_1 (mm)	d_2 (mm)	F (leg) (kgf/mm^2)	F (foundation bolt) (kgf/mm^2)
5.323×10^6	1.771×10^5	200	15	2	2	1	150	340	314.1 (M20)	75	30	24.6	23.6

Table 6.6.3-3 (Cont'd). Example of stress evaluation results of four-leg vertical cylindrical container.

3. Evaluation results

(Units: kgf/mm^2)

Part	Material	Stress	Calculated stress	Allowable stress
Barrel shell	SUS304	Primary general membrane stress	$\sigma_0 = 0.3$	$S_0 = 19.2$
		Primary stress	$\sigma_1 = 2.1$	$S_0 = 19.2$
Legs	SS41	Combined stress	$\sigma_s = 2.8$	$f_t = 24.6$
		Compressive/flexural combined stress (buckling evaluation)	$\frac{\sigma_{sr}}{f_{br}} + \frac{\sigma_{st}}{f_{bt}} + \frac{\sigma_{sc}}{f_c} \leq 1$	
			0.11	
Anchor bolts	SS41	Tensile stress	$\sigma_b = 1.3$	f_{tb} = [illegible].7
		Shear stress	$\tau_b = 1.2$	f_{sb} [illegible].6



Since all stresses are below allowable stresses, the system is safe.

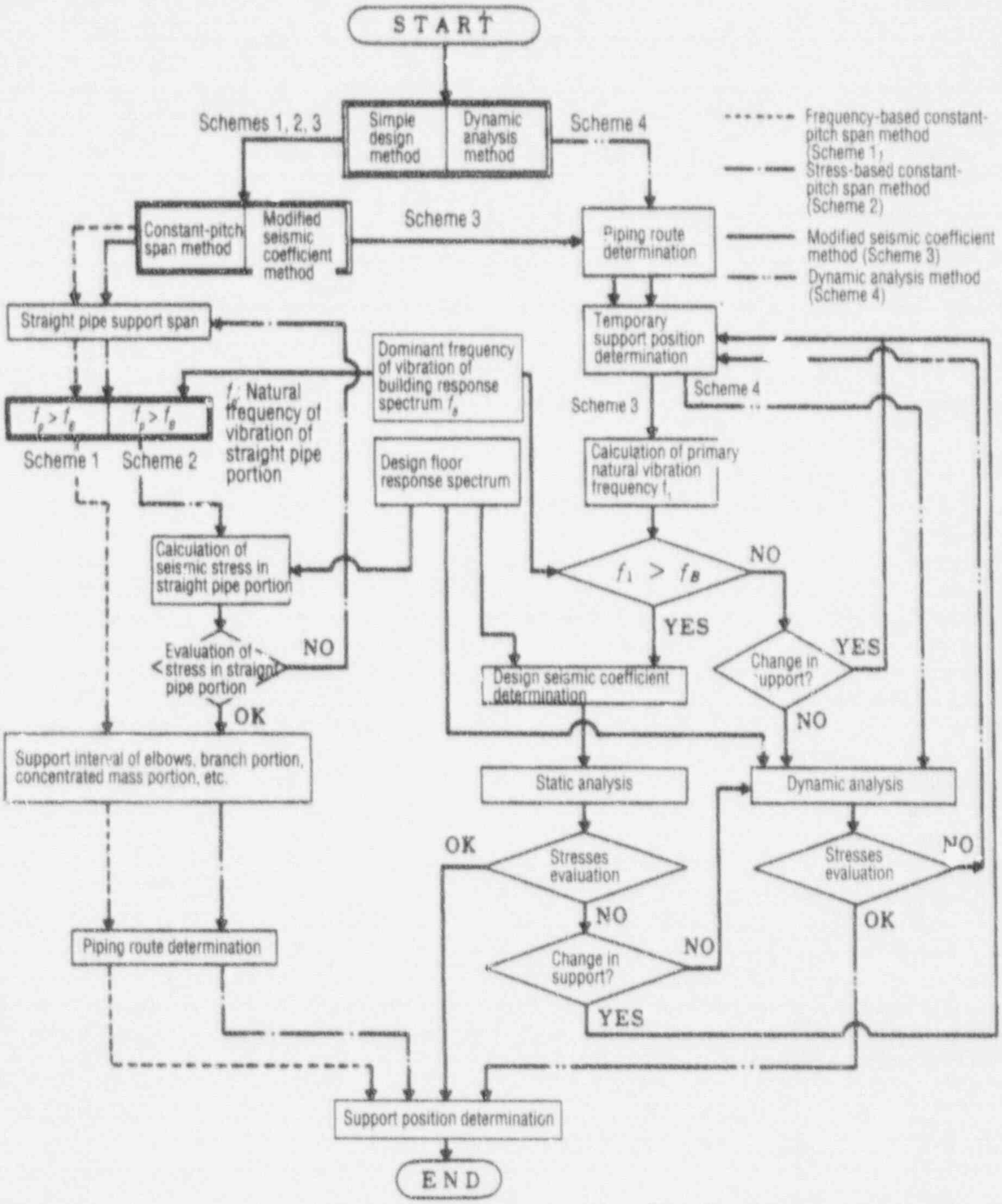


Figure 6.6.3-40. Design procedure in Class B piping simple design method and dynamic analysis method.

Tabale 6.6.3-4. Standard design methods of pipings.

Piping class		Standard design method	
Operating temperature	Diameter	Dynamic analysis method	Simple design method
High	Large	□	○
	Small	□	○
Low			○

○ Design method used in principle.

□ Dynamic analysis is performed when it is difficult to apply the simple design method (Class C piping not included).

b. Determination of support points [H-K-7]

Among the simple design methods, for the vibration frequency-based constant-pitch span method and the stress-based constant-pitch span method, support points are determined with the support interval determined beforehand according to natural frequency of vibration, piping diameter, etc. For the special portions, such as bend portion, concentrated mass portion, branch portion, etc., their specific characteristics are considered when the support span is determined. Among the simple design methods, for the modified seismic coefficient method and dynamic analysis method, the piping system is represented by a multiple discrete mass system model; first, temporary support positions are set; then, the primary natural frequency of vibration is determined for the modified seismic coefficient method, and several higher-order natural frequencies of vibration are determined for the dynamic analysis method. The conventional eigenvalue analysis method is used in this scheme. Trial-and-error is performed until the stress evaluation condition is satisfied at the temporary support positions. In this way, the final support positions are determined.

(a) Frequency-based constant-pitch span method

(i) Guideline

{1} In order to prevent excessive vibration of the piping system due to earthquake, the support intervals of the piping system should be smaller than the standard support span determined beforehand on the base of the standard frequency of vibration.

{2} The standard frequency of vibration is determined at the sufficiently safe side for the stress generated in the piping in earthquake with respect to the allowable value.

(ii) Support of straight pipe portion

{1} Support in direction perpendicular to the piping axis

The relation between pipe diameter and length in the case when the two ends are assumed to be simply supported is determined to ensure that the primary natural frequency of vibration becomes the standard frequency of vibration. The actual support span should be smaller than the support span determined in this case. As an example, Table 6.6.3-5 shows the standard support span when the standard vibration frequency is 20 Hz.

Table 6.6.3-5. Examples of standard support interval when the two ends are simply supported (standard vibration frequency: 20 Hz) [Units: m].

Nominal diameter (A)	With insulation		Without insulation
	Water	Steam	Water
15 A	1.36	1.39	1.56
20 A	1.56	1.60	1.75
25 A	1.78	1.85	1.95
40 A	2.16	2.28	2.31
50 A	2.39	2.53	2.56
65 A	2.74	2.91	2.89
80 A	2.96	3.17	3.10
100 A	3.34	3.64	3.48
150 A	3.99	4.45	4.11
200 A	4.54	5.14	4.65
250 A	5.01	5.72	5.12
300 A	5.44	6.27	5.54
350 A	5.70	6.59	5.83
400 A	6.11	7.08	6.23
450 A	6.50	7.54	6.61
500 A	6.82	7.97	6.93
550 A	7.07	8.37	7.18
600 A	7.45	8.77	7.55
650 A	7.68	9.14	7.78
750 A	8.10	9.84	8.20
800 A	8.30	10.17	8.39
900 A	8.77	10.81	8.86

(For Sch 40)

{2} Support in piping axial direction
When the straight pipe portion is long and the motion in the piping axial direction is not restrained, support in the axial direction is needed.

(iii) Support of bend portion

For a bend portion, the vibration frequency decreases in the direction perpendicular to the bending plane (out-of-plane direction). Hence, near the bend portion, support is provided to suppress the out-of-plane vibration to ensure that the length of the support interval is within the standard support span. Figure 6.6.3-41 shows an example of the relation between bend angle and decrease in vibration frequency [6.6.3-4].

(iv) Support of concentrated mass portion

{1} In the case of a concentrated mass, such as a valve the support span is determined by multiplying a reduction factor times the support span of the straight pipe portion to ensure that the vibration frequency of the support interval is higher than the standard vibration frequency (see Figure 6.6.3-42). However, it is also possible to directly support the concentrated mass portion at a position as near as possible.

{2} In particular, in the case when an electrical valve or pneumatic valve is attached, support should be provided to prevent generation of excessively large torsional moment in the piping due to the eccentric load in the driving portion, so that no excessively large acceleration takes place during earthquake.

(v) Support of branched portion

In the case when there exists branch pipe, the support span is determined by multiplying a reduction factor times the support span of the straight pipe portion to ensure that the vibration frequency of the support interval is higher than the standard vibration frequency (see Figure 6.6.3-43). However, it is also possible to directly support the branched portion at a position as near as possible.

(b) Support according to stress-based constant-pitch span method

(i) Guideline

The piping system is divided into straight pipe portions, bend portions, branched portions, concentrated mass portions and other standard structural elements. For each element, the support span is determined to ensure that the natural vibration frequency of the element and the seismic stress value are within the allowable ranges.

For the overall piping system, the support points are determined in consideration of the combination of the various elements.

(ii) Support of straight pipe portion

As shown in Diagram 6.6.3-1, each pipe is represented by an equally distributed load continuous beam model supported at 3 points with a support span of l. The maximum support span is derived by performing dynamic and static analysis. The span adopted for support should be smaller than the maximum span.

{1} Design seismic force

{a} Static seismic force
See Table 6.6.3-6.

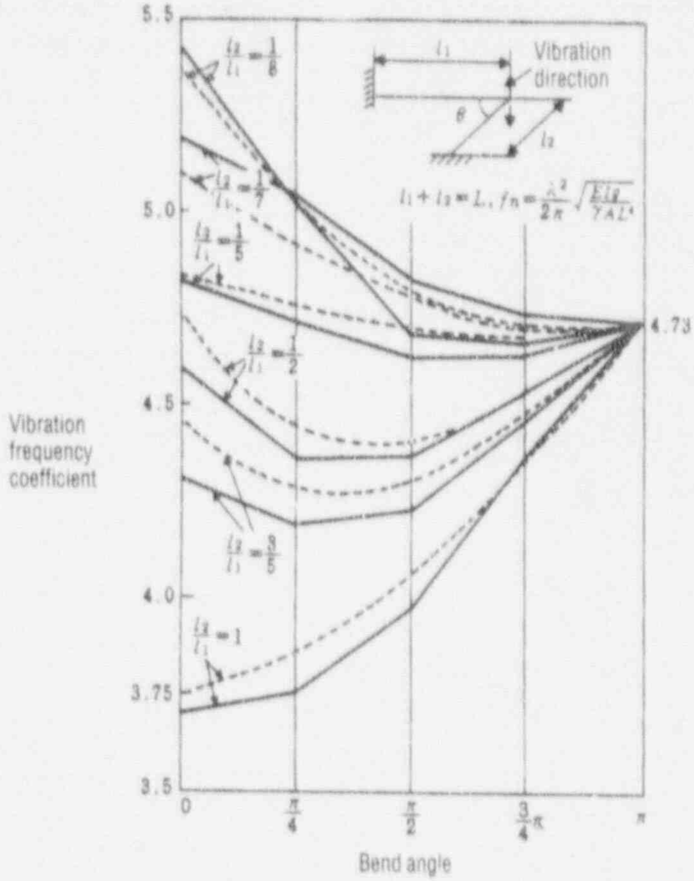
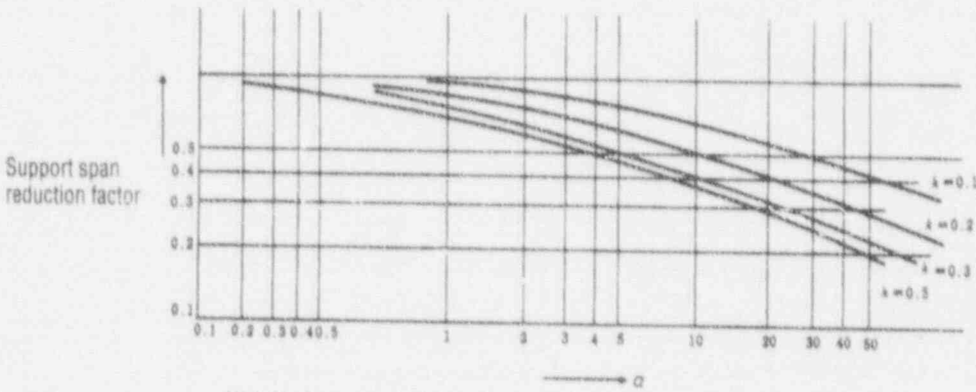


Figure 6.6.3-41. Example of bend a~gle vs. vibration frequency coefficient.



$\alpha = \dfrac{\text{Weight of concentrated mass}}{\text{Weight of constant-pitch span length}}$

β = Support span reduction factor
κ = position of concentrated load between support points
(κ = 0.5 when the load is at the center, κ = 0.25 when the load is at 1/4 the span.)

Figure 6.6.3-42. Example of span reduction in the case of concentrated mass system.

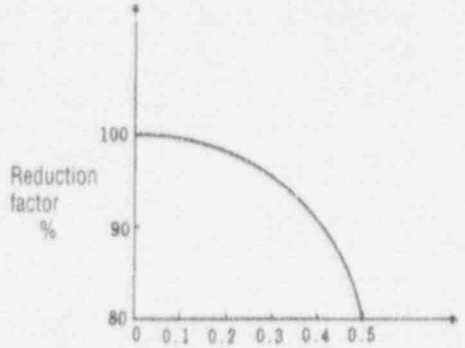
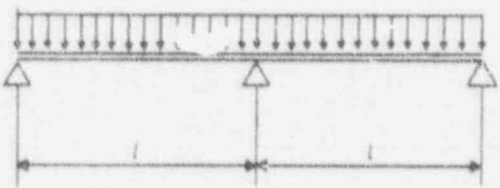


Figure 6.6.3-43. Example of reduction factor for branched portion.



(In this model, the support points are assumed to be restrained only in the direction perpendicular to axial direction, and they are free with respect to axial direction and rotation.)

Diagram 6.6.3-1.

Table 6.6.3-6. Design seismic force (see Table 6.6.4-1).

Class	Horizontal	Vertical
B	$K_h(1.8\ C_i)$	—
C	$K_h(1.2\ C_i)$	—

{b} Dynamic seismic force

The dynamic seismic force is determined from the eigenvalue of piping and the design floor response spectrum (1/2 S_1). For Class C piping, however, the dynamic seismic force is not considered.

{2} Analysis

For each pipe, the stress is determined from the design seismic force. In addition, the effects of the internal pressure and self weight are taken into consideration to calculate the maximum support span by using the trial-and-error method.

{a} Consideration of pipe weight

The weight of a pipe is the sum of the self weight of the pipe and the weight of water as the internal fluid. In addition, for piping with a thermal insulating material layer applied, its weight should also be taken into consideration.

{b} Piping stress

The stresses generated in a pipe include seismic stress, and stress due to internal pressure and self weight. The sum of these stresses should be lower than the allowable stress.

{c} Vibration frequency of piping system

In principle, the natural vibration frequency of the overall piping system should be out of the frequency region near the peak of the response spectrum of the building floor.

(iii) Support of bend portion

The bend portion of piping is represented by a model of equally distributed load beams pin-connected together and with their two ends fixed (see Diagram 6.6.3-2).

{1} The natural vibration frequency should be higher than the natural vibration frequency of the maximum support span of the straight-pipe portion.

{2} The bending moment when a seismic force is applied should be less than the bending moment due to the seismic force for the straight-pipe portion with the maximum support span.

In this way, the support span is determined.

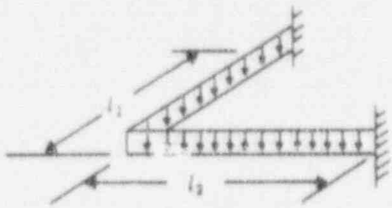
In addition, when piping system, support structure, etc., are to be designed, if it is necessary to increase l_1 and/or l_2, the following requirements should be satisfied simultaneously by arranging support structures to restrain the out-of-plane vibration (see Diagram 6.6.3-3).

(iv) Support of concentrated mass portion

When a valve or other weight is attached to the piping, the piping is represented by a model shown in Diagram 6.6.3-4 as a continuous beam having its two ends supported and having the concentrated load at an arbitrary position.

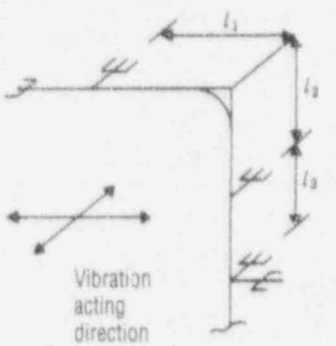
The support span is determined to satisfy the following requirements:

{1} The natural vibration frequency should be higher than the natural vibration frequency of the maximum support span of the straight-pipe portion.



Note 1: $l_1 + l_2 < l_E$
l_E is support interval which satisfies ② below.

Diagram 6.6.3-2.

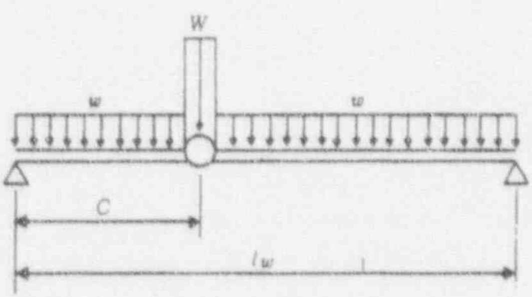


$l_1 + l_2 \leqq l_E$

$l_2 + l_3 \leqq l_0$

l_0 is the maximum support interval at the straight pipe portion.

Diagram 6.6.3-3.



Note 1: l_w is the support interval of the concentrated mass portion.
Note 2: C is the length from the supported end to the concentrated load point.
Note 3: w is the weight per unit length of pipe.
Note 4: W is the concentrated load.

Diagram 6.6.3-4.

{2} When a seismic force is applied, the sum of bending moments of the concentrated load and uniformly distributed load should be less than the bending moment due to the seismic force of the straight pipe portion with the maximum support span.

In addition, it is desirable that the support point be as near the discrete mass portion as possible.

(v) Support of branched portion

As shown in Diagram 6.6.3-5, the branched portion of piping can be represented by a beam model having the three support ends of the T-shaped portions simply supported and with the branch pipe pin-connected to main pipe for conservative estimate.

The support span is determined to meet the following requirements:

{1} The natural vibration frequency should be higher than the natural vibration frequency of the straight-pipe portion with the maximum support span.

{2} The bending moment when a seismic force is applied is less than the bending moment due to the seismic force at the straight-pipe portion having the maximum support span.

In addition, it is desirable that the support point be as near the branch point as possible.

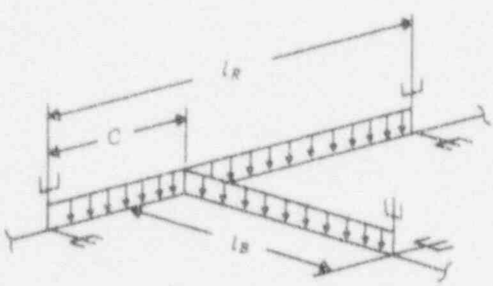
(c) Modified seismic coefficient method

(i) Guideline

The support points and support scheme are determined to ensure that the stress of the overall piping system is within the allowable range against the seismic coefficient which is determined conservatively in consideration of the natural vibration frequency of the overall piping system and the frequencies of the building/structure that support the piping.

(ii) Design seismic coefficient

In principle, the seismic coefficient determined on the base of the seismic force calculated from the design floor response spectrum (1/2 S_1) with respect to the primary natural vibration frequency of the piping system is used. However, for convenience's sake in design, when the static seismic force is larger than the aforementioned seismic force or when Class C piping is designed, the static seismic coefficients from Table 6.6.3-6 may be used.



Note 1: l_R is the length of the mother pipe of the T portion.
Note 2: C is the length from the support point of another pipe to the mounting point of the branch pipe.
Note 3: l_B is the length of the branch pipe.
Notte 4: l_0 is the maximum support length of the straight-pipe portion.

Diagram 6.6.3-5.

(iii) Analysis and evaluation

According to the design seismic coefficient defined above, the supporting points are determined by trial-and-error to ensure that the combined stress of the seismic stress statically applied to the entire piping system and the stress due to internal pressure, self weight and other mechanical loads is below the allowable stress.

(d) Dynamic design method

(i) General items

The seismic response analysis and stress analysis are performed for the entire piping system, and the support points and support scheme are determined to ensure that the stress generated in the piping system is below the allowable stress. In the case when the static seismic force is larger than the dynamic seismic force, for convenience's sake in design, the static analysis method which gives results on the safer side is usually adopted.

(ii) Design seismic force

{1} Static seismic force
The static seismic force is listed in Table 6.6.3-6.

{2} Dynamic seismic force
The dynamic seismic force is derived from the eigenvalue of the piping system and the design floor response spectrum (1/2 S_1). However, for Class C, the dynamic seismic force is not considered.

(e) Other items that should be considered

(i) Piping running between different buildings/structures

For the portion of piping running between different buildings or other structures, the relative displacement of the two buildings/structures should be taken into consideration.

(ii) Connection portion to equipment

In principle, support should be made as near the equipment as possible. Also, when the operating temperature of the equipment is high, the thermal expansion of the equipment should be taken into consideration. In addition, the nozzle reaction force acting on the equipment should be within the allowance range.

(iii) Outdoor piping

In this case, the behavior of the soil during earthquake, the relative displacement between building/structure and ground, and the thermal expansion of the piping should be taken into consideration. The flexibility of piping with respect to the support structure and the flexible joint should also be taken into consideration.

(iv) Adjacent pipings

Arrangement should be made to ensure that there is no mutual interference between pipings caused by displacement during earthquake.

(v) Support structure

The purpose of the support structure is to provide restraint against seismic force. It should be designed to have a necessary stiffness.

The support structure should have a sufficiently high strength against seismic force, thermal expansion of piping, and self weight of piping.

c. Stress evaluation

The stress due to earthquake is combined with the stress due to self weight, internal pressure, and other loads, and the result should be lower than the allowable limit. The stress evaluation procedure is shown in Figure 6.6.3-44.

The general parts for which the stress evaluation should be performed are as follows:

{1} Anchor portion
{2} Nozzle portion
{3} Elbow portion
{4} Valves and other concentrated mass portions
{5} Support mounting portion

The standard stress evaluation results are tabulated. As an example, these results for Class B piping are listed in Table 6.6.3-7. In this table, SP, SP_m, SM_a, SM_b, and SM_c are defined in Item 56 of "Notification No. 501." They represent stresses due to the following loads, respectively:

P: Highest pressure in operation
P_m: Highest pressure acting on the inner surface
M_a: Bending moment generated by mechanical loads of the piping (self weight and other long-term loads)
M_b: Bending moment generated by mechanical loads of the piping (injection reaction force of relief valve or safety valve, and other short-term loads)
M_c: Bending moment generated by displacement of support point and thermal expansion due to heat of pipe.

Also, SS(SB) represents the seismic stress generated by S_B seismic force; $S_a(C)$ and $S_a(D)$ represent the allowable stresses shown in Item 5-6-2-C and D of "Notification No. 501."

(3) Other equipment

a. Pumps/blowers

The aseismic design procedure is shown in Figure 6.6.3-45. In principle, calculation of the natural period of each piece of equipment is performed using a model of a single mass system under appropriate support conditions suitable for the shape of the equipment. Typical equipment models are shown in Figure 6.6.3-46. In this case, for the motor portion of vertical-shaft equipment and the horizontal-shaft equipment, the structure can be taken as a single rigid body, and the natural period need not be calculated. In the following, the standard calculation methods of natural period and stress are presented.

(a) Vertical pump

Calculation conditions

{1} The weight of the pump is divided into the upper portion above the mounting plane and the lower portion below the mounting plane. For each portion, the weight is taken as concentrated at the corresponding center of gravity.

{2} The pump is fixed by anchor bolts, etc., on a sufficiently rigid foundation or flange.

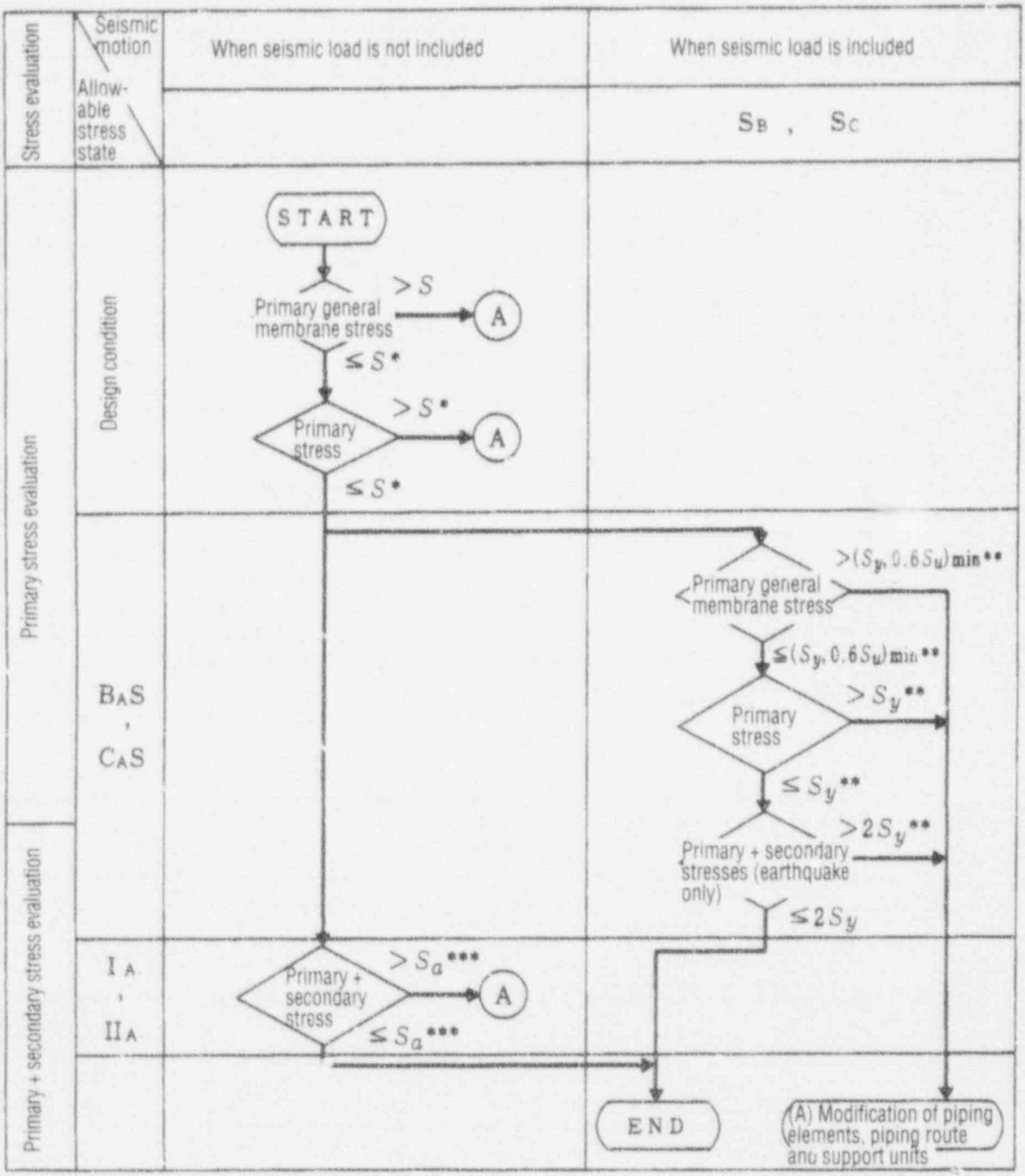


Figure 6.6.3-44. Procedure of stress evaluation of Class B and C piping.

Table 6.6.3-7. Example of Class B piping stress analysis results.

Evaluation point	Allowable stress state	Primary and secondary stress (kgf/mm^2): Stress due to internal pressure {1} SP (SP_m)	Stress due to self weight {2} SM_a	Short-term mechanical load stress and regional stress {3} SM_b	Secondary stress* {4} SM_c	Primary stress evaluation (kgf/mm^2): Calculated stress {1}+{2} {1}+{2}+{3}	Allowable stress 1.0 S 1.2 S 1.0 Sy**	Primary + secondary stress evaluation (kgf/mm^2): Calculated stress {1}+{2}+{4} {1}+{2}+{3}+{4} SS (SB)	Allowable stress Sa Sa 2.0 Sy
	(I_A, II_A)	4.8	0.4	—	2.3	5.2	12.2	7.5	30.5
	(I_A, II_A)	5.3	0.4	—	2.3	5.7	14.6	8.0	32.9
	B_AS	5.3	0.4	1.2	—	6.9	25.2	2.4	50.4
	(I_A, II_A)	4.8	0.5	—	1.5	5.3	12.2	6.3	30.5
	(I_A, II_A)	5.3	0.5	—	1.5	5.8	14.6	7.3	32.9
	B_AS	5.3	0.5	1.4	—	7.2	25.2	2.8	50.4
	(I_A, II_A)	4.8	1.0	—	12.0	5.8	12.2	17.8	30.5
	(I_A, II_A)	5.3	1.0	—	12.0	6.3	14.6	18.3	32.9
	B_AS	5.3	1.0	3.0	—	9.3	25.2	6.0	50.4
	(I_A, II_A)	4.8	0.4	—	1.7	5.2	12.2	6.9	30.5
	(I_A, II_A)	5.3	0.4	—	1.7	5.7	14.6	7.4	32.9
	B_AS	5.3	0.4	1.8	—	7.5	25.2	3.6	50.4
	(I_A, II_A)	4.8	0.3	—	3.0	5.1	12.2	8.1	30.5
	(I_A, II_A)	5.3	0.3	—	3.0	5.6	14.6	8.6	32.9
	B_AS	5.3	0.3	1.5	—	7.1	25.2	3.0	50.4

*(I_A, II_A) represents stress due to displacement of support point and heat expansion caused by heat, B_AS represents stress due to relative displacement in earthquake.

**For austenitic stainless steel and high-nickel alloy, 1.0 S_y or 1.2 S, whichever is larger.

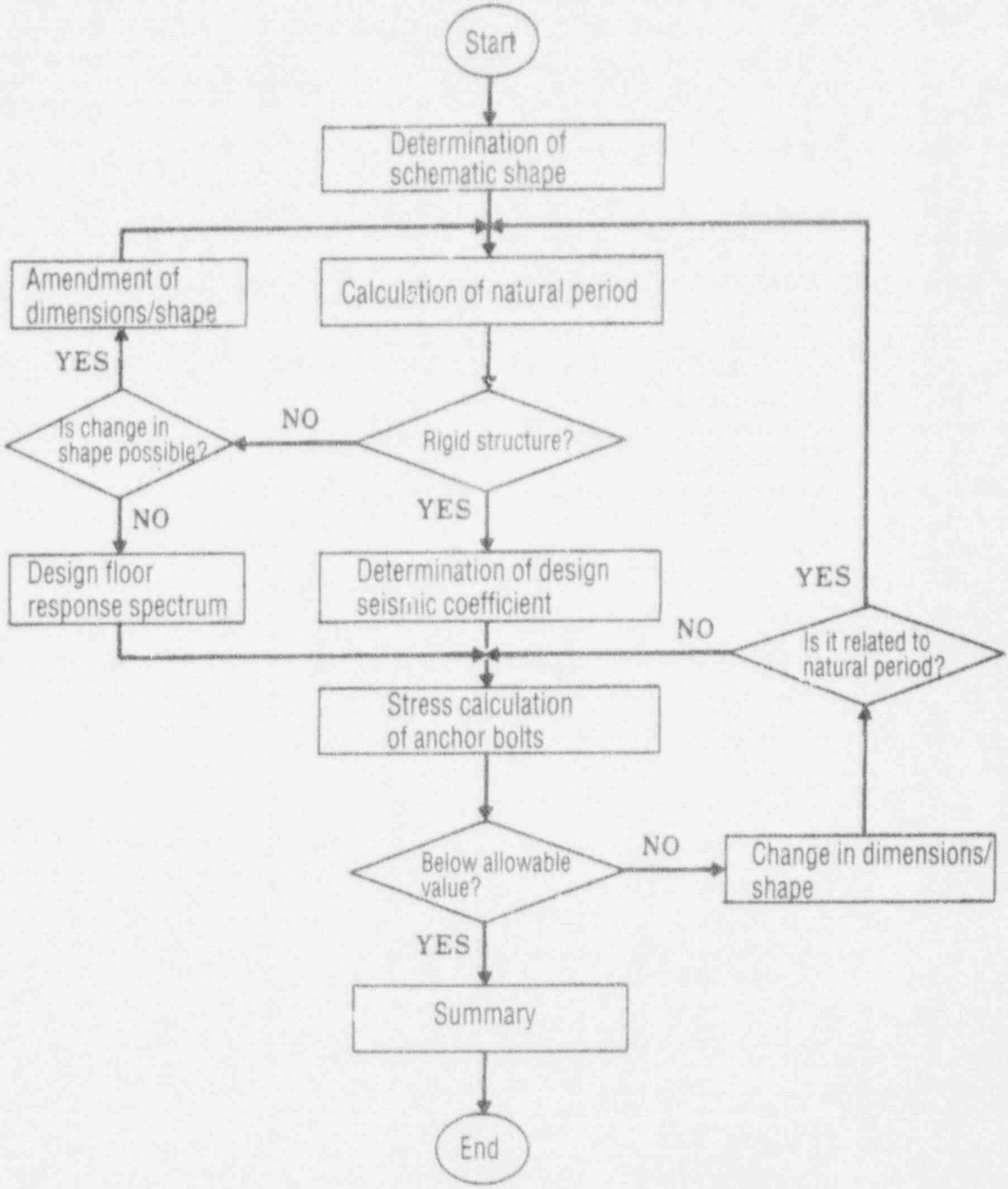


Figure 6.6.3-45. Aseismic design procedure of pumps/blower.

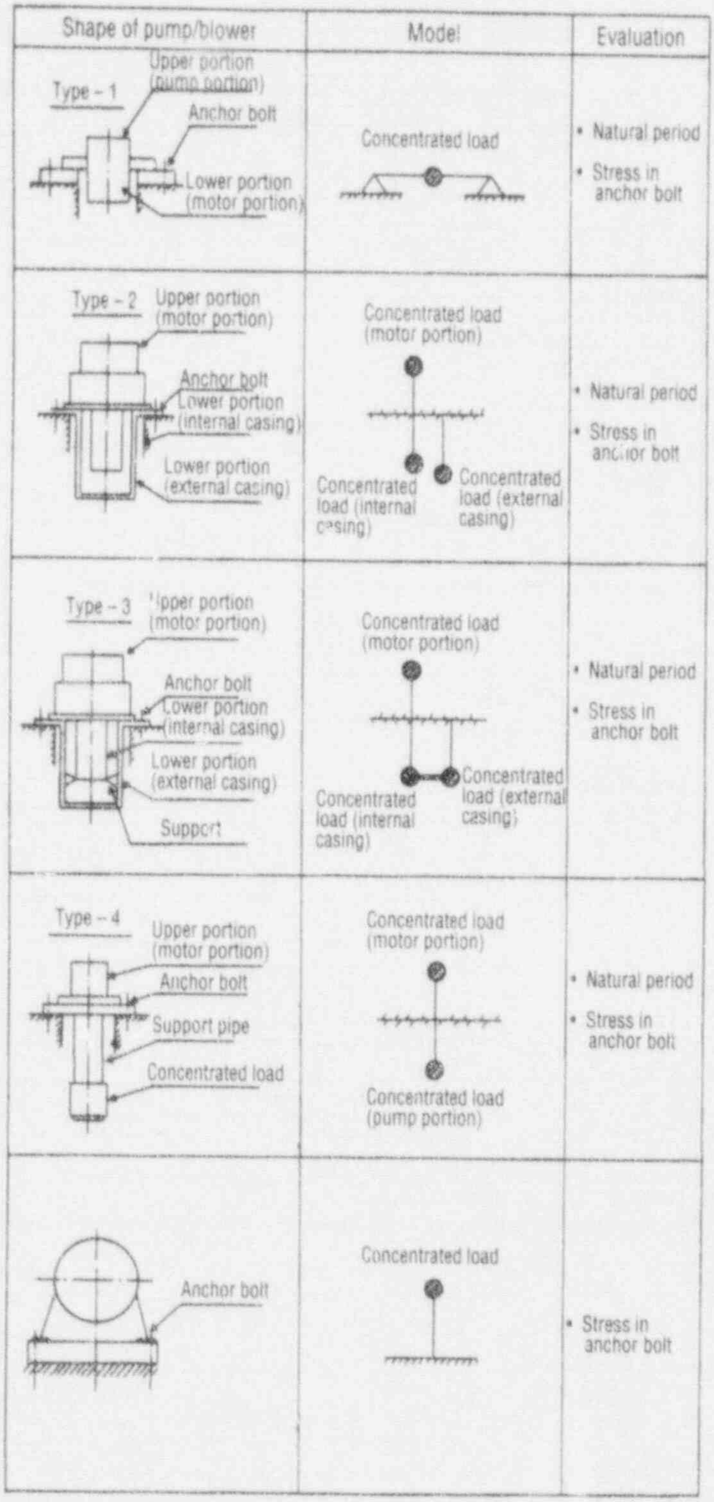


Figure 6.6.3-46. Examples of natural period calculation models and evaluation methods.

Definitions of symbols (for common symbols, please see section 6.6.2(1)a)

Symbol	Explanation of symbol	Units
C_p	Seismic coefficient due to pump vibration	—
l	Distance between pump shaft center and anchor bolts	mm
l_1	Span of beam in calculation model (1)	mm
D	Pitch circle diameter of anchor bolts	mm
d	Nominal diameter of anchor bolts	mm
n	Number of anchor bolts	—
n_f	Number of evaluated anchor bolts with action of tensile force	—
Q_b	Shear force acting on anchor bolts	kgf
A_{si}	Minimum effective shear sectional area	mm^2
G_i	Shear modulus of elasticity	kgf/mm^2
E_i	Longitudinal modulus of elasticity	kgf/mm^2
I_i	Moment of inertia	mm^4
T_i	Natural period	s
W_i	Weight	kgf
	i = 1: Upper portion above mounting plane	—
	i = 2: Lower portion below mounting plane (internal casing)	—
	i = 3: Lower portion below mounting plane (external casing)	—
h_i	Distance	mm
	i = 1: Distance between mounting plane and the center of gravity of the upper portion	—
	i = 2: Distance between mounting plane and the center of gravity of the lower portion (internal casing)	—
	i = 3: Distance between mounting plane and the center of gravity of the lower portion (external casing)	—
H	Distance between mounting plane and the support position	mm

{3} The seismic force is assumed to act in the horizontal direction on the pump, and the vertical direction is not considered.

(i) Calculation of natural period

As shown in Figure 6.6.3-47, calculation is performed for the single discrete mass system as a beam fixed on the mounting plane.

The natural period for calculation model (1) is

$$T = 2\pi\sqrt{\frac{W_1}{g}\left(\frac{l_1^3}{48E_1I_1} + \frac{l_1}{4A_{s1}G_1}\right)} \tag{6.6.3-503}$$

The natural period for calculation models (2), (3), (4) is

$$T_i = 2\pi\sqrt{\frac{W_i}{g}\left(\frac{h_i^3}{3E_iI_i} + \frac{h_i}{A_{si}G_i}\right)} \tag{6.6.3-504}$$

The natural periods for calculation model (5) are as follows:

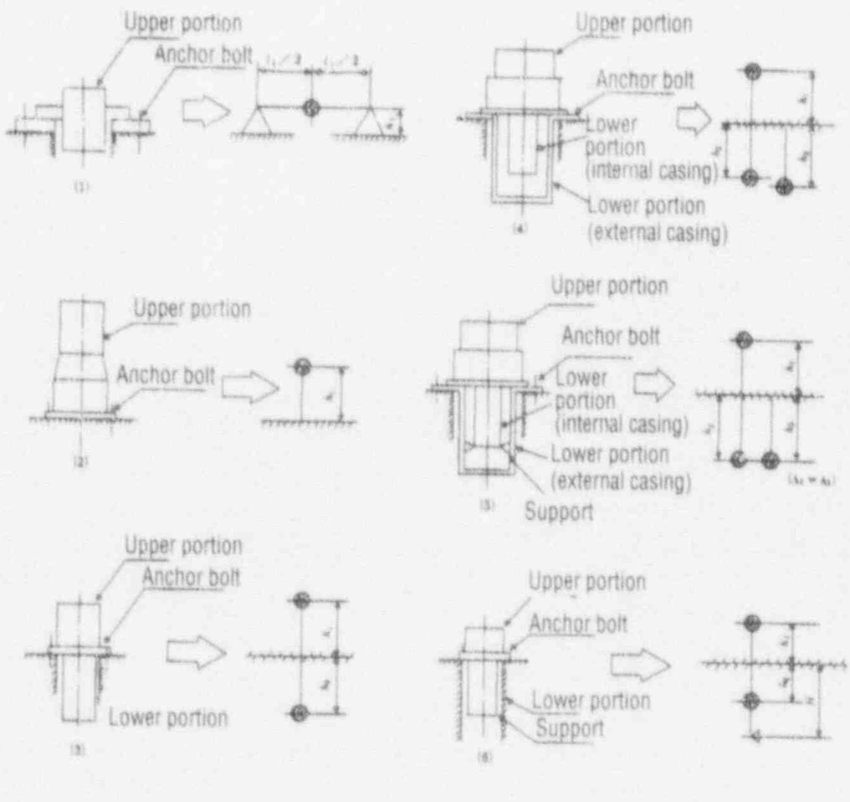


Figure 6.6.3-47. Calculation models.

{1} For the upper portion

$$T_1 = 2\pi\sqrt{\frac{W_1}{g}\left(\frac{h_1^3}{3E_1I_1}+\frac{h_1}{A_{s1}G_1}\right)} \qquad (6.6.3\text{-}505)$$

{2} For the lower portion

$$T_2 = 2\pi\sqrt{\frac{(W_2+W_3)}{g}\frac{\left(\frac{h_2^3}{3E_2I_2}+\frac{h_2}{A_{s2}G_2}\right)\left(\frac{h_3^3}{3E_3I_3}+\frac{h_3}{A_{s3}G_3}\right)}{\left(\frac{h_2^3}{3E_2I_2}+\frac{h_3^3}{3E_3I_3}+\frac{h_2}{A_{s2}G_2}+\frac{h_3}{A_{s3}G_3}\right)}} \qquad (6.6.3\text{-}506)$$

where h_2, and h_3 ($h_2 = h_3$) are the height of center of gravity of the entire portion below the mounting plane.

