November 19, 1982

Docket No. 50-409
LS05-82-11-059

Mr. Frank Linder
General Manager
Dairyland Power Cooperative
2615 East Avenue South
LaCrosse, Wisconsin 54601

Dear Mr. Linder:

SUBJECT: SEP TOPICS III-6, SEISMIC DESIGN CONSIDERATIONS
AND III-11, COMPONENT INTEGRITY
LACROSSE BOILING WATER REACTOR (LACBWR)

Enclosed is our draft safety evaluation for the seismic design of the LaCrosse Boiling Water Reactor. The staff's review is based on submitted analyses and