The natural periods of calculation model (6) is as follows:

{1} For the upper portion

$$T_1 = 2\pi\sqrt{\frac{W_1}{g}\left(\frac{h_1^3}{3E_1I_1}+\frac{h_1}{A_{s1}G_1}\right)} \qquad (6.6.3\text{-}507)$$

{2} For the lower portion

$$T_2 = 2\pi\sqrt{\frac{W_2}{g}\frac{1}{K}} \qquad (6.6.3\text{-}508)$$

where

$$\frac{1}{K} = \frac{h_2^3}{3E_2I_2}+\frac{h_2}{A_{s2}G_2}-\frac{\left\{\frac{h_2}{A_{s2}G_2}+\frac{h_2^2(3H-h_2)}{6E_2I_2}\right\}^2}{\frac{H^3}{3E_2I_2}+\frac{H}{A_{s2}G_2}} \qquad (6.6.3\text{-}509)$$

(ii) Calculation of stresses

{1} Depending on the shape of the installation portion, there are two models for anchor bolt stress calculation (see Figures 6.6.3-48 and 6.6.3-49). The stresses acting on the anchor bolts are calculated with respect to the tensile forces and shear forces due to seismic coefficient, pump vibration and pump rotation moment.

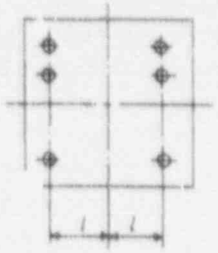


Figure 6.6.3-48. Square calculation model.

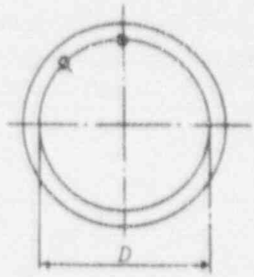


Figure 6.6.3-49. Circular calculation model.

{a} Shear stresses

The shear force of the anchor bolts is calculated as the shear force acting on the total number of anchor bolts.

Shear force (Q_b)

$$Q_b = \sum_{i=1}^{3} W_i(C_H + C_p) \tag{6.6.3-510}$$

Shear stress (τ_b)

$$\tau_b = \frac{Q_b}{nA_b} \tag{6.6.3-511}$$

where sectional area A_b of anchor bolt is

$$A_b = \frac{\pi}{4} d^2 \tag{6.6.3-512}$$

i = 1: Upper portion above mounting plane
i = 2: Internal casing
i = 3: External casing

{b} Tensile stress

The tensile force acting on the anchor bolts in the case of square calculation model is evaluated based on the most conservative assumption; i.e., the base plate is assumed to rotate at one row of anchor bolts, and the other row of anchor bolts are assumed to resist this movement.

Tensile force (F_b)

$$F_b = \frac{\sum_{i=1}^{3} W_i(C_H + C_p)h_i - \sum_{i=1}^{3} W_i(1 - C_p)l}{n_f l} \tag{6.6.3-513}$$

The tensile force acting on the anchor bolts in the case of the circular calculation model is calculated for the bolts farthest from the rotation point as the most strict condition, based on the assumption that the anchor forces are proportional to the distance from the rotation point.

Tensile force (F_b)

$$F_b = \frac{\sum_{i=1}^{3} W_i(C_H + C_p)h_i - \sum_{i=1}^{3} W_i(1 - C_p)\frac{D}{2}}{\frac{3}{8} n_f D} \qquad (6.6.3\text{-}514)$$

Tensile stress (σ_b)

$$\sigma_b = \frac{F_b}{A_b} \qquad (6.6.3\text{-}515)$$

(iii) Evaluation methods

{1} Evaluation of natural period
Based on the natural period in item (i), the horizontal design seismic coefficient is evaluated.

{2} Stress evaluation of anchor bolts
It is performed according to section (`` and section "6.6.3(1)b Skirt-support vertical cylindrical container."

(b) Horizontal pump

<u>Calculation conditions</u>

{1} The weight of the pump is assumed as concentrated at the center of gravity.

{2} The pump is assumed as fixed by anchor bolts on a sufficiently rigid foundation or frame.

{3} The seismic force is assumed to act on the pump in the horizontal direction, while the vertical direction is not considered.

{4} The overturning directions are determined according to the calculation models shown in Figures 6.6.3-50 and 6.6.3-51. The direction with the strictest condition is selected for performing aseis' c evaluation.

(i) Calculation of natural period

A horizontal pump is taken as a large block-like structure, with its center of gravity located near the center of the block and with its lower surface fixed using anchor bolts. As a result, it can be taken as a rigid body as a whole. The natural period is very small and can be ignored in calculation.

(ii) Calculation of stresses

{1} Stress calculation of anchor bolts

The stresses in the anchor bolts are calculated with respect to the tensile forces and shear forces caused by seismic force, pump vibration and moment due to pump rotation.

Definitions of symbols (for common symbols, see section 6.6.3(1)a)

Symbol	Definition of symbol	Units
C_p	Seismic coefficient due to pump vibration	—
M_p	Moment due to pump rotation	kgf/mm
l_1	Distance between pump shaft center and anchor bolt	mm
l_2	Distance between pump shaft center and anchor bolt ($l_1 \leq l_2$)	mm
d	Nominal diameter of anchor bolts	mm
n	Number of anchor bolts	—
n_f	Evaluation number of anchor bolts with tensile force	—
h	Distance from mounting plane to center of gravity	mm
W	Weight acting on the mounting plane	kgf
Q_b	Shear force acting on anchor bolt	kgf

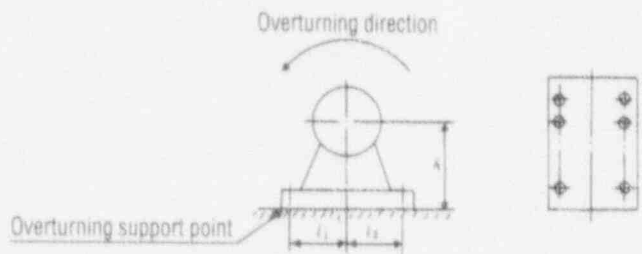


Figure 6.6.3-50. Calculation model (overturnng in direction perpendicular to the axial direction).

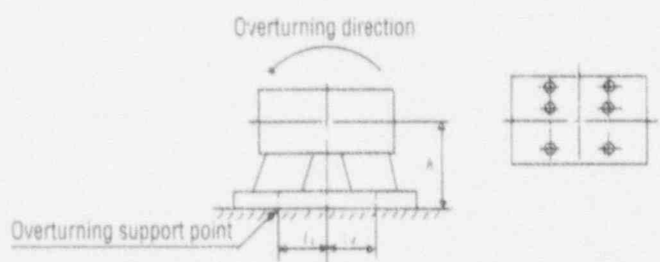


Figure 6.6.3-51. Calculation model (overturning in axial direction).

{a} Shear stress

The shear force acting on the total number of anchor bolts is calculated.

Shear force (Q_b)

$$Q_b = W(C_H + C_p) \tag{6.6.3-516}$$

Shear stress (τ_b)

$$\tau_b = \frac{Q_b}{nA_b} \tag{6.6.3-517}$$

where sectional area A_b of the bolt is as follows:

$$A_b = \frac{\pi}{4} d^2 \tag{6.6.3-518}$$

{b} Tensile stress

The tensile force on the anchor bolts is calculated as acting on bolts on one side as the strictest condition, on consideration of overturning shown in Figure 6.6.3-50 and 6.6.3-51 with the other side of bolts used as rotation point.

Tensile force (F_b)

$$F_b = \frac{W(C_H + C_p)h + M_p - W(1 - C_p)l_1}{\frac{1}{2} n_f (l_1 + l_2)} \tag{6.6.3-519}$$

Here, moment M_p due to pump rotation is for the calculation model shown in Figure 6.6.3-50. It does not exist in the case when the pump and the motor share the same base or when the calculation model is as shown in Figure 6.6.3-51.

Tensile stress (τ_b)

$$\sigma_b = \frac{F_b}{A_b} \tag{6.6.3-520}$$

(iii) Evaluation methods

{1} Evaluation of natural period

Based on the natural period in item (i), the horizontal design seismic coefficient is evaluated.

{2} Stress evaluation of anchor bolts

It is performed according to section "6.6.3(1)b Skirt-support vertical cylindrical container."

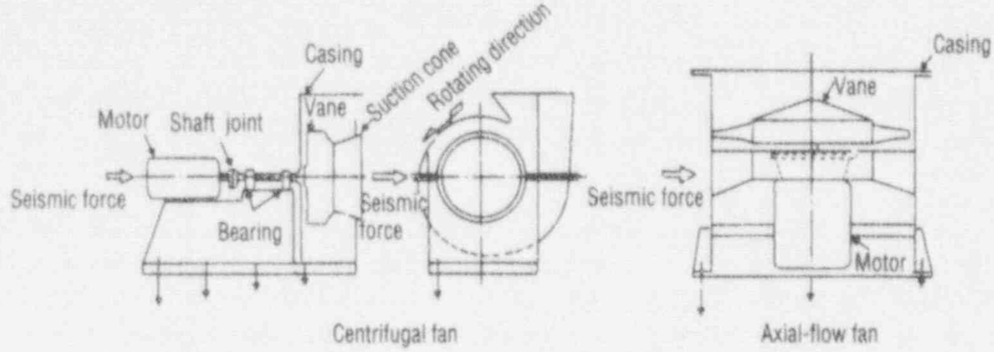


Figure 6.6.3-52. Fan (blower).

(c) Aseismic design method of fan (blower) (see Figure 6.6.3-52)

(i) General items

Since the main body of a fan (blower) has very rigid vibration characteristics, its aseismic design scheme is based on (b) horizontal pump, in principle. That is, the calculation conditions are as follows:

{1} The weight of the fan (blower) is taken as concentrated at the center of gravity, and the seismic force is assumed to act at this position.
{2} The base is fixed using anchor bolts, etc., on a sufficiently rigid foundation or frame.

The stress evaluation is performed only for the anchor bolts under the aforementioned conditions.

b. Duct and cable tray

(a) Duct

The aseismic design of a duct is usually performed using the following methods: in one method, depending on the aseismic class suitable for the duct, dynamic analysis or static analysis is performed to calculate the seismic load and to evaluate the strength; in another method, the span of the support is made shorter than the allowable buckling limit length of the duct. In this case, for a duct in Class B or higher class, the dynamic seismic force or static seismic force is used, depending on the natural frequency of vibration. For a Class C duct, however, there is no need to calculate the natural frequency of vibration.

Calculation of the support interval from the natural vibration frequency or allowable buckling limit is performed using the theoretical formula derived under the assumption that the duct is a beam with its two ends simply supported, with an appropriate safety margin taken into consideration.

(b) Cable tray

Just as in the case of a duct, for the cable tray, evaluation is performed according to its aseismic class. For a Class A cable tray, dynamic analysis or static analysis is performed to calculate the seismic load and to evaluate the strength, or the frequency-based contact-pitch span method is used; for a Class C cable tray, the stress-based constant-pitch span method is used to select the support points. In this case, the various schemes used also depend on the piping system.

c. Crane

For the main body (girder) of the crane, evaluation is performed for a model of a beam hav... ends simply supported. In this case, since the vibration characteristics and stress generated depend on the trolley's position, it is necessary to take them into consideration when evaluation is performed. In addition, since equipment of high aseismic classes is located below the crane, it is necessary to ensure that the crane does not fall by the seismic motion corresponding to the aseismic class of the equipment. In this case, when the strength evaluation is performed for the support portion in consideration of the sliding of the crane, the following scheme is usually used. That is, in the strength calculation of the support portion, for the running direction, the seismic coefficient at the support portion cannot be larger than frictional coefficient between the rail and the wheel. On the other hand, for the transverse direction (direction perpendicular to the running rails), since the girder is taken as a rigid body, evaluation is performed by assuming that the seismic response acceleration with respect to the afore... tioned seismic motion of the building at the mounting position is transferred to the girder.

d. Condenser

The condenser is taken as a flexural shear beam or FEM with fixed lower end for calculation of the natural vibration frequency. In this case, evaluation of stiffness is performed in consideration of the barrel plate, ribs of side plate, and ribs of front/rear plates. As a result, when it is found to be a rigid structure or for a Class C condenser, static analysis is performed. Usually, stress evaluation is performed of the interested points in the structure, such as reinforcing parts, etc.

6.6.4 Support structures

(1) Outline of support structures

For the equipment/piping system support structure, the support scheme is determined usually in consideration of the load conditions as well as the functions required of the equipment/piping system, configuration, service/maintenance features, construction features, etc. As a result, the following various structural forms may be used:

{1} Legs, skirt
{2} Skeleton structure (electrical panel, air conditioning unit, etc.)
{3} Frame structure (piping frame restraint, support structures of air conditioning duct, cable tray, etc.)
{4} Other (piping seismic support, etc.)

When these support structures are designed, the following items are taken into consideration to ensure an appropriate strength against the load transferred from the body of the equipment/piping system.

{1} When the seismic load on the body of the equipment/piping system is to be calculated, amplification of the seismic input due to the support structure is evaluated appropriately and determined. When it is determined that the stiffness of the support structure is higher than that of the body of the equipment/piping system, it is also possible not to consider the amplification of the seismic input due to the support structure.

{2} When the stress of the support structure is evaluated, not only the seismic load, but also the effects of the self weight and pressure or other mechanical load on the body of the equipment/piping system are taken into consideration.

{3} The support structure should have the necessary support function. In addition, it should not hamper the designated movement of the equipment/piping system under various load conditions.

{4} In principle, the support structure is reliably fixed by using embedded fixtures set in the building/structure, steel structure, or anchor bolts.

{5} The structure should be appropriate to ensure the access route and the space for maintenance and service.

{6} When the thermal transfer from the equipment/piping system body is significant, the effects on the nearby structures should be taken into consideration for the structure.

{7} In the case when separation is required from the other equipment/piping system in consideration of the equipment/piping system, this requirement should be taken into consideration.

Stress evaluation is performed as follows [H-K-7] for the anchor bolts and embedded fixtures which transfer the load applied on the aforementioned support structures to the building or other concrete structures.

(2) Anchor portion

a. General items

(a) The equipment is fixed by anchor bolts, etc., on a sufficiently rigid foundation or flange.

(b) When load due to earthquake or other external force is applied to the anchor bolt, the tensile force and shear force are taken into consideration.

(c) The base plate should be able to withstand the concrete reaction force and the pullout restraint force from the anchor.

b. Stress calculation of anchor portion

The methods for calculating the stresses in the base plate and anchor bolts that form the anchor portion are as follows:

(a) Stress calculation of base plate

(i) Guideline

Evalvation is made by calculation to ensure that the lower end surface of the base plate can withstand the concrete reaction force (f_c) and the pullout restraint force (N) from the anchor bolts.

(ii) Guideline of calculation

{1} The concrete contact surface of the base plate is taken as a reinforced concrete column; the anchor bolts acted upon by tensile stress are taken as tensile reinforcing bars.

{2} According to the "Reinforced Concrete Structure Calculation Standards of Architecture Institute of Japan" [6.6.4-1], the tensile force (N) of foundation bolts, the concrete extreme fiber compressive stress (f_c) and the neutral axis are calculated.

{3} Confirmation is made to see if N and f_c are within the allowable ranges, and to see if the stress in the base plate due to N and reaction force f_c is within the allowable range.

(b) Stress calculation of anchor bolts

(i) Guideline

The horizontal force acting on the structure is resisted by the frictional force between the bottom steel part and concrete due to bolt fastening; in the case when the fastening force can be neglected, this force should be resisted by the shear force of the bolts. Figure 6.6.4-1 shows the classes of the anchor bolts according to their shapes.

(ii) Calculation guidelines

{1} In the case when the fastening force can be neglected, the overall horizontal force on the structure is resisted by the effective number of bolts. On the other hand, when the friction due to the fastening force is taken into consideration, the horizontal force of the structure as a whole should not exceed the resistance due to the friction of the structure as a whole.

{2} When the shear stress and tensile stress in the anchor bolts are combined in action, they are combined in the evaluation. The calculation methods of stress and the allowance stress are determined according to the "Steel Structure Design Standards" [6.6.4-2] and "JEAG 4601-Supplement-1984."

{3} Evaluation of concrete bond strength and shear strength against bolt tensile force is performed according to "JEAG 4601-Supplement-1984."

{4} When anchor portion of nuclear reactor containing vessel has two rows of bolt cricles in the circumferencial direction, evaluation of this part is sometimes performed with the following assumptions.

{a}The anchor bolt circle is defined as the circle with the average diameter of the two circumferential rows of bolt circles. Also, a anchor bolt is taken as an equivalent cylinder having an area equal to the total sectional area of the bolt.

{b}The stress varies linearly from the maximum tensile stress to the maximum compressive stress.

c. Expansion anchor

The expansion anchor is arranged by drilling holes in concrete using a punching machine. Chemical anchor, hole-in anchor, or other scheme may be adopted. They usually are arranged at locations where a light load is applied. For equipment, the chemical anchor scheme is usually used; for electrical appliances, the hole-in anchor scheme is usually used. When this operation is to be implemented, the form/dimensions should be selected after conducting tests with regard to the actual working condition, or after confirming appropriate margin of the catalog specifications.

(3) Embedded metallic parts

Table 6.6.4-1 lists the classification of embedded metallic parts according to their shapes.

There is as yet no established standard evaluation method for embedded metallic parts. Usually, evaluation is performed in the same way as the anchor portion.

The evaluation methods adopted for the present design are listed in Table 6.6.4-2 and 6.6.4-3.

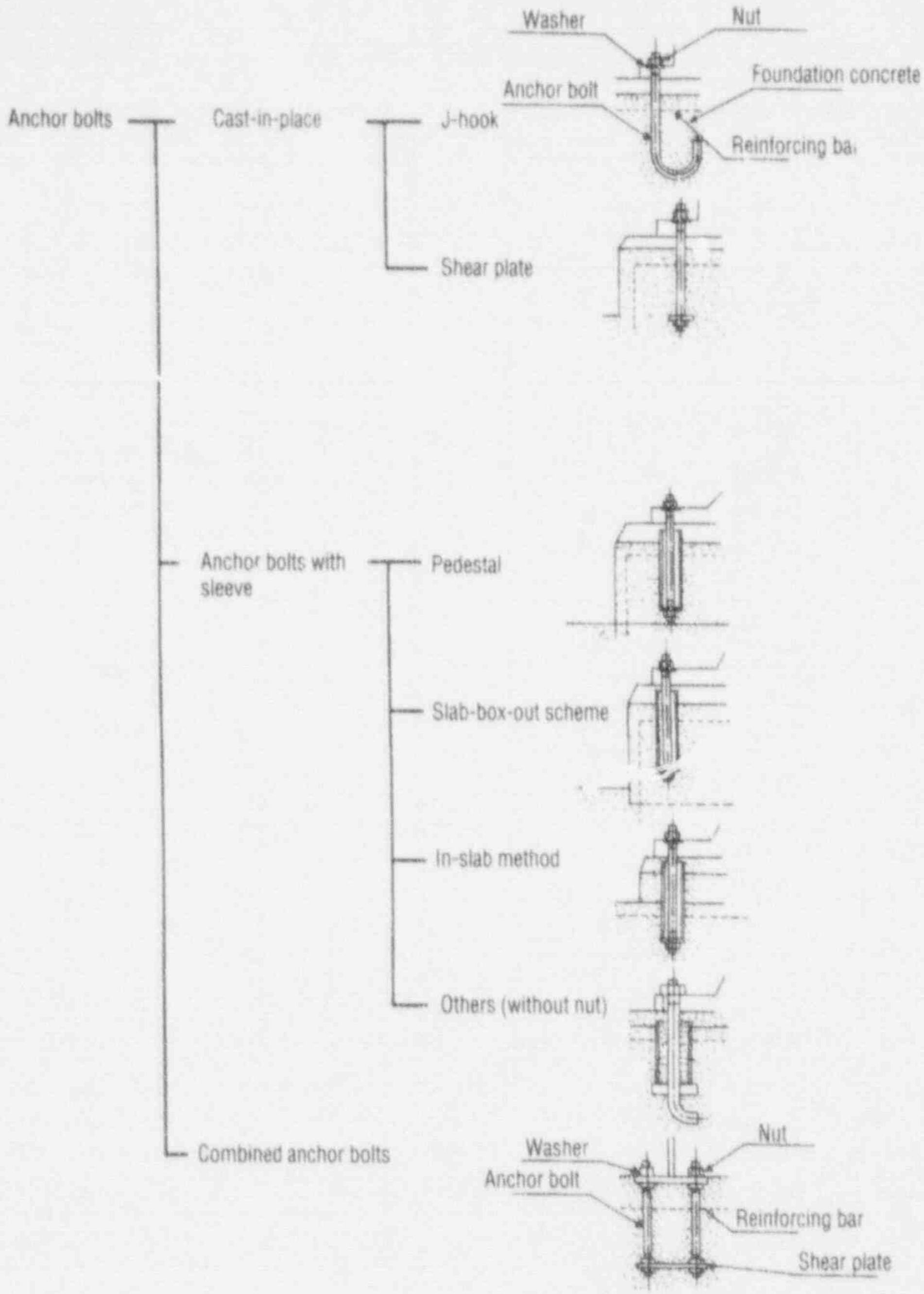


Figure 6.6.4-1. Classification of anchor bolts according to their shapes.

Table 6.6.4-1. Classification of embedded metallic parts according to their shapes.

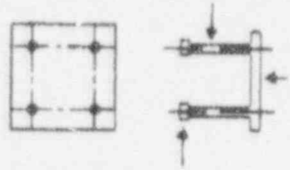
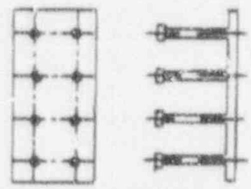
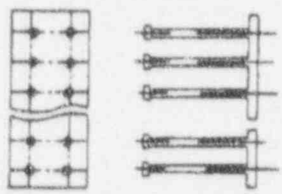
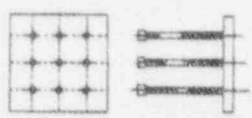
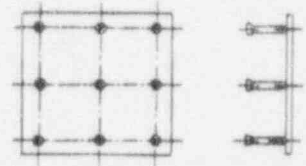
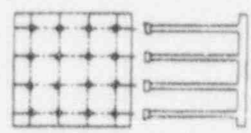
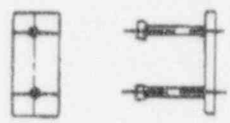
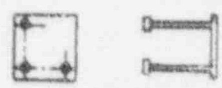
Type	Shape
I	
II	
III	
IV	
V	
VI	
VII	
VIII	

Table 6.6.4-2. Evaluation methods of embedded metallic parts (Part 1).

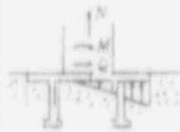
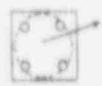
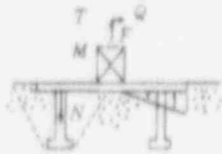
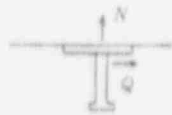
Item	Design standard	Evaluation method
Load transfer	- Column foot design - (H-K-7)	 With respect to the loads of axial force N (tensile/compressive), shear force Q, bending moment M, the load transfer is evaluated according to the design standards listed in the left column. In this case, the strength evaluation for the structural element of the standard embedded metallic parts is as the lower left side from the double line (with only N and Q taken into consideration); the item for addition evaluation for special metallic parts is as the lower right side (with N, Q, and M taken into consideration). However, for special metallic parts, the plate design is evaluated in (3).
Embedded plate	- Steel Structure Design Standards - JEAG 4601-Supplement-1984 (Chapter of Allowable Stress) - "Notification No. 501" - Building Standard Law	(1) When support is large (load applied on a large area). The bending stress at the central portion is checked with the assumption that the stud portion is fixed and concentrated load is acting. Tensile load  (2) When support is small (load applied on central point). The bending stress at the central portion is checked with the assumption that the plate is fixed along the circle which connects the studs.  Concentrated load (tensile) (3) The bending stress due to concrete bearing reaction force is checked at point X. 
Studded bolt		(1) Round rod portion. Tensile load + shear load  (2) Top portion. As shown in the left figure, when a tensile load is applied, the shear strength at the top portion is checked at the portion indicated by the broken lines. (3) Welded portion. It is acceptable if the load of the round rod portion is less than 50% of the allowable load. (For the welded portion, the strength is guaranteed to be greater than 80% of that of the stud material.) —

Table 6.6.4-3. Evaluation methods of embedded metallic parts (Part 2).

Item	Design standard		Evaluation method
Load transfer	- Column foot design - (H-K-7)		- Axial force F (tensile, compressive) - Bending moment M - Shear force Q - Torsional moment T - Magnitude of support is considered - Mounting position of support is considered
Embedded plate	- JEAG 4601-Supplement-1984 (Chapter of Allowable Stress) - "Notification No. 501" - Steel Structure Design Standards		- Flexural stress due to tensile force in bolt - Flexural stress due to bearing force of concrete - Magnitude of support is considered - Mounting position of support is considered
Studded bolt	Same as above		- Combined stress due to tensile force N and shear force Q

6.7 Confirmation of functions of Class As and A equipment during earthquake

The functions of the equipment for which the appropriateness with respect to seismic motion should be confirmed include pressure resistance, strength and other static functions, as well as dynamic functions needed for rotation, opening/closing, and other operations during or after earthquake. Among these functions, for the static functions, the strength evaluation methods described in section "6.5.2 Class As and A equipment" or vibration tests for strength and deformation may be used to confirm the appropriateness. In the present section, we will discuss fundamental idea of the methods which confirm whether the equipment maitain the required active function during and/or after earthquakes.

6.7.1 Active equipment

In a nuclear power plant, the equipment which is needed to maintain the active functions during and/or after design basic earthquake ground motion include the following:

{1} Those which are needed to make emergency shutdown of the nuclear reactor and to maintain the safe shutdown state;

{2} Those which are needed to protect the public from radioactive hazard during a nuclear reactor accident. More specifically, the equipment include control rod driving mechanism, emergency core cooling pump, nuclear reactor containing vessel isolation valves, etc.

For this equipment, in past research work, typical equipment which was selected for every type of equipment to confirm the active functions during the design basis earthquake ground motion by vibration test or detailed analysis. For the actual active components, they are judged to be similar to the components that had been confirmed in past research work, confirmation of maintaining the active functions is performed by checking if the acceleration inputted to the active components is less than the acceleration confirmed in past research work. In addition, confirmation may be performed based on the response results which is derived from response analysis at each part of the components, in that case fundamental idea of confirmation should be as follows. This scheme, however, is for existing equipment. If the earthquake conditions, equipment structures, etc., are significantly different from those of the existing equipment, specific detailed evaluation is needed.

(1) Control rod driving mechanism

For the control rods and control rod driving mechanism, it should be confirmed that the control rods can be inserted into the core within a period needed for reactor safety evaluation during earthquake.

(2) Pumps and motions

The strength of anchor bolts and mounting bolts and the integrity of bearings both in pumps and in motors should be confirmed. For those with a long shaft, the aforementioned evaluation should be performed using a multiple discrete model for analysis.

(3) Turbine for driving pump

Just as in the case of a motor, the strength of the anchor bolts and the rotating function of the bearings should be confirmed. Confirmation of active functions is performed together with confirmation for the pumps driven by the turbine.

(4) Emergency diesel generator

Since the main body has a sufficiently high stiffness, evaluation could be made only for the anchor bolts. Also, it should be confirmed that the bellows in exhaust pipe are able to follow the displacement between the diesel engine and the diesel generator building during earthquake, sloshing of the lubricating oil sump tank and appropriateness of the speed gear.

(5) Valves

There are many types of valves. The method for checking the function maintenance should be checked by comparison of response acceleration with that of the sample established and evaluation of the strength for the anchor bolts.

(6) Other equipment

Other equipment with active function requirements include fan, damper, refrigerator, air compressor for control, etc. Just as in the case of valves, the function maintenance should be checked by comparison of response acceleration with that of the established acceptable acceleration values, and evaluation of the strength for anchor bolts. In addition, it is necessary to make sure that the fuel assembly does not hamper insertion of control rods and can be maintained in a coolable shape.

6.7.2 Electrical instrumentation and control equipment

Usually, the electrical instrumentation and control equipment is connected to various types of boards, apparatus, etc., and it is difficult to confirm its ability to maintain functions for the entire system at the same time. Hence confirmation of the function maintenance is performed by evaluating each board, apparatus, etc., and summarizing the results to guarantee the functions of the overall system of equipment. The equipment can be classified mainly into four types: boards, devices, apparatus, and circuits. Their abilities to maintain function are checked as follows. The definitions of the various types of equipment and corresponding examples are shown in Table 6.7-1.

(1) Boards (see Figure 6.7-1)

A board is an assembly of many apparatuses. Hence, its structure and functions must be appropriate to resist the design seismic motion.

In the case when an analysis model is available and analysis can be performed easily, the "scheme using analysis" can be adopted. On the other hand, when an analysis model is not available or, although an analysis model is available, seismic property can be evaluated experimentally, the "scheme using vibration test" can be adopted.

Based on the vibration analysis or vibration test, judgment is made of whether it is a rigid structure. If it is a rigid structure, the integrity of structure is confirmed by static analysis, on the other hand, if it isn't a rigid structue, the integrity of the structure and function should be confirmed by dynamic analysis or vibration test.

The following two methods can be used in the vibration test. In the first method, the boards installed the actual apparatuses are used for vibration test; In the second method, the boards installed the dummies that can simulate the actual apparatuses in vibrational characteristics and structures are used for vibration test. In the case of using the dummies, confirmation for the installed apparatus is performed by measuring the response acceleration at the mounting point of the dummy and comparing that with the verified function maintaining acceleration in vibration test for the apparatus itself. In the case of analysis, confirmation is performed by comparing the response spectrum calculated at the mounting point of the apparatus with the spectrum verified in verification test for the apparatus itself.

If the performance evaluated in the above is not appropriate, the practical counter measures such as amendment of the design should be taken.

Table 6.7-1. Classification, definitions, and examples of electrical measurement/control equipment.

Class	Definition	Examples
1. Board	It is a portion of the electrical instrumentation system. It is a structure made of steel beams, steel plates, etc., and containing apparatus, cables, etc., with the functions of processing and controlling signals in the electrical system and instrumentation system, protection of control of operation system, switching and conversion of power, etc.	Central control panels, locked power boards, power center, control center, instrumentation rack, on-site operation panel, static invertor battery charger, etc.
2. Device	A portion of electrical instrumentation equipment for conversion of electrical power or conversion of energy	Transformer, diesel generator, motor for auxiliary equipment, motor/generator, batteries, etc.
3. Instrument	Elements in the electrical instrumentation for performing detection, conversion, operation, control, etc., of the signals or electrical power to realize functions of electrical system and instrumentation system. They are mounted on panels or installed at prescribed locations.	Various types of detectors, signal emitters, protective relays, control relays, operators, switches, breakers, meters, transformers for instrumentation, current transformers, etc.
4. Circuits	When circuits which include electrical wires, cables, conductors, etc., are contained in the structure made of steel plate or other material to support and protect them, we say, the structures are included in the circuits.	Cable tray, bus duct, electrical conduit, cable penetration, conduits for instrumentation, etc.

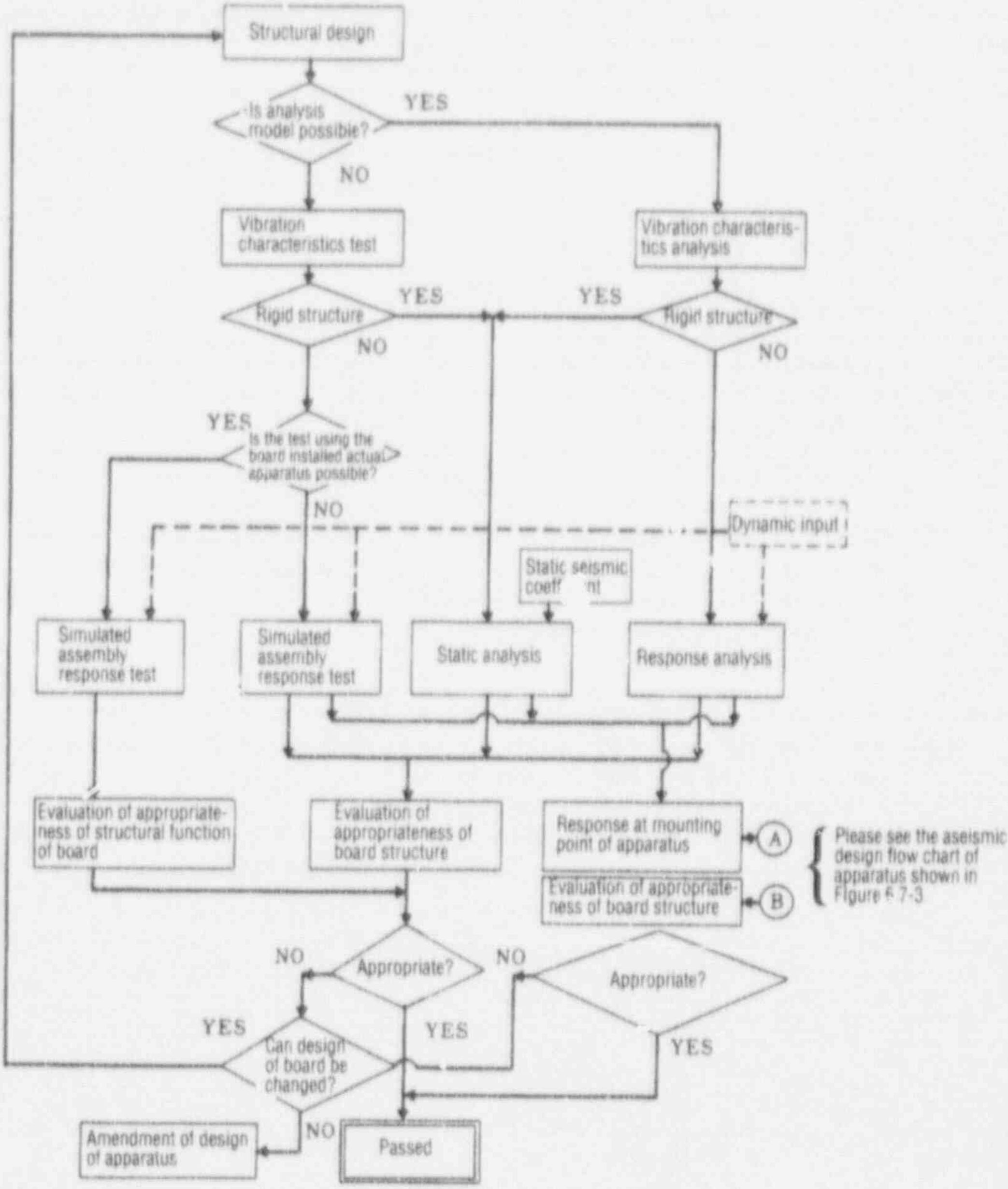


Figure 6.7-1. Flow chart of aseismic design of board.

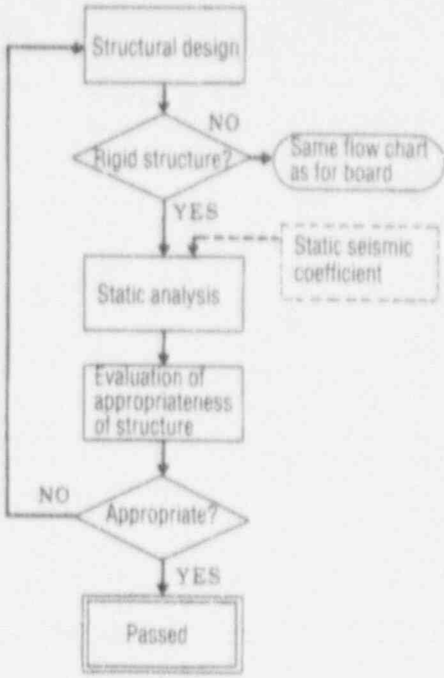


Figure 6.7-2. Aseismic design flow chart of devices.

(2) Device (see Figure 6.7-2)

The device is usually a rigid structure, and its function can be maintained as long as the structure is kept perfect. Hence, the seismic evaluation is performed by static analysis to confirm the structural appropriateness. If the device is not a rigid structure, however, the structural appropriateness can be confirmed according to the same flow chart as the panel.

(3) Instrument (see Figure 6.7.3)

Evaluation of instrument is performed in two aspects: structure and function. As instruments are usually mounted at different positions, verification test for the instrument itself is performed to determine limiting input earthquake motion that its functional/structural integrity is maintained in advance. Then the seismic resistant capability of instrument mounted is confirmed by comparing the response spectrum due to design basis earthquake ground motion at the mounting position with the spectrum verified in verification test for the instrument itself. It is also possible to confirm the seismic resistant capability by vibration test for the instrument. In the vibration test, the instrument should be excited by earthquake motion due to design basis earthquake ground motion at the mounting position of the instrument.

Among instruments for those that can be taken as rigid bodies, such as a transformer for instrumentation, as long as the structure is perfect, the function can be maintained. In this case, confirmation of structural/functional integrity is performed in the same way as for devices.

(4) Circuits (see Figure 6.7-4)

For circuits, as long as the structure is sound, the functions can be maintained. As a result, only structural evaluation is needed. In this case, the structural appropriateness is assessed using the dynamic analysis method depending on the aseismic class and natural frequency of vibration, or by using the static analysis method in the case

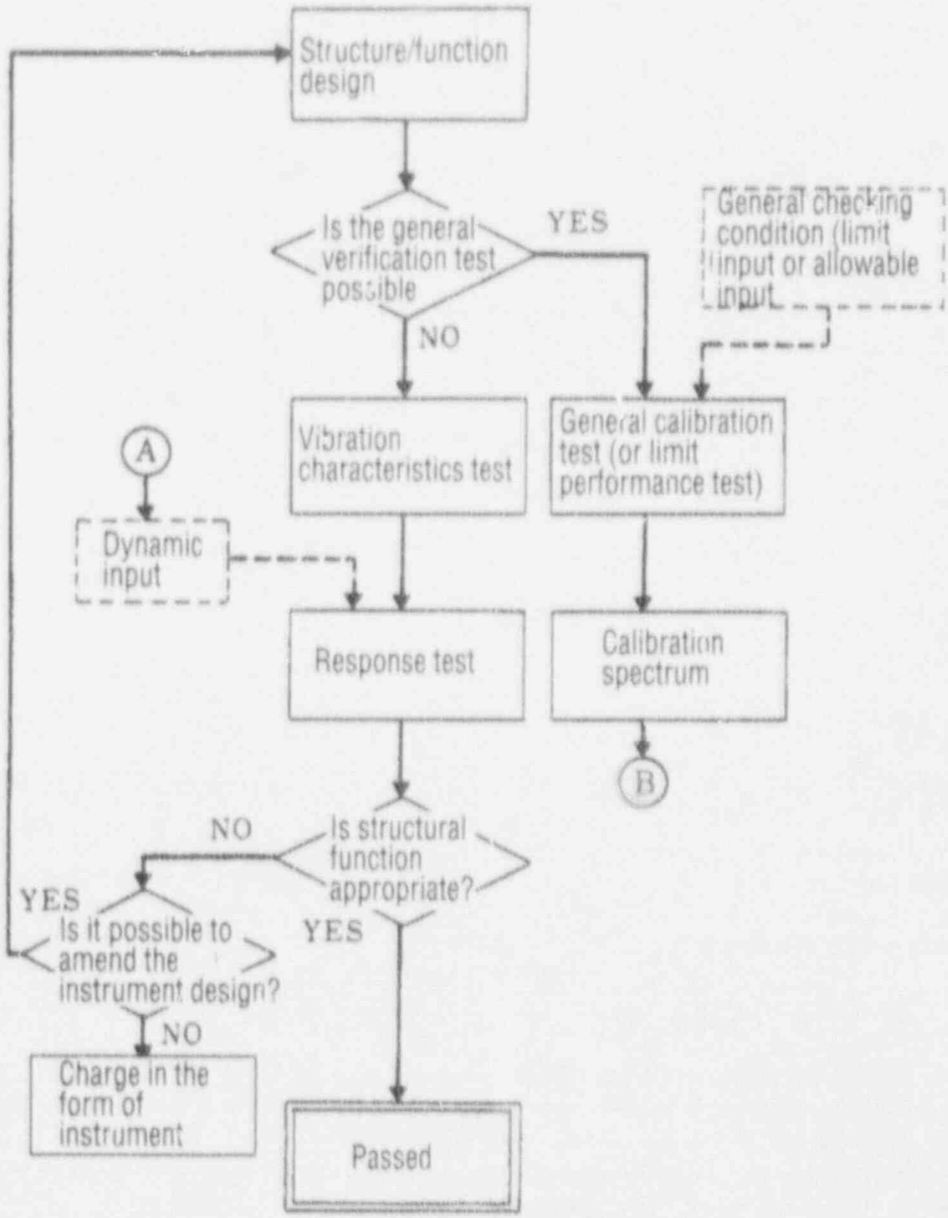


Figure 6.7-3. Flow chart of aseismic design of instrument.

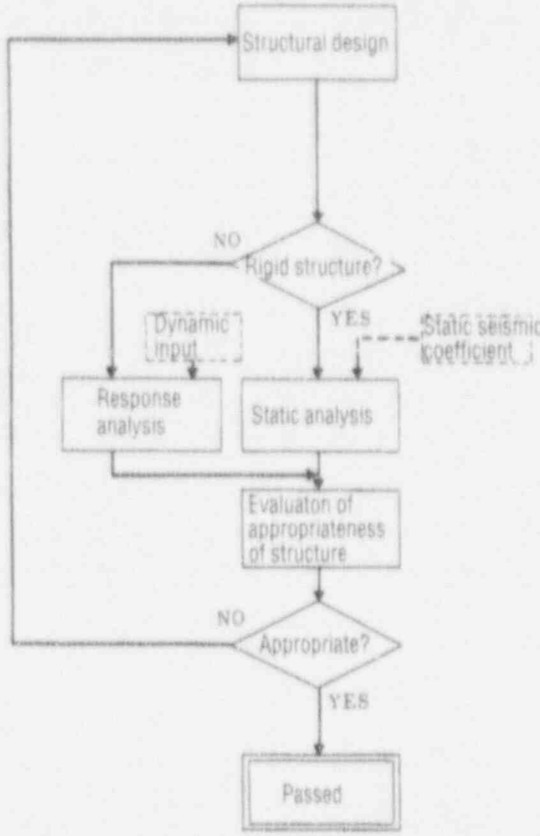


Figure 6.7-4. Flow chart of aseismic design of circuits.

of a rigid structure. In the case when the circuit is installed between buildings or between building and ground outside the building, the structure should be able to absorb the relative displacement during earthquake.

Besides, following erroneous operation are acceptable:

a. Erroneous operation to the safer side.
b. Erroneous operation in earthquake when the following two conditions are met simultaneously.

{1} The safety of the plant is not degraded even in erroneous operation in earthquake.
{2} The function recovers after earthquake.

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Chapter 7. Prospects of future technical topics

In recent years, the reliability of nuclear power generation has been improved based on actual construction and operation. As the techniques for consolidating the achieved results of nuclear power generation for the future development are being improved, efforts are made to improve the economic effect and safety. According to the conclusion reached at the "Conference on Improvement of Nuclear Power Generation" (Chairman: Masao Mukasaka, director of International Energy Policy Forum), "in order to improve nuclear power generation, it is necessary to establish an information processing system, to set up and improve the operation/service system, to develop high-level light water reactor technique, to increase overall efficiency of utilities and manufacturers, and to improve the official business administration in the country. In this way, high reliability and a better economic effect can be realized." In this way, it is expected that the overall design of the nuclear reactor facilities will be further streamlined. Aseismic design is a part of the above.

It is believed that the safety margin of the aseismic design of the nuclear reactor facilities is the result of accumulating the measures taken to ensure safety for the various aspects of soil-building-equipment, starting from determination of the standard earthquake motion in light of the various uncertain factors. For the actual nuclear reactor facilities, there is as yet no experience of a major earthquake. In order to supplement knowledge in this respect, many theoretical studies and tests have been carried out to increase the reliability of seismic design technology. At the present stage, efforts are being made to pinpoint the uncertain factors in the various special fields where safety consideration should be made. The survey/research work is now performed at various government and civil institutions.

These investigations and research are mainly within the range defined as the basic items required in the Evaluation Guidelines of Aseismic Design of Nuclear Reactors in Power Plants used at present (referred to as "Examination Guidelines" hereinafter). They are useful to improve and streamline aseismic design. It is expected that the results of these studies may upgrade aseismic design to a higher and more streamlined level.

On the other hand, efforts are also made on topics which are not included in the present "Examination Guidelines" and involve changes in the fundamental technical features, such as building the nuclear reactor buildings on a soft ground to reduce the seismic input into the facility, adoption of passive and active control systems or base isolation structures for the nuclear reactor building, as well as for structures and equipment within the nuclear reactor building. Although there are many problems to be solved to realize them, since they involve a radical change in the conventional practice and amendment of the "Examination Guidelines," it is still necessary to promote them from the viewpoint of variety of siting and economic effect.

The aforementioned Chapters 2 through 6, although certain new findings are included, are mainly a summary of the present seismic design of the nuclear reactor facilities. In the present chapter, however, emphasis is set on the prospects of the future technical topics for upgrading and streamlining aseismic designs in the various fields.

7.1 Earthquake and basic earthquake ground motion

As far as earthquake and basic earthquake ground motion, which are the basis of aseismic design, are concerned, since an earthquake itself is a natural phenomenon, it is difficult to perform experimental research on it at present. However, in order to improve the reliability, efforts are being made to establish an evaluation method, with the updated knowledge and technical know-how included.

In the following, we will present several items for which further research is needed.

7.1.1 Formation of standard earthquake catalog

In order to determine the basic earthquake ground motion, it is necessary to survey the past earthquakes, active faults and seismo-tectonic structure. One method involves using an earthquake catalog which is formulated by a survey of past earthquakes and lists the time, focal position, and magnitude of each earthquake. However, although the data for the earthquakes taking place since the early Meiji [about 1870] era are relatively complete due to measurement using instruments, for the so-called historical earthquakes taking place before the early Meiji era, there are many uncertain points for their data listed in the earthquake catalog. As a result, for each nuclear power plant site, an independent detailed investigation is usually performed on the historical earthquakes which have had relatively large influence on the site. In the future, it is necessary to form a more reliable catalog for the entire country.

7.1.2 Evaluation of seismo-tectonic structure

The evaluation of seismo-tectonic structure is performed on the basis of a map which determines the largest possible scales of earthquake that can take place at the various regions in Japan. This map is mainly formed on the base of the states of the major earthquakes that took place in the past. In the recent years, the research work in this field has achieved a rapid progress. As a result, based on recent observations of earthquakes as well as various results of geophysical observation and data on active faults, it is necessary to establish an evaluation method based on plate tectonics and can reflect these results.

7.1.3 Evaluation of earthquake ground motion characteristics

In recent years, a great effort has been made to understand seismic motion characteristics of hard bedrocks and amplification characteristics of seismic motion in bedrocks by collecting records of earthquake observation in horizontal and vertical alleys for the bedrocks corresponding to the grounds of nuclear reactor building sites with a reliable management system, and by using other records available both in Japan and abroad. As a result, the characteristics of the seismic motion have been gradually clarified. In the future, an even more rational evaluation method is to be established. In addition, when investigation is to be made on the Quaternary-era bed in addition to the present bedrock, it is necessary to establish an evaluation method for the seismic motion in a soft ground.

7.1.4 Evaluation of seismic motion based on fault model

To evaluate the long-period component seismic motions based on the seismic fault model, a simple model known as the Haskel model has been established and can represent seismic motion. On the other hand, for the mechanism of generation of the short-period component, there are still various ideas on the physical parameter that should be used to make a specific representation of the nonuniform rupture phenomenon. There is as yet no unified scheme in this respect. Consequently, in the case when the site is near a seismic focal region, there are several different means for evaluating the seismic motion using the fault model. It is necessary to establish a standard evaluation method in the future on the basis of the results of further research.

7.1.5 Vertical seismic motion

At present, for vertical seismic motion, aseismic design is performed with static consideration of the vertical seismic load. However, it is believed to be necessary in the future to perform a dynamic seismic response analysis with the vertical seismic motion as input. For horizontal seismic motion, a large amount of research results has accumulated on the frequency characteristics, etc. On the other hand, for vertical seismic motion, it is still in the initial stage of research. Hence, it is necessary to establish an evaluation method of the characteristics of the vertical seismic motion in the future.

7.2 Geological/ground survey

In Chapter 3, the present techniques of geological/ground survey for construction of nuclear power plants were summarized with the recent knowledge and technical know-how included. For the following items, however, further research is to be performed to obtain a better understanding.

7.2.1 Evaluation of fault activity

In order to determine the basic earthquake ground motion at the site, it is necessary to perform a reliable evaluation of the scale and activity of the active faults. For this purpose, much effort has been performed up to now. However, as far as the evaluation of activity is concerned, further research and development are still needed to improve the reliability and rationality. In particular, for the seabed fault, an effective method to determine the age of the seabed layers and a standard method for reading the record of maritime sonic survey are yet to be established. At present, with the improvement in the reliability of the analysis of substances within the fault, the evaluation of the activity of the fault in the bed can be made more rational by combining the conventional geological and topographical methods. On the other hand, although it is possible to estimate the presence of a fault from the geological point of view, it is rather difficult to clarify its presence from the surface geological survey. Hence, a better survey/evaluation method is to be established.

7.2.2 Survey method/evaluation method of gravelly bed

It is believed that the gravel ground has a rather high support strength when it is used as the support ground of a large-size foundation structure. As a matter of fact, in Japan, many skyscrapers are built on gravel ground. In other countries, some nuclear power plants are built on gravel ground. However, although it is believed that the seismic stability of the gravel ground is high, there are still many unclarified points concerning the survey/test method of the gravel ground and the safety evaluation method of the gravel ground in the case of earthquake.

Consequently, it is necessary to develop in the future sampling methods and on-site test methods as the gravel ground survey methods, evaluation methods of liquefaction resistance, prediction methods of deformation behavior during earthquake, evaluation of embedment effect of foundation to ensure the foundation safety, evaluation of foundation bottom shear resistance, evaluation of side earth pressure during earthquake, and countermeasures against underground water.

7.2.3 Evaluation of joint bedrock and discontinuous plane in bedrock

The topics yet to be solved in respect to evaluation of the mechanical characteristics of bedrock include establishment of a method of evaluating the mechanical characteristics of bedrock having joints and other geological separating planes and development of an effective survey method of the discontinuous planes in the bedrock.

For the former topic, it is necessary to establish a method of finding statistical data concerning the distribution and properties of the separating planes and to establish a method of evaluating the mechanical characteristics on the basis of formulation of the system of the various mechanical test methods corresponding to the states of the separating planes.

For the latter topic, it is necessary to develop a technique for clarifying the bed structure in the order of several tens of cm for a range to about 200 m underground, and a technique for automatically drawing the obtained data in three-dimensional form, so that a more rational survey can be performed for the discontinuous plane in the bedrock.

7.2.4 Evaluation of tensile strength of bedrock

For seismic stability analysis of bedrock, tensile stress occurs in a wide range in the bedrock in some cases. For a hard bedrock, of course, there is a tensile strength which can be evaluated in design. However, in the process of evaluating stability, in almost all the cases, the tensile strength of the bedrock is not yet taken as a resistance. This is because it is difficult to evaluate the separating plane of the bedrock and its effect on the tensile strength.

Consequently, progress is expected with respect to the best evaluation method of the bedrock at the site based on the theoretical explanation related to the geological survey and mechanical evaluation of the separating plane, etc., as well as the methods for utilizing these results. In addition, together with these research studies, many tests of the tensile strength of bedrock are to be implemented to clarify the effects of stress path, ground condition, dimensional effects, etc., on the tensile strength, by finding the correlation between the tensile strength and the other properties.

7.2.5 Correlation between static properties and dynamic properties

Although there is no special problem for a hard bedrock, for a relatively soft bedrock or weak layer, it is necessary to perform a detailed aseismic stability evaluation by a dynamic analysis. In this case, in order to streamline the methods of bedrock survey/test, it is desirable to clarify the correlation among the physical characteristics, static strength/deformation characteristics, and dynamic strength/deformation characteristics of the bedrock for each type of bedrock.

More specifically, there are the following items.

(1) The result of bed survey/tests performed in the past are summarized to form a database, so that the correlation among the mechanical properties and physical properties can be understood for the various bedrocks.

(2) The parameters with the highest degree of correlation are selected appropriately to formulate the correlation between the physical properties and the mechanical properties and the correlation between the static properties and dynamic properties.

(3) Based on the aforementioned results, the survey/test method of the bedrock is streamlined.

7.3 Stability evaluation of ground and aseismic design of underground structures

In Chapter 4, the present methods for evaluation of aseismic stability of ground for construction of nuclear power plants are summarized, with the updated knowledge and technical know-how included.

However, for the following items, it is necessary to perform further research with the purpose of further improving the reliability and efficiency of the aseismic stability evaluation of ground.

7.3.1 Seismic coefficient for ground

In order to determine the seismic coefficient for the ground, the following methods may be used to be consistent with basic earthquake ground motion S_2: (1) the method in which the maximum dynamic shear stress distribution in the ground is used; (2) the method in which the maximum value of the instantaneous acceleration of the earth mass on the slip plane at various depths. For the seismic coefficient determined using these methods, the seismic motion which varies in time is replaced by a static one. This may lead to a more severe condition than the actual ground vibration under rapidly repeating cyclic loads. From this point of view, it is necessary to clarify the relation between the peak value and the effective value that actually affects the stability and to set an appropriate seismic coefficient for a more rational aseismic evaluation.

In addition, for the underground distribution of the seismic coefficient and the effect of the soil type, at present, research is being performed by the Underground Seismic Intensity Division, Nuclear Power Civil Committee, Japanese Society of Civil Engineers as a topic of joint research in the field of electrical power. The results of this research are expected.

7.3.2 Earth pressure during earthquake

In order to perform a rational evaluation of the earth pressure in an earthquake, earthquake observation of the underground structure and modal vibration test are performed. For example, the earthquake observation is performed for LNG underground tank, etc. Up to now, data for medium and small earthquakes have been obtained. According to these observation data and modal vibration tests, the value and distribution profile of the earth pressure usually depend on the relative stiffness between the ground and structure as well as on the contact state of the structure (attached on rock or not). Also, the earth pressure may be simulated to a certain degree by performing analysis using an equivalent linear method. However, for evaluation of earth pressure in the case of a major earthquake with residual displacement generated in the bedrock, more precise property representation and analytical methods should be used. Based on these methods, a rational evaluation method of earth pressure during earthquake that fits better with the actual situation will be developed.

7.3.3 Large deformation problem

When a weathered soft bedrock or a soil ground is selected as the site, if the assumed seismic force is large, an analysis should be performed to predict the deformation and to determine the measures against the deformation. The earthquake damage to a structure is mainly caused by the repeated action of the shear stress, which leads to a decrease in the strength of the ground, accumulation of deformation, and generation of excessive pore pressure.

As far as the earthquake deformation is concerned, there are still many problems to be clarified. Research should be made on the following aspects: (1) formulation of the accumulated behavior of the shear deformation and volume change in ground materials, estimation of decrease in strength, and evaluation of excessive pore pressure; (2) development of a nonlinear response analysis method using direct integration; (3) clarification of the deformation mechanism and acquirement of actual data in vibration table test; etc. Based on an organic coordination of the results of this research, a progress in the design method and its application will be achieved.

7.3.4 Limit-state design of important underground structures

In the conventional technique, the seawater duct, water intake pit, water intake tower, and other reinforced concrete structures are designed on the basis of the allowable stress method, which, however, may not be rational. It is important to perform the design by accounting for the functions required for the structure as well as the seismic load level of S_1 or S_2. For this purpose, for the S_2 earthquake, it is necessary to establish a large-deformation analytical method which can ensure that the strength capacity of the cross section following the concept of the ultimate strength design, and to use it in the actual design on the basis of the model tests which prove the appropriateness of the method.

Japanese Society of Civil Engineers (JSCE) is planning to adopt the limit-state design method. It is believed that as this design method is applied to the important underground reinforced concrete structures, it is possible to reduce the cross section and the amount of reinforcing bars used. In this way, the design of these structures will become more rational.

7.4 Aseismic design of buildings and structures

Research on the nuclear power generation techniques involves various fields. For the aseismic design, there are also various research fields with emphasis on the tests and measurements of seismic motion, nuclear reactor facility's vibration characteristics, aseismic structures, etc., as well as various analytical and evaluation schemes. Recently, as a major theme shared by many parties, research has been carried out as Electrical Power Joint Research.

In addition, under a contract with the Ministry of International Trade and Industry, the Nuclear Power Engineering Test Corporation is carrying out a large-scale test on the interaction between the nuclear reactor building and ground as well as the restoring force characteristics of the shear walls.

As far as standardization of aseismic design is concerned, progress is being made in the field of standardization of the seismic design method performed as the Light Water Reactor Modification Standardization Aseismic Design Survey (First-Third).

In the future, in order to improve the economic effectiveness of nuclear power plants, a more rational aseismic design is desired. In the following, we will present the major topics related to streamlining the aseismic design of the building/structures of the nuclear reactor facilities.

7.4.1 Handling of soil-structure interaction in design

For a long time, people have known the importance of an appropriate evaluation of the soil-structure interaction in aseismic design of the nuclear reactor facilities. Up to now, many theoretical and analytical research studies have been performed to address this problem both in Japan and abroad. Also, actual research works have been performed in Japan with respect to vibration experiment and earthquake observation.

As far as the interaction problem is concerned, relatively simple problems, such as the behavior of a rigid foundation on a uniform semi-infinite elastic ground, have been fully clarified in theory, with results in good agreement with the actually measured results. In addition, for the case with a large foundation deformation, the case with a complicated foundation shape, or the case when the ground has a layered form or irregular form, FEM and other methods of discretization are developed as powerful means for analysis. However, in many cases, they become three-dimensional problems and require a very long time for direct computation. A similar situation takes place in the case when the building is buried deeply and in the case when the effect of the interaction with an adjacent building must be taken into consideration. In the future, it is important to find methods which can further clarify these problems in a simpler way for handling in design.

7.4.2 Earth pressure during earthquake

The earth pressure acting on the underground wall of a building during an earthquake is affected by the stiffness of the soil, the vibration characteristics of the structure, the characteristics of the seismic motion, etc., and varies in a complicated way. Since there are few examples of buildings damaged by earth pressure in earthquakes, the mechanism has not been fully clarified, and a rational evaluation method that fits well with the actual situation is not yet established.

The earth pressure during earthquake should be evaluated as a result of the dynamic interaction among the support ground and the fill earth on the periphery of the building. In this case, it is necessary to evaluate it with attention paid to the geometrical nonlinearity such as the stress-strain relation of the support bedrock and fill earth, the nonlinearity of the material damping, and the slip and separation at the contact plane between the building and the ground. As far as the actually measured data of the earth pressure acting on the actual structure in an earthquake are concerned, since the history of observation is relatively short, sufficient data are not yet available.

In order to address this problem, it is desirable to perform various analytical discussion from the design point of view with the results of experiment and measurement.

7.4.3 Restoring force characteristics

Recently, many experiments have been performed on the restoring force characteristics, hysteretic characteristics and the major parameters that affect these characteristics for box-shaped, cylindrical, and other shapes of shear walls of nuclear reactor buildings. As these parameters are sorted, they are expected to play their roles in the future design for improving the reliability of the analysis evaluation.

The future topics of the reinforced concrete shear walls include experimental classification of the effects of strain rate, variation in axial force, two-directional force application or other loading application method on the restoring force characteristics, dynamic experiments using a vibration table, clarification of the effects on stiffness and restoring force characteristics for walls with openings from both experimental and analytical approaches, etc. Also, for composite shear walls with box-shaped and cylindrical walls connected at the floor, accumulation of experimental data and analytical research are desired. The difference between the test specimens used in experiment and the actual building's shear walls, e.g., the effects of the difference in their dimensions (scaling effects) and the effects of the wall thickness ratio, etc., should be further clarified. It is also necessary to further clarify the relation between cracks in the shear wall and the requirement for the ability of a wall to maintain its function.

For the structural parts other than the shear wall, i.e., floor, foundation mat, partitioning walls, SRC structure, etc., it is necessary to clarify experimentally the restoring force characteristics and strength in order to achieve designs with a higher degree of freedom.

7.4.4 Investigation of function maintenance

For investigation of function maintenance, the first important task is to find out how to relate the functional requirements for the various portions of the building to the structural behavior of the components. This is a basic problem.

Evaluation of the building can be divided into an overall system and various partial systems. For the overall system, the evaluation means of responses under basic earthquake ground motion S_2 have been nearly established, and the index for the safety margin can be calculated. In the future, efforts should be made to further rationalize the system, and it is necessary to perform a more detailed investigation of the relation between the behavior of the overall system and the functional requirement of the building.

For the various partial systems, at present, design is performed on the sufficiently safe side by using a relatively simple evaluation method. However, in consideration of the variety of structural shapes and loading conditions, as well as the plastic behavior and the ultimate strength, in order to establish a more rational design scheme it is necessary to perform extensive experimental and analytical research. In the plastic analysis, sufficient attention should be paid to the boundary conditions when analyses are performed for each portion separately.

7.4.5 On seismic safety margin

According to the "Evaluation Guidelines," the horizontal strength capacity of a building/structure should be attained with an appropriate safety margin, depending on the importance, with respect to the required horizontal strength capacity calculated according to the scheme defined in the Building Design Code. In addition, for Class As building/structures, with respect to the combination of the long-term load, load during operation and seismic force due to standard seismic motion S_2, the building/structure should have a sufficient margin of deformation ability as an overall structure, and an appropriate safety margin with respect to the ultimate strength of the building/structure. For this safety margin, there is no quantitative representation of the lowest limit depending on the importance.

In this respect, specific evaluation should be made of the reliability of the evaluation method of the strength capacity or ultimate strength of the building/structure, the deformation ability, scatter in strength and deformation ability of the structure, reliability of the evaluation method of seismic force caused by S_2 earthquake, as well as requirement of maintenance of the safety role of building/structure and maintenance of function in the plant facility. On the basis of this investigation, the lower limit value of the margin can be determined.

7.4.6 Dynamic analysis of vertical motion

At present, for both building and equipment, a static evaluation method is performed on the vertical seismic force, i.e., the seismic load caused by the vertical component of the seismic motion. According to the experience of major earthquakes in the past, most damage is caused by horizontal motion, while the damage caused to a building/structure due to the vertical motion is quite limited.

For the building/structure of a nuclear power plant, judging from its structural features, the amplification is usually small for the seismic response in the vertical direction. However, people in the USA and West Germany still perform dynamic analysis for the response to vertical motion. In consideration of this tendency, it is believed that we should also take the vertical motion into consideration by combining it with the response of horizontal motion to find its effect on the design by understanding the mechanism of the response to vertical motion of the nuclear power plant facilities. For this purpose, various research has been performed in the survey on standardization of the aseismic design of the light water reactor modification/standardization program and in the form of electrical power common research. In the research performed up to now, much effort has been performed on the analytical models of building/structure, effects of the foundation mat stiffness on the interaction between building and ground, etc.

Since the vertical motion response of the building/structure has a significant influence on the vertical motion response of the equipment/piping system, it is necessary to perform sufficient investigation of this problem in the future. Many researchers have addressed the problem of the vertical component of the seismic motion used as input to vertical motion response analysis. Also, recently, many measurement records have been obtained. However, a method is yet to be determined for the standard seismic motion used in the design evaluation of nuclear power facilities.

7.4.7 Base-isolated structures

Research on the base-isolation has been carried out in Japan for a long time. Recently, this structure has been adopted for some buildings in New Zealand and the USA. France is the first country which formally adopted the idea of base-isolation for the nuclear power plants. There are cases where the base-isolation has been adopted for all nuclear reactor buildings in some PWR plants in South Africa and France. The foremost merit in adopting the base-isolation in France is that with this earthquake-proof method, the plant can be designed as a standard design which does not depend on the specific ground conditions and can be built anywhere. As adoption of the base-isolation increases the freedom of aseismic design, it is believed that it will be further studied by various countries in the future.

The techniques for using base-isolation in nuclear power plants include the total isolation technique, which isolates the entire building, and the partial isolation technique, which isolates only a portion of the important equipment. They are selected according to the plant configuration and aseismic design conditions.

When base-isolation is adopted, it is necessary to perform sufficient investigation of the reliability of the isolation devices, maintenance, economical effectiveness of the plant as a whole, etc. In order to evaluate the earthquake strength of a plant with base-isolation, it is important to clarify the effect of the long-period component of the seismic motion.

7.4.8 Site on Quaternary-period ground

In Japan, up to now, the nuclear reactor buildings and other important buildings and structures have been built on a rock site in principle. Recently, however, as a measure to enlarge the selection range of the sites of nuclear power plants, investigation is also made of building nuclear reactor facilities on Quaternary-period ground.

A topic to be investigated in order to evaluate the aseismic safety when the nuclear reactor facility is built on a Quaternary-period ground concerns the design seismic motion. This is related to the amplification of seismic motion in the Quaternary-period soil. Also, in order to obtain the basic data for evaluating the safety of the soil and earthquake strength of the structure, it is important to determine the survey of ground to be performed for evaluating the properties of the soil.

In consideration of the design of building and structure, it is necessary to pay a great deal of attention to forming the model of interaction between the building and soil to perform seismic response analysis and to treat the problem of nonhomogeneity of soil. For a softer soil, the earthquake response acceleration of the nuclear reactor building can be reduced, and the seismic input to the equipment/piping system can be reduced. These are advantages. On the other hand, the displacement in earthquake is increased, and the interaction between the building and the soil becomes dominant in the earthquake response. Hence, sufficient attention should be paid to the evaluation. Also, since the strength of the support ground is lower than that of the rock bed, in some cases, it is necessary to reduce the weight of the building.

7.5 Aseismic design of equipment/piping systems

As far as the aseismic design of equipment/piping systems is concerned, a detailed description is presented on the importance classification, load combination and allowable limit in "Technical Guidelines of Aseismic Design of Nuclear Power Plants: Volume of Importance Classification/Allowable Stress, JEAG 4601, Supplement—1984." These guidelines are believed to be sufficient to implement aseismic design.

On the other hand, the above guidelines do not provide a detailed description of aseismic analysis (earthquake response analysis, seismic stress, strength analysis, etc.). This is because rapid progress in technology is being achieved in this respect, and a fixed form of guideline could hamper further development in the technology in this respect. Recently, however, it has become desirable to standardize the important portion in the seismic analysis technology as the standards for design due to the following reasons:

- Many examples have been accumulated in seismic analysis, and the importance of standardization has been increased. At present, many items have almost become standards and are used routinely.

- On the other hand, with further development in technology, new knowledge is obtained and may lead to better methods. However, if the conventional method is made standard, it would become difficult to adopt the aforementioned better methods. That is, although new knowledge is obtained with the technical progress, it would be difficult to adopt it. This would lead to a decrease in the speed of technical development.

In this section, we will discuss not only the standard routine methods in design, analysis and techniques (standard techniques at present) for aseismic design of the equipment/piping systems, but also the new findings in design methods, as well as the techniques which are expected to be standardized and adopted (powerful techniques, based on new findings) and the items of design, analysis and techniques (future techniques) which are expected to have a significant technical development.

7.5.1 Present standard techniques

In this respect, there are the following items:

(1) Method for determining the design floor response spectrum
(2) Spectral modal analytical method of Class As and A equipment/piping systems
(3) Time history response analysis method of Class As and A equipment
(4) Method of analysis of seismic stress and strength of Class As and A equipment/piping
(5) Seismic analysis method of Class B and C equipment/piping systems

For these items, the present aseismic design techniques are described in detail in Chapter 6 "Aseismic design of equipment/piping systems."

The contents of Chapter 6 can be used as the Guidelines of the Electrical Society of Japan (Civil Guidelines). However, there is still room for further improvement in the following respects: for item (1), the ±10% broadening method in the period of the design floor response spectral model; for item (2), spectral modal analytical method (SRSS). Also, for item (3), since the time history response analytical method (ground-building-equipment coupled system) depends on the time history response analytical method of the ground-building interaction system, further development is expected.

For a small-size light-weight equipment system, an independent vibration system can be assumed using the floor motions as input according to the above items (1) and (2). It is desirable that this technique be standardized for general application.

For item (4), the stress/strength analytical method and for item (5), Class B and C analytical method, the degree of completion is high. It is believed that they may not limited to light water reactors and may be used in a wider range.

7.5.2 High technology based on new knowledge

In this respect, there are the following items:

(1) Design damping constant
(2) Evaluation method of earthquake strength of support structural portion (including anchor portion)
(3) Evaluation method of aseismic safety of active equipment
(4) Other items reflecting high technology in design

As far as the design damping constant (item (2)) is concerned, the conservative values are used. However, according to the various tests and experimental research performed recently, the values listed in Chapter 6 are now used as the design damping constants of piping system, cable tray, electrical panel, air conditioning duct, etc.

These values are used in the S_1 earthquake linear response analysis. They are the lower limit values considering the scatter in test data.

The conventionally used value of the damping constant for vessels is 1%. Since the vessels are usually the rigid structure with a low earthquake response amplification, the influence of the damping constant is small. As a result, almost no testing research is performed on the damping. However, in the case when a rigid structure is difficult to achieve for a large-size vessel, or for the future design based on dynamic analysis, it is desirable that further data be collected in the future for damping values.

Due to the properties of the damping mechanism of the equipment system, there is a large scatter in the data. For the elastic design of the system, it is appropriate to adopt its lower limit value. However, in the case of evaluation of the reliability of the entire system, it may be necessary to adopt other methods (such as the method of consideration of the mean value-standard deviation vrlue). Related to this aspect, there remains certain items for investigation on the value to be adopted for S_2 earthquake-linear response analysis.

For the aseismic evaluation of the support structural portion in item (2) and the dynamic equipment in item (3), the main point is to find an evaluation method with higher reliability. At present, survey is performed with emphasis on experimental research. The evaluation method will be established in the near future.

For the related S_2 earthquake linear response analysis, however, it is still necessary to perform further evaluation of the value that should be adopted.

For the seismic evaluation of (2) support structure and (3) active equipment, efforts are mainly made to develop evaluation methods with higher reliability. At present, survey is being made on the basis of various experimental research. The appropriate evaluation method might be developed in the near future.

7.5.3 Techniques to be used in the future

In this respect, the following items are taken into consideration:

(1) Evaluation cf stiffness of support structures (including anchorage).

(2) Seismic response analysis method of equipment/piping system having support structure with gaps.

(3) Seismic response analysis methods with the three-dimensional input of the piping system taken into consideration (including multi-input method, SRSS method, etc.).

(4) Adoption of addition damping mechanism of equipment/piping system (pure damper).

(5) Base-isolation of support structure of equipment/piping system.

(6) Evaluation of the elastoplastic response of Class As and A equipment/piping system, in particular, response characteristics of support structures.

(7) Evaluation of safety margin up to the limit for maintaining functions of Class A... A equipment/piping system (including maintenance of functions of pressure portion and dynan... quipment).

(8) Stochastic evaluation of seismic safety.

In order to further improve the rationality of a portion of the seismic analysis method (such as increase in damping, decrease in response spectrum broadening rate, etc.), it is necessary to perform analysis with a higher reliability with respect to factors important to ensure assumptions on the safe side, such as effects of deviation in the natural frequency of vibration, deviation in the response amplitude, etc.

From this point of view, for the aforementioned items (1) stiffness evaluation of support structure, (2) analysis of support structure with gaps, (3) multi-input method, SRSS method, etc., it is desirable to evaluate the effects of the analysis methods on the seismic response, relationship with the safety margin for the conventional methods (items in sections 7.5.1-2), and to reflect the results in the guidelines.

Adoption of additional damping mechanism (pure damper) as item (4) is accompanied with development of damping mechanism (improvement of hydraulic dampers, etc.) and development of seismic analysis codes for the discrete mass-discrete damping constant system. As the softer design for piping system is being considered for future application, this survey is believed to be effective and desirable.

For item (5), base-isolation of vibration of equipment/piping system, it is desirable to evaluate the method of partial isolation of the containment vessel's internal structure, emergency equipment system, etc., and the isolation device of the equipment system with soft structure-large displacement absorption structure, such as piping/wiring, etc. The essential advantage of the base-isolation is in the establishment of standard configuration of the equipment/piping system independent of the site and seismic conditions. As a result, it is desirable to perform evaluation to realize practical application of the base-isolation of the overall nuclear reactor building.

For the final items of (6) elastoplastic response evaluation, (7) evaluation of safety margin up to the function maintaining limit, and (8) stochastic evaluation of seismic safety, in addition to the intrinsic problems of the equipment/piping system, there are also problems with respect to the reliability of the seismic motion-soil-building floor response. Consequently, it is very difficult to obtain a correct evaluation, which, however is the most important factor in the seismic safety evaluation of the nuclear power equipment.

At present, efforts are made to evaluate the seismic motion-soil-building floor response as an intrinsic problem of the equipment/piping system. In addition, research programs are also carried out actively for aspects related to the seismic motion-soil-building [floor] response portion.

Postscript

In Japan, 33 nuclear power plants are in operation, and 17 nuclear power plants are under construction or on drawing boards (as of July 1986). In other words, we will soon have 50 nuclear power plants in operation. There is no doubt of the importance of the nuclear power plants as a type of energy source in Japan.

On the other hand, however, the developers of nuclear power plants have always been worrying about the fact that Japan is a country with frequent earthquakes. As a matter of fact, special aseismic design measures have been taken for nuclear power facilities starting from Tokai No. 1 Power Plant, which is a gas-cooled reactor whose construction started in 1960, and Tsuruga No. 1 Unit, the first light-water reactor whose construction started in 1967.

At the time when Tokai No. 1 Power Plant was designed, the dynamic analysis method was not yet established. More emphasis was put on making radical changes in the building's structural planning and core structure from the viewpoint of aseismic design, while a static method was adopted for the analysis. On the other hand, Tsuruga No. 1 Unit was designed originally in the USA. Dynamic analysis, which had not been adopted in the USA, was fully adopted, using earthquake records (El Centro, Golden Gate wave) as the input wave, with SR model adopted for the SSI analysis, and with a flexural-shear type multiple discrete mass model taken as the model of the building. In order to perform design for equipment, floor response curves were formed; for the building, equipment, etc., based on the importance classification, the design seismic force was also changed correspondingly. In other words, the basic items of the aseismic design used at present were already established for Tsuruga No. 1 Unit.

While Japanese take the aforementioned measures to ensure the aseismic design of nuclear power plants, foreign countries are falling behind to take appropriate measures in this respect. In the USA, nuclear power plants located in middle/east regions are now in operation almost without any aseismic design. At present, Americans are paying attention to this problem by evaluating the seismic safety of the facility on the basis of the internal margin by using probability theory. In the UK, which owns 35 gas-cooled reactors in operation and has never been troubled by earthquake, people also begin to consider the effects of earthquake, which seldom takes place in England, and they have decided that for the light-water reactors to be built in the future, the 0.25 G (SSE) seismic motion will be taken into consideration. Hence, compared with the various foreign countries, Japan is a pioneer with respect to the aseismic design of nuclear power plants, with appropriate measures taken from the initial period when the nuclear power plants were first introduced to Japan.

In the initial period, the light-water reactors in Japan were introduced from the USA, with specifications of the aseismic design made by the [Japanese] electrical companies while specific analysis/design was implemented in the USA. The manufacturers and construction design engineers in Japan imitated the analysis/ design method in the starting stage. This is really an ironic phenomenon. Then, "Technical Guidelines of Aseismic Design of Nuclear Power Plants, JEAG 4601-1970" (Japan Electrical Association) was published in 1970 after two years of evaluation since 1968. This book plays the role of a textbook and has made great contribution to the education of the related engineers. For example, the mechanical engineers usually are accustomed to using the seismic load defined in the Building Standard Law in machines with an acceleration of 0.2-0.3 G as the conventional value. They are psychologically reluctant to accept the fact that the response value of the mechanical system with a small damping is over 10 times the aforementioned value. With the aid of the aforementioned guidelines, however, the engineers understand that a higher response may take place and that the aseismic design of nuclear power facilities must be performed in consideration of the higher seismic load they have never experienced.

Since Tokai No. 1 Power Plant, it has become a rule that once the site of a nuclear power plant is determined, the history of earthquakes and the damage caused by them in the vicinity be surveyed. While the American seismologists explain the cause of earthquake as due to active faults according to the elastic rebound theory (H. F. Reid, 1910), the Japanese seismologists believe that the faults are manifestations of earthquakes on the ground surface. Dialogue did not exist among seismologists and geologists. Since the late 60s, however, the ideas of plate tectonics have been widely accepted, and the relation between earthquake and active fault cannot be ignored. As a result, in the aseismic design of the nuclear power plant, survey of the active faults has become an important item with the active faults taken as the hypocenters of the earthquakes that are assumed for the aseismic design. Since the sites of the nuclear power plants are always selected in the coastal area, survey of the active faults should be performed not only on the land but also on the seabed. Since survey and judgment of the seabed geology are difficult, this is still a challenging job at present. In "Regulatory Guide for Aseismic Design of Nuclear Facilities" (Japan Atomic Energy Commission) and "Introduction to Safety Examination of Geology/Soil of Nuclear Power Plants" (Nuclear Reactor Safety Special Examination Council), both published in 1978, the items concerning survey/evaluation of active faults are described.

Although the aseismic design of nuclear power plants was implemented from the very beginning stage as pointed out above, the aforementioned "Examination Guideline" was the first standard published in written form. It was the result of a tedious preparation process. In 1958, the Ministry of International Trade and Industry set up a "Committee on Safety Standard of Nuclear Power Station," which published "Primary Report on Safety Standards" in 1961. A portion of this report has been incorporated into the aseismic design. However, it was only in the form of a report.

Then, the "Earthquake Countermeasure Subcommittee" in the aforementioned Safety Standard Committee was asked to continue examination of the aseismic design by the Ministry of International Trade and Industry. This subcommittee published "Report of Survey on Aseismic Design of Nuclear Power Plants" in 1965. In the preface of this report, it was pointed out that "the most rational aseismic design is based on dynamic analysis. However, there is yet no publicly acknowledged result of the s[illegible]titative method for performing this analysis in Japan. Consequently, at present, it is difficult to publish th[illegible] on the aseismic design for the nuclear power plants."

In order to prepare a standard for the aseism[illegible]n of nuclear power plants, the Ministry of International Trade and Industry asked the Japan Electrical Association to publish the aforementioned "JEAG 4601-1970" as a civil guideline in 1970. Afterwards, a portion concerning importance classification and allowable stress was added to it. The evaluation was started in 1976. In 1984, as a supplement to "JEAG 4601-1970," the "Technical Guidelines of Aseismic Design of Nuclear Power Plant: Classification of Importance Level/Allowable Stress Edition, JEAG 4601-Supplement-1984" was published.

On the other hand, according to the new aseismic design method for buildings, for which the study started in 1972, an amended edition of the "Implementation Law of Bulding Standard Law" was published in 1981. In addition, the aforementioned "Examination Guideline" drafted in 1978 was amended in July 1981. In this amended edition, the horizontal seismic coefficient was replaced by story shear coefficient, and the formulas for calculating the horizontal seismic force and the required horizontal strength are presented in the commentary.

Since 1975, the Ministry of International Trade and Industry has been engaged in establishing the plan for the improved light-water reactor standard. The purpose is to improve the reliability and operating efficiency of the light-water reactor by using the independent technology of Japan, and finally to establish the Japanese type of light-water reactor. As a measure taken in this respect, the aseismic design is standardized. Before 1980, this work was directly performed by the Ministry of International Trade and Industry. In the period of 1981-1985, it was performed under commission at the Nuclear Power Engineering Corporation. As a result of this work, standard design seismic motion was established, and the aseismic design of buildings and equipment as well as standardization of the analysis methods were realized.

Although the aforementioned "Introduction to Safety Examination of Geology/Soil of Nuclear Power Plant," published in 1978, listed the items of survey for the vicinity of the site and the site itself, it did not describe the survey method and judgment standard. Hence, when survey is to be implemented, the specific guidelines should be drafted. Also, among the facilities of the nuclear power plants, the civil structures have features that are different from those of the equipment/piping, and the soil stability problem is out of the scope for design engineers responsible for equipment design. In order to establish systematic guidelines for the aseismic design of the nuclear power plant in the civil engineering field on the basis of existing methods, upon request by the Ministry of International Trade and Industry, the Japanese Society of Civil Engineers started evaluation of the survey/test methods of geology/soil and seismic stability of soil. As a result of this work, a report titled "Evaluation Method of Survey/Test Method of Geology/Soil and Seismic Stability of Soil of Nuclear Power Plant" was published in 1985. This report summarizes the experiences in actual nuclear power plants concerning the survey/test method of geology/soil and their representation, as well as the seismic safety evaluation methods for foundation soil of nuclear power plant, peripheral slope, and important underground structures. It synthesized these experiences and summarized them, with many examples and data presented.

Since the "Technical Guideline of Aseismic Design of Nuclear Power Plant, JEAG 4601-1970" was published in 1970, 17 years have passed. During this period, about 30 nuclear power plants were constructed, which contributed to the accumulated aseismic design experience. At the same time, there is significant progress in the technology of aseismic design and development in research and development. There are countless examples in these respects. While the companies actively push forward the program, the reviewers also made efforts to establish the evaluation methods. As a result of these efforts, the standards become more consistent and comprehensive.

In order to perform the present amendment of JEAG 4601, an Aseismic Design Division was established in the Special Committee on Nuclear Power, Survey Committee for Electrotechnical Standard, Japan Electrical Association. All of the technical results obtained in the aforementioned background are summarized in forming this new edition of "Technical Guideline of Aseismic Design."

Attached data

Attached data 1

Licensing and Related Laws

The regulation of the practical nuclear reactor for power generation from the viewpoint of prevention of excessive radioactive exposure as described in section "1.1.1 Purpose of aseismic design" in Chapter 1, is mainly performed on the basis of the "Electricity Utilities Industry Law" as well as the "Law for the Regulations of Nuclear Source Material, Nuclear Fuel Material and Reactors," which are based on the "Atomic Energy Fundamental Act." Table 1-1 lists these laws and the guidelines of the Nuclear Power Safety Committee related to aseismic design. In the following, we will present a brief explanation of the various items related to aseismic design up to the stage of pre-service inspection with reference to these laws.

When the electric power company selects the planned site and performs the various surveys and evaluations, the Ministry of International Trade and Industry (referred to as "Ministry of Trade" hereinafter) makes examination of the environment and holds the first public hearing to listen to the opinions of the local residents and government. On the basis of these works, the construction program is sent for examination by the Electric Power Development Arrangement Council. If the program obtains the consent of the governors and various prefectures, the program is included in a National Base Program on Electric Power Development, and sent to the Prime Minister for approval. Afterwards, the electrical company acquires the application for reactor construction permit (or license for change in the case of expansion) from the Ministry of Trade.

In the stage of the basic planning for the construction license, examination is performed on the soil safety, seismic motion, and tsunami. For the major technical items, it is necessary to listen to the opinions of the Technical Advisory Committee on Nuclear Power Generation under the Ministry of Trade. In addition, the Ministry of Trade asks the opinion of the Nuclear Power Safety Committee with respect to the examination results. In this case, the major technical items are surveyed/examined by the Special Examination Committee on the Safety of Nuclear Rectors under the Nuclear Power Safety Committee. At this stage, the Nuclear Power Safety Committee performs the second public hearing to listen to the opinions of the local residents and government again. Then, the Nuclear Power Safety Committee reports to the Ministry of International Trade and Industry. At this stage, the consent of the Prime Minister is obtained. After the report is accepted, i.e., after the so-called double check is completed, the Ministry of International Trade and Industry issues the licence for construction.

In the next stage, the electrical company applies for construction permit, with the detailed design examined by the Ministry of International Trade and Industry. Just as in the case of safety examination, for the major technical items, it is necessary to listen to the opinions of the Nuclear Power Technical Advisory Council. In this detailed design stage, design of various equipment according to the basic guidelines, structural appropriateness and function maintenance during earthquake are examined. At this time, a construction license for the buildings should be obtained according to the Building Standard Code. Usually, the license for the nuclear reactor buildings is issued by the Ministry of Construction based on Clause 38[1] of the aforementioned code. The Ministry of Construction makes his decision on the basis of the "Seismic Examination Report" (building structure) furnished by the Technical Advisory Council of the Ministry of International Trade and Industry and the examination of the Architecture Technical Examination Council of the Ministry of Construction.

For the construction of a nuclear power plant, in each stage, examination before application is performed on the basis of the Electricity Utility Industry Law. According to the Implementation Rules of Electricity Utility Industry Law (Clause 37, No. 4), each engineering stage is divided into 5 items (A)-(E). Among these items, the

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[1]Clause 38 of Building Standard Law points out that in the case when special construction materials or structural methods are used, approval of the minister of construction is needed.

portion related to aseismic design is item (A), which is related to the structure and strength. More specifically, bedrock inspection and inspection of the seismic support structure of the equipment/piping of each equipment system are performed. For the items and contents in this respect, please see Appendix 2, "Test/Inspection."

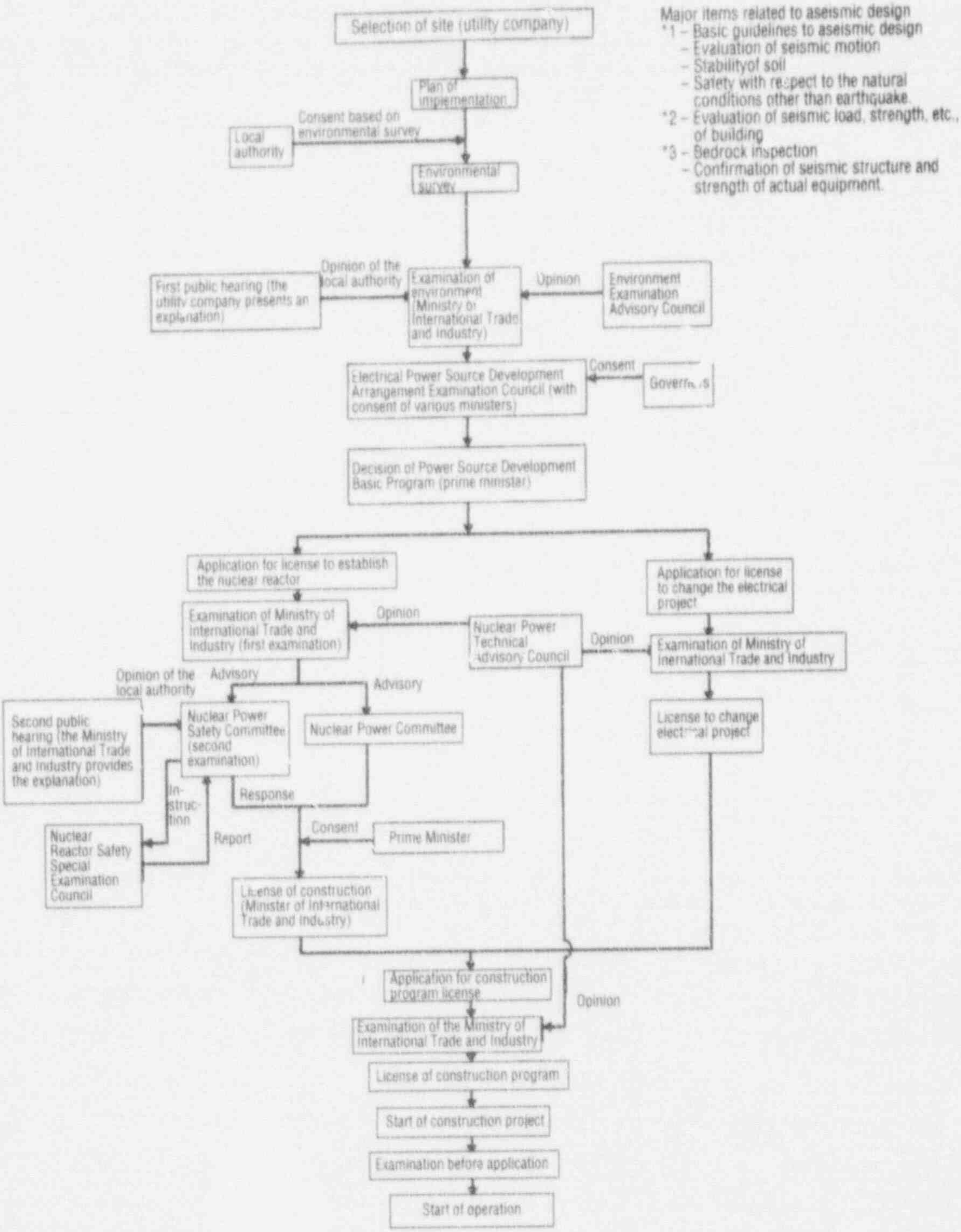


Figure 1-1. Flow chart of procedure from site selection to operation for nuclear reactor facility.

Table 1-1. Laws and guidelines related to aseismic design.

	Type	Summary
Items related to basic design	Law of Regulations of Nuclear Raw Materials, Nuclear Fuel Materials and Nuclear Reactor, Clause 23 (License of Establishment)	In the application for license of establishment, description is made of the seismic structure.
	On Examination Guideline of Nuclear Reactor Site and Standards for Its Application (Guidelines) (Nuclear Power Safety Committee)	In principle, for the condition of the site, there is the following requirement: "not only should there be no phenomenon in the past that may become the cause of major accidents, there should be no such phenomenon ever in the future. In addition, there should be few phenomena that would proliferate a hazard." Earthquake is also included in these phenomena.
	Safety Design Examination Guideline of Light-Water Nuclear Reactor Facility for Power Generation (Nuclear Power Safety Committee)	The consideration of design with respect to natural phenomena has the following major requirements: for facilities important in safety, aseismic design classification is made according to the importance level; it is necessary to make the design able to withstand the design seismic motion which is believed to be most appropriate according to the past records and on-site survey of the site and its peripheral region.
	Examination Guideline of Aseismic Design of Nuclear Reactor Facility for Power Generation (Nuclear Power Safety Committee)	When safety examination is performed for aseismic design, in order to evaluate the appropriateness of the design guidelines, the following guidelines are used for examination. (1) Basic guideline: With respect to any imaginable seismic force, it should not become the cause of a major accident. (2) Importance level classification for aseismic design: The nuclear reactor facilities are classified according to the level of importance from the viewpoint of the influence on the environment of radioactive rays that may be generated in an earthquake. (3) Aseismic design evaluation method: According to the level of importance of the facility, the calculation method of the seismic force is determined. (4) Seismic motion evaluation method: The method for determining the seismic motion at the rock outcrop surface on the site used as the seismic motion for aseismic design is determined.

Table 1-1 *(Cont'd)*. Laws and guidelines related to aseismic design.

	Type	Summary
		(5) Combination of loads and allowable limit: The basic methods for combining seismic load and other loads according to the importance level of the building/structure and equipment/piping system as well as the methods for determining the allowable limit are determined. (6) Explanation: "Judgment Standards for Evaluation of Active Faults," etc.
	Examination Guideline of Radiation Measurement in Accident of Light Water Nuclear Reactor Facility for Power Generation (Nuclear Power Safety Committee)	The portion of radiation measurement system in Class 1, i.e., the radiation measurement system which provides information for assessing the function of the radiation barrier, is designed as aseismic Class A.
	Introduction to Safety Examination of Geology/Soil of Nuclear Power Plant (Nuclear Power Safety Committee)	The standards of survey range, survey items, survey methods, etc., for geology and geological structure of the site and its periphery, as well as strength characteristics and deformation characteristics of the bedrock on the site, are determined.
Items related to detailed design	Clause 41 (Engineering Program) of Electrical Business Law	The calculation sheets of the seismic design of nuclear power equipment is attached to the Application for Engineering Program License.
	Minister's Instruction of Technical Standards on Nuclear Power Equipment (Ministry of International Trade and Industry)	(1) Clause 4 (establishment of protective facilities, etc.): In the case when there might be damage caused by landslide, fault, avalanche, flood, tsunami/high tide, differential settlements of foundation soil, etc., appropriate measures are taken to set up protective facility, to improve foundation soil, etc. (2) Clause 5 (aseismic property): Evaluation is performed to ensure that the public is not exposed to radioactive hazard when earthquake takes place. (3) Clause 9 (material and structure): The materials and structures of the containers and piping of the nuclear reactor facilities must conform to the standards defined in the following publication: Notification No. 501 of the Ministry of International Trade and Industry "Technical Standards of Structures of Nuclear Power Equipment for Power Generation" (October 30, 1980).

Table 1-1 *(Cont'd)*. Laws and guidelines related to aseismic design.

	Type	Summary
		(4) Clause 22 (emergency shutdown equipment): Equipment is set up to ensure that in the case when the nuclear reactor cannot operate safely due to earthquake, the state is detected and the operation of the nuclear reactor is quickly turned off automatically.
	Technical Guideline of Aseismic Design of Nuclear Power Plant, JEAG 4601-Supplement-1984, Importance Level Classification/Allowable Stress Edition (Technical Guideline of Japan Electrical Society)	Chapter of Importance Level Classification (1) Basic items: Items required for safety in earthquake, definition of importance level classification, classification of functions, specific classification examples, etc. (2) Reference data: Relation between operating state and earthquake, maintenance of function of dynamic equipment in earthquake. Chapter of Allowable Stress (1) Basic items: Guideline of allowable stress determination for facilities of various classes; combination of operating state and standard seismic motion; classification table of allowable stresses. (2) Allowable pressures of facilities: Allowable pressure table and explanation of equipment of various classes of facilities. (3) Reference data: Background of allowable stress determination, aseismic design evaluation method, etc.
Items related to inspection	Clause 43 (Examination before application) of Electrical Business Law	Each stage of the construction is subject to examination of the Ministry of Trade; the facility can be used only after it passed the examination. The item related to aseismic design is item (A): Structure, strength, and leakage.

References

[1-1] Sachimi Ibe: "Examination of aseismic design of nuclear power plant; Genshiryoku Kogyo, Vol. 26, No. 6, June 1980, pp. 22-55.

[1-2] Planning Division, Public Business Department, Resources/Energy Bureau, Ministry of International Trade and Industry (ed.): Electrical Power KOROKU Method, 1984 edition, published by Denryoku Shimposha.

[1-3] Nuclear Power Division, Public Business Department, Resources/Energy Bureau, Ministry of International Trade and Industry (ed.): Encyclopedia of Nuclear Power Generation, 1985 edition, published by Denryoku Shimposha.

Attached data 2: Test/inspection

Introduction

In the attached, we will present a summary of the tests related to the aseismic designs of the building/structure and equipment/piping of nuclear power plants, as well as the pre-service inspection performed during the construction process.

As far as the test methods in the soil survey are concerned, we will summarize the test methods in the various stages of design in an implementation of "Chapter 4. Stability evaluation of ground and aseismic design of underground structures."

2.1 Test/inspection in soil survey

Table 2.1-1 summarizes the tests usually performed to evaluate the physical characteristics and mechanical characteristics of the soil for a nuclear power plant. For more details, please see the reference listed in the same table. Table 2.1-2 summarizes the items and contents of the inspection before application for the foundation of the nuclear reactor containment vessel in item (A), pre-service inspection.

Table 2.1-1. Test methods in soil survey.

Type	Test purpose	Test content		References
Rock tests	The physical characteristics and mechanical characteristics of the rocks that form the foundation ground are surveyed; the data are used for soil stability evaluation and structural design of the nuclear reactor facility.	Physical test	Samples in boring core or pit are used for measurement of specific gravity, water content, water absorptivity, effective porosity, etc.	- JISM 0302 (Compressive strength test method of rocks) - JISM 0303 (Tensile strength test method of rocks) - JISA 1110 (Test methods of specific gravity and water absorptivity of coarse aggregate) - Soil Quality Test Method (Soil Engineering Institute) - Nippon Kogyo Kaishi (February 1964) - Engineering Properties of Rocks and Their Application in Design and Operation (Soil Engineering Institute) - Major Points of Velocity Measurement in Rock Test (Physical Prospecting Technical Association) - Rock Standard Test Method in National Railway Bureau (Draft) (Teddo Gijutsu Kenkyu Hokoku, No. 668, 1969) - Civil Engineering Test Standard (Draft) (Ministry of Construction) - Survey/Test Method of Geography/Soil of Nuclear Power Plants and Evaluation Method of Seismic Stability of Soil (JSCE)
		Ultrasonic velocity measurement	The propagation velocity of ultrasonic waves in samples of boring core or pit is measured.	
		Uniaxial compression test	Samples in boring core or pit are used. The dimensions of samples usually have a diameter of about 50 mm and a height of about 100 mm.	
		Triaxial compression test	Samples in boring core or pit are used. The dimensions of samples usually have a diameter of about 50 mm and a height of about 100 mm.	
		Tensile test	Samples in boring core or pit are used. The dimensions of samples usually have a diameter of about 50 mm and a height of about 100 mm. Split-cylinder test is usually performed.	
		Uniaxial (triaxial) creep deformation test	Mainly performed for soft rocks. Samples in pit are used.	
Bedrock tests	The mechanical characteristics and wave propagation characteristics of the foundation bedrock are surveyed; the results are used for soil stability evalulation and structural design of the nuclear reactor facility.	Bedrock deformation test	Plate load test. Load plate has a diameter of about 30 cm. Up to the maximum load, 3-5 load levels are divided. On each load level, stepwise loading/deloading is performed. The standard loading rate is set as 5 $kgf/cm^2/min$.	- Guideline of In Situ Deformation/Shear Test of Bedrock (JSCE) - Soil Test Method (Soil Engineering Institute) - Soil Survey Method (Soil Engineering Institute) - Engineering Properties of Rocks and Their Application in Design and Operation (Soil Engineering Institute) - Survey/Test Method of Geology/Soil of Nuclear Power Plant, and Evaluation Method of Seismic Stability of Soil (Soil Engineering Institute)

Table 2.1-1 *(Cont'd)*. Test methods in soil survey.

Type	Test purpose	Test content		References
Bedrock tests		Bedrock shear test	Block shear test or rock shear test. Vertical load: over 4 stages. 0.5 kgf/cm^2/min is taken as the standard shear load rate.	
		Bearing strength test	Plate loading test. Load plate diameter: about 30 cm. The limit bearing force is derived.	
		Bore-hole load test	With the boring hole wall loaded by hydraulic pressure, the deformation characteristics of bedrock are surveyed.	
		Test of elastic wave velocity in pit	Pit-wall elastic wave survey (refractive wave method) Interpit elastic wave survey (direct wave method)	
		PS logging	Boring hole is used. Vibration source: P-wave - explosion, hammer falling; S-wave - plate knocking	
		Dynamic deformation test	Plate load test. Dynamic load is applied. Load plate has a diameter of about 30 cm.	
		In situ water permeation test	Boring hole is used. Usually, Rudion [transliteration] test.	
		Schmidt rock hammer test	The repulsion degree of the bedrock is surveyed. The measurement interval is about 0.5 m. The measurement points are about 9 points/location.	
Soil test	The physical characteristics and mechanical characteristics of class (D) bedrock, surface soil, fault rupture belt and other weak strata distributed on foundation soil and peripheral slopes are surveyed; the results are used for soil stability evalulation and structural design of nuclear reactor facilities.	Physical test	Measurement of specific gravity, water content ratio, grain size, liquifaction limit, plastic limit, etc.	- Soil Test Method (Soil Engineering Institute) - Soil Survey Method (Soil Engineering Institute) - JIS A 1202 (Specific gravity test of soil particles) - JIS A 1110 (Specific gravity/water absorptivity test of conglomerate) - JIS A 1203 (Soil water content test) - JIS A 1204 (Soil grain size test)
		Triaxial compression test	The dimensions of the sample usually are as follows: Diameter: 50-100 mm; Height: 100-200 mm	
		Dynamic triaxial compression test	The dimensions of the sample usually are as follows: Diameter: 50-100 mm; Height: 100-200 mm	

Table 2.1-1 *(Cont'd)*. Test methods in soil survey.

Type	Test purpose	Test content		References
Soil test		Indoor water permeation test	Constant water level method, varied water level method	- JIS A 1205 (Liquid Property Limit Test of Soil) - JIS A 1206 (Plastic Property Limit Test) - Foundation of Soil Dynamics (Kashima Publishing Co.) - Survey/Test Methods of Geology/Soil of Nuclear Power Plant, and Evaluation Method of Seismic Safety of Soil (JSCE)
		Consolidation test	The dimensions of the sample are usually as follows: diameter, about 60 mm; thickness, about 20 mm.	
		Standard penetration test	Boring hole is used. N-value is measured.	
		Single-plane shear test	The dimensions of the sample are usually as follows: diameter, about 60 mm; thickness, about 20 mm.	
		Simple shear test	The dimensions of the sample are usually as follows: diameter, about 50 mm; thickness, about 20 mm.	
		Dynamic simple shear test	The dimensions of the sample are usually as follows: diameter, about 50 mm; thickness, about 20 mm.	
Other tests	The mechanical characteristics of the foundation soil are surveyed; the data are used for soil stability evaluation and structural design of the nuclear reactor facility.	Initial soil pressure measurement	Over coring method, AE method	- Soil Survey Method (Soil Engineering Institute) - Bedrock Mechanics for Civil Engineers (JSCE) - Engineering Properties of Rock and Applications in Design/Operation (Soil Engineering Institute) - Measurement and Analysis of Bedrock (Soil Engineering Institute)
		Bedrock creep test	Mainly performed for soft rocks. Plate load test. Load plate diameter: about 30 cm. Loading time: 1-3 months.	
		Uniaxial (triaxial) creep damage test	Performed for relatively soft rocks. Samples in pit are used.	

Table 2.1-2. Inspection before application for foundation of nuclear reactor containment vessel.

Item	Content
Inspection of geology of foundation ground and ground properties	(1) Classes and distribution states of rocks and bedrocks; development degree and distribution state of joints, seams, etc.; properties, scales and distribution states of faults, rupture belts, weak strata, etc. (2) General properties of rocks that form the foundation ground; mechanical characteristics of bedrock.[1]
Inspection of construction	(1) State of construction which may change the conditions of the rupture belts, weak strata, etc. (2) Underground water level and state of operation of water drainage equipment. (3) Assessment of depth of foundation bed. (4) State of leanness of ground surface.

[1]Assessment is performed to ensure that there exists no significant difference between the prediction made for acquiring the license and the actual state after the foundation bedrock is dug; depending on the state of the ground, it is performed with appropriate test items and test amount.

2.2 Test/inspection for buildings/structures

(1) Purpose of test/inspection

When the nuclear power plant equipment is to be actually used, it is important to perform testing/inspection to confirm that the equipment is suitable for equipment conditions and poses no safety problems. Hence, from the viewpoint of seismic safety evaluation, testing/inspection for the seismic support structure should be performed at appropriate stages for the typical equipment with aseismic safety Class As and A. In this case, testing/inspection is performed with the purpose of confirming that the operating state of the seismic support structure of the equipment, the vibration characteristics of the facilities, etc., fit with the aseismic design conditions and various standards and are free of safety problems.

(2) Pre-service inspection

Inspections must be performed on all the equipment before they are used. And the equipment can be operated only after the results pass the regulations defined in Utility Industry Law in each stage of the construction process. These inspections are called "Pre-Service Inspection".

Each of the construction stage, as described in the Utility Industry Law's Implementation Rules, has five items of inspection, which are denoted from (A)–(E) in the Law. Structure/strength test must be carried out for aseismic design and they are described in item (A). The facilities as the objects and the test and inspection methods are as follows.

Assessment of the structure/strength is performed for the nuclear reactor containment facility and soil. Inspection is performed of their materials, structures and strength for the nuclear reactor building and internal concrete. The inspection method and assessment are performed according to JASS 5N and the regulations defined in the related JIS just as for conventional reinforced concrete structures. Inspection of foundation soil is described in Section "2.1 Test/inspection in soil survey."

(3) Other tests

a. Forced vibration test

The vibration characteristics of the nuclear reactor containment facility are an important property which determines the magnitude of the seismic load acting on the facility. The vibration characteristics are assessed as one of the properties/functions by performing forced vibration test.

The objects include the nuclear reactor building, internal concrete, containment vessel, etc. In the test, forced vibration is performed by an installed vibration machine on a portion of the facility, and the vibration characteristics (vibration frequency, vibration mode, damping constant, etc.) are measured.

As an example, attached Figure 2.2-1 illustrate the case of a nuclear reactor building (BWR MARK-II type). In the test, two large-sized vibrating machines are set on the fuel exchange floor of the building to perform forced vibration in the horizontal direction, and the aforementioned vibration characteristics are determined by measuring the displacement amplitude and phase difference of each floor.

b. Earthquake observation

In order to find the seismic response characteristics of the nuclear reactor building and to assess the seismic response analysis method, earthquake observation is usually performed.

In the test, seismographs are set up in the building and on its peripheral ground to observe the seismic motion. In this way, the input seismic motion, maximum seismic responses (mainly acceleration), natural frequency of vibration, damping constant, etc., can be measured. On the basis of earthquake observation and simulation analysis, the analysis model and analysis method of the nuclear reactor building are evaluated.

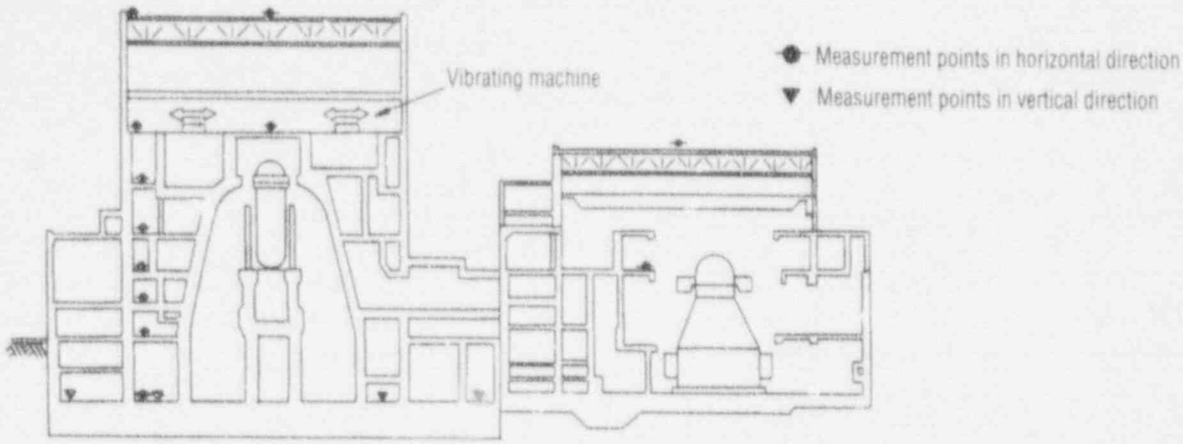
As an example of the earthquake observation, attached Figure 2.2-2 illustrates the case of nuclear reactor building (PWR 2 LOOP).

c. Interaction test using rigid foundation

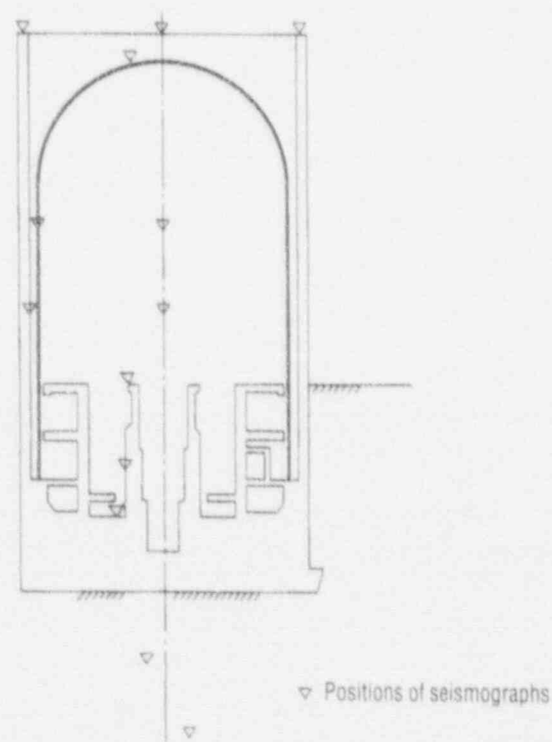
In order to assess the dynamic interaction between the soil and building experimentally, interaction test using a rigid foundation is performed in some cases.

In this case, vibrating machine test is performed to assess the vibration characteristics of the rigid foundation and its surrounding soil. The test using rigid foundation differs from the aforementioned building forced vibration test in that the test is simpler in modeling, and the test can be implemented in an ideal state. By performing the interaction test and simulation analysis, the stiffness and damping characteristics of the spectral range important for the aseismic design can be assessed, and the appropriateness of the soil-building interaction model can be assessed.

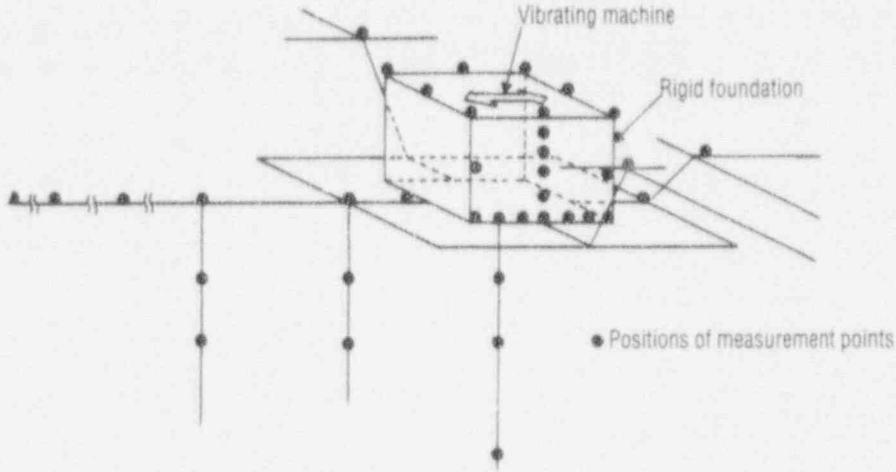
Attached Figure 2.2-3 illustrates an example of the interaction test using a rigid foundation. The test is performed by using a vibrating machine to apply a forced vibration of a rigid foundation (a concrete block measuring 15 m x 15 m x 13 m) set on hard bedrock, and measuring the responses of the rigid foundation and its surrounding soil using displacement gauges mounted on them.



Attached Figure 2.2-1. Example of forced vibration test of nuclear reactor building (BWR MARK-II type).



Attached Figure 2.2-2. Example of earthquake observation of nuclear reactor building (PWR 2 LOOP).



Attached Figure 2.2-3. Example of interaction tests using a rigid foundation.

2.3 Test/inspection of equipment/piping system

1. Test

The strength design of nuclear power plants can be divided into design by analysis and design by test. In addition, as will be pointed out later, in order to assess the maintenance of performance/functions required of the pumps and other dynamic equipment; both analysis and test are performed. In this section, we will discuss the tests for design and the tests for design confirmation, which refer to the tests performed during design and the tests performed to confirm the design by using actual equipment, etc.

(i) Tests for design

The material strength is the basis for evaluating the seismic properties of the equipment and piping system. This is true not only for the seismic load but also for the other loads. The materials used for the important equipment/piping systems which are required to have high strength in the nuclear power plant are defined in Notification No. 501 of the Ministry of International Trade and Industry "Technical Standards of Structures of Nuclear Facilities for Power Generation." When other materials are used, however, tests may be needed to confirm the yield strength, tensile strength, etc., of the materials. In particular, in the case when analysis is not performed on the support structures, according to Notification No. 501, in order to confirm the strength of the support structures, testing is performed by making several test samples, and the allowable load is determined for them.

As pointed out in Chapter 6, the seismic force is determined by analysis. One important parameter is the damping constant. Usually, conventional values are adopted for the damping constants of the equipment/piping system. However, for certain structures/materials used in the nuclear power plant, the appropriateness of the damping constants of the equipment/piping system is assessed by performing vibration test before they are actually used.

(2) Tests for confirming design

For the pumps, valves, dampers, fans, and other dynamic equipment, as well as relays, etc., among the electrical measurement/control apparatus described in "Technical Guideline of Aseismic Design of Nuclear Power Plant: Importance Classification/Allowable Stress Edition, JEAG 4601-Supplement-1984" by Japan Electrical Association, in some cases, it is difficult to assess the behavior and function maintenance ability during an earthquake. In this case, it is necessary to perform the vibration test by using a vibrating machine or a shaking table. Also, it is necessary to perform the same evaluation on the inserting property of the control rods during earthquake.

In addition, the vibration characteristics of the nuclear power equipment are an important factor in the calculated seismic load. In order to assess them, the vibration test must be performed. The objects for the test are selected appropriately considering the importance of equipment.. Depending on the equipment, tapping, wire cutting or other free vibration test or forced vibration test by a vibrating machine is performed. In this way, the vibration characteristics (natural vibration frequency, damping constant, etc.) are assessed.

2. Inspection

When equipment is to be used for practical application, it is important to confirm that the equipment is appropriate for the design conditions without safety problems. Hence, from the viewpoint of seismic safety evaluation, at appropriate stages of construction, inspection of the items important for the seismic support structures is performed mainly for the typical equipment of aseismic importance Classes As and A to make sure that the seismic support structure of equipment is appropriate for the various standards without safety problems. When the inspection is implemented, it is important to confirm that the equipment/piping systems are usually the standard equipment, and that the equipment as inspection objects is selected on the basis of assessment of the appropriateness of the various tests and inspection.

In addition, during the application period, regular inspection is performed at the important parts of the equipment/piping support structures to ensure their safety function.

(1) Inspection before operation

The inspection of equipment/piping support structures is mainly performed for the structures/strength of the equipment/seismic support structures. The contents of the inspection are as follows:

(1) Inspection items

Items of equipment/piping support structures mainly related to the operating state, such as mounting position, restraint direction and adjustment scheme of piping support structures, structural strength (parts, welding, etc.), interferences of other structures, etc.

(2) Inspection method

The appropriateness of the construction management is confirmed by the construction management records, etc. Also, for the construction state, visual observation and actual measurement are performed to make sure that there are no safety problems.

(3) Inspection period

In principle, the period after the corresponding system is installed is the most suitable period for the inspection.

(2) In-service inspection

During the period of operation, regular inspection is performed for important parts among the equipment/piping support structures, and visual observation is performed to make sure that the appropriateness of these parts is maintained. Implementation of the inspection is performed with reference to "Inspection During Application Period for Equipment of Light-Water Nuclear Power Plant, JEAC 4205-1986" drafted by Japan Electrical Association.

Attached data - 3. Earthquake detecting equipment

In a nuclear power plant, if something happens and the nuclear reactor cannot operate safely, the state is detected by a safety protection system and the operation of the nuclear reactor is shut down automatically. During an earthquake, if certain abnormal phenomena caused by the earthquake take place so that the nuclear reactor cannot be operated safely, the nuclear reactor is finally shut down by this safety protection system.

In the case when the seismic motion is greater than the design seismic motion for the equipment which is important for safety, in order to effectively ensure the safety, the nuclear reaction should be shut down. For this purpose, earthquake-detecting equipment is arranged as a safety protection system, which can shut down the nuclear reactor when the earthquake intensity is above a certain level.

For the earthquake-detecting equipment, the location of the seismometer, the scram level and the earthquake scram logic circuit are as follows:

(1) Location of seismic trigger

The location for the seismic trigger of the earthquake-detecting equipment should be determined by considering the object for which the seismic motion is to be detected; and the selected location should be easy for maintenance/inspection and should be able to ensure high reliability. As a result, the location is determined on the same floor where the equipment is placed.

More specifically, in a building which contains equipment important to safety, the seismic trigger is set in the horizontal direction on the lowest story of the building to detect the seismic motion input to the building. In some cases, a seismic trigger in the horizontal direction is also set on a typical floor among the upper floors, and a seismic trigger in the vertical direction is set on a typical floor.

(2) Scram level

This is the predetermined value at which the nuclear reactor is shut down automatically by the earthquake detection device. From its purpose, it is necessary to detect it in a reliable way in the case when a seismic motion (about S_1 seismic motion) corresponding to the design seismic strength of the equipment which is important to safety takes place.

(3) Earthquake scram logic circuit

The earthquake detection device is a safety protecting system which can automatically stop the nuclear reactor quickly as the earthquake takes place. Based on the basic design guideline of the safety protecting system, as shown in Figure 3-1, the earthquake scram logic circuit may have the form of "double '1 out of 2'" or the form of "2 out of 3."

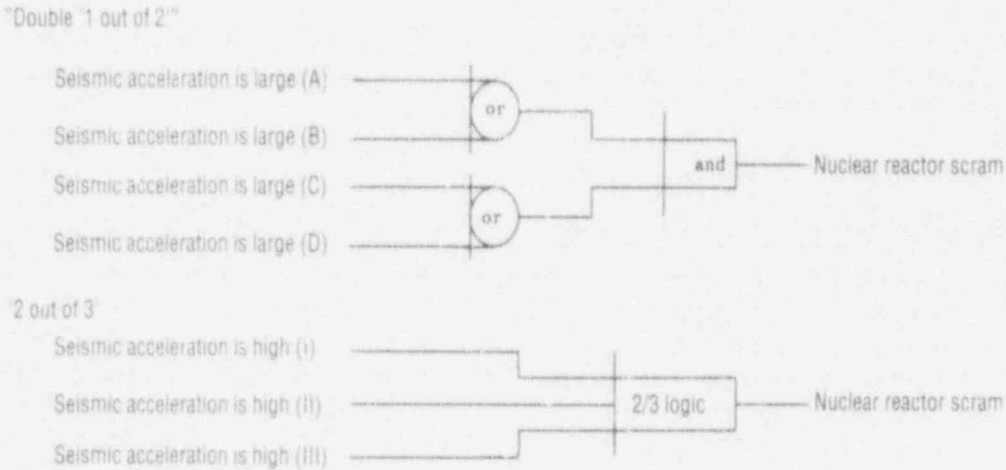


Figure 3-1. Earthquake scram logic circuit.

Attached data 4. Inspection/service after earthquake

When an earthquake takes place during the operation period, since the equipment is designed according to the seismic force corresponding to their respective aseismic importance, the influences on the different items of equipment are also different from each other. Hence, when the operation is to be continued or when the operation is to be restarted, it is important to confirm the integrity of the various equipment from the viewpoint of ensuring the safety of the nuclear power plant. On the other hand, from the viewpoint of power supply, it is undesirable to interrupt power generation frequently or to suspend power generation for a long time. As a result, in consideration of the fact that each piece of equipment is designed according to its respective aseismic importance, the inspection scope and content are defined corresponding to the magnitude of the earthquake that takes place.

As an example, the contents of inspection after the earthquake are as follows, depending on the magnitude of the earthquake that takes place:

(1) In the case when the earthquake has an intensity equal to or higher than Class C design seismic intensity, it is necessary to check if there is any abnormal phenomenon in the equipment by monitoring the various alarms in the central control room and by walk-down.

(2) In the case when the earthquake has an intensity equal to or higher than Class B, it is necessary to strengthen the above step in checking the presence/absence of abnormal phenomena and to confirm the appropriateness of the engineering safety by performing operation testing.

(3) In the case when the earthquake has an intensity equal to or higher than Class A, it is necessary to perform careful inspection of the equipment, including that within the containment vessel, and to check function-maintaining ability of the equipment that is important for ensuring safety.

In addition, as indices of the inspection after earthquake, it is possible to install devices that can display the acceleration when the acceleration becomes higher than a certain level at typical locations in the central control room.

In addition, for the aforementioned contents, detailed survey is needed according to the specific local conditions of the power plant, etc.

Appendix

Appendix 1. List of various tests and research

Introduction

As far as the aseismic design of the nuclear power plant is concerned, up to now, numerous tests and researches have been carried out or are being carried out in the government, electric power company, plant maker, construction companies, etc. In this appendix, we present a list of the major research efforts in the industry.

- With respect to the sources, the research items are classified into the following groups A, B and C:

  A: Items which have been published in periodicals of societies, technical journals, etc. [(A) indicates that the items have been partially published in periodicals of societies, technical journals, etc.]
  B: Items which have not yet been fully published, with only their abstracts published.
  C: Items which have only their titles published.

- The reference numbers in the main text are defined as follows:

  K-C-1 — (Chapter 2, Items related to seismic motion)
  K-T-1 — (Chapters 2, 4, Items related to geology, soil, civil structures)
  K-K-1 — (Chapter 5, Items related to buildings/structures)
  K-KI-1 — (Chapter 6, Items related to equipment/piping system)

- Symbols in the list have the following meanings.

  - DK (P, B) represents PWR-BWR Electrical Power Joint Research
  - DK (B) represents BWR Electrical Power Joint Research
  - DK (P) represents PWR Electrical Power Joint Research

- References are listed in the "Notes," column, with the following abbreviations:

  - KDK: Nippon Kenchikugakkai Taikai Gakujutsu Koen Kogaishu [Proceedings of Symposium of Architecture Institute of Japan].
  - KKS: Nippon Kenchikugakkai Kantoshibu Kenkyuhokokushu [Reports of Research, Kanto Branch of Architecture Institute of Japan]
  - KP.H: Nippon Kenchikugakkai Rombun Hokokushu [Transactions of Architecture Institute of Japan].
  - GAK: Researches Based on Annual Program of Safety Research of Nuclear Power Facilities of Nuclear Power Industry Safety Research Special Division, Nuclear Power Safety Committee.

(Chapter 2. Items related to earthquake/seismic motion)

	Number	Research item		Major content	Period	Note
DK	K-C-1	Research on seismic motion characteristics (part 1, part 2)	A	The purpose is to perform earthquake observation on the same bedrock as the rock on which the nuclear power plant is constructed, to accumulate the effective basic data for seismic engineering, and to establish the method for evaluating the design seismic motion.	1977-1985	KDK (Kinki), September 1980, Part 1
DK	K-C-2	Research on seismic motion characteristics of bedrock (part 1, part 2)	A	The purpose is to evaluate the propagation characteristics of seismic motion by vertical array earthquake observation within bedrock with hard soil as the object, and to obtain the basic data of design seismic motion used for analysis of the amplification characteristics of seismic motion in bedrock and the response of the soil-building system as needed in the aseismic design of nuclear reactor facilities.	1981-1985	KDK (Hokuriku), September 1983, part 1, part 2
DK	K-C-3	Research on seismic motion characteristics of Neogene-period sedimentary rocks	A	For the Neogene-period sedimentary rocks, by performing vertical array observation, the propagation characteristics of seismic motion are evaluated, and the basic data for the amplification characteristics of seismic motion in bedrock, the dependence of stiffness and damping property of bedrock on strain, and the design seismic motion used in the response analysis of the soil-building system are obtained.	1981-1985	Butsuri Tansa, Vol. 39, No. 2, May 1986
DK	K-C-4	Survey and research of historical earthquake data (part 1, part 2, part 3)	B	In order to improve the reliability of the earthquake catalog which is used as the basis for determining the standard seismic motion used in the aseismic design of nuclear power plant, the historical records of the historical earthquakes are surveyed and used in compiling the earthquake catalog.	1981-1985	
DK	K-C-5	Survey and research on seismotectonics	C	In order to rationalize the evaluation of the design standard seismic motion, plate tectonics is surveyed. Based on the recent earthquake observation and geophysical observation as well as the data of active faults, the conventional seismic tectonic structure is reinvestigated. In addition, based on results of the recent seismological research (fault model), the design seismic motion is evaluated.	1985-1986	

(Chapter 2 Items related to earthquake/seismic motion)—*Cont'd*

	Number	Research item		Major content	Period	Note
DK	K-C-6	Research on standard seismic motion evaluation method based on earthquake observation	C	Earthquake observation was performed at the eastern portion of Kanto, Izu, and Fukushima-ken. The observation records are accumulated for evaluating the seismic motion. Also, the existing data in Japan and abroad are used in analysis to evaluate the seismic motion for aseismic design of nuclear power plant.	1986-1990	

(Chapters 3, 4. Items related to earthquake, ground, and civil structures)

	Number	Research item		Major content	Period	Note
Electrical Power Central Research Laboratory (EPCRL)	K-T-1	Research on fault activity	A	The purpose is to perform survey/research on the distribution state and activity of the major real faults, analysis of tissue and structure of fault rupture belt, measurement of activity age by substances in the fault, displacement measurement of fault/soil, and standardization of evaluation method of fault activity.	1977-1990	Oyo Chishitsu 22-1, pp. 67-86. EPCRL Research Report Nos. 377011, 380004, 380044, 381029
Electrical Power Central Research Laboratory	K-T-2	Research on classification of soft bedrocks	A	A scheme is formed for classification of bedrocks of sedimentary rocks of Tertiary period and Quaternary period.	1983-1984	JSCE: "Survey/test method of geology/soil of nuclear power plant and evaluation of method of seismic stability of soil"
Electrical Power Central Research Laboratory	K-T-3	Research on mechanical characteristics of mudstone	A	By performing laboratory tests, the static strength, deformation, creep characteristics, dynamic strength and deformation characteristics of mudstone are evaluated and explained in a unified way.	1980-1983	EPCRL Research Report Nos. 382011, 382012, 382013, 382014, 382059, 383004

(Chapters 3, 4. Items related to earthquake, ground, and civil structures)—*Cont'd*

	Number	Research item		Major content	Period	Note
Electrical Power Central Research Laboratory	K-T-4	Research on mechanical characteristics of fault rupture belt materials	A	By performing laboratory tests, the static/dynamic strength and deformation characteristics of fault rupture belt material are investigated, and a new scheme for survey/test method is prepared.	1982-1985	EPCRL Research Report No. 384033
Electrical Power Central Research Laboratory	K-T-5	Evaluation method of stability of dense sandy soil in earthquake	A	By performing laboratory experiment concerning the dynamic strength characteristics of dense sand, evaluation of the effects of sand particle structure and disturbance on the dynamic strength, evaluation of the in situ dynamic strength of a dense sandy soil using standard penetration test, and evaluation of the stability of a dense sandy soil in earthquake are performed.	1981-1982	EPCRL Research Report Nos. 383025, 383026
Electrical Power Central Research Laboratory	K-T-6	Prevention of liquefaction of saturated sandy soil by gravel pile	A	Research is performed on the method for preventing liquefaction in earthquake due to the water evacuating effect. This research provides the specific design scheme for this method.	1980-1984	EPCRL Research Report Nos. 382010, 382058, 383006, 383060, 384002
Electrical Power Central Research Laboratory	K-T-7	Evaluation method of scatter in soil properties	A	In order to establish a safety evaluation method of soil in consideration of the scatter of the properties of the soil, property scatter analysis, analysis program, and dispersion influence evaluation, etc., are implemented.	1982-1985	EPCRL Research Report Nos. 384004, 384025, 384026
Electrical Power Central Research Laboratory	K-T-8	Seismic stability of large-scale slope of nuclear power plant	A	A model slope is used for slope destruction and vibration destruction experiment and numerical simulation. The static and dynamic destruction phenomena, comparison of analysis results, and stability evaluation method are analyzed, and the evaluation method for the seismic coefficient is discussed.	1977-1982	EPCRL Research Report Nos. 381030, 382020, 382021, 382022, 380057, 381003

(Chapters 3, 4. Items related to earthquake, ground, and civil structures)—*Cont'd*

	Number	Research item		Major content	Period	Note
Electrical Power Central Research Laboratory	K-T-9	Evaluation of elastic damping of bedrock of block vibration experiment	A	Quantitative determination of elasticity and damping property of bedrock at nuclear power plant site in block vibration experiment.	1983-1985	EPCRL Research Report (permission by Electrical Business Association is needed)
Electrical Power Central Research Laboratory	K-T-10	Small-region elastic wave test method	A	Method for in situ measurement of small-region elastic wave velocity is developed. The shear test locations, properties of weak layers, etc., are clarified.	1982-1984	EPCRL Research Report No. 382043
Electrical Power Central Research Laboratory	K-T-11	Research on seismic property of underground structure	A	The aseismic design methods of LNG tanks, intake shaft intake pit, and other underground structures are clarified by experiment, observation and analysis.	1980-1984	EPCRL, A Review
Electrical Power Central Research Laboratory	K-T-12	Aseismic evaluation of important outdoor civil structures of nuclear power plant (part 1) Response displacement method and dynamic analysis	A	The seismic response design method of important outdoor civil structures of the emergency water intake system of nuclear power plant is classified.	1980-1984	EPCRL Research Report
Electrical Power Central Research Laboratory	K-T-13	Seismic property of ground piping system (evaluation of seismic property of cooling water intake piping for condenser of nuclear power plant)	A	The behavior of ground piping system, i.e., emergency auxiliary equipment cooling water intake piping, is evaluated.	1971-1974	EPCRL Research Report No. 74004

(Chapters 3, 4. Items related to earthquake, ground, and civil structures)—*Cont'd*

	Number	Research item		Major content	Period	Note
Electrical Power Central Research Laboratory	K-T-14	Analytical evaluation of behavior of underground structure in earthquake	A	The behavior of facilities for emergency cooling of auxiliary equipment (intake pit. seawater piping duct) set on soft soil.	1977-1982	EPCRL Research Report No. 382030
Electrical Power Central Research Laboratory	K-T-15	Streamlining of evaluation method of bedrock engineering	B	The in situ test data of bedrock are collected and analyzed, and laboratory experiment is performed for the simulated rock materials. In this way, a practical method for making a simple engineering judgment on the seismic property of the foundation bedrock in the nuclear power plant is proposed.	1984-1990	EPCRL Research Report (to be published in Civil Engineering Institute)
Electrical Power Central Research Laboratory	K-T-16	Reduction in effective seismic input	B	By performing in situ observation, in situ test and laboratory model experiment, the seismic effect of the foundation in earthquake due to the burying effect is clarified	Same as above	Same as above
Electrical Power Central Research Laboratory	K-T-17	Applicability of base isolation systems	B	Model experiment that simulated the base isolation systems is implemented; a method for numerical simulation of the base-isolation system is developed.	Save as above	Same as above
Electrical Power Central Research Laboratory	K-T-13	Proposal of design method in consideration of the nonlinear behavior of structure	B	By evaluating the dynamic mechanical characteristics of the reinforced concrete, and clarifying experimentally the ultimate strength of the cross section of the seawater duct, the analysis method is improved	Same as above	Same as above

(Chapters 3, 4.   Items related to earthquake, ground, and civil structures)—*Cont'd*

	Number	Research item		Major content	Period	Note
DK	K-T-19	Research on evaluation of soil	B	In order to streamline the aseismic design of nuclear power plant equipment, the importance of the object structure and its behavior in earthquake are taken into consideration in deriving the basic guidelines for the fault survey, soil survey, and test method, which are formulated in standard forms.	1980-1982	
DK	K-T-20	Research on stability evaluation method of nonhomogeneous foundation ground	B	For a nonhomogeneous foundation ground containing fault ruptured belt and other weak strata, the behavior in earthquake and the effect of the weak strata are clarified, and the safety evaluation method of the nonhomogeneous foundation ground is investigated.	1983	

(Chapter 5.   Items related to building/structures)

DK (P, B)	K-K-1	Research on streamlining the seismic force design for nuclear reactor facilities	C	In order to determine the appropriate design seismic force in a nuclear reactor facility, the present method of calculation of static seismic force of the underground portion is improved. In addition, by earthquake measurement and simulation analysis of the vibration test results, the appropriateness of the analysis method is proved. On the basis of past research results, a dynamic analysis method with a high enough precision and reliability that can relax or even delete the regulation of the static seismic force is developed.	1984-1986	
(P, B)	K-K-2	Research on evaluation method of seismic margin of nuclear reactor building	C (A)	The ultimate strength and restoring force characteristics of the nuclear reactor building are clarified; the evaluation method of the seismic margin with respect to S_2 earthquake is established. In consideration of the reliability of the evaluation, a lower limit of the safety margin is proposed.	1981-1986	KDK, 1985 (Structure I), pp. 823-826
(P, B)	K-K-3	Experimental study on the structural characteristics of large-diameter bar joint	C (A)	With the wall plates acted upon by in-plane shear forces, such as shear wall, etc., taken as the objects, the structural characteristics of the various lap joints are assessed experimentally, and the appropriateness of the lap joints is assessed. In addition, experiments were performed to investigate the joint forms for reducing construction time of reinforcing bar assembly; the large-diameter bar lap splice method that can be adopted, and the applicable limit were proposed.	1982-1984	KDK: 1984, pp. 1765-1774; 1985, pp. 583-596

(Chapter 5. Items related to building/structures)—*Cont'd*

	Number	Research item		Major content	Period	Note
(B)	K-K-4	Research on response behavior of nuclear reactor buildings	C	In order to make a more correct evaluation of the vibration behavior of the nuclear reactor building in earthquake including the high-frequency region, vibration experiment and analysis of a contracted model of the nuclear reactor building, analysis/evaluation of the input seismic motion, and evaluation of the input reduction effect are performed.	1980-1983	
(B)	K-K-5	Research on seismic stability of nuclear reactor building	C (A)	Research and development is performed for the soil-structure interaction which is used for evaluating the uplifting of nuclear reactor building embedded deeply, dynamic earth pressure distribution, and behavior in the deformation process of the backfill soil.	1980-1984	KDK: 1984, pp. 689-696; 1985 (Structure I), pp. 165-174
(B)	K-K-6	Experimental research on the thermal stress of reinforced concrete structure	C (A)	Basic experiments on the thermal stress in the reinforced concrete structure were performed. On the basis of past research, the streamlining method and the sectional design method are established.	1981-1983	KDK: 1983, pp. 1549-1544; 1984, pp. 2321-2326
DK (B)	K-K-7	Research on new structural method of reinforced concrete and steel slab	C	In a reactor building, steel girders, which are embedded in concrete slab, can be used to support the deck plates to form a composite section. For this structure, the structural characteristics, operation property, and economy are investigated. The feasibility of this new construction method is assessed.	1983-1984	
(B)	K-K-8	Research on cutting cost by adopting high-strength concrete	C	In the case when a high-strength concrete is adopted for the nuclear power plant building, the problems in design and operation are evaluated, and experiments are performed to assess the material characteristics and the structural characteristics. In this way, the effect in cutting the cost is evaluated in an overall way, and design data are accumulated.	1983-1985	

(Chapter 5. Items related to building/structures)—*Cont'd*

	Number	Research item		Major content	Period	Note
(B)	K-K-9	Research on streamling the design of a deeply buried foundation of nuclear power plant building	C	In order to perform appropriate evaluation of the soil pressure in earthquake acting on the reactor building's underground walls and the behavior of the underground portion in earthquake (embedment effect), observation of soil pressure in conventional case and in earthquake and analysis evaluation are performed, and a rational soil pressure calculation method is developed.	1984-1985	
(B)	K-K-10	Research on streamlining the seismic analysis model of nuclear reactor building	C	For the dynamic analysis model with a high reliability including the high vibration frequency region, evaluation is performed mainly using the FEM model to streamline the seismic analysis.	1984-1985	
(B)	K-K-11	Experimental research on analysis method of vertical seismic motion	C	With the overall building and quipment/piping system taken as the objects, the vertical seismic motion is determined, and the analysis model and analysis method with respect to the vertical seismic motion are evaluated. In this way, research is performed to establish an appropriate dynamic analysis method for the building and equipment/piping system with respect to the vertical seismic motion.	1980-1982	
(P, B)	K-K-12	Research on aseismic capability of nuclear reactor facility set up on Quaternary stratum. Research on aseismic property of Shinritsu-type nuclear reactor building (name changed after 1983)	C	Survey/evaluation were performed on the aseismic property of nuclear reactor facility which does not stand on rock; the design method for assessing its possibility of realization was established, and the experimental test was performed.	1981-1984	

(Chapter 5. Items related to building/structures)—*Cont'd*

	Number	Research item		Major content	Period	Note
DK	K-K-13	Research on appropriate evaluation method of soil-structure interaction	C	Appropriate evaluation method is established for the soil-building interaction of nuclear reactor building for the following items: - Soil constants for dynamic analysis - Dynamic interaction between building bottom surface and soil - Embedment effect of building - Uplifting of foundation during earthquake.	1985-1987	
Nuclear Power Engineering Corporation	K-K-14	Test on dynamic interaction between building and soil	B (A)	Vibration experiments are performed using a large model simulating BWR and PWR buildings. The contents include experiment of foundation body, experiment of upper structure mounted on foundation, experiment of interaction among buildings, and experiment on side surface restraint effect due to difference in burying depth of foundation.	1980-	KDK. 1984, pp. 2301-2312
Nuclear Power Engineering Corporation	K-K-15	Test of restoring force characteristics of nuclear reactor building	A	For BWR-MARK-II type building and I/C of PWR 4 LOOP type building, small-model and partial-model test were performed to their basic restoring force characteristics data, and whole model test was performed to verify restoring force characteristics for design. In addition, tests are also performed to evaluate the scale effect.	1980-1984	KDK: 1982, pp. 957-970; 1984, pp. 2331-2338
Electrical Power Central Research Laboratory	K-K-16	Experiment for a model of a nuclear power plant concrete containment vessel, which include internal pressure by LOCA and horizontal forces acting simultaneously	A	Experiments were performed by using cylindrical specimens to evaluate the safety in the ultimate state and the behavior before the ultimate state when internal pressure and seismic force act simultaneously during a RCCV accident; the experimental results and the analysis results are compared with each other in the evaluation.		KDK: 1978, pp. 1827-1834; 1979, pp. 1381-1382; 1980, pp. 1847-1850; 1981, pp. 1417-1418

(Chapter 5. Items related to building/structures)—*Cont'd*

	Number	Research item		Major content	Period	Note
Electrical Power Central Research Laboratory	K-K-17	Test for 1/15 PCCV model under action of internal pressure, thermal, and horizontal load	A	in order to assess the behavior under a combination of major loads assumed in design of PCCV, a 1/15 model is used to perform tests under active of internal pressure, thermal and horizontal seismic load.	October 1979-1980	Prestressed concrete, January 1981, pp. 68-78
Tokyo Electric Power Co.	K-K-18	Experimental research on strength and restoring force characteristics of reinforced concrete shear walls	A	Experiments were performed to evaluate the behavior of shear walls of BWR building, such as box wall, cylindrical wall, conical wall and their combination during earthquake, and for evaluating the effects of openings.	June 1974-the 1974 fiscal year (studied by Tokyo University)	KDK: 1976, pp. 1577-1582; 1978, pp. 1619-1628. KKS: 1978, pp. 177-188.
Tokyo Electric Power Co.	K-K-19	Experimental study of reinforcing method around the openings in a reinforced concrete shear wall	A	For a single opening in a wall, the effect of the reinforcing method of the periphery of the opening on the performance of the shear wall with the opening in earthquake is evaluated, and the ultimate strength is ca[illegible]ulated.	(Performed by Tokyo University on commission)	KKS: 1979, pp. 129-132; 1980, pp. 157-16[illegible] 1981, pp. [illegible]21-132 KDK: 1979, pp. 1495-1498; 1980, pp. 1643-164[illegible]; 1981, pp. 16[illegible]-1632; 1982, pp. 1487-1488
Kansei Electric Power Co.	K-K-20	Experiment of horizontal force acting on cylindrical reinforced concrete shear wall	A	Experiment was performed on the behavior of cylindrical shear wall in earthquake when a horizontal force acts on the external shield wall of a PWR building; the various properties were evaluated.	1977-1978	KRH April 1980, pp. 57-67

(Chapter 5. Items related to building/structures)—*Cont'd*

	Number	Research item		Major content	Period	Note
Kansei Electric Power Co.	K-K-21	Research on shear strength of cylindrical wall made of prestressed concrete	A	This research was performed to obtain the data for designing a PWR building PCCV. With the internal pressure and prestressed force varied, two series of tests were performed for the cylindrical wall: torsional force test and horizontal force test. The shear strength was evaluated.	1977-1978	KDK, 1979, pp. 1392-1402
Kansei Electric Power Co.	K-K-22	Horizontal force experiment of 1/8 model of prestressed concrete containment vessel	A	In order to assess the integrity and strength of PCCV in the case of combined load in earthquake, experiment was performed with combination of internal pressure and horizontal seismic force for an accurate 1/8 scale model.	1978-1979	KDK, 1980, pp. 1851-1860
Nippon Nuclear Power Co.	K-K-23	Dynamic and static experiments of the structural strength of prestressed concrete containment vessel	A	In order to assess the vibration characteristics and aseismic capacity of a PCCV experimentally, vibration test and static horizontal force test were performed. Also, on the basis of the static experimental results, the restoring force characteristics model was set up to perform response analysis. The results are compared with the experimental results.		KDK, 1980, pp. 839-852
Japan Atomic Power Co.	K-K-24	Research on seismic property of internal concrete structure of PWR reactor's containment vessel.	A	In order to assess the strength of the internal concrete structure of PWR type 4 LOOP building, horizontal force experiment was performed for a 1/10 contracted model to evaluate the restoring force characteristics.		KDK, 1982, pp. 947-954
Power Reactor and Nuclear Fuel Development Co. (PNC)	K-K-25	Research on the internal concrete structure of reactor building	A	A three-dimensional FEM nonlinear analysis program suitable for evaluating the nonlinear behavior of structures having complicated shapes, such as the internal concrete structure has been developed. Its content is as follows: in order to develop a technique that can predict the behavior of a structure acted upon by horizontal force or thermal and horizontal load simultaneously from elastic response to failure, a program was developed, and, at the same time, simulation analysis for various experiments was performed to evaluate the reliability and applicability of the	1980-1986	KDK, 1985 (Structure I), pp. 869-888

(Chapter 5. Items related to building/structures) –*Cont'd*

	Number	Research item		Major content	Period	Note
				analysis method. In addition, analysis is also performed in consideration of the temperature dependence of material constants of a concrete structure at a high temperature over 100°C as related to thermal deformation and stress. The effects on deformation and stress are evaluated.		
Obayashi Corp.	K-K-26	Direct calculation method of floor response spectrum	A	A direct calculation method of floor response spectrum is proposed. From the target spectrum of seismic motion, instead of a simulated seismic spectrum, a floor response spectrum is directly calculated in this method.	1985-1986	KDK, 1985 (Structure I), pp. 757-760; Obayashi Corp. Technical Research Institute
Obayashi Corp.	K-K-27	Research on the maximum composite response of horizontal motion and vertical motion in earthquake	A	In order to evaluate the maximum combined response of horizontal motion and vertical motion inputs in earthquake, analysis is performed to determine the equivalent maximum response amount as a single input.	1982-1984	Nippon Jishin Kogaku Shimpojiumu [Symp. of Seismic Eng. of Japan] 1982, pp. 1129-1136; KDK, 1984, pp. 659-660
Kajima Co.	K-K-28	Structural experiment and analysis on behavior of reinforced concrete reactor building in earthquake	A	In order to evaluate the behavior in earthquake of BWR-MARK-II type building, horizontal force test of the half model of the building is performed. The results are compared with the results of FEM analysis and earthquake response analysis.		KDK: 1977, pp. 1727-1732 KRH: August 1978, pp. 35-42; September 1978, pp. 37-44
Shimizu Co.	K-K-29	Horizontal force experiment of a model of reactor building	A	With a BWR-MARK-II advanced reactor building taken as the object, models are formed for the major seismic elements of outer box, inner box and shield wall; and horizontal force experiment is performed [illegible] btain the basic data for determining the restoring force characteristics.	1979-1980	KDK: 1980, pp. 1839-1842; 1981, pp. 1419-1420

(Chapter 5. Items related to building/structures)—*Cont'd*

	Number	Research item		Major content	Period	Note
Taisei Co.	K-K-30	Strength and deformation of reinforced concrete shear wall having many small openings	A	The characteristics of a shear wall having many small openings are evaluated in a horizontal force experiment. The ultimate strength calculation method for the shear wall having randomly arranged openings is evaluated.	1979-1983	KDK: 1980, pp. 1645-1646; 1981, pp. 1623-1628; 1983, pp. 1539-1542; 1984, pp. 2343-2346 Konkurito Kogaku, January 1984, pp. 91-105
Taisei co.	K-K-31	Research on foundation uplifting of reactor building	A	The base mat uplifting is taken as a three dimensional motion; model experiment using a three-axis vibration table and analysis evaluation using a newly developed spatial analysis method are performed. Various features concerning streamlining of the analysis method of the foundation uplifting problem have been clarified.	1981-	KDK: 1983, pp. 1591-1592; 1985 (Structure I), pp. 799-800 KRH, June 1984, pp. 32-39 Taisei Corp. Technology Research Center, 1985, pp. 137-146 SMIRT (8th), 1985 K5/9, pp. 203-208
Science and Technology Agency and Constr. Co. (Toda, Sato, Nishimatsu, Kumagai, Maeda, Hazama, Fujita)	K-K-32	Research on building restoring force characteristics	A	In order to obtain data concerning the restoring force characteristics of a reactor building, horizontal force experiments were performed for small-sized model and partial model. For the small-sized model, the composite effect of a box wall and a cylindrical wall and the effect of the half symmetrical part model were evaluated. For the partial model, the effects of flanges of shear wall, heavily reinforced concrete, openings, concrete compressive strength, etc., were evaluated.	1982-1985	KDK: 1983, pp. 1495-1522; 1984, pp. 2363-2394; 1985, (Structure I), pp. 835-868

(Chapter 5. Items related to building/structures)—*Cont'd*

	Number	Research item		Major content	Period	Note
Takenaka Komuten Co.	K-K-33	Research on in-plane thermal stress in opening portion of reinforced concrete containment vessel	A	Internal pressure test and thermal stress test were performed on a circular plate made of reinforced concrete and having an opening. It has been found that they are effective for clarifying sleeve thrust force property, elastic analysis result of thermal stress, time variation of thermal propagation in the sleeve, and thermal stress analysis method of the periphery of the opening portion using nonlinear FEM analysis.	1982-1983	SMIRT (8th), 1985 H6/9, pp. 289-295
Takenaka Komuten Co.	K-K-34	Research on dynamic interaction between buildings	A	In order to evaluate the seismic safety of reactor building in consideration of the adjacent building influence on the soil-structure interaction, an evaluation method of soil spring, a stiffness evaluation method of building, and earthquake response analysis method have been developed.	1975-1982	KDK, 1977, pp. 629-632, pp. 811-814 SMIRT (6th), 1981 K2/9

(Chapter 6. Items related to equipment/piping system)

	Number	Research item		Major content	Period	Note
Building Research Institute	K-K-1	Research on non-linear response analysis	C	With a large-sized model of a nuclear reactor building, the restoring characteristics of the building are assessed, and a more rational aseismic design method is developed. A nonlinear analysis method for equipment/piping is developed, and its appropriateness and limit of application have been evaluated by analysis and experiment.	1981-1985	Genanken [Nuclear Safety Research], July 8, 1983, p. 49
National Disaster Prevention Science & Technical Center	K-K-2	Research on evaluation of seismic safty margin of equipment/piping system	C	Vibration experiment is performed for equipment/piping system important for safety; the safety margin in earthquake is confirmed.	1982-1986	Genanken [Nuclear Safety Research], September 4, 1985, p. 169
Nippon Nuclear Power Research Laboratory	K-K-3	Research on evaluation method of damage probability caused by earthquake	C	The damage probability of building, equipment, piping, etc., due to earthquake is evaluated for each aseismic design importance class; the aseismic design margin and the accident probability in a destructive earthquake are assessed.	1981-1986	Genanken [Nuclear Safety Research], July 8, 1983, p. 50
Nippon Nuclear Power Research Laboratory	K-K-4	Research on developing method for emergency operation during earthquake and inspection method after earthquake	C	In order to prevent nuclear reactor accident in earthquake, the emergency operation content is evaluated systematically. The inspection items and methods performed after an earthquake takes place are determined to check if the nuclear power plant can be restarted for operation.	1981-1982	Genanken [Nuclear Safety Research], July 8, 1983, p. 50
Nuclear Power Engineering Corporation, Ministry of International Trade and Industry	K-K-5	Survey on formation of standards for aseismic design (mechanical system)	B	Survey on the aseismic design analysis method of the major nuclear power genertion equipment—equipment system, design floor response spectrum determination method, damping constants of specific equipment, equipment aseismic design and determination of model, dynamic evaluation method of seismic property of equipment.	1976-1985	

(Chapter 6. Items related to equipment/piping system)—*(Cont'd)*

	Number	Research item		Major content	Period	Note
Nuclear Power Engineering Corporation, Ministry of International Trade and Industry	K-K-6	Test for proving seismic reliability	B	For the major nuclear power generation equipment of PWR and BWR, such as reactor containment vessel, primary cooling system piping, pressure containment, in-core structures, etc., seismic reliability tests were performed and evaluated.	1980-1988	
Same as above	K-K-7	Vibration test of piping system	B	General vibration test of piping system that simulates the actual equipment, single-part vibration test of support parts related to the general vibration test, vibration test of the frame system of piping-support and multi-input vibration test of piping are implemented; the seismic analysis code of the piping system is evaluated and the basic data for modifying the seismic analysis code of the piping system are obtained.	1980-1984	
DK	K-KI-8	Research on maintenance of function of dynamic equipment in earthquake	A	Vibration test is performed for the typical type of the dynamic equipment needed for maintaining function to obtain the data for correlation between function maintenance limit and affecting factors. On the basis of these data, the method for assessment is developed.	1980-1982	SMIRT (8th), 1985, No. K14/1-14/4
DK	K-KI-9	Experimental test of seismic property of electrical instrumentation equipment	B	Vibration test is performed for board, pressure/pressure-difference transducers for monitoring in accident, and indicators for monitoring in accident. It is experimentally proven that the function is good enough for the seismic motion for an intermediate seismic coefficient (300 Gal).	1980-1981	
DK	K-KI-10	Research on damping characteristics of piping in nuclear power plant	A	in order to amend the damping constant conventionally used in the aseismic design of piping systems, vibration test/analysis of the piping model is performed, and reasonable values are determined.	1978-1981	ASME, 1983, JUN The 4th National Congress on Pressure Vessel and Piping Technology

(Chapter 6. Items related to equipment/piping system)—*(Cont'd)*

	Number	Research item		Major content	Period	Note
DK	K-KI-11	Experimental research of analysis method for vertical seismic motion	B	For the overall building and equipment/piping, the dynamic response analysis method for the vertical seismic motion is established.	1980-1982	
DK (P)	K-KI-12	Experimental research on maintenance of function of active equipment in earthquake	A	For the dynamic equipment important for safety and with requirement on strength and dynamic function in earthquake (longitudinal ECCS pump, certain types of valves for emergency diesel, fan blower, compressor, etc.), maintenance of function is assessed experimentally and the conventional analysis method and evaluation standard are established.	1980-1982	SMIRT (8th), 1985, No. K14/1-14/4
DK	K-KI-13	Research on experimental evaluation of mechanical snubber	B	General test evaluation is performed for the durability and reliability of the mechanical snubber which has a better service property than the hydraulic snubber.	1980-1981	
DK	K-KI-14	Research on actual ability to maintain function in case of vertical vibration of core fuel and CRD during earthquake.	B	The vibration characteristics of the fuel assembly in the case of action of acceleration in the horizontal and/or vertical directions in earthquake are clarified. In addition, the scram inserting property of the control rod drive device (CRD) is confirmed.	1980-1981	
DK	K-KI-15	Study of seismic property of internal pump	B	A simulated seismic load is applied to the internal pump's motor casing portion; the appropriateness of the internal pump in earthquake is assessed, and the strength against the earthquake is evaluated	1981-1984	

(Chapter 6. Items related to equipment/piping system)—*(Cont'd)*

	Number	Research item		Major content	Period	Note
DK	K-KI-16	Research on damping characteristics of nuclear power equipment (equipment/piping)	A	In order to prevent excessive design in the aseismic design, the damping constant and accompanying evaluation method are improved according to survey of the data in the publications in Japan and other countries, as well as the experimental data accumulated.	1979-1980	ASME, 1983, JUN The 4th National Congress on Pressure Vessel and Piping Technolgy
DK	K-KI-17	Research on establishment of a rational aseismic design method of equipment/piping system	B	In order to establish a rational aseismic design method for the equipment/piping system of the nuclear power plant, a better method is established for evaluation of the seismic input to the equipment/piping system and analysis of the equipment/piping system. As a result, the cost of the construction is reduced.	1983-1985	
Research Laboratory of Manufacturer	K-KI-18	Verification of analysis method of piping system	A	By performing vibration test of needle model and small-sized piping model, the vibration characteristics of three-dimensional piping are measured; the results are compared with the analysis results obtained using the analysis program for verification of the program.	1960-1971	Hitachi Review, Vol. 52, No. 10, 1970
Same as above	K-KI-19	Verification of shell vibration analysis program	A	Vibration test is performed for the reduced model of the nuclear reactor containment vessel which has a thin shell structure, and for a simple-shaped tank model. The results are compared with the analysis results of the analysis program so that the program is verified and the appropriateness of the simple analysis method is evaluated.	1962-1983	Kikaigakkai Rombunshu, Vol. 51, No. 462, 1985
Same as above	K-KI-20	Test of fluid dynamic property	A	Rise in the free liquid level and rise in pressure on internal wall surface in core and tank are measured. The appropriateness of the design method and analysis program is checked; at the same time, the interactive effect of water and shell can be evaluated.	1962-1983	Toshiba Review, Vol. 28, No. 5, 1973

(Chapter 6. Items related to equipment/piping system)—*(Cont'd)*

	Number	Research item		Major content	Period	Note
Research Laboratory of Manufacturer	K-KI-21	In-water vibration test of in-core structures	A	Reduced models of core shroud and other in-core structures are made to vibrate in water. In this way, the pseudomass effect of water and the damping constant of the structure are measured.	1969-1972	Hitachi Review, Vol. 53, No. 11, 1971 Toshiba Review, Vol. 27, No. 8, 1972
Same as above	K-KI-22	Test of fuel assembly vibration	A	Vibration test is performed in air and water for fuel assembly with actual dimensions supported in the same way as in the actual machine. In this way, the analysis constants, such as equivalent stiffness, equivalent length, damping constant, etc., are derived.	1969-1975	Toshiba Review, Vol. 33, No. 8, 1978
Same as above	K-KI-23	Test of vibration of rod group in water	A	Vibration test is performed in water for reduced model that simulates the fuel assembly. By measuring the behavior of the fuel assembly as a group and the mutual vibration connection, the appropriateness of the analysis program and the appropriateness of the simple design formula are verified.	1970-1978	Kikaigakkai Rombunshu, Vol. 49, No. 440, 1983 Toshiba Review, Vol. 36, No. 7, 1981 Kikoron: No. 700-17, 1970; No. 710-4, 1971
Same as above	K-KI-24	Experimental test of inserting property of control rods	A	To verify the insertion function of the control rods in earthquake, the static and dynamic insertion characteristics of the control rods are experimentally studied using actual control rods, control rod drive mechanism, and simulated fuel assembly.	1969-1976	Mitsubishi Genshiryoku Giho, No. 2 Toshiba Review, Vol. 28, No. 5, 1973

(Chapter 6. Items related to equipment/piping system)—*(Cont'd)*

	Number	Research item		Major content	Period	Note
Research Laboratory of Manufacturer	K-K-25	Test of duct equivalent stiffnes	A	Static and dynamic tests are performed for the actual-sized model of thin-wall structure, such as air conditioning duct; the equivalent stiffness is measured and the analysis method by analysis program is established.	1975-1979	Kukichowa, Eisei-kogakukai Kinki Shibu Gakujutsu Kenkyu Happyo kei Rombunshu [Proc. of Symp. of Kinki Branch of Air Conditioning/Hygiene Engineering Society], No. 22, 1983
Same as above	K-KI-26	Experimental test of seismic property of electrical parts	A	For various electrical parts and panels, vibration test is performed using a vibration table to make sure that the panel is of rigid structure. In addition, it is confirmed that the response acceleration of the electrical parts is not over the allowable limit.	1970-1983	Hitachi Review, Vol. 57, No. 7, 1977 Toshiba Review, Vol. 28, No. 5, 1973
Same as above	K-KI-27	Seismic test of control rod drive device	A	By using a real-size model of the control-rod drive device, the scram characteristics test and design method in earthquake are assessed.		Toshiba Review, Vol. 27, No. 8, 1972
Same as above	K-KI-28	Seismic analysis of fuel assembly	A	Development of nonlinear response behavior accompanied with impact and verification using model experiment.	1970	SMIRT (2nd), 1973, K6/10
Same as above	K-KI-29	Seismic response analysis of nuclear reactor containment vessel	A	Development of seismic response analysis method of thin-wall shell having asymmetric mass and verification test using inelastic model.	1972	Mitsubishi Genshiryoku Giho, No. 3
Same as above	K-K-30	Seismic experiment of electrical valve	A	Experimental test is performed for the vibration characteristics, strength, and function maintenance property of electrical valve.	1974-1975	Mitsubishi Genshiryoku Giho, No. 8

(Chapter 6. Items related to equipment/piping system)—*(Cont'd)*

	Number	Research item		Major content	Period	Note
Research Laboratory of Manufacturer	K-KI-31	Vibration characteristics test of primary cooling equipment	A	Assessment of the vibration characteristics of primary cooling equipment of 1/4 scale plastic model.	1972-1976	SMIRT (5th), 1979 K13/8
Same as above	K-KI-32	Vertical seismic analysis of fuel assembly	A	Development of analysis method of fuel assembly in vertical direction and assessment test using a real-size model	1978	Mitsubishi Genshiryoku Giho, No. 20
Same as above	K-KI-33	Test of vibration characteristics of cable tray and electrical wire conduit	C	Assessment of vibration characteristics and damping of cable trays and electrical wire conduits with typical shapes.	1980	

Appendix 2. Improvement of standardization programs

Introduction

In order to establish the light-water reactor technology, the Ministry of International Trade and Industry had performed the improvement of the technology and the standardization for consolidation of improved technology.

For the improved standardization program, the first survey was performed from 1975 to 1977; the second survey was performed from 1978 to 1980; and the third survey was performed from 1981 to 1985. As a portion of this program, standardization of the aseismic design of the first survey and the second survey was performed by the Ministry of International Trade and Industry, while the third survey was performed by the Nuclear Power Engineering Corporation under commission from the Ministry of International Trade and Industry.

In the first program, feasibility study of plant was performed for the fundamental arrangement of PWR and BWR. In the second survey, the equipment arrangement program is determined for the basic configurations and basic structures of the standard PWR and BWR buildings. Efforts were made for the specific item such as aseismic design methods from the second program.

In the second survey, the standard seismic motions (S_1 and S_2) are determined for the modified standardization survey of the nuclear reactor building due to near and distant earthquakes at the low-frequency earthquake region and the high-frequency earthquake region. The seismic motions are used as the basic earthquake ground motion in the later modified standardization survey.

In this section, we will summarize the major items for the aseismic design of the second and third modified standardization surveys. For the buildings/structures, however, due to the relation with the citation of the main text, only those related to the third survey are presented.

- The numbers cited in the table are defined as follows:

  H-K-1 — (Chapter 5, Items related to building/structure)
  H-KI-1 — (Chapter 6, Items related to equipment/piping system

Appendix 2. Survey of improved standardization for building system.

No.	Item	Main content
H-K-1	Basic earthquake ground motion and design seismic motion	For the effects of bedrock characteristics on the design response spectrum, the earthquake observation records obtained for various bedrocks are used for statistical analysis; depending on the shear wave velocity, the reduction factor of the design response spectrum is derived as taken as the standard design method. As far as the shape of the vertical motion response spectrum is concerned, the bedrock array observation records in Japan are used to perform statistical analysis to assess the vertical response spectral profile.
H-K-2	Formation method of simulated seismic wave	It has been found that by taking the phase information of the simulated seismic wave into consideration, it is possible to form a simulated seismic wave that can satisfy the response spectrum corresponding to the damping constant. In this way, a standard simulated seismic wave can be formed.
H-K-3	Calculation method of static seismic loads of building and structure	Using BWR and PWR plants as examples, simple calculation formulas are developed for the fundamental natural period (T) and story shear distribution coefficient (A_i). In addition, evaluation is made of structural characteristic coefficients (D_s) based on the existing experimental results. In addition, evaluation is made of the underground portion of the building and underground structure. These results are summarized to establish a standard design method for calculating the static seismic force of the building and structure.
H-K-4	Evaluation method of soil constants for dynamic analysis	The actual state of soil survey is assessed, and the methods for handling the scatter of soil constants and soil strain level are studied. It has been found that the scatter can be evaluated using the mean value, and that the strain level almost does not enter the nonlinear region for the assumed seismic wave (M 7.0, Δ = 20). Also, as related to the interaction with the foundation, the stiffness and damping are evaluated with stiffness and damping treated using a discretization method. These results are summarized to establish a standard evaluation method for evaluation method of soil constants for dynamic analysis.
H-K-5	Seismic response analysis method of soil structure interaction model	Based on a rational simplified method (D method) for homogeneous and isotropic soil, a method which is applicable for layered soil is developed and its problems for application are clarified. As a result, it is found that it is difficult to determine the reduction coefficient of the damping constant in case of layered soil in the region of a_0 = 1-3. Based on these evalluation results, the appropriate simplified method (D method) for homogeneous soil is taken as the standard design method.

Appendix 2. Survey of improved standardization for building system—*(Cont'd)*.

No.	Item	Main content
H-K-6	Evaluation method of restoring force characteristics	Evaluation has been made of the validity of the proposed skeleton curve, shape of hysteresis loops, FEM elastoplastic analysis of overall model of BWR type building, etc. As a result, for the skeleton curve proposed in the second Promotion and Standardization Committee of LWR, the names and symbols of the skeleton curve are partially changed, and the applicable range for the ultimate shear strength (τ_u) of PWR 4 LOOP PCCV and the hysteresis characteristics proposed are clarified.
H-K-7	Allowable limit for several functions of building and structure in earthquake resistant design	Based on the survey of the allowable limit values of the reinforced concrete structure, and endurance function of leakage, a specific building is selected as the object for studying the function mentioned above. In addition, evaluation is made of the applicability of endurance function of leakage.
H-K-8	Method for evaluating the support function of building and structure	Based on the survey results, the supporting functions are evaluated. As examples, the supporting functions of the nuclear reactor building and BWR turbine building are studied.
H-K-9	Evaluation of response analysis method in vertical direction	Survey is made of the existing references related to the vertical response analysis. With regard to the contact ratio of the base, dynamic response analysis is performed for simultaneous horizontal + vertical inputs.

Appendix 2. Survey of improved standardization (equipment/piping system).

No.	Item	Main content
H-K1-1	Calculation method of static seismic force of equipment system	With BWR and PWR reactor buildings taken as the examples, C_1 distribution derived by dynamic analysis and C_1 distributions using various static analysis methods are compared with each other to find out the appropriate static analysis method. In addition, for various outdoor tanks with different sizes, the C_1 distribution derived by dynamic analysis and the C_1 distributions derived by the various static analysis methods are compared to each other to find the appropriate static analysis method.
K-KI-2	Formation method of design floor response spectrum	The factors that affect the variation in the floor response spectrum in the period axial direction are extracted. While sensitivity analysis is implemented, piping system response analysis is performed to assess the safety of the present method. As a result, although the design floor response spectrum has a broadening rate of $\pm10\%$ in principle, the widening rate can also be reduced according to appropriate evaluation.
K-KI-3	Evaluation method of damping in equipment/piping system	For the damping constants of piping, electrical panel, and cable tray, the various vibration test results are assorted and analyzed. Instead of the values conventionally used in the design, the damping constants based on test results are proposed.
H-KI-4	Coupled/decoupled analysis of building-equipment system	The coupled/decouled response analysis of the two discrete masses system of building-equipment system are compared with each other. The selection standards for the coupled analysis and decoupled analysis are assorted, and the appropriateness of the conventional method is displayed.
H-K-5	Vertical response calculation of equipment	By using the dynamic vertical seismic coefficient derived from the vertical dynamic response analysis of the building, test calculation of the equipment/piping system is performed. After comparison with the present method, it is found that there is no problem with respect to the conformity.
K-KI-6	Evaluation method of sloshing	Comparison among several methods is performed. As a result, it is found that although the simple calculation method of the velocity potential theory of Hausner can be found to perform the design in a simple way, the value obtained is rather conservative. On the other hand, the method of time history response calculation using FEM gives more appropriate values.
H-K-7	Standard design method of Class B and C equipment/piping systems	Aseismic design methods have been assorted for Class B and C vessels/tanks, pump floor, piping, duct, tray, and anchorage.
H-KI-8	Aseismic evaluation method of dynamic equipment	Based on the procedure of evaluation of the existing research results, test calculation is performed for the various typical dynamic equipment. Based on the result of this calculation, the modified program is studied as a standard procedure of evaluation.
H-KI-9	Evaluation method of equipment/piping anchorage	For the strength calculation formula based on the present regulations, the applicability with respect to the experimental values is studied. It is found that the pullout strength is in good agreement with the ACI standard formula, with a small scatter.

Appendix 2. Survey of improved standardization (equipment/piping system)—*(Cont'd)*.

No.	Item	Main content
H-KI-10	Combination of stresses in vertical and horizontal directions	Results obtained by the present method, the SRSS method and the absolute sum method, in which the combination of the vertical dynamic seismic coefficient with the horizontal seismic forces is considered, are compared with each other. As a result, it is found that while the present method gives safer results, the SRSS method with a combination of the vertical/horizontal directions provides more appropriate results.
H-KI-11	Seismic analysis method of major equipment	For equipment in high safety classes, i.e., aseismic Classes As and A, the conventional schemes of the earthquake response analysis and stress analysis are assorted, and the standard calculation method and evaluation method are specifically discussed for each item of equipment. In addition, the damping constant in S_2 earthquake is discussed in consideration of high stress and high strain level.
H-KI-12	Limit load of equipment system	For the determination method of limit load of the main equipment, while the limit load is derived using the stress evaluation method, the standard load is calculated. In addition, the limit load is included with respect to the load derived using the standard design floor response curve.
H-KI-13	Equipment support structure and limit load	Evaluation is made to see the degree of streamlining for the equipment used in a low earthquake frequency region as compared to that used in a high frequency region (in particular, with respect to the support structure). The conclusion is that it is important to have the floor response spectrum decreased.
H-KI-14	Floor response spectra for standard design	The sway/rocking model is used to perform seismic response analysis for a standard nuclear reactor building. The design floor response spectrum with respect to seismic motion for standard soil is prepared.
H-KI-15	Evaluation of standardization of seismic calculation manual	Evaluation is performed of the necessary items described in the seismic calculation manual. Assortment is performed to simplify the calculation manual and to improve the operation efficiency. For the Class B equipment with standardized design method, the examples of description are presented.
H-KI-16	Seismic evaluation method of standardized equipment and machines	As an index of the aseismic design of the piping system, for types of limit spectrum methods are set, and the appropriate method as the seismic property evaluation method during the process of standardization is extracted. In addition, during the standardization of the piping system, the method is discussed that allows variation in the layout; the effectiveness of the method is confirmed.
H-KI-17	General evaluation of aseismic design of equipment system	With reference to representative examples, evaluation is performed of the engineering judgment content, its reason, and its effect on the stress evaluation.

Appendix Table 2-1. Summary of survey on modified standardization (building system).

	Major items	Minor items	Method of evaluation	Results or proposals
Design seismic motion	Basic earthquake ground motion and design seismic vibration	Determination of spectrum corresponding to bedrock characteristics	The measured bedrock data are assorted with M-Δ, V_S taken as the parameters (regression analysis).	For the conventional Osaki spectrum, the agreement in the case of $V_S = 0.7$ km/s is good. As V_S increases, the agreement decreases. It is proposed that the behavior be represented by a period dependent formula.
		Study of vertical motion spectrum shape	By statistical evaluation of array system observation data on three bedrock sites, the spectrum shape is evaluated.	It is confirmed that the second Improvement and Standardization Program is appropriate.
		Transformation method form standard seismic motion to input seismic motion to model	One-dimensional wave propagation theory, FEM scheme	The types of the transformation methods are exemplified.
			As the two-dimensional problems for evaluating the influence of topography, evaluation is made using FEM for P, S_V waves and using the Green's function for S_H waves.	Examples of calculation are presented for the case when there exists terrace topography in the periphery of the building (the height of the terrace and the angle of the slope are taken as the parameters).
	Generation method of simulated seismic wave	Suitability of spectrum with respect to two damping constants ($h = 0.01, 0.05$)	Comparison of the time histories and spectral characteristics of waves depending on the phase characteristics definition.	A mixture of pulse phase, exponential functional phase, and uniform random number phase has a good correspondence.
Calculation method of static seismic force for building/structure	Calculation method of static seismic force of building and structure	Seismic regional coefficient (Z)		$Z = 1.0$ for all of Japan
		Vibration characteristic coefficient (R_t)	General judgment is made in consideration of reduction in input seismic motion (α), vibration characteristics of building (β), and reduction due to embedment depth (γ) ($R'_t = \alpha \cdot \beta \cdot \gamma \cdot R_t$).	It is determined as $R'_t = 0.8$.
		Fundamental natural period (T)		Eigenvalue analysis is performed.

Appendix Table 2-1. Summary of survey on modified standardization (building system)—*(Cont'd)*.

	Major items	Minor items	Method of evaluation	Results or proposals
Calculation method of static seismic force for building/structure	Calculation method of static seismic force of building and structure	Story shear distribution coefficient (A_i)		The modal method using the vibration system with eigenvalue analysis in consideration of soil influence.
		Structural characteristic coefficient (D_S)	Test calculation is made using the formula according to Building Standard Law and the guideline of energy equivalence of structure.	For reinforced concrete structure with shear wall as the major aseismic element, it is set as $D_S = 0.45$.
		Shape characteristic coefficient (F_{es})		It is set as $F_S = 1.0$.
		Treatment of underground portion	Evaluation of difference in response characteristics due to embedment using FEM	The portion below the soil surface with embedding effect is taken as the underground portion. The underground seismic coefficient formula according to the "Building Standard Law" is applicable
		Safety margin		The lower limit value is taken as 1.6 for Class A, 1.2 for Class B, and 1.0 for Class C.
Earthquake response analysis method	Evaluation method of soil constant for dynamic analysis	Elastic parameters of soil	Study of numerical differences due to difference among the various survey methods. Evaluation of variabilities corresponding to the period derived by analysis from the elastic wave test and the period derived from the observed earthquake records. Study of the soil strain level in the case of earthquake by dynamic analysis of hard soil.	By elastic wave tests (PS logging, elastic wave exploration etc.). The mean value is taken for the above test results. For a hard soil, there is almost no decrease in the stiffness; hence, the aforementioned test value may also be used.
		Stiffness and damping of soil spring of sway/rocking model	The stiffness/damping of soil spring obtianed from the existing results of vibration exprimental tests for block foundation and nuclear reactor buildilng.	Good correspondence to the theoretical value. However, it is necessary to consider the effect of the portion where the stratification influence is significant.

Appendix Table 2-1. Summary of survey on modified standardization (building system)—*(Cont'd)*.

	Major items	Minor items	Method of evaluation	Results or proposals
Earthquake response analysis method	Seismic response analysis method of soil-structure model	Method of treatment of stiffness and damping of soil spring of sway/rocking model	Response comparison is implemented for results obtained using the conventional method (static spring, damping), frequency dependence method, constant soil spring method, and rational simplified method (D method).	D method, which can display the dependence of damping on frequency, is useful for design.
		Evaluation of sway/rocking model in consideration of embedment	In order to form the input wave, one-dimensional wave teary or two-dimensional FEM are used. In order to evaluate the soil spring, the existing D-method or side-surface spring method is used.	
		Comparison of discretization	Comparison is made between the grid type model and FEM model.	If appropriate measure are taken for forming the model, the two show the same response.
Safety function evaluation method of building/structure	Evaluation method of restoring force characteristics	Appropriateness of skeleton curve	Comparing the existing structural experimental data and FEM elastoplastic analysis results with the values of the second Improvement and Standardization Programs.	Proposal is made for various evaluation formulas at the first turning, second turning and maximum yield strength points in trilinear skeleton curves of M-ϕ, Q-γ relations.
		Appropriateness of hysteresis rule		M-ϕ relation: peak-oriented Q-γ relation: origin-oriented
	Allowable limit in function maintenance of building/structure for seismic motion	Determination of allowable limit value of structure	Using a safety factor of 3 in energy absorption	As the allowable limit values, $\mu = 2$ for shear ductility factor and $\mu = 10$ for bending.
		Allowable limit value required by equipment system	The request from the equipment/piping system is considered as characteristics of the structure.	Determination of allowable value for maintaining leak proof function.
	Evaluation method of support function of building/structure	Assessment of deformation/strength	Assortment of basic guidelines. Survey of existing plant status.	Evaluation methods are summarized and the test calculation is presented for the typical building.

Appendix Table 2-1. Summary of survey on modified standardization (building system)—*(Cont'd)*.

	Major items	Minor items	Method of evaluation	Results or proposals
Study of vertical response analysis method	Study of vertical motion response analysis method	Contact ratio	Nonlinear analysis is performed considering uplift, and comparison is made with the conventional evaluation method.	The results are much larger than those in the conventional evaluation method.
		Floor response spectrum	Linear analysis and nonlinear analysis (parameter: contact ratio to ground) is compared.	For the design seismic motion, the effect on the floor response spectrum is small.

Appendix Table 2-2. Survey of improved standardization (equipment/piping system).

	Major items	Minor items	Method of evaluation	Results or proposals
Calculation method of design seismic force	Calculation method of static seismic force in equipment system	Application method for equipment/structure in building	With the BWR and PWR nuclear reactor buildings taken as examples, the C_1 distribution derived from dynamic analysis results is compared with the C_1 distributions according to various static analysis methods.	The vibration mode of the eigenvalue analysis result of the building is used for modal analysis.
		Application method for outdoor equipment	For the va[illegible]or tanks of different sizes, th[illegible]n derived from dynami[illegible]ompared with t[illegible] C_1 distr[illegible]g to various static analy[illegible]	As a simplified method, the tank is simulated by two-discrete mass model, and the primary mode is used for the modal method.
	Formation method of design floor response spectrum	Study of the necessary broadening rate for various factors	The factors that affect the variation of the floor response spectrum in the period axial direction are extracted, and sensitivity analysis is implemented.	The floor response spectrum can be enclosed by $\pm 10\%$ broadening. The fluctuation width of the affecting factors is quantitatively displayed.
		Summary of proposals for the necessary broadening rate	Piping system response analysis is performed to assess the safety of the present method, and the sensitivity analysis results are summarized.	For the design floor response spectrum $\pm 10\%$ broadening is performed. Based on an appropriate evaluation, however, it is possible to decrease the broadening rate.
			Regression analysis is performed for the various vibration test results; the damping constant in earthquake is calculated.	Instead of the conventionally used values, the damping constant determined by various pipings and support conditions is assigned.
Earthquake response analysis method	Evaluation method of damping of equipment/piping system	Damping constant of piping system	The vibration test results of the actual equipment are assorted, the damping constant in earthquake is calculated.	Instead of the conventionally used values, damping constant based on the vibration test results is presented.
		Damping constant of panel and cable tray	The decoupled/coupled response analysis results of single-discrete mass system of equipment-building are compared and evaluated.	The selection standards of coupled analysis/decoupled analysis are assorted; it is found that the present method is appropriate.

Appendix Table 2-2. Survey of improved standardization (equipment/piping system)—*(Cont'd)*.

	Major items	Minor items	Method of evaluation	Results or proposals
Earthquake response analysis method	Coupled/decoupled analysis of building-equipment system	Study of coupled/decoupled standards of two-discrete mass system	Effects of the decoupling position and the difference in the overall system model on the response values are evaluated.	In consideration of the support system of the equipment to be studied, the separation position is selected.
		Decoupled analysis using actual equipment model	From the dynamic vertical seismic coefficient derived using the vertical dynamic response analysis of the building, test calculation of the equipment/piping system is performed. The results are compared with those of the present method.	The vertical motion response has no great difference from the present static vertical seismic coefficient. There is no problem for the agreement with the present method.
	Equipment's vertical motion response calculation	Vertical response analysis of major equipment/piping system	Comparison is made among Hausner theory, velocity potential theory and FEM.	For the simple calculation method, although the design is simple, the results obtained are rather conservative.
	Sloshing evaluation method	Comparison of conventional methods		
Structure design method	Standard design method of Class B and C equipment/piping	Standard evaluation method of containers, piping/duct, and pump/fan	Evaluation of standard aseismic design methods of Class B and C containers, tanks, pump/floor, piping, duct, tray, and anchorage	Standard design method are assorted for the Class B and C vertical four-leg support tank, skirt support tank, flat bottom tank, horizontal single-barrel cylinder container, leg-support vertical cylindrical container, vertical/horizontal pump, floor and anchorage; the standard design procedure is summarized for piping, duct, and tray.
	Seismic evaluation method of active equipment	Test calculation of typical equipment	Based on the evaluation procedure for the existing research results, test calculation is performed for the various types of typical equipment.	For the existing evaluation procedure, the items needed for evaluation are extracted, and the evaluation examples are presented.
		Modification of standard evaluation sequence	For the standard evaluation procedure, the conformity of various types of equipment and the modification scheme are investigated.	The scheme of modifying the existing evaluation procedure is studied as a standard method.

Appendix Table 2-7 Survey of improved standardization (equipment/piping system)—*(Cont'd)*.

	Major items	Minor items	Method of evaluation	Results or proposals
Structure design method	Evaluation method of equipment/piping anchorage	Comparison of evaluation methods	Evaluation is made of the applicability of the strength calculation formula based on the present regulations with respect to the experimental values.	For both anchor bolt and embedding plated, good agreement with ACI regulation formulas has been found, and the dispersion width is also small.
	Combination method of vertical/horizontal stresses	Comparison of combination methods using practical piping system model	Results are compared between the present method using vertical static seismic coefficient and the method in which absolute sum is derived for the combination of dynamic vertical seismic coefficient and horizontal [seismic coefficient] as SRSS.	SRSS method is appropriate as a method with combination of vertical and horizontal directions.
	Seismic analysis method of major equipment	Stress evaluation method of Type 1 containments Type 1 piping, Type 2 containment's penetration portion and major equipment	With respect to equipmetn with high safety level, such as aseismic Class As and A, seismic response analysis method, stress analysis method and other conventional methods are assorted with emphasis put on the discrete mass system spectral modal analysis.	For each item of equipment, the standard calculation method and evaluation formulas are presented. Also, study is made of using a damping constant in S_2 earthquake in consideration of high stress/strain level.
Standardization of seismic load	Limit load of equipment system		In order to set the limit load of the major equipment, the limit load is derived according to the stress evaluation mthod, and, at the same time, calculation of the standard load is investigated.	The design method of the limit load is presented. In addition, it is pointed out that the limit load is also included in the load derived using the standard design floor response c[illegible]
	Equipment support structure and limit load		Study [illegible] performed to find the degree at which the equipment for the low earthquake zone can be rationalized as compared to those for the high earthquake zone (in particular, with respect to the support structure).	Although it is [illegible]ible to fully streamline the support structure of equipment/piping for the lower earthquake zone, it is found as an important conclusion that the floor response spectrum can be reduced after performing detailed study of the nuclear reactor building.

Appendix Table 2-2. Survey of improved standardization (equipment/piping system)—*(Cont'd)*.

	Major items	Minor items	Method of evaluation	Results or proposals
Standardization of seismic load	Standard design floor response curve	Formation of design floor response spectrum	With the aid of sway/rocking model, the seismic response analysis of the standard reactor building is performed to form a response spectrum of the floor on which the equipment is installed.	For each of the standard seismic motions, including horizontal motion and vertical motion, the design floor response spectrum is derived for soils with V_S = 500, 1,000 and 1,500 m/s, respectively.
Others	Study of standardization of seismic calculation manual	Standardization of document format	Study is performed of the items needed for seismic calculation manual; assortment is performed to simplify the calculation manual and to improve the efficiency of operation.	Examples are presented for Class B equipment with standardized design method.
	Seismic evaluation method of established model of earthquake and machines	Method of determining limit spectrum	As index of seismic property of the piping system, four types of limit spectral methods are set up. The features of these methods and their application schemes are evaluated.	The constant value limit spectrum method or the mode group constant value limit spectrum method is found appropriate for evaluation the seismic property when the model is to be established.
		Standard routine of established model of piping system	Study is performed of the method which allows variation in the layout when the model of the piping system equipment is to be established.	Assessment is made of the effectiveness of the method of making the primary natural period smaller than the target period.
	Overall evaluation of aseismic design of equipment system	Degree of influence of factors on stress evaluation	The aseismic design method is surveyed for the equipment/piping; the engineering judgment content for the design is studied.	The arguments for the engineering judgment in the aseismic design of equipment/piping and the effects of the engineering judgment on the stress evaluation are discussed with reference to typical examples.

Appendix 3. Aseismic specifications of various power plants.

		Power Plant A	Power Plant B	Power Plant C	Power Plant D
	Reactor Type	BWR: MARK-II	BWR: MARK-I	PWR: 3 LOOP	PWR: 4 LOOP
1.0	Support ground of nuclear reactor facilities				
1.1	- Geology	Mudstone	Sandstone/mudstone laminate	Sandstone, conglomerate, clay slate	Granite
1.2	- Density	1.73 t/m^3	2.1 t/m^3	2.7 t/m^3	2.4 t/m^3
1.3	- Propagation velocity of shear wave	0.49 km/s	0.7 km/s	1.8 km/s	1.6 km/s
2.0	Design standard				
2.1	- General features (earthquake determination method, static seismic force, vertical seismic coefficient, etc.)	Examination Guideline of Aseismic Design[1] 1978	Examination Guideline of Aseismic Design, 1981	Examination Guideline of Aseismic Design 1978	Examination Guideline of Aseismic Design 1981
2.2	- Classification of importance degree	Same as above	Same as above	Same as above	Same as above
2.3	- Allowable limit/equipment-piping system	Same as above	Same as above	Same as above	Same as above
2.4	/Building-structures	Same as above, Standards of Architecture Institute of Japan	Same as above, Standards of Architecture Institute of Japan	Same as above, Standards of Architecture Institute of Japan	Same as above, Standards of Architecture Institute of Japan
3.0	Basic earthquake ground motion (for design of Class As and A nuclear reactor facilities				
3.1	- Assumed position (rock outcrop surface)	GL. − 180 m (= EL. − 168 m)	GL. − 20 m (= EL. − 14 m)	GL. − 31.5 m (= EL. − 18.5 m) (Building foundation bottom surface)	GL. − 17 m (= EL. − 10 m)

Appendix 3. Aseismic specifications of various power plants—*(Cont'd)*.

		Power Plant A	Power Plant B	Power Plant C	Power Plant D
	Reactor Type	BWR: MARK-II	BWR: MARK-I	PWR: 3 LOOP	PWR: 4 LOOP
3.2	- Maximum design earthquake → Standard earthquake S_1/maximum accelerator (simulated seismic wave)	180 Gal	450 Gal	188.5 Gal	365 Gal
3.3	/Maximum acceleration of response spectrum[2]	450 Gal (Standard response spectrum)	1,530 Gal (Standard response spectrum)	525 Gal (Standard response spectrum)	1,000 Gal (Standard response spectrum)
3.4	- Extreme design earthquake → Basic earthquake ground motion S_2 /maximum acceleration (simulated seismic wave)	270 Gal & 370 Gal [3]	600 Gal	371.8 Gal & 370 Gal [3]	532 Gal
3.5	/Maximum acceleration of response spectrum[2]	668 Gal & 1,137 Gal [3] (Standard response spectrum)	2,030 Gal (Standard response spectrum)	1,086 Gal & 1,137 Gal [3] (Standard response spectrum)	1,400 Gal (Standard response spectrum)
3.6	Safety evaluation/analysis method of nuclear reactor facility support soil	Conventional method (slip-surface method, etc.)	FEM	FEM	FEM
3.7	/Soil seismic coefficient	—	0.2	0.2	0.2
4.0	Seismic analysis of nuclear reactor building				
4.1	- With analysis method/S_1 [4]	Grid-type model, elastoplastic time history response analysis	Grid-type model, elastoplastic time history response analysis	Sway-rocking model, elastic time history response analysis	Sway-rocking model, elastic time history response analysis
4.2	With analysis method/S_2	Same as above	Same as above	Sway-rocking model, elastoplastic time history response analysis	Sway-rocking model, elastoplastic time history response analysis

Appendix 3. Aseismic specifications of various power plants—*(Cont'd)*.

		Power Plant A	Power Plant B	Power Plant C	Power Plant D
	Reactor Type	BWR: MARK-II	BWR: MARK-I	PWR: 3 LOOP	PWR: 4 LOOP
4.3	- For S_2/base shear coefficient[5]	0.59[6]	0.74	0.68	1.1
4.4	/Maximum response acceleration at top portion	0.9 G[6]	2.0 G	2.2 G	2.8 G
5.0	Seismic analysis of major equipment				
5.1	- Nuclear reactor containment vessel/analysis method	Grid-type model, coupled to building, elastic time history response analysis	Grid-type model, coupled to building, elastic time history response analysis	Grid-type model, coupled to building, elastic time history response analysis	Grid-type model, coupled to building, elastic time history response analysis
5.2	/Maximum response acceleration during S_2	0.7 G[6]	1.2 G	4.0 G	3.3 G
5.3	- Nuclear reactor containment/ analysis method	Grid-type model, coupled to building, elastic time history response analysis	Grid-type model, coupled to building, elastic time history response analysis	Floor response spectrum, modal analysis	Floor response spectrum, modal analysis
5.4	/Maximum response acceleration during S_2	0.9 G[6]	1.8 G	2.0 G[7]	2.9 G[7]
5.5	- Main steam piping (within nuclear reactor containment vessel)/ analysis method	Floor response spectrum, modal analysis	Floor response spectrum, modal analysis	Floor response spectrum, modal analysis	Floor response spectrum, modal analysis
5.6	/Maximum acceleration of floor response spectrum during S_2[7]	1.9 G	5.6 G	5.0 G	4.0 G

[1]Examination Guideline of Aseismic Design Concerning Nuclear Power Generation Facilities (same for the items in the right-hand columns in this line).
[2]Maximum value for 5% damping and in C.1–0.3 sec.
[3]Determined according to shallow-focus earthquake and other earthquake.
[4]For a seismic force based on basic earthquake ground motion S_1.
[5]Upper surface of foundation mat.
[6]Value based on static seismic force.
[7]Maximum value of floor response spectrum within the major characteristic period.

Appendix 4. Recent survey report of intra-plate earthquakes

This portion of survey reports is prepared as a reference for "Chapter 2. Earthquake and basic earthquake ground motion." It summarizes the major earthquake features, source parameters, damage states, earthquake phenomena, etc., for four typical earthquakes in Japan and three typical earthquakes in other countries, among the major inland-type earthquakes that took place in the past 10 years.

The content of this appendix is based on citations from the references. As far as the fault parameters are concerned, different researchers provide different data. Hence, only the typical data are cited.

The earthquakes under our consideration are as follows:

(1) 1974: Off Izu Peninsular Earthquake

(2) 1975: Central Oita-ken Earthquake

(3) 1983: Tottori Earthquake

(4) 1984: Western Nagano-ken Earthquake

(5) 1976: Northern Italy Earthquake

(6) 1979: Imperial Valley Earthquake

(7) 1983: Coalinga Earthquake

No. 1 Name of earthquake: 1974 Izu Peninsular Offing Earthquake

Origin time: 8:33'27" a.m., May 9, 1974 (Japan standard time)

Epicenter: 138°48'E, 34°34'N

Magnitude: 6.9 Focal depth: 10 km

Summary of earthquake [1,2]

The earthquake took place at the south end and offing of Izu Peninsula. The seismic motion with fierce vertical movement hit hard on the south portion of the peninsula, with loss of human lives and damage to buildings and civil structures. For the single-fold seismograph set at Irozaki Monitoring Station, both the vertical and horizontal displacement components were off scale, and it is thus impossible to estimate the amplitude, period, etc. As far as the amplitude is concerned, both the horizontal and vertical components of the amplitude were certainly over 6 cm. The duration of the earthquake was about 15-20 sec.

The features of this earthquake include many earth cracks, generation of faults, landslide, falling rocks, etc., at the damage region. The southern part of the Izu Peninsula has a complicated topography with low mountains 100-300 m above the sea distributed in it. There exist many faults and discontinuous liens of topography related to the faults among them. It is believed that several faults were active in this earthquake. The landslides and falling rocks were induced by the movement of these faults.

The damages caused by the earthquake were surveyed on May 13 by Shizuoka-ken Hazard Countermeasure Headquarters as follows: killed and missing persons: 29; injured persons: 74; completely destroyed houses: 121; half-destroyed houses: 242; partially damaged houses, 1,274; broken road sites: 69; landslide sites: 91. A prominent feature of this earthquake damage is that the houses were damaged not only by simple vibration, but also by fault activities, which cause foundation shear, separation/ rotation between foundation and columns, etc. In addition, many houses were crushed by landslides. Most of the killed and missing persons were due to landslide crushing. It is believed that the landslide and road damage are also related to the faults. The damage was concentrated at the southern portion of the Izu Peninsula. In particular, the damage was serious for the western region around Irozaki. The eastern region seemed to have lesser damage.

Seismic intensity by JMA	According to [3]

Maximum seismic intensity: 5, at Irozaki

Seismic intensity distribution

Figure 1 illustrates the distribution of the seismic intensity. At Irozaki, near the epicenter, the earthquake was as strong as having a seismic intensity of 5 refined by the Japan Meteorological Agency. In addition, the seismic motion propagated widely in central Japan with the following seismic intensity distribution: 4 at Shizuoka, Mishima, Yokohama, Tateyama, Oshima, Shinshima; 3 at Hamamatsu, Nagoya, Iida, Suwa, Kawaguchiko, Kofu, Chichibu, Maebashi, Tokyo, Choshi, Miyakejima, etc.; and 2 at Osaka, Hikone, Karuizawa, Kumagaya, Mitto, Onahama, Shirakawa, Chiba, Irago, Tsu, etc.

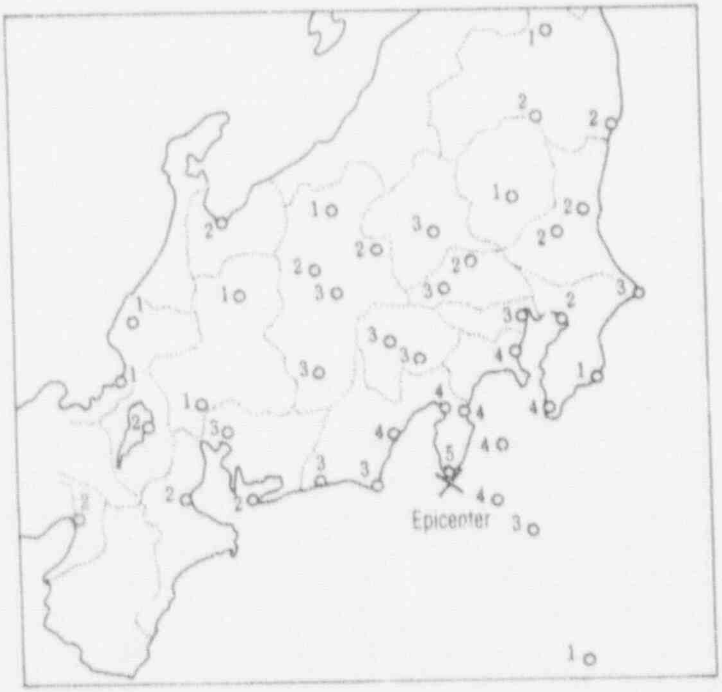


Figure 1. Seismic intensity distribution.

Fault parameters	According to [2]	
Fault length: 25 km	Width: 15 km	Depth: 10 km
Dip angle: 75°NE	Dip direction: N43°W	
Mo: 1.1×10^{26} dyne·cm	Source displacement: 1.0 m	
Slip type: right strike slip	Rise time: 2.0 s	
Rupture velocity: 2.5 km/s	Stress drop: 57 bar	

Figure 2 shows the fault positions. According to several sets of data, several seismic faults developed in the region from Irozaki to Mera through Iruma. They are all large-angle right transverse deviation faults, with directions of WNW near Irozaki, NW near Iruma and NNW near Mera. Although there is a certain variation in the direction, the general pattern is an arc shape stretched in the SW direction. The distance between A and G is about 7.0 km; the distance between A and Z or W is about 9.5 km; also, since the fault extends to the seabed in the southeastern direction from A, the length is over 10 km. Among these faults, for the portion in A-G (Irozaki, Iruma fault), although it is impossible to determine whether it is a single fault sequence or is made of 2-3 faults arranged in stepping form, in consideration of the relation between its position and epicenter and the earthquake magnitude, it is believed to be the major fault related to the focal plans of the earthquake. The slip displacement is as large as 30-40 cm in the Irozaki, Iruma region at the southeast portion of the fault line; while the slip displacement decreases drastically for the northwest portion. As far as the slip direction is concerned, it is mainly right transverse slip. At the Irozaki Iruma region, there is also a southbound upward component with a slip displacement about half that of the transverse slip amount.

The other seismic faults, i.e., H-I, J-K, L, M-N, etc., are minor faults parallel to the aforementioned major faults. They are also right transverse slip faults, with slip displacement much smaller than that of the Irozaki, Iruma fault. In addition, they do not extend for a great length. It is believed that these minor faults converged underground to the fault at the Irozaki, Iruma region.

Fault positions

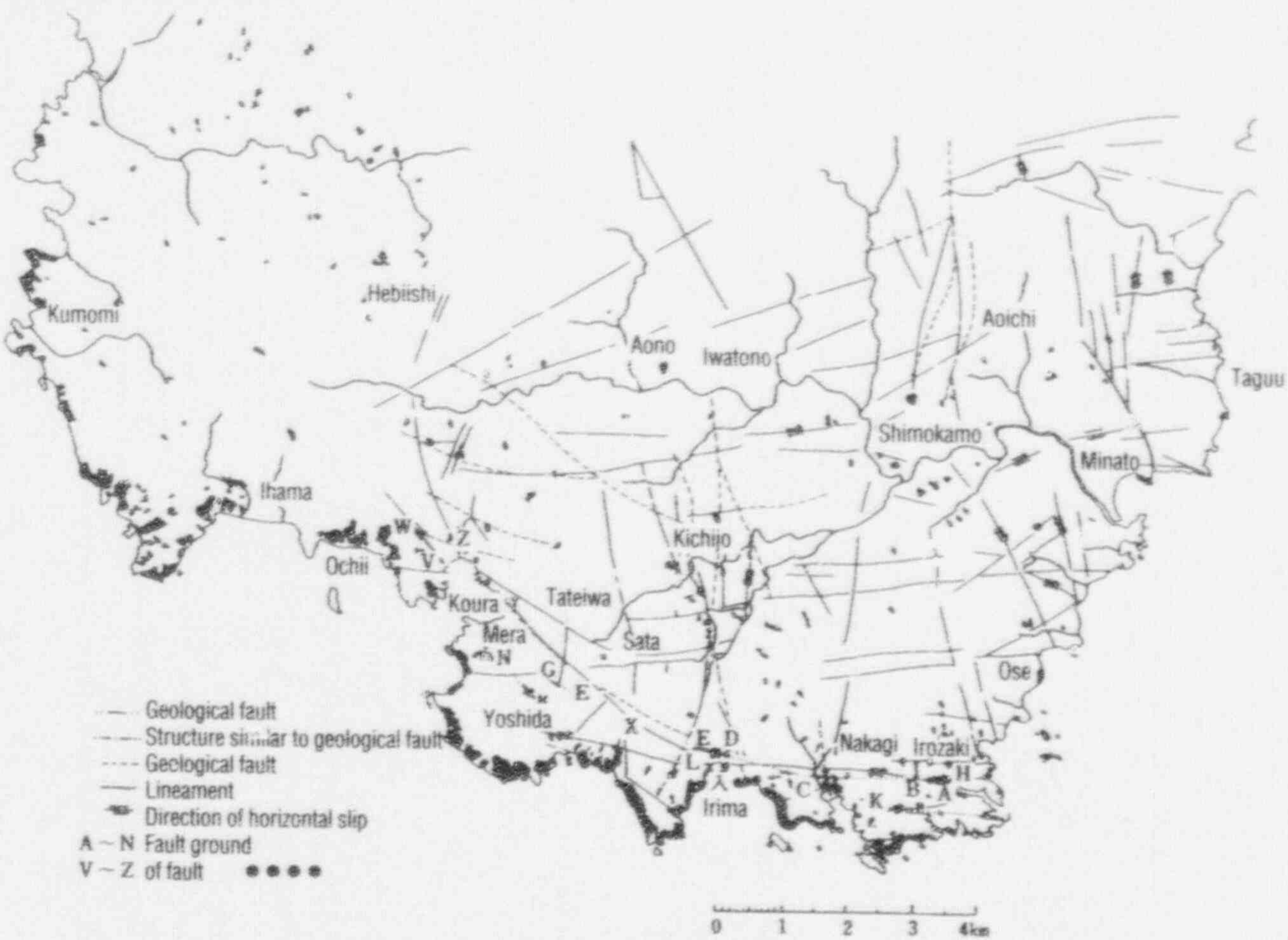


Figure 2. Fault positions.

Damage caused by earthquake According to [1,2]

Features of damage

The features of damage are as follows:

(1) The damage is concentrated in a relatively limited region.

(2) The causes for loss of human lives are landslide and cliff collapse.

(3) The most prominent feature is crushing of structures by landslide and precipice collapse, followed by the damage caused by soil and foundation slip. On the other hand, damage caused by vibration of the structure itself are less significant.

Distribution of damage

The damage is concentrated at the south portion of the Izu Peninsula. In particular, the damage is serious at the western region around Irozaki. On the other hand, the damage is less significant at the eastern region. Figure 3 shows the damage distribution of various structures.

(i) Slope collapse

(1) Complete collapse of beach precipice near the focus
The coastal line from near Okuirosaki to Mera via Nakaki, Iruma, and Yoshida.

(2) Minor landslide directly caused by seismic faults
Northern part of Nakaki, Iruma, Southern part of Mera, Northern part of Mera to Eastern part of Koura.

(3) Landslides with different magnitude caused by the existing faults
Sata, Kichijo, Shimokamo and vicinity

(4) Others, collapse of unstable slope
villa formed land in northern part of Kichijo

(ii) Buildings/structures

1) Regions with direct damage caused by the seismic faults
Irozaki, Iruma regions

(2) Regions which are coastal low land with concentrated dam[illegible], although they are not composed of very soft soil
Koura, Mera, and Tauma regions

(3) Damage in weak soil
Street land of Shimoda-shi

(4) Damage on slopes
Nakaki, Ochii, Ihama, and a portion of Shimokamo

(5) Damage to land and mountains

(iii) Civil structures

For civil structures, except for the blockage of roads by falling sand and rocks due to collapse of slopes, the damage is concentrated in the various regions of Irozaki, Nakaki, Iruma, and Koura. Among them, the damage to the former 3 regions is limited to the vicinity of the seismic faults.

Typical damage

(i) Collapse of slopes

The amount of collapsed soil reached 10^4 m^3. At the Nakaki region where 27 persons were killed or are missing, collapse of the eastern slope of Mt. Ochihata was due to the relatively thin layer of collapsed materials deposited on the shallow-valley slope formed in ancient landslides 10^2-10^3 years ago, and a certain amount of in situ weathered materials. At a portion of the landslide surface, the existing fault plane in the NNE direction was exposed.

(ii) Buildings/structures

The damage to the different types of structures was as follows:

(1) For the RC buildings, the harm level was low, and there was almost no fatal damage to the main structure.

(2) For 2-story or 3-story houses and shops made of steel structure finished with mortar, there was significant damage on the outer walls, such as cracks, peeling, etc.

(3) For the 2-story buildings at Iruma region made of RC blocks, some collapsed, while some were almost unharmed.

(4) For wood houses, there was significant movement of roof tiles as well as significant peeling and cracks on the mortar outer walls.

(5) In addition, as far as the stone structures are concerned, the monitoring tower near the beacon at Irozaki was completely destroyed, other structures were also seriously damaged. Also, damage was also significant for stone treasure houses.

(iii) Civil structures

The damage to different types of civil structures was as follows:

(1) Damage to roads

(a) Traffic roads were blocked due to collapse of hillside, precipice, or slope above the roads.
Several sites in the Irozaki-Nakaki section of Shimota-Ishimuro-Matsuzaki railway line, Chogano, Ochii.

(b) Partial collapse of road surface due to settlement of filled soil or damage to filled soil wall.
North part of Nakayama region and the region near Koura of Margaret Line.

(c) Harm due to seismic faults
Northwestern part of Irozaki region, northern part of Nakayama region.

(2) Bridges

For the Tengo Bridge at the northern part of Koura region of Maragaret Line, the bolts for bonding the girders to shoes were detached; the concrete at the bridge table mounting portion of the fall preventive device was peeled off. In addition, it was found that the joints of the high rail near the fixed end were detached and the soil portion with the bridge platform settled.

In addition, for the simple pedestrian bridges (widths: 2-3 m, span between supports: 4-5 m, made of concrete plates) near the seismic fault at the Iruma region, the bridge platforms slipped and the bridges were in a state close to falling.

(3) Retaining walls

About 20-30% of the pertaining wall in Ihama region collapsed. They were all dry walling gravel. In the Iruma region, the unreinforced concrete retaining walls were cut due to the seismic faults.

(4) Harbor structures

At Koura Harbor, the concrete surface of loading land cracked and sank; the retaining bank, which is rather new, slipped and inclined by several centimeters.

(5) Tunnel

Cracks developed in the Nakaki Tunnel located in the northern part of the Nakayama region near the seismic fault.

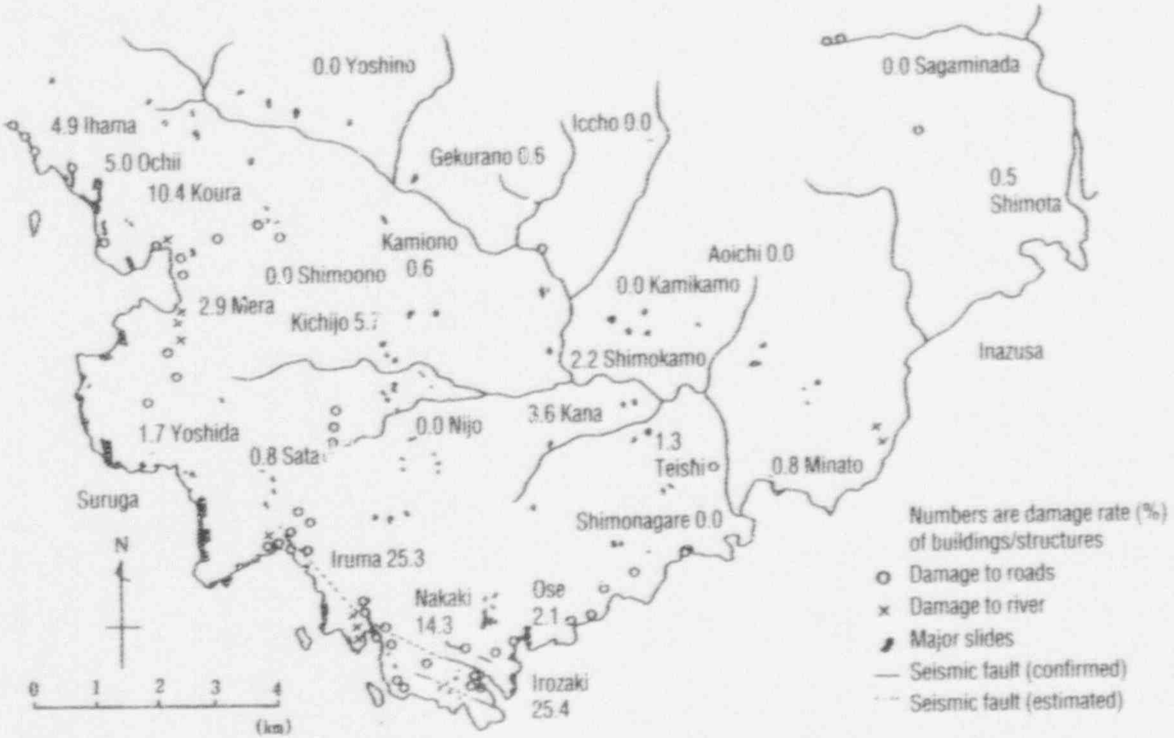


Figure 3. Schematic diagram of damage to various structures.

Earthquake records	According to [5, 6, 7]

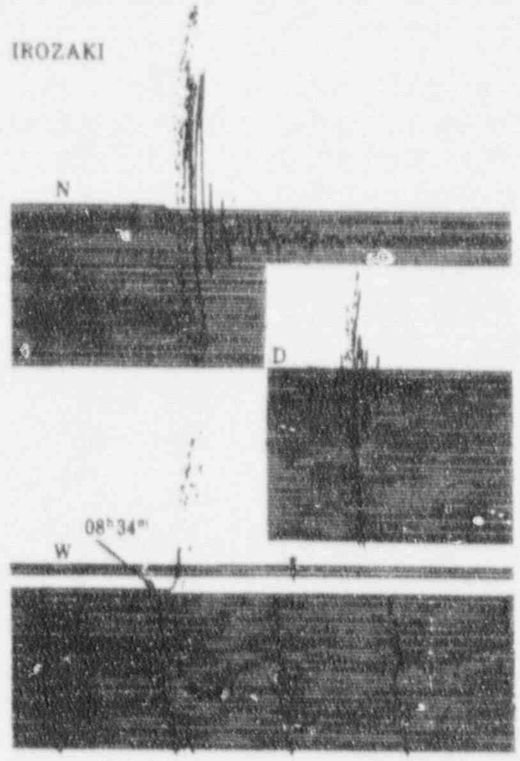


Figure 4. Seismographic records at Irozaki.

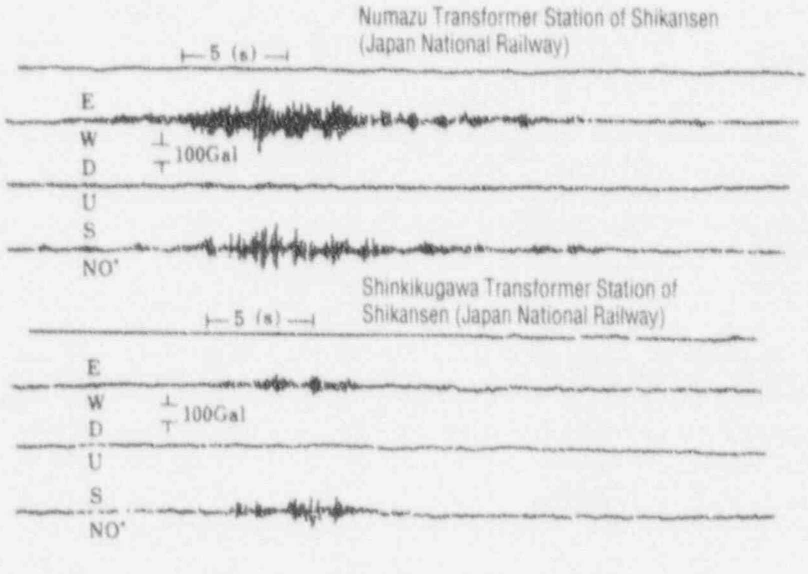


Figure 5. Records of seismograph.

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No. 2 Earthquake name: Central Ohita-ken Earthquake in 1975

Time of earthquake: 2:35'51.0", April 21 (Japan standard time)

Epicenter: 131°20 E, 33°08'N

Magnitude: 6.4* Depth of focus: 0 km

Summary of the earthquake [1,2]

Kyushu is a region where major earthquakes are less frequent as compared to other regions in Japan. According to the publication of the Ministry of Construction (July 25, 1952), for all of the nine prefectures [Shu] and 7 counties [ken] including Yamaguchi-ken, the horizontal seismic coefficient is 0.8 times the standard value. In particular, major earthquakes are rare in Ohita-ken and Fukuoka-ken. For example, among the earthquakes over a certain magnitude described in "Annual of Natural Sciences" (1973 edition), there are only two cases within Ohita-ken: one in the year of 1596 (33.3°N, 131.7°E, M = 6.9), another in the year of 1597 (33.3°N, 131.7°E, M = 6.4). In several cases, this region is affected by earthquakes with foci outside the county. The foci of these earthquakes are in Ehime-ken near Hyuga-nada and Bungo Channel; the magnitudes are over 7.

The title earthquake had its focus in the volcano region of Kuju, Aso, etc. Typical examples of recent earthquakes due to this focus include the earthquake on January 22, 1975 (M = 5.5), and the earthquake on January 13, 1975 (M = 6.1), with focus near Mt. Aso. The title earthquake had the highest magnitude among the earthquakes having foci in this volcano region.

The epicenter of this earthquake is near Yufuinmachi, Ohita-gun, Ohita-ken. Serious damage took place at a portion of Yufuinmachi and Shonaimachi of Ohita-gun, Kuju-cho of Kusu-gun, and Naoirimachi of Naoiri-gun in Ohita-ken. The region with damage was limited. The regions with particularly large damage were located in the WNW-ESE direction. Certain houses were totally destroyed in Teratoko and Okunameshi of Kuju-machi, Uchiyama of Shonai-machi, Shiote of Naoiri-machi, etc. The region where houses were half-damaged is an elliptical region with a major diameter of 28 km and a minor diameter of 12 km, with the major diameter in the WNW-ESE direction. In the region outside the aforementioned region, the degree of damage to houses and the falling rate of tombstones decreased rapidly. Landslides, falling rocks, ground cracks, and road cracks took place at many sites.

The seismic motion was in a short impact form, with the transverse vibration lasting for a short time. Also, there was no verified example of fault displacement in the ground surface layer, and there was no report of fire after the earthquake.

Death: nobody; totally destroyed houses: 76.

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*All the data were published by the Meteorological Agency.

Seismic intensity distribution	According to [1]
Maximum seismic intensity: IV at Ohita, Aso	

Distribution of seismic intensity

Figure 1 shows the distribution of seismic intensity in a wide range as published by the Meteorological Station of Fukuoka District.

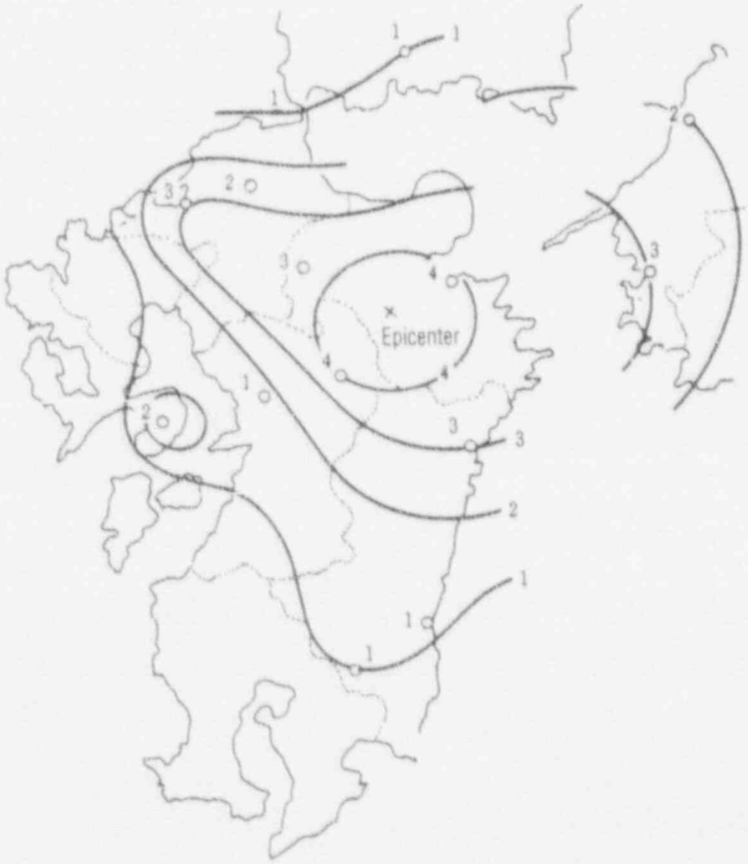

Figure 1. Distribution of seismic intensity in a wide range. Data courtesy of Meteorological Station of Fukuoka District.

Fault parameter	According to [3]	
Fault length: 10 km	Width: 20 km	Depth: 0 km
Dip angle: 75°SW	Dip direction: N50°W	
Mo: —	Fault displacement: 0.1 m (left transverse slip), 0.3 m (normal fault)	
Slip type: —	Rise time: —	
Rupture velocity:	Stress drop: —	

<u>Fault position</u>

The fault displacement of the ground surface due to the earthquake was not determined. Figure 2 shows the focal position together with the known distribution of active faults near the focus.

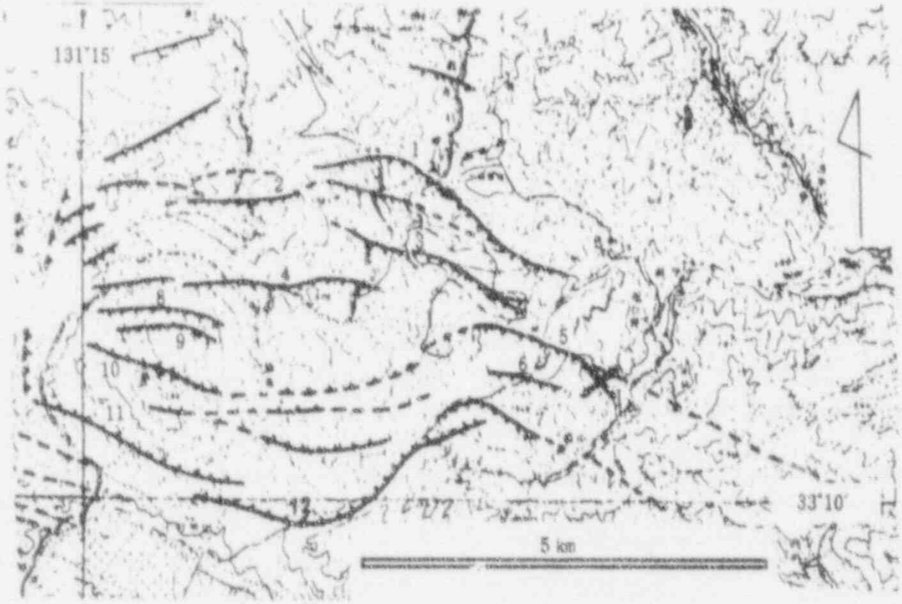

Figure 2. Focus and distribution of active faults near the focus. The fault displacement at the ground surface due to this earthquake is not determined.

Features of damage

The damage caused by this earthquake included damage and falling of buildings, ruptures of roads, retaining walls, bridges, etc., landslide, damage to railway, etc. This damage was limited to Shonai-machi, Kuju-machi, Yufuin-machi, and Naoiri-machi. In addition, the major damage region was relatively narrow. A prominent feature of the earthquake damage was that the falling and damage (complete damage) of the wood houses were relatively less serious.

On the other hand, damage of high-stiffness objects, such as retaining walls, roads, old stone treasure houses, etc., was significant, such as the reinforced concrete buildings at Yufuin-machi which were seriously damaged. These are believed to be the features of the damage caused in hilly regions by medium-level inland-type shallow earthquakes. In this respect, for the Imaichi Earthquake in 1949, the Mt. Hidaka Earthquake in 1970, the Izu Peninsular Offing Earthquake in 1974, etc., although they differ from each other with respect to the damage status, they all have the common feature that for the seriously damaged region, the acceleration was extremely large, and structures with high stiffness could be damaged by the amplitude of this acceleration. On the other hand, for the wood houses, a large strain was left, while the damage of roof tiles was rather significant.

Summary of damage

Table 1 lists the damage to humans and objects as surveyed by Disaster Treatment Headquarters of Ohita-ken at 4 p.m., April 24. As pointed out above, in this earthquake, the damage was concentrated in Shonai-machi, Kuju-machi, Yujuin-machi, and Naoiri-machi. The seriously damaged regions were further restricted. It is necessary to present the detailed damage states by further dividing these regions.

- Damage to buildings

The seriously damaged regions included Uchiyama, Naono, and Asono regions of Shonai-machi, Okue and Yuhira regions of Yufuin-machi; Terasho region of Kuju-machi; Muda and Okufutaishi regions of Chi-machi; and Shiote region of Naoiri-machi.

Buildings that were seriously damaged include Kuju Lakeside Hotel, Yamanami Highway Fee Collector Station, and sports warehouse of Terasho Division of Tachinoya Primary School of Kujimachi. In addition, damage also took place at Odanoike Rest Home, Kariba Pavilion, Yamashimo Lakeside Village, Yamashimo Clubhouse of Kuju Country Club, etc.

Since Kuju Lakeside Hotel is located near the focus, it was seriously damaged with a portion of the guest rooms collapsed. However, there were few injured persons. The acceleration can be estimated according to the state of objects falling in the hotel: for seat stands [sic] (for 6 seat stands, 5 fell, 1 leaned): 0.44 G; for clubhouse lockers (about 1/3 of them fell): 0.29 G. On the other hand, screens (0.36 G) and vertical ashtrays

(0.37 G, 0.82 G) did not fall. Judging from the deformation of the hotel and its nearby buildings, it is believed that a seismic acceleration hit here in the N10°W-S10°E direction.

- Damage to roads

According to a survey by Ohita Engineering Division, Kyushu Region Construction Bureau, Ministry of Construction, and Second Repair Division, Facility Maintenance Department, Japan Road Corp., there was almost no damage to the National Highways No. 10 (Daibu-shi-Nakatsu-shi) and No. 210 (Yufuin-machi-Hita-shi), while for the 18.1 km section between Yufuin-machi and Kuji-machi of the Beppu Aso Highway (Japan Road Corp.), there were 3 sites of road collapse (7,500 m^3), 2 sites of collapse of cutting slope (280 m^3) and cracks of road (numerous).

- Damage to highway bridges

With slightly damaged bridges excluded, there were four highway bridges damaged: Kono Bridge, Asahi Bridge, Ogiyama Bridge, and an unnamed bridge in the southern part of Ogiyama region.

Table 1. Damage status.

Harm to humans	Seriously injured	3 persons
	Slightly wounded	19 persons
Damage to houses	Completely damaged	58 houses
		268 persons
		56 subsections
	Half damaged	93 houses
		387 persons
		91 subsections
	Partially damaged	2,089 sections
		7,938 persons
		1,980 subsections
Nonresidence buildings (stone treasure houses, warehouses, storage houses)		Completely destroyed: 30 buildings
		Half destroyed: 68 buildings
Damage to education facilities of schools, etc.		Completely damaged: 1 school
		Half damaged: 13 schools
		Others: 22 cases
Report on simple water channels		5 facilities
Damage to rivers		6 sites
Damage to roads		182 sites
Damage to bridges		3 sites
Damage to farmland		218 ha, 1,366 sites
Damage to forests (landslides, land slip)		94 sites
Damage to railway facilities		28 sites
Damage to communication facilities		2 sites
Total amount of financial loss due to damage		¥2,935,000,000

(Note): According to survey by Disaster Treatment Headquarters of Daibu-ken as of the afternoon of April 24, 1975.

Earthquake records	According to [6]

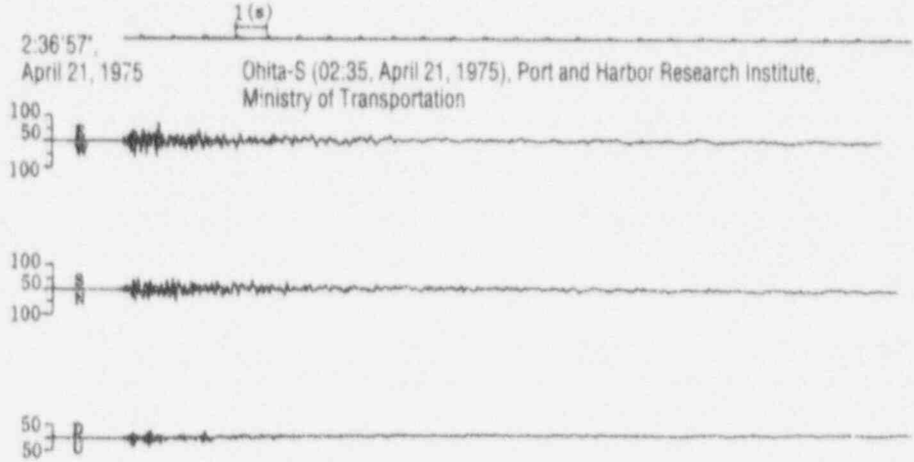

Figure 3. SMAC records recorded at Beppu Port. (Characteristics period: 0.14 sec, sensitivity 12.5 Gal/mm, braking critical damping, distance to epicenter: 30 km).

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No. 3 Earthquake name: Tottori Earthquake in 1983

Origin time: 1:51 a.m., October 31, 1983

Epicenter: 133°57'E, 35°26'N

Magnitude: 6.3* Depth of focus: 10 km

Summary of earthquake [1,2]

In this earthquake, only slight damage took place in a limited region. However, for the city hall of Kurayoshi-shi located 10 km from the epicenter, significant damage took place due to eccentricity of the building.

It is believed that this earthquake took place in the interior of the aftershock region of the Tottori Earthquake of September 1943, with the fault direction in the conjugate direction that is nearly orthogonal to the Yoshioka/Kano fault. Based on the survey questionnaire for the local residents, it was found that the region with high seismic intensity is distributed along the peripheral active fault.

It was found that the amplitude was high along the E-W direction near the epicenter and along the S-N direction in the region separated from the focus by a certain distance.

*All data were published by Meteorological Agency.

Seismic intensity distribution in various regions	According to [2, 6]
Maximum seismic intensity: 4 at Tottori	

Distribution of seismic intensity

Figure 1. Seismic intensity distribution of Tottori Earthquake in 1983.

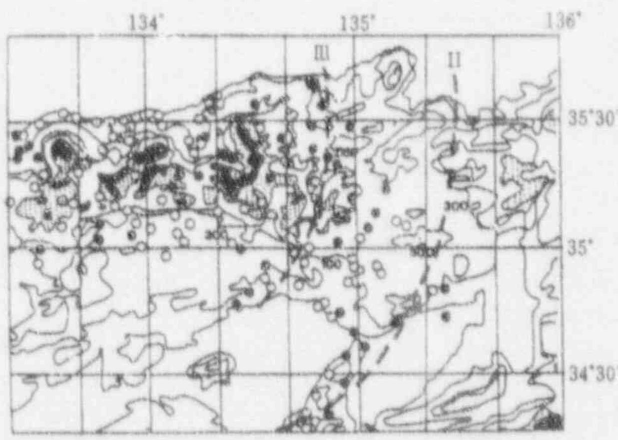


Figure 2. Distribution of seismic intensity and topography. Solid circle indicates I ≥ 3 and open circle indicates I > 2. Contours indicate 100, 300, 500, 700 and 900 m. (Seismic intensity distribution of questionnaire survey)

(Note): The seismic intensity is larger for the nearby Yamasaki fault and other active faults as well as the basic periphery.

Fault parameters	According to [1, 4, 5]	
Fault length: — km	Width: — km	Depth: — km
Dip angle: Almost vertical	Dip direction: About N60°E	
Mo:	Fault displacement:	
Slip type: Strike slip	Rise time: —	
Rupture velocity: —	Stress drop: —	

Source mechanism

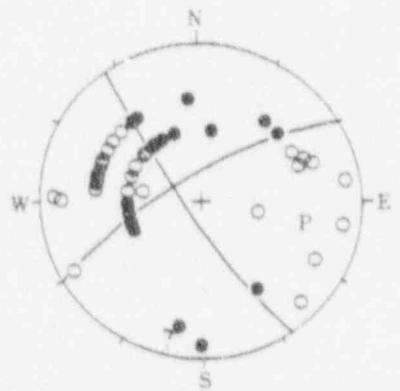


Figure 3. Projection of upper hemisphere.

Aftershock distribution

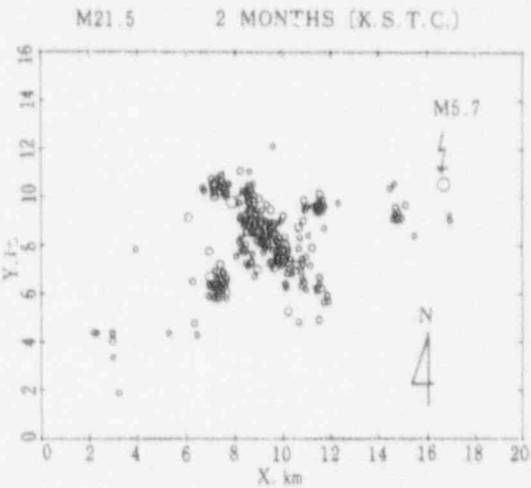


Figure 4. Magnitude over 1.5.

Fault position

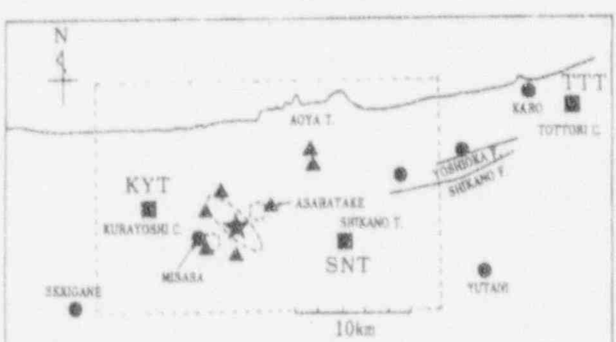


Figure 5. Locations of observation points near aftershock region. ■: Constant observation point of minute tremors; ▲: Temporary observation point of minute tremors; ●: Spring and underground observation point.

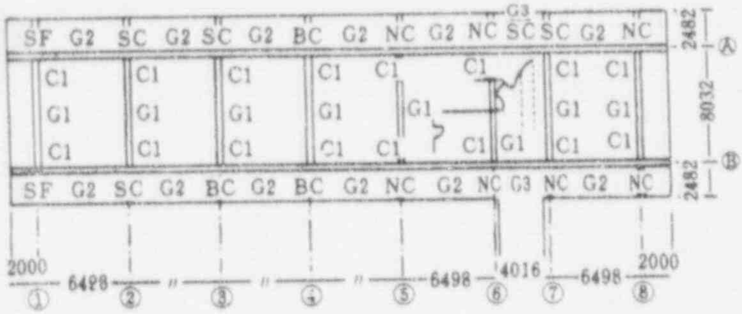
Earthquake damage	Due to [6]

Features of damage

Sight damage in limited regions were reported.

Typical damage

Significant damage took place for the secondary floor of the eastern portion of Kurayoshi City Hall, a 3-story RC building located less than 10 km from the epicenter.



Plane view of secondary of the eastern portion of City Hall and damage to the various columns (SF: shear failure; EC: bending cracks; SC: shear cracks; NC, no crack)

Figure 6. Damage state of the eastern portion of Kurayoshi-shi City Hall.

Damage took place due to torsional vibration caused by eccentricity. Judging from the state of falling of nearby tombstones, it is believed that the vibration in the E-W direction was dominant, and that this component acted in the shorter edge direction, which is one of the reasons for the damage. For the passage in the second floor from the road, the south end was in a fixed form, while the north end was deformed to one side. At any rate, it was effective to evaluate the damage from the defects of the building.

Earthquake phenomenon	According to [6, 7]

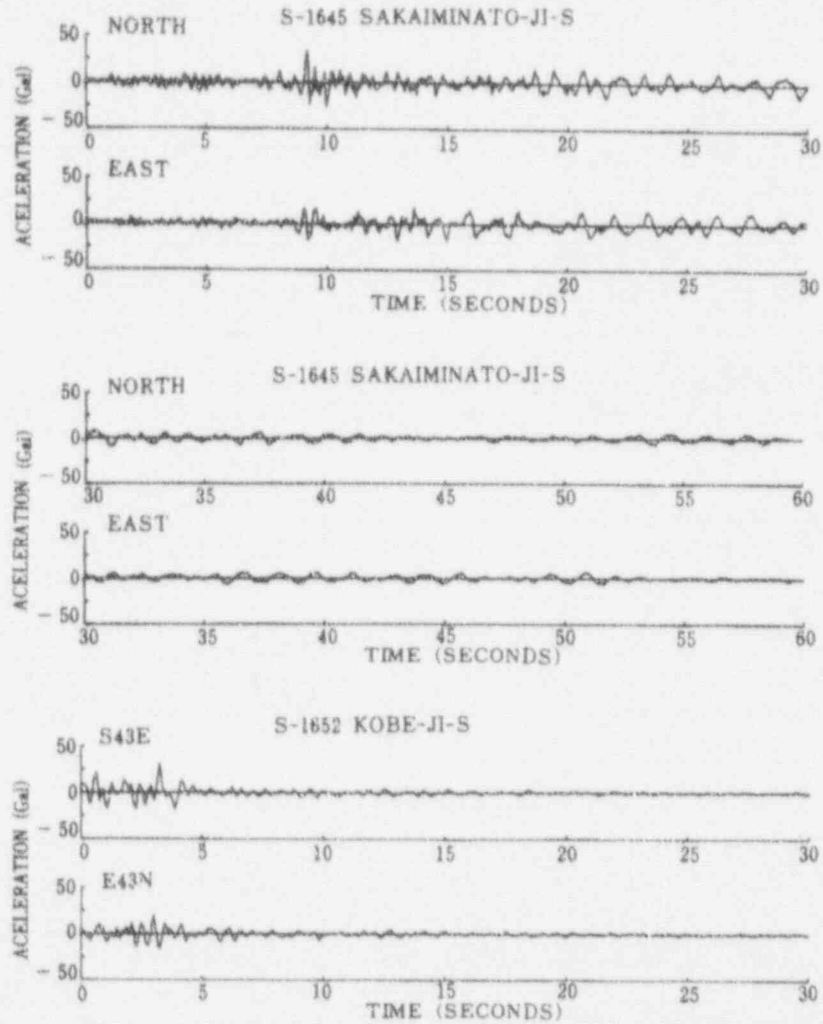


Figure 7. Acceleration records at Sakaiminato and Kobe (by Port Technical Research Institute).

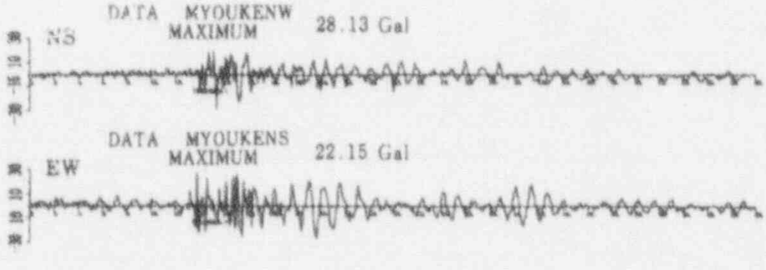


Figure 8. Acceleration records of Myouken Bridge (Civil Research Institute).

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No. 4 Earthquake name: Western Nagano-ken Earthquake in 1984

Time of tremor: 8:48'49.4", September 14, 1984

Epicenter: 137°33.6'E, 35°49.3'N

Magnitude: 6.8 Depth of focus: At rather shallow place within 10 km

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Summary of earthquake [1,2]

At 8:48, September 14, 1984, a strong earthquake with a magnitude of 6.8 and with the focus at southeast region of Mt. Mitake, Otaki-mura, Kiso-gun Nagano-ken took place. Its felt tremor region was as wide as reaching Fukushima-ken in the Tohoku Region as well as Tottori-ken, Okayama-ken, etc., of the Chugoku Region. Since the focus of this earthquake was shallow and it took place in a hilly region, a large-scale landslide of Mt. Mitake as well as slope collapse and avalanche of sand and stone took place. 29 persons were killed, and serious damage was caused to rivers, roads, bridges, forest, homes, etc.

Many aftershocks took place after the main earthquake. The largest aftershock with a magnitude of 6.2 took place at 7:14, September 15, 1984, at the west end of the aftershock region of the earthquake. Its aftershocks also formed an aftershock region, with a characteristic distribution similar to the aftershock region of the main earthquake.

Seismic intensity distribution in different places	According to [1]
Maximum seismic intensity: 4 at Kofu, Iida, Maizuru, Suwa	

Seismic intensity distribution

The seismic intensity at different places are as follows:

Seismic intensity 4 (Intermediate tremor): Kofu, Iita, Maizuru, Suwa

Seismic intensity 3 (Weak tremor): Tokyo, Yokohama, Maebashi, Shizuoka, Fukui, Kyoto, Toyama, Nagano, Tsu, Wajima, Hamamatsu, Irago, Kumagaya, Toyooka, Nagoya, Osaka, Gifu, Matsumoto, Safukiyama, Takayama, Nara, Omaezaki, Mishima, Yokkaichi, Hikone

Seismic intensity 2 (Light tremor): Oshima, Tottori, Kanazawa, Chiba, Okayama, Tateyama, Utsunomiya, Takada, Tsuruga, Karuizawa, Chichibu, Kawaguchiko, Fushiki

Seismic intensity 1 (Minute tremor): Mito, Niigata, Onahama, Ueno, Saigo, Owase, Aikawa, Wakayama, Ajiro, Matsushiro, Yonago, Kobe, Irozaki

The felt-tremor region is rather wide.

Figure 1. Seismic coefficient distribution published by Meteorological Agency.

Fault parameters	According to [1]	
Fault length: 12 km	Width: 8 km	Depth: —
Dip angle: —	Dip direction: About N70°E	
Mo: 2.9×10^{25} dyne·cm	Fault displacement: 1.0 m	
Slip form: Right strike slip	Rise time: —	
Rupture velocity:	Stress drop: 32 bar	

As the mechanism of the main shock, the right strike slip type fault with direction of N70°E was derived from the initial motion push-pull distribution diagram. In addition, for the maximum aftershock on September 15, a left slip fault in direction N20°W and conjugate to the fault of the main shock was derived. Judging from the range of aftershock distribution during one day after the principal tremor, the fault is believed to have a length of 12 km and a width of 8 km. The average displacement of the fault in a range of about ±20% is estimated to be about 1 m. The seismic moment is derived as about 2.9×10^{25} dyne·cm, and the static stress drop amount is derived as about 32 bar.

Earthquake 'amage	According to [1]

The earthquake that took place in the western part of Nagano-ken was a shallow-focus inlan' type earthquake in hilly region. The damage was characterized by the feature that the focal region had large-scale slope topography and that the geography and soil had volcanic properties. Figure 2 shows the dist... n of damage in the region. Since Otaki-mura, Kiso-gun, Nagano-ken is located at the epicenter, it was nit n ...est in the earthquake. Also, various damage took place in the surrounding villages and cities in Nagano-ken and Gifu-ken. As far as the degree of damage is concerned, the degree was rather low except Otaki-mura, which was hit hard with serious damage.

The features included large-scale avalanche of sand and stone and slope collapse at the Mt. Mitake Geography. The slope collapse caused by topography, geology, and soil conditions led to harm to humans, as well as damage to structures, roads, tunnels, and bridges.

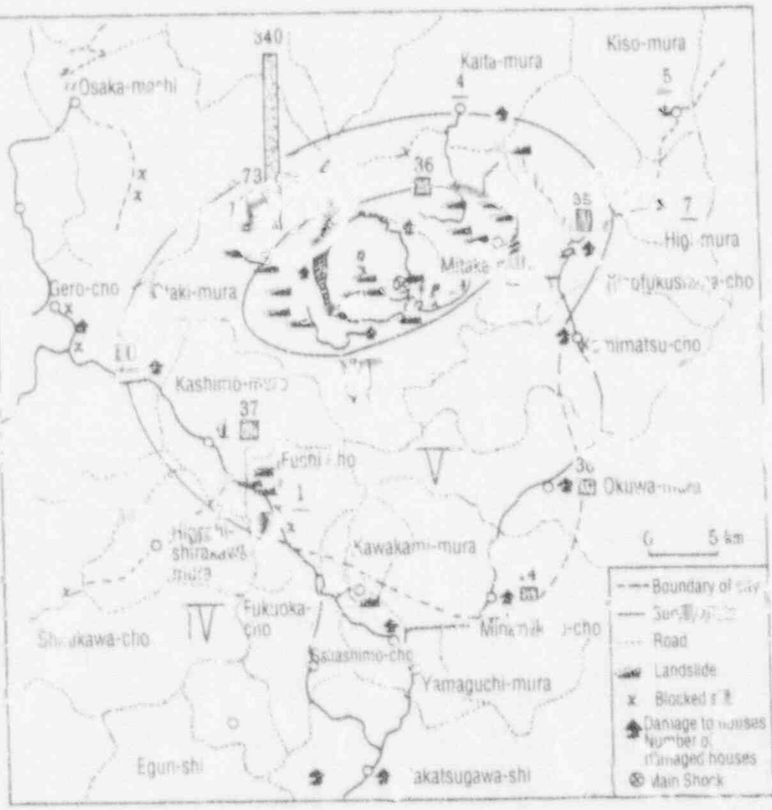

Figure 2. Distribution of seismic intensity and damage status in western part of Nagano-ken and eastern part of Gifu-ken.

Major damage: During the principal tremor, a large-scale avalanche of sand and stone took place on the south side of Mt. Mitake. The large amount of sand and stone flew into the Otakigawa River to form a dam and partially blocked it. In addition, the roads were buried and the forest was damaged. In addition, slope collapse also took place at Takikoe, Matsukoe, Kiyotaki, and Mitake Plateau, with damage caused by the sand and stone.

Harm to humans: 29 persons were killed by avalanche of sand and stone and slope collapse. Also, 4 persons were seriously wounded, many others were slightly wounded.

Civil structures: Damage to roads, bridges, tunnels, etc., was mainly caused by the damage to retaining walls due to collapse of sand and stone of the slope topography. As sand and stone slid, the roads and tunnels were buried, and the bridges were hit and damaged.

Wood houses: Large-scale damage, including complete destruction and half destruction of houses, was limited to Otaki-mura. It was all caused by avalanche of sand and stone. The partial-damage region is in Nagano-ken around Otaki-mura, with such damage phenomena as roof tile, furniture falling, and cracks of window glass.

The reinforced concrete structures and steel-frame structures, such as the large buildings of schools and social activity centers, were not completely damaged. The features in the damage include destruction of the expansion joints in the reinforced concrete structure and buckling of the braces of the steel-frame structures.

Damage to retaining walls: Since the topography is charaterized by slopes, retaining walls are arranged to maintain the step differences for roads, houses, and farm land. As these retaining walls were damaged, the roads were blocked, the houses settled unevenly and became inclined. Damage was particularly significant for the portion using filled soil retaining walls.

Buildings' internal space, buildings' periphery: Although the buildings/houses themselves were not harmed, damage nevertheless took place to falling, shifting, dropping of various equipment set in the internal/external regions of the living space. In particular, for furniture, equipment, propane gas tanks, petroleum storage tanks, and screens, as they were set up without considering earthquakes, damage took place in many cases.

Earthquake phenomenon	According to [1]

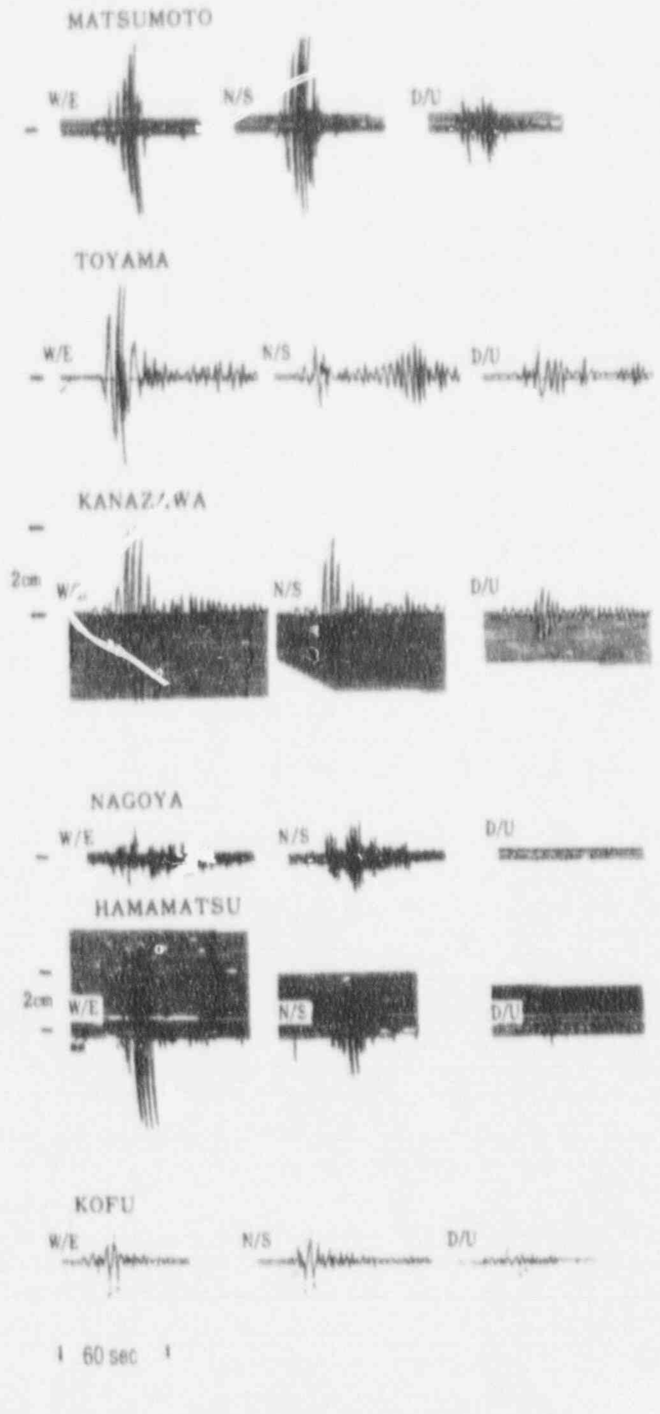

Figure 3. Records of JMA 1-fold seismograph at various observation points (courtesy Meteorological Agency).

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No. 5 Earthquake name: Northern Italy (Friuli) Earthquake in 1976

Time of tremor: 20:00'11.6", May 6, 1976 (UTC)

Epicenter: 13.26°E, 46.35°N

Magnitude: 6.5 (Ms) Depth of focus: 9 km

Summary of earthquake[1,2]

968 persons were killed (on June 18). The catastrophic region runs 80 km in the E-W direction and 70 km in the S-N direction. In particular, with center at Gemona on the Tagiamento River, more than 50% of the houses were destroyed in an area running 45 km in the E-W direction and 20 km in the S-N direction.

According to the Italian government, the value damaged was about 1200 billion lire (¥440 billion).

Since this region used to have few earthquakes, many buildings were constructed without seismic consideration, and were thus seriously damaged. On the other hand, since the ground is good, damage to civil structures was less serious. Several seismic faults running in the N20°E direction have been confirmed. Damage and soil settlement from tops of precipices were significant. Also, at Gemona, liquefaction was observed in certain portions.

Note

The time of tremor is published in NEIS.

Seismic intensity distribution in different places	According to [1]
Maximum seismic intensity: X (modified Mercalli scale) at Gemona	

Seismic intensity map

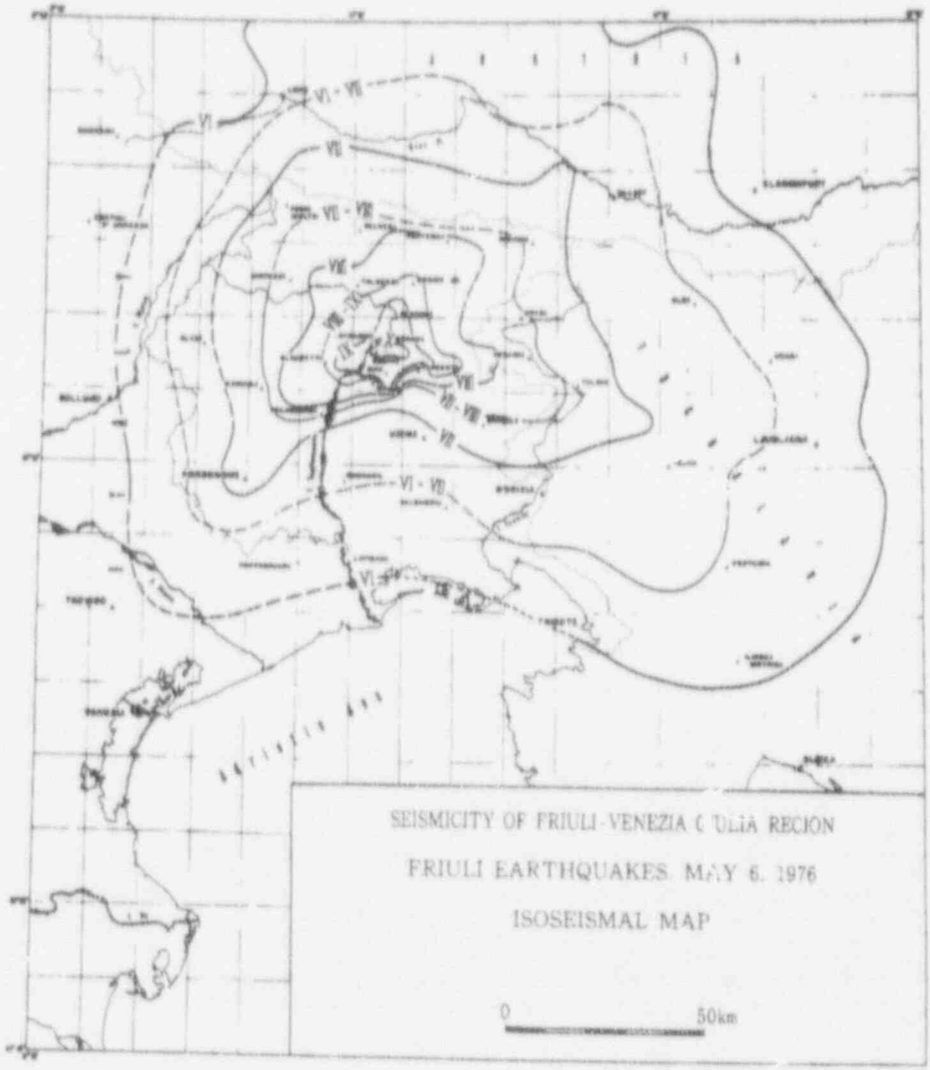
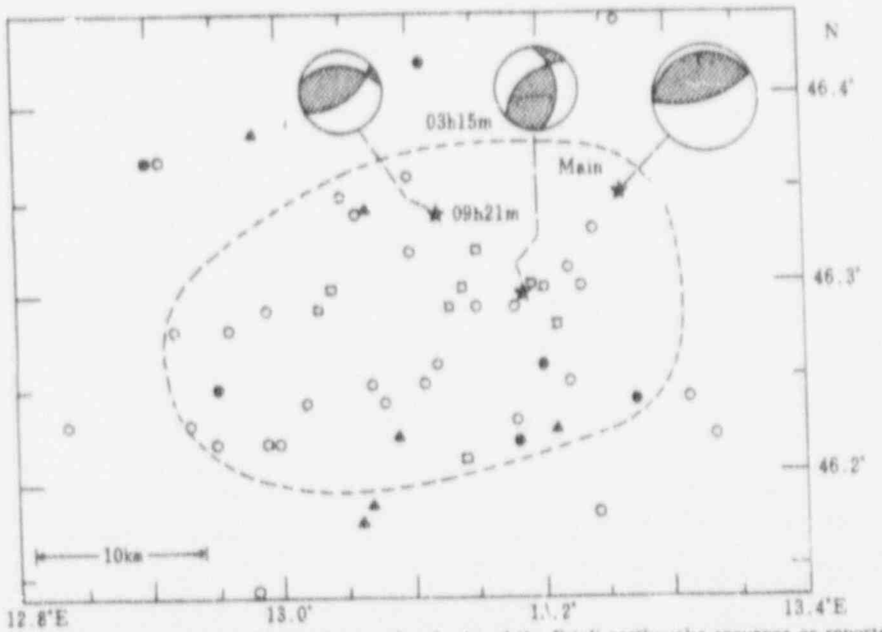


Figure 1. Seismic intensity (MM scale) map of Friuli Earthquake on May 5, 1976.

Fault parameters	According to [1, 3]	
Fault length: 30 km	Width: 15 km	Depth: 8 km
Dip angle: 75°S	Dip direction: N14°W	
Mo: 3×10^{25} dyne·cm	Fault displacement: 0.33 m	
Slip type: Thrust fault	Rise time: 1.0 s	
Rupture velocity: 2.9 km/s	Stress drop: 12 bar	

<u>Fault position</u>



Epicenters of the main shock and larger aftershocks of the Friuli earthquake sequence as reported by the ISC. Large star indicates the main shock. Closed circles represent aftershocks which occurred within the first 24 hr. and open circles are later aftershocks up to September 11. Smaller stars are epicenters of the two largest earthquakes on September 15. Open squares are aftershocks of these events. Focal mechanisms are determined in this paper with compressional quadrants shaded. Heavy dashed line denotes the approximate limit of aftershock concentration. Triangles are accelerograph stations.

Figure 2. Source mechanism of main shock and aftershocks.

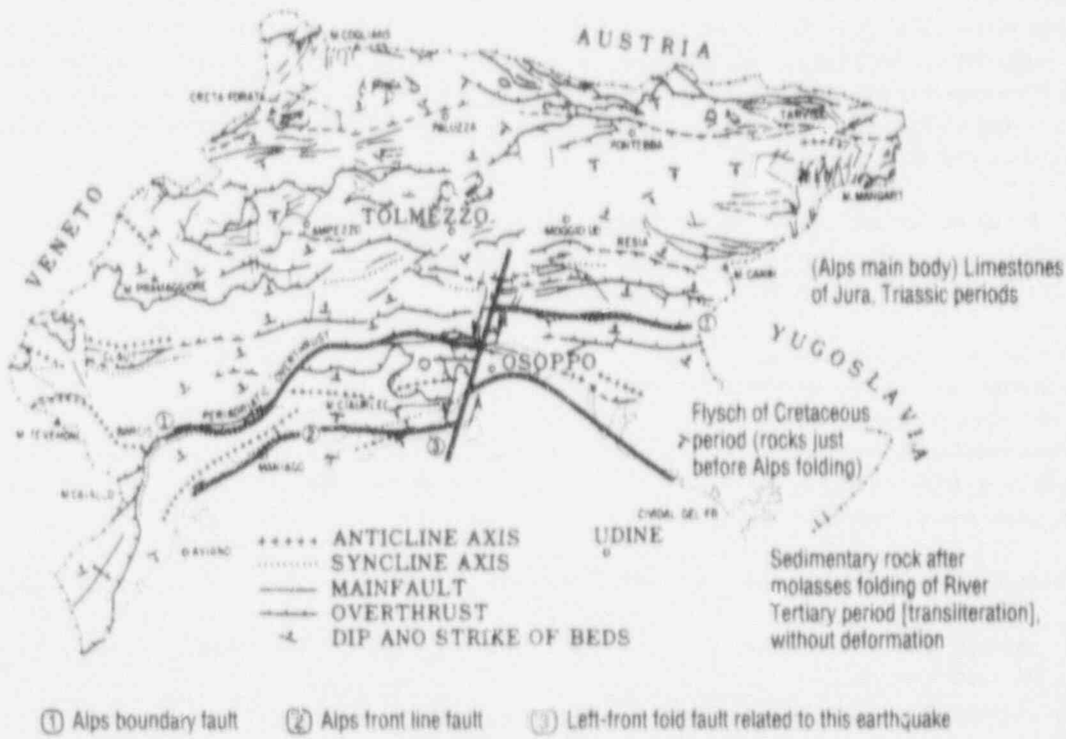


Figure 3. Tectonics near the catastrophic region.

Earthquake damage According to [1]

<u>Features of damage</u>

The 3 regions of Venzone, Gemona and Osoppo were hit seriously by the earthquake. Almost all of the ancient buildings in these areas were destroyed. Considering the scale of this earthquake, the damage was extraordinarily serious. In addition to these three areas, significant damage to buildings as well as rock collapse and landslide also took place at Buija Majano, S. Daniele, Tarcento, Vedronza, Lusevera, and Moggio/Voline in the north, Flagagna on the west side of Tagliamento, etc. The damage range of the earthquake ran about 80 km in the E-W direction and about 70 km in the S-N direction. In particular, over half of the houses were destroyed in an area running 45 km in the E-W direction and 20 km in the S-N direction. At Venzone and Osoppo, over 90% of the population lost their homes.

As far as the seismic intensity of the earthquake is concerned, the seismic intensity (Mercalli scale) is X for the periphery of Gemona, which stretches in a slender shape from NNE to SSW. The region with a seismic intensity of VII stretches in this direction to the east along the Alps. Damage also took place in this range.

For Gemona, Osoppo, Moggio, etc., which were hit hard by the earthquake, since they are hilly regions having no houses, no serious disaster was caused. However, at Brauline, Carnin north of Venzon, Tarcento, Flagogna, and other regions with large-scale rock collapse, cracks developed in the foundation rock, with large gaps for some cracks and with slips for some others. In particular, in the areas hit hard by the earthquake, it is believed that such cracks were developed on the bed of rock (bedrock), which, together with the existing joints, cracks, and faults, may cause tremors again in the future.

There are two features for the topography with serious harm:

i) Among the alluvial fan land formed by the Tagliamento River, the region along the hills was hit hard. In particular, the disaster was serious for Venzone, Gemona, and Osppo [sic; Osoppo].
ii) Among the residential areas in the hilly regions, those near precipices or on steep hills were hit hard.

The regions with many old buildings made of stones and bricks were hit hardest. If this factor is excluded, the damages are found concentrated at the alluvium adjacent to hills. This is because, in such a region, the thickness of the alluvium is in agreement with the resonance periods of the buildings. In addition, since the alluvium has its two sides surrounded by hills, i.e., hard soils, there is no way for the seismic energy to be released. As a result, the seismic energy is concentrated and causes a large tremor. This idea, however, is yet to be verified.

Distribution of damage

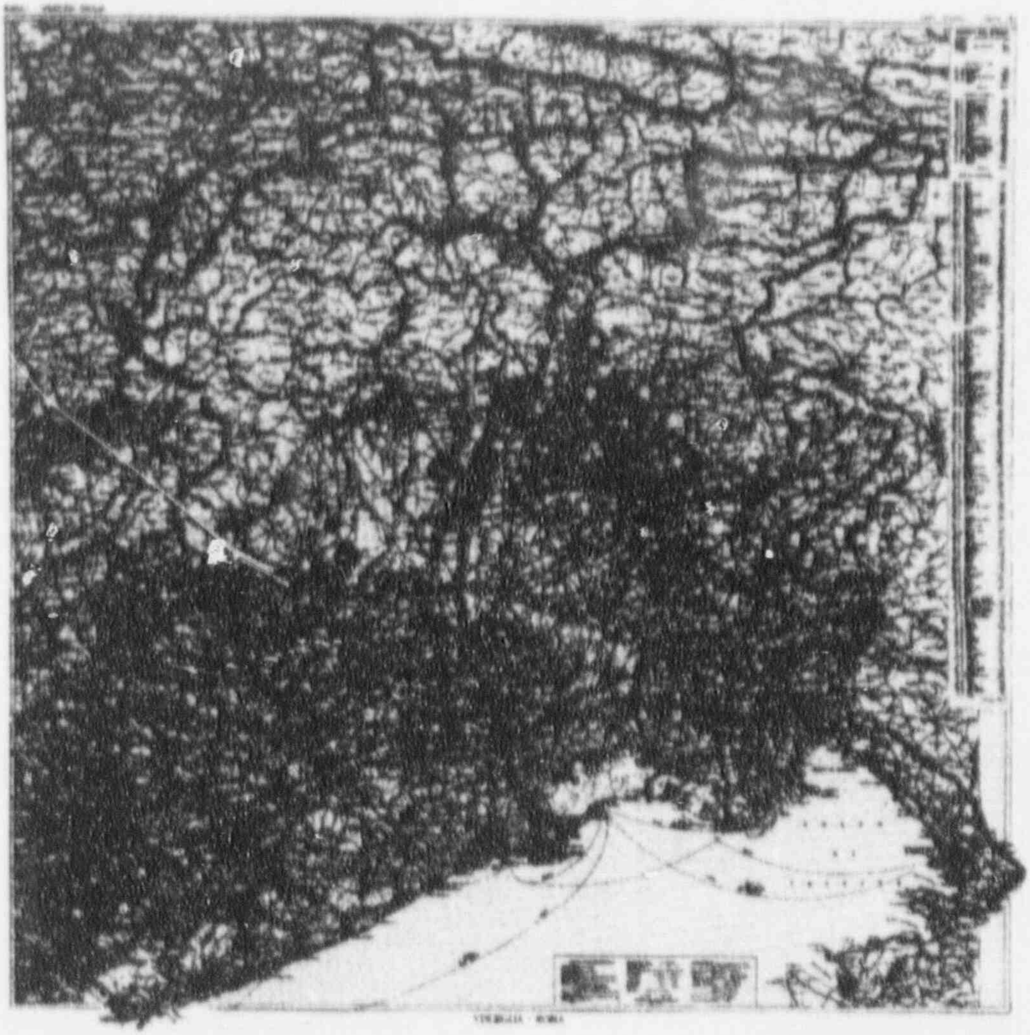


Figure 4. Distribution of damage. The solid dots in the figure represent the total number of buildings in the cities with damaged buildings; the blank circles with the solid dots represent the number of damged buildings.

Typical damage

The types of buildings in the disaster region can be divided as follows: (i) old houses manually made of stones and bricks (mainly of 2 or 3 stories); (ii) new apartment buildings made of bricks by constructors; (iii) RC high-rise apartment/office buildings; (iv) one-story plant buildings made of RC/CP; (v) churches/bell towers, and other special buildings. All of these buildings were it. In particular, damage to (i) and (iv) was significant. For types (ii) and (iv), the damage was more significant for those with thinner walls. In particular, the piloti-type [transliteration] buildings and buildings using hollow bricks were weak to damages.

The catastrophic disasters took place at Majano, where a 6-story apartment building collapsed, causing the death of about 80 persons. Also, a barracks at Gemona collapsed and killed about 60 soldiers.

Figure 5. Apartment building with the same shape as that destroyed (Majana [sic]).

Figure 6. PC/RC plant (plant near Osoppo).

Earthquake records	According to [1]

Typical seismic waveforms

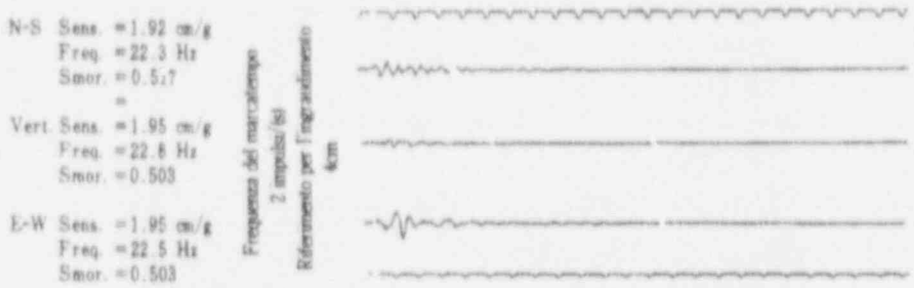


Figure 7. Preshock recorded at Tolmezzo.

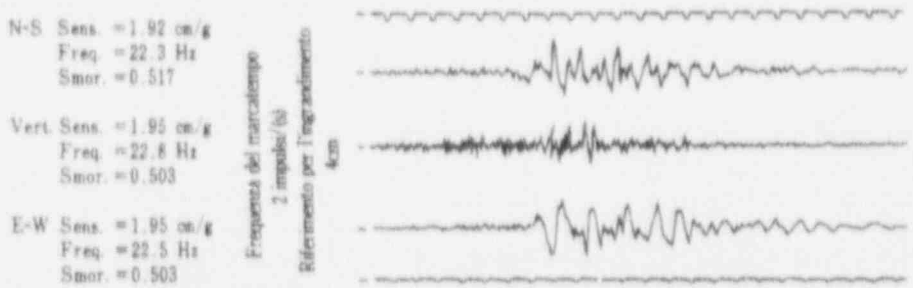


Figure 8. Main shock recorded at Tolmezzo.

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No. 6 Earthquake name: Imperial Valley Earthquake in 1979

Time of tremor: 23:16'54.29", October 15, 1979 (Greenwich standard time [1])

Epicenter: 115.33°W [2], 32.63°N

Local magnitude: 6.6 [1] Depth of focus: 10 km [3]

Summary of earthquake [4,5]

As shown in Figure 1 [5], a transform fault runs through the Imperial Valley in Southern California. This transform fault is composed of the San Andreas fault, transform fault, and the Cerro Prieto fault at the boundary between the North American plate and the Pacific Ocean plate. As a result, earthquakes including the Imperial Valley Earthquake (El Centro Earthquake), take place frequently and repeatedly. This region has thus attracted many seismologists, geologists, and engineers for topographical survey with a purpose to clarify the mechanism of the earthquakes.

This earthquake, the largest one in recent years on the Imperial fault near the border between USA and Mexico, on October 15, 1979, was monitored by many seismographs and strong-motion seismographs. The obtained data are so abundant that they have never been available to the researchers in the seismological engineering field.

The moment magnitude was 6.5; the epicenter was located in northern Mexico. The range was so wide that even structures at El Centro in California and its vicinity were damaged. There were ground surface motions for the four fault belts.

This earthquake caused a monetary loss of $21.10 million and wounded 73 persons. No deaths were reported in the USA. The small number of wounded persons is due to the fact that the most populated regions are not located in the strongest tremor portion. The two most significant damage cases are as follows: partial collapse of the 6-story Imperial County Service Building made of concrete and complete collapse of a steel frame water tank located south of Brawley.

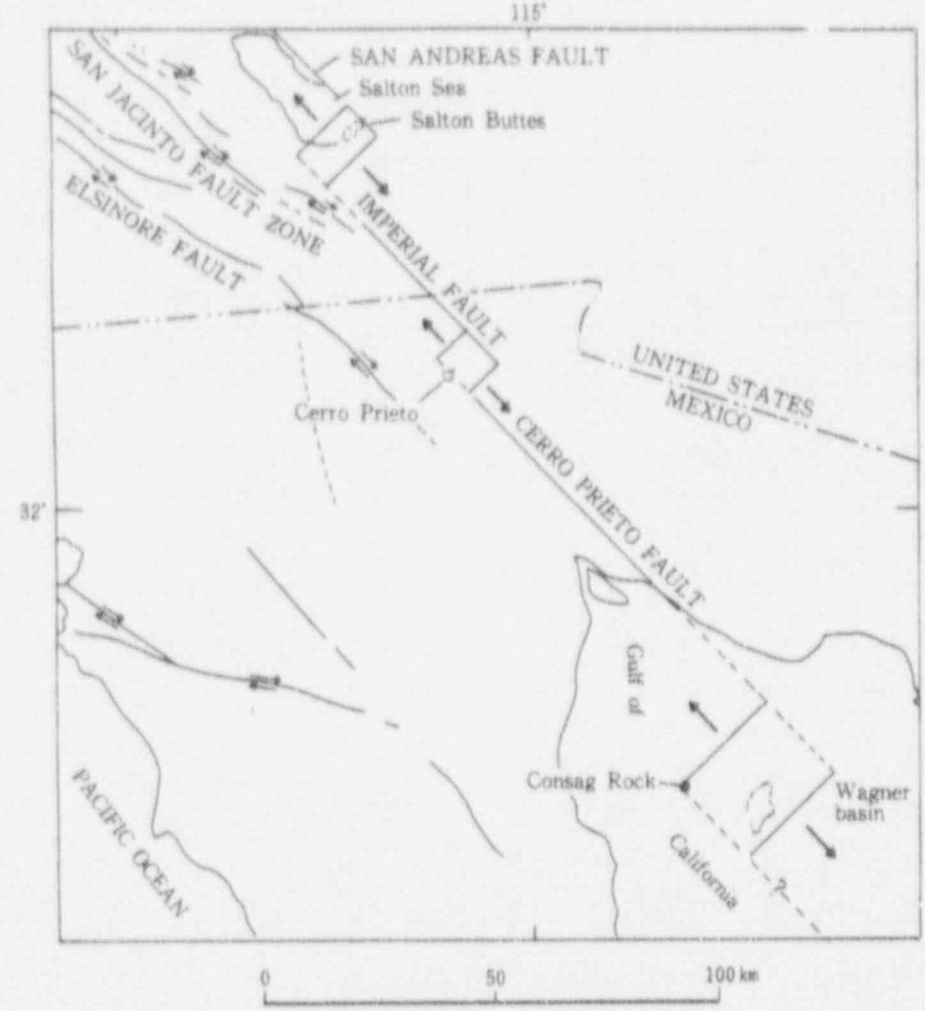

Figure 1. Transform fault.

Fault parameters	According to [3, 6, 7]	
Fault length: 35 km	Width: 10 km	Depth: 10 km
Dip angle: 90°	Dip direction: S53°W	
Mo: 7.0×10^{25} dyne·cm	Fault displacement: 67 cm (Note)	
Slip type: Right transverse slip	Rise time: —	
Rupture velocity: 2.5 km/s	Stress drop: 5–10 bar	

Fault position

Based on the well-surveyed topography at the Imperial Valley area, it has been found that this earthquake was caused by the motions of the Imperial fault and Brawley fault.

Figure 2. Focus and active faults.

(Note): According to $Do = Mo/\mu LW = 7.0 \times 10^{25}/3 \times 10^{11} \cdot 10 \times 10^{5} \cdot 35 \times 10^{5}$.

Features of damage

The focus of this earthquake was located in Mexico. However, the fault generated on the ground surface runs through American cities, such as El Centro, Brawley, Imperial, Holtville, and Calexico. The material damage is concentrated in the region within several km from the fault. Since the population density is low in this area, nobody was killed. (However, 9 persons were killed in a similar earthquake at nearly the same place in 1940.)

- Damage to buildings

The most serious damage in this earthquake took place at the 6-story Imperial County Service Building. The piloti columns at the east end portion of the first floor of this building were crushed and the building was damaged. Except for this building, there is no building higher than three stories in this region. Damage to other buildings was mostly nonstructural damage, such as broken window glass, etc. Weakly anchored mobile homes received a certain amount of damage.

According to Reference [7], the maximum acceleration of the seismic motion in the vicinity of Imperial County Service Building was 0.27 G for the vertical motion and 0.24 G for the horizontal motion in 2° and 92° directions as marked clockwise from the north direction.

- Damage to tanks

Three of the ten high-rise water tanks in this region were damaged by the earthquake. The most serious damage took place in the form of collapse of a 100,000-gallon (378.5 kL) tank located several km from the fault. This tank was designed to withstand a horizontal seismic acceleration of 0.1 G. In addition, it was reported that the brace parts and horizontal parts were damaged for a 100,000-gallon tank on the side opposite to array No. 9.

Most of the ground tanks are used for storage of oil. For some of them oil leakage took place due to sloshing; tank buckling and mounting pipe damage also took place. In addition, one corn silo at Holtville fell and was damaged.

- Damage to roads and bridges

Almost all of the damage to roads took place at locations where the fault trace traverse through them. However, the fault caused by the earthquake measures only up to 50 cm in the horizontal direction and up to 30 cm in the vertical direction. Hence, its influence on traffic was small. For Interstate Highway 8, although transverse slip of 7-1/2" (18-1 cm) took place, the traffic was not interrupted. On Hebver Rd., road surface settled due to liquefaction of the soil near the fault line. On Harris Rd. south of Brawley, 6" (15 cm) vertical slip took place. They were all caused on or near the fault line.

As far as damage to bridges is concerned, on an Interstate Highway 86 [bridge] which crosses New River, abutments made of concrete were either damaged or deformed with a step difference. However, it was not on the extension liner of the fault.

- Damage to lifelines

For the soil-filled portion of the All American Canal, a water channel for irrigation with a length of 10 miles (16 km), landslide and slip over 4 feet (1.2 m) took place.

The power supply in this region is provided by Imperial Irrigation District, which has four sets of power generation facilities. This equipment was designed to withstand a horizontal seismic coefficient of 0.2. However, during the earthquake, a horizontal acceleration over 0.5 G and a vertical acceleration over 0.9 G were recorded at a differential array station located 0.85 km to the southeast. In the power plants, although torsion of steel parts, yield of bolts, etc., were observed for the boiler, equipment and piping system, no serious damage was observed. For the two plants in operation, one recovered operation 1 hour later, the other recovered operation 6 hours later.

Earthquake records	According to [3]

Typical seismic waveform

Figure 3 shows the velocity waveform derived by processing the horizontal acceleration waveforms measured in the direction 230° clockwise from north at El Centro Array. The two velocity waveforms in each group refer to the velocity waveforms at the observation points located almout equidistant from the fault line. The horizontal motion in direction 230° is the seismic motion in the direction orthogonal to the fault line. Judging from the relationship between the focal position and the alley observation position, the component of the seismic wave in this direction is believed to be SH wave.

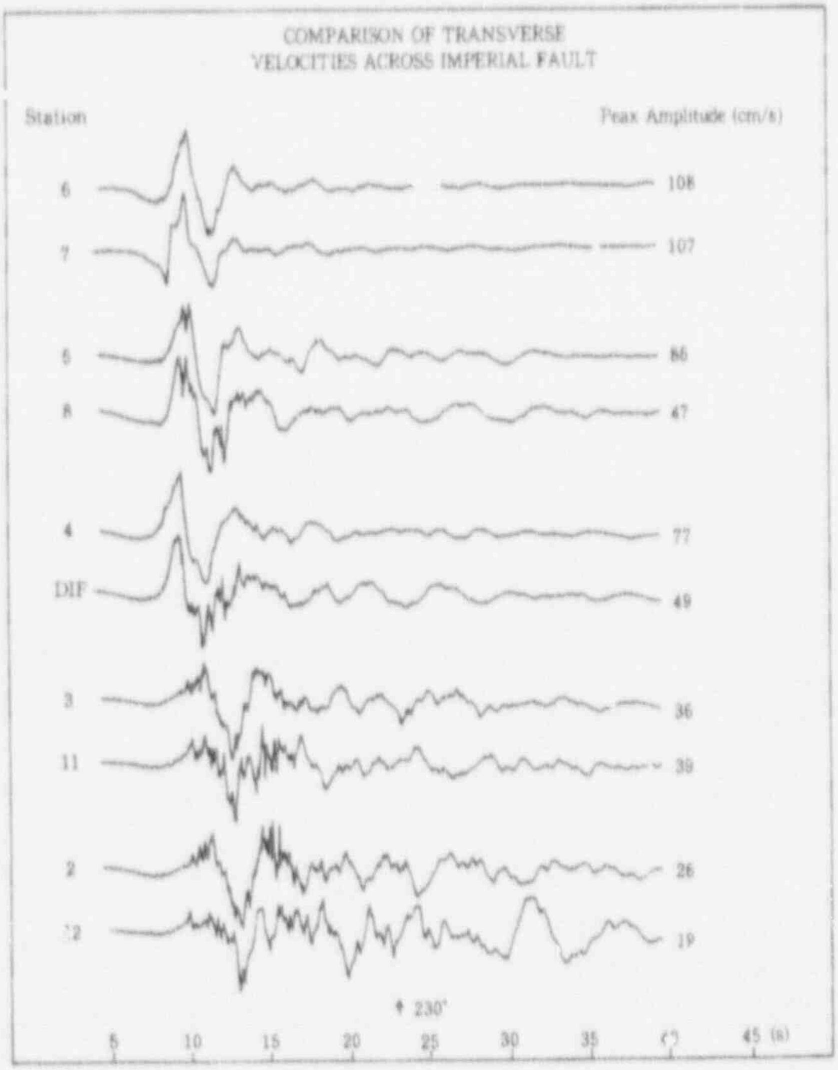

Figure 3. Velocity of waveforms recorded at El Centro (SH velocity waveforms at observation points located equidistant from the fault line).

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No. 7 Earthquake name: Coalinga earthquake in 1983

Time of tremor: 23:42'37.85", May 2, 1983 (Greenwich standard time)

Epicenter: 120°17.5'W, 36°23.99'N

Local magnitude: 6.5 Depth of focus: 10.5 km

Summary of earthquake [1]

At 23:42'37.85", May 2, 1983 (Greenwich standard time), an earthquake with a local magnitude of 6.5 took place. The focus was located at a spot about 30 km northeast of the San Andreas fault and about 10 km northeast of Coalinga at the central portion of California. The city of Coalinga nearest to the epicenter is an "oil city" which has been developed from hay and farmland, with a population of about 7,000 and with many oil rigs for petroleum in its hilly suburbs. Tanks for storage of crude oil have been set up at many sites.

In this earthquake, the old buildings of the main street at the center of Coalinga were completely destroyed. Damage took place to about 150 buildings, including low-story buildings, shops, houses, hospitals, etc. The financial loss is estimated to be at least 33 million dollars.

As an earthquake which took place at a site remote from the San Andreas fault, the earthquake at Coalinga has a rather large scale and has a source mechanism different from the mechanism of the San Andreas fault.

Seismic intensity at various places	According to [3]
Maximum seismic intensity: VIII (modified Mercalli scale) at oil field	

Seismic intensity distribution

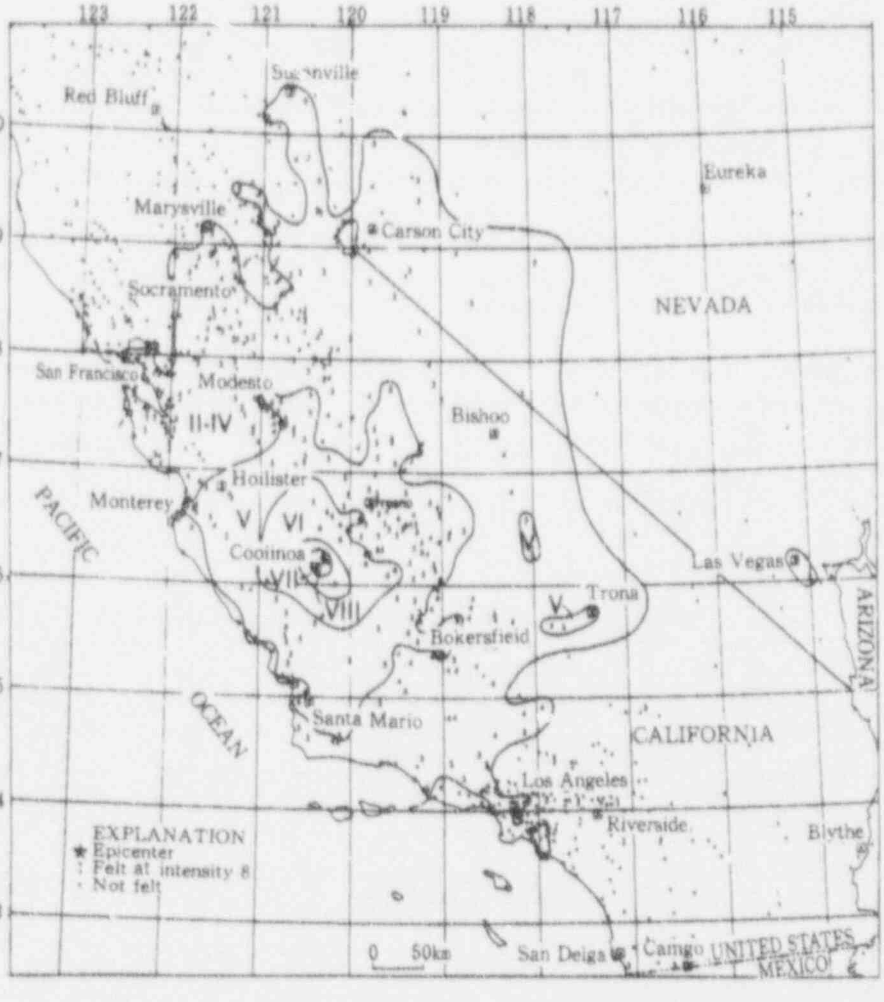


Figure 1. Seismic intensity distribution.

Fault parameters	According to [3]	
Fault length: —	Width: —	Depth: —

Dip angle: —	Dip direction: —
Mo: —	Fault displacement: —
Slip type: —	Rise time: —
Rupture velocity: —	Stress drop: —

Judging from the diagram of projection of the initial motion of the main shock to the lower hemisphere, a nodal surface inclined to the northeast at 67° in the N53°W direction can be identified clearly. The inclination of the second nodal surface is between 23° and 26°. Its direction, however, can be determined only between N20°W and N80°W. Assuming that the first nodal surface indicates the fault plane, it becomes a large-angle reverse fault with its northeast side rising; assuming that the second nodal surface indicates the fault surface, it becomes a small-angle thrust fault with its southwest side rising. It is believed that in the actual situation, it is quite possible that the second small-angle thrust fault is present. Table 1 lists the fault models proposed. Figure 2-b and Figure 3 illustrate the plane view of the fault and the fault diagram corresponding to Table 1, respectively. It is believed that the possibility for model d is rather high.

Table 1. List of fault models proposed.

Strike/Dip	Slip 1 Direction	Slip (m)	Vertical Depth To: Top of Fault (km)	Vertical Depth To: Base of Fault (km)	Fault Width (km)	Moment $M_0 \times 10^{25}$ (dyne·cm)	Model Fit	Figure 3 Symbol
N53°W67°NE	reverse	1.3	3.0	13.2	11.0	6.5	good	a - - -
N53°W67°NE	reverse	1.8	4.0	11.2	8.0	6.0	good	a —
N53°W67°NE	reverse	2.3	5.0	10.5	6.0	6.0	good	a ·····
N53°W67°NE	reverse w/20°rt-lat.	2.2	4.5	11.0	7.0	6.5	good	b ·····
N53°W67°NE	reverse w/15°rt-lat.	1.8	4.0	11.5	8.0	7.0	good	b —
N53°W23°SW	thrust	2.5	10.5	13.2	10.0	9.0	poor	c - - -
N53°W23°SW	thrust	1.4, top 2.0, base	4.5	7.2	9.0	5.0	good[3]	c ·····
N53°W23°SW N53°W23°SW	reverse thrust	1.0, top 2.0, base	5.0	12.0	11.0	7.5	fair	d —

Fault position

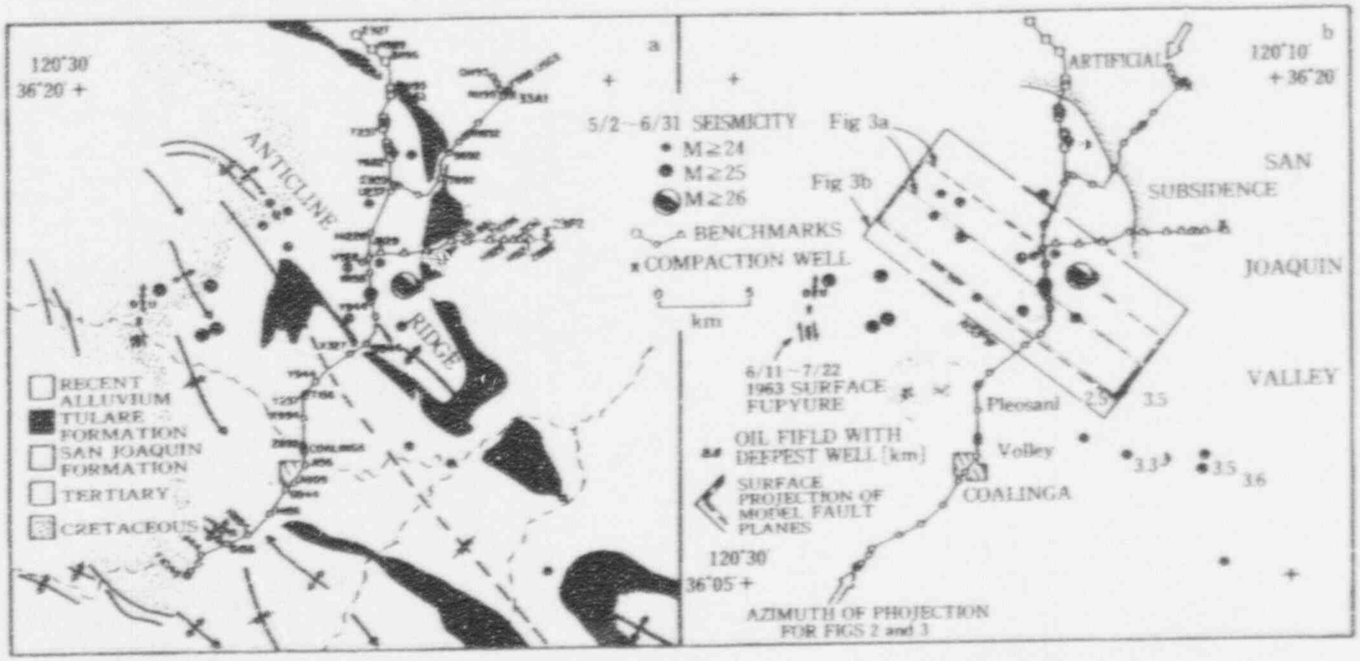


Figure 1. (a.) Map of the leveling route, compaction recorder wells, geology and structural features simplified from Fowkes [1982], and M_L ≥4 aftershocks. (b.) Oil fields, showing the deepest well locations, the Nunez fault surface rupture, and the model fault planes projected to the surface.

Figure 2. Fault position diagram (plane view).

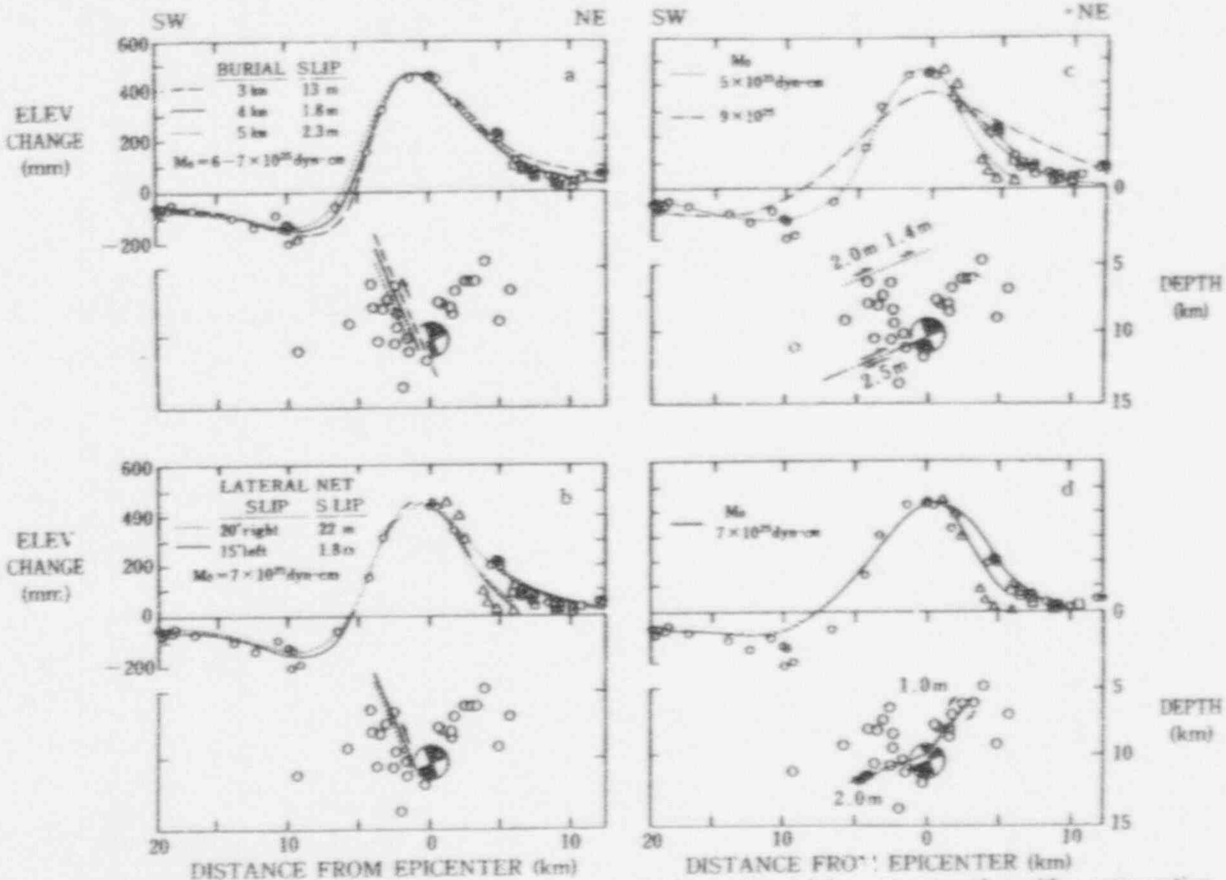


Figure 3. Earthquake elevation changes from Figure 2c compared with predicted deformation, together with a cross-section showing the model faults and the first four days of $M_L > 3$ aftershocks, and the mainshock. (a, b.) Reverse faults with 67°NE dips. (c, d.) Thrust faults dipping 23°SW.

Figure 3. Fault position diagram (cross-sectional view).

Features of damage

The downtown area of Coalinga was hit hardest. The city authority closed this area to prevent looting and secondary disasters from June 6, i.e., one month after the earthquake, damaged buildings were started to be pulled down. Almost all of the damaged buildings were made of brick, blocks and wood. Most of these buildings were rather old (built before 1920). The damage features were as follows:

a) Damage to buildings made of brick or blocks was in the form of out-of-plane falling. According to the authority of Tarzeon [transliteration] in Fresno County, in buildings built before 1920, the bonding portion between upper wall and roof was weak.

b) Damage to buildings made of bricks usually was in the pattern of out-of-plane falling. On the longer walls, there were no cracks caused by in-plane shear force.

c) Damage to wood houses was more frequent for the old type structure (with columns sitting on foundation) and less frequent for the new type structure. As a result, it is believed that the seismic intensity within Coalinga city was IX in the Modified Mercalli scale (corresponding to an acceleration of about 200-430 Gal).

d) Since there are almost no reinforced concrete buildings in this region, it is difficult to estimate the intensity of the seismic motion from the damage state.

Based on the aforementioned features, it is estimated that the maximum seismic acceleration in the city of Coalinga was about 300-450 Gal.

Disaster distribution

Figure 4. Disaster distribution of bridges.

Earthquake records	According to [1, 2]

The strong-motion records of the Coalinga Earthquake on May 2 were recorded at 37 sites under control of the U.S.G.S. The record nearest to the epicenter was observed at Pleasant Valley Pump Station, about 9 km from the epicenter (see Figure 5). The other observation points include Bear Valley Observation Network (75-100 km from epicenter) and Fresno Hospital (75 km from epicenter). In these cases, since the distances to the epicenter are large, they cannot be used to evaluate the seismic motion at the epicentral region.

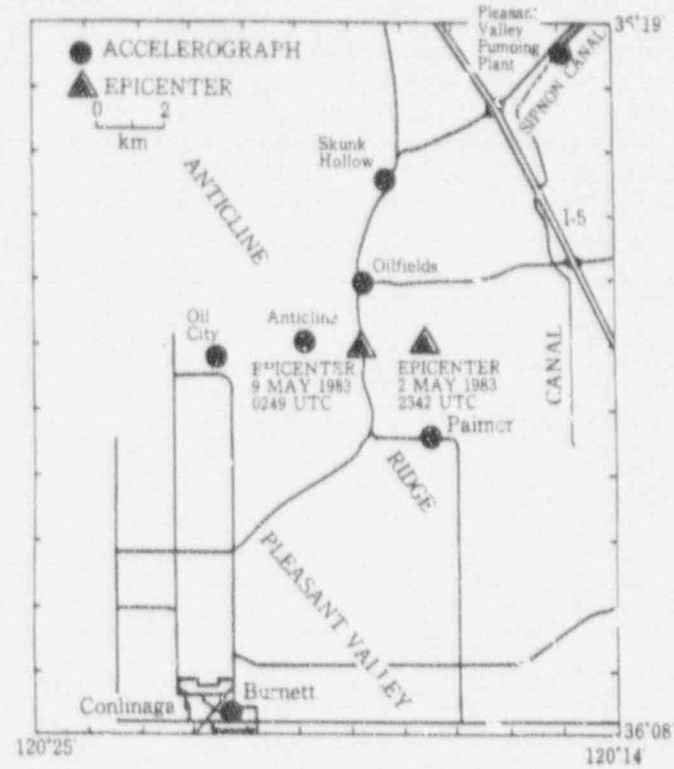
The Pleasant Valley Pump Station, where the strong-motion record was made nearest to the epicenter, is a water feeding facility for feeding water from the Califor[illegible] queduct to the Coalinga Aqueduct. Records were made at two sites: underground cell (5.2 m below grou[illegible]ce) and switch yard (on surface). The switch yard is located 85 m southwest of the plant and is on the edg[illegible] slope with an elevation of about 21 m from the plant le[illegible]. Hence, the position of the strong-motion seism[illegible]ph in the underground cell is located about 26 m below the level of the switch yard. The apparatus at the switch yard is set on a 4'-square concrete block, with a small metallic cover.

According to the analog records, the switch yard had a maximum acceleration of 0.45 G in the horizontal direction and 0.37 G in the vertical direction, the underground cell had a maximum acceleration of 0.33 G in the horizontal direction and 0.22 G in the vertical direction. Although strong-motion seismographs were also set on the first floor and roof of the pump station, they were not well triggered and no record was obtain[illegible]

Figure 6 shows the acceleration waveforms after computer processing of the digitized data from the analog data (film recording). Table 2 lists the maximum amplitudes of vibration. The duration of the main shock of this earthquake was about 10 sec.

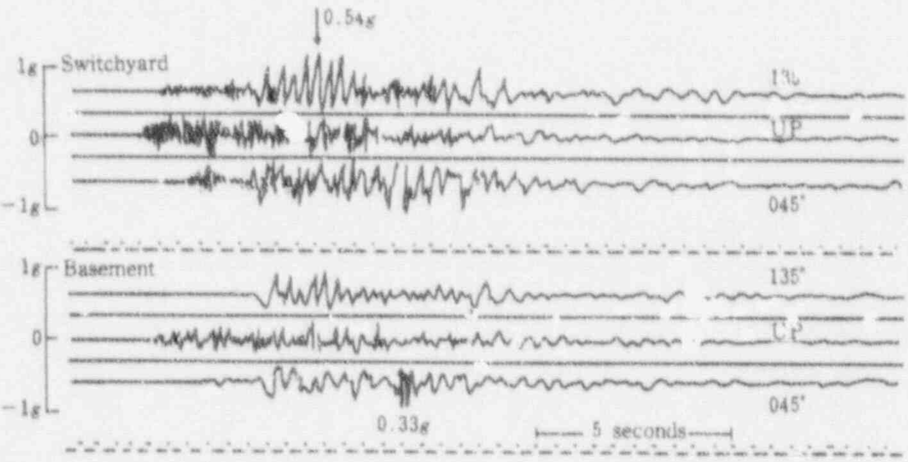
Table 2. Strong-shock records (maximum values) of Pleasant Valley Pump Station.

	Switch			Underground cell		
	Acceleration (Gal)	Velocity (kine)	Displacement (cm)	Acceleration (Gal)	Velocity (kine)	Displacement (cm)
Horizontal 135°	514.4	39.2	5.05	267.3	21.7	3.86
Horizontal 45°	440.6	50.0	15.5	306.7	36.7	10.[illegible]
Vertical	371.1	16.4	7.58	216.3	15.5	7.94



Instrument locations and epicenter of main shock ($M_L = 6.5$) and large aftershock ($M_L = 5.1$).

Figure 5. Locations of instruments.



Strong-motion records from Pleasant Valley pumping plant: main shock.

Figure 6. Strong-motion records from Pleasant Valley Pump Station.

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Appendix 5. Basic references/reference books

Introduction

These references, some of which have already been listed in the main text, are tutorial and reference materials which are believed indispensible for aseismic design in various fields.

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Appendix 6. List of summaries of seismic-related codes at the Institute of Nuclear Safety of Nuclear Power Engineering Corporation

Introduction

The Institute of Nuclear Safety, Nuclear Power Engineering Corporation has performed improvement and assortment for the safety analysis codes used for analysis calculation of the safety of nuclear power facilities when safety examination is performed by the government of Japan. SAN (Seismic Analysis Nuclear) series codes are prepared as the analysis codes related to seismic design. In the following, summaries of the codes will be presented. As pointed out above, these codes are used only by Japan government and they are confidential.

1. SANWAV (Formation/analysis code of seismic wave)

(1) Purpose

With respect to the input earthquake ground motion used for seismic response analysis, a simulated seismic wave suitable for the design response spectrum is formed. In addition, correction of the seismic wave, various spectral analyses, and wave propagation analyses in layered soil are performed.

(2) Features

a. The methods of forming simulated seismic waves include the method using random number phase, the method using actual seismic wave phase, and the method using mixed phase of random number and exponential function.

b. Analyses of the seismic wave include various spectral analyses of response spectrum, power spectrum, Fourier spectrum, nonstationary spectrum, auto and cross-correlation functions, transfer function, etc., as well as appropriate analysis with respect to the design response spectrum.

c. The wave propagation analyses in the layered soil can be performed by linear or equivalent linear one-dimensional wave propagation analyses based on the wave theory, and by nonlinear wave propagation analyses using a discrete mass model. As the nonlinear hysteresis characteristics, it is possible to use the Ramberg-Osgood model and Hardin-Drnevich model.

2. SANDEL, SANDEP, (Earthquake records, database)

(1) Purpose

In order to perform evaluation of the basic earthquake ground motion used in the seismic design, a database of the various historical earthquakes and active fault information is stored and used for various analyses to evaluate the effect of earthquake on the site.

(2) Features

a. The earthquake data in the Usami Catalog and the data published by Japan Meteorological Agency are used as the historical earthquake data. The data in "Active Faults in Japan—Sheet Maps and Inventories" are used as the active fault information. The seismic wave data include 36 earthquakes and 478 spectra.

b. It can make use of the stored data to perform indexing/tabulating according to the time sequence or magnitude sequence assigned, and it can form epicenter distribution map, focus depth distribution map, magnitude vs. epicenter distance diagram, etc. In addition, it can calculate and draw the response spectrum at base rock, maximum velocity and acceleration spectra and statistical expected values at the site.

3. SANFALT (Seismic motion analysis code using fault model)

(1) Purpose

When the basic earthquake ground motion is to be evaluated, the epicentral region and its seismic motion can be evaluated based on the fault model.

(2) Features

a. The code covers Sato's method, Ishida's method, and Kobayashi/Midorigawa's method, which are analysis methods with emphasis put on the short-period component using fault method.

b. The data used are the fault parameter of the earthquakes taking place near Japan when the references were published.

c. Analysis can also be performed for the case when earthquakes are induced by multiple faults near each other in time or space (multiple shocks).

d. In addition to the ability to calculate the amplification characteristics of the surface layer of soil, it can also extract the amplification characteristics of the site from the observed plural record of actual earthquakes.

4. SANSHL (Analysis code of axi-symmetric structure)

(1) Purpose

Seismic response analysis and stress analysis with respect to the static loads including thermal load are performed for nuclear reactor containment vessel, cylindrical tank and other axi-symmetric structures; also, creep/crack analysis is performed for the concrete containment vessels.

(2) Features

a. By Fourier series expression, it is also possible to treat non-axisymmetric loads.

b. As dynamic analysis, it is possible to perform spectral response analysis, time history response analysis, complex response analysis, and oval mode analysis.

c. Both stationary and nonstationary thermal conduction analysis can be performed.

d. It is possible to perform creep analysis of concrete structures and to calculate the thermal stress in consideration of the decrease in rigidity due to cracks in the cross section based on Gurfinkel's method.

5. SANSTR (General structural analysis code)

(1) Purpose

In order to assess the behavior of the nuclear reactor building and the related structures, two-dimensional and three-dimensional linear stress analysis are performed using the finite element method to calculate the stress, deformation, etc.

(2) Features

a. It can perform stress analysis and provide graphic output for models with arbitrary combinations of beam elements, plate bending elements, thick shell elements, and solid elements.

b. It can analyze concentrated load, uniformly distributed load, body force, thermal load, forced displacement, etc.

c. It can perform uplift analysis of foundation supported by elastic soil.

d. It can represent the three-dimensional structure using equivalent beam elements by means of condensation technique (stiffness evaluation).

6. SANREF (Building restoring force characteristics analysis code)

(1) Purpose

In order to evaluate the hysteresis characteristics and the ultimate strength of the nuclear reactor building structure, the finite element method is used in this code to perform nonlinear stress analysis when static external forces act on a reinforced concrete continuous structure.

(2) Features

a. It has a two-dimensional analysis function that can treat the in-plane deformation using plane strain elements and a three-dimensional analysis function that can treat the out-of-plane deformation using shell elements.

b. In the two-dimensional analysis, cyclic loading is possible. In the three-dimensional analysis, it is possible to treat walls, slabs, cylindrical structures, and their composite structure. It is assumed that the external forces increase monotonically.

c. As the external forces include nodal load, edge distributed load, body force, etc. can be handled.

7. SANSSI (Soil-structure interaction analysis code)

(1) Purpose

Seismic response analysis is performed in consideration of the soil-structure interaction by using the two-dimensional finite element method.

(2) Features

a. It is possible to perform analyses of the seismic response in the frequency domain of the soil-structure interaction system, and forced vibration analysis.

b. The semi-infinity of the soil can be taken into consideration by using the viscous boundary at the bottom and the transfer boundary at both sides.

c. With the aid of the viscous boundary in the out-of-plane direction, the three-dimensional effect can be represented in a pseudo way.

d. With the aid of the equivalent linearization method, the nonlinearity of the properties of the soil can be taken into consideration.

8. SAMLUAM (Lumped mass model analysis code)

(1) Purpose

Seismic response analysis of lumped mass model of structure composed of bending-shear beams and lumped masses.

(2) Features

a. It can perform time history response analysis, frequency response analysis, and spectral response analysis.

b. In the time history response analysis, it is possible to treat the nonlinear problems for building material and foundation uplift.

c. When the soil is dealt with as a grid-model, it is possible to consider viscous boundaries for the bottom, both sides and in out-of-plane direction.

9. SANRAI (Soil complex stiffness analysis code)

(1) Purpose

In order to assess the vibration characteristics of the structure, the soil spring is calculated in consideration of the dynamic interaction of the soil.

(2) Features

The soil stiffness below the foundation on a half-space elastic ground can be calculated as a function of frequency.

10. SANSOL (Layered soil-structure interaction analysis code)

(1) Purpose

Frequency vibration response analysis and seismic wave input response analysis are performed for the structure embedded in three-dimensional layered ground to assess the characteristics of the interaction of soil-structure system.

(2) Features

a. Thin-layer element method suitable for solution of three-dimensional wave propagation equation is used.

b. The frequency response displacement of the foundation and soil with respect to the vibration excitation can be calculated.

c. In the process of calculation of the frequency response displacement, the complex stiffness of the soil can be calculated.

d. Response analysis of the foundation and soil can be performed caused by the seismic wave input.

11. SANGRS (Ground stability analysis code)

(1) Purpose

In order to evaluate the seismic stability of the ground which directly supports the various facilities of the nuclear power plant, nearby cut slope surfaces, and natural slope surface, the finite element method is used to perform soil stability analysis.

(2) Features

a. It is possible to perform arc slip analysis by using a simplified dividing method, modified Fellenius method and Bishop method.

b. By using a two-dimensional FEM model, initial stress analysis, analysis in excavation process, analysis after building is completed, seismic analysis (static or dynamic analysis), etc., are implemented for the ground. As they are combined, the local shear safety coefficient and the slip safety factor of ground slippage can be calculated.

c. With the aid of a two-dimensional FEM model, it is possible to perform stationary and nonstationary percolation flow analysis in consideration of the saturated and unsaturated regions.

12. SANPIP (Piping system analysis code)

(1) Purpose

For piping and equipment consisting of beam-elements, finite element method is used to perform static and dynamic linear analysis.

(2) Features

a. It can perform stress evaluation analysis of the piping system on the base of "Notification No. 501."

b. The mode for spectral response analysis can correspond to several methods (such as mode synthesis using square root of sum of square or absolute sum) as the synthesis method of vertical seismic load.

c. Stress analysis due to thermal expansion and temperature distribution analysis can be performed.

13. SANNAMI (Tsunami analysis code)

(1) Purpose

In order to evaluate the effects of tsunami on the nuclear power plant facilities located in a coastal area, tsunami analysis is performed using a finite difference method, etc.

(2) Features

a. It can use the time histories of the seabed dislocation amount, or the sea water level of the surface, or the forced water level vibration in the open boundary of the analysis as the source of the tsunami. It can select the wave source model according to the analysis conditions. When the time history of the seabed dislocation is given, it can use the fault parameters to calculate the dislocation amount.

b. The analysis code using the finite difference method has a function for finely dividing the calculation region. As the position nears the coast, the lattice size is reduced in sequence.

c. The finite difference analysis code can perform run up and wave over top analyses of the tsunami.

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This document is a translation, in its entirety, of the Japan Electric Association (JEA) publication entitled "Technical Guidelines for Aseismic Design of Nuclear Power Plants - JEAG 4601-1987." This guideline describes in detail the aseismic design techniques used in Japan for nuclear power plants. It contains chapters dealing with: (a) the selection of earthquake ground motions for a site, (b) the investigation of foundation and bedrock conditions, (c) the evaluation of ground stability and the effects of ground movement on buried piping and structures, (d) the analysis and design of structures, and (e) the analysis and design of equipment and distribution systems (piping, electrical raceways, instrumentation, tubing and HVAC duct). The guideline also includes appendices which summarize data, information and references related to aseismic design technology.

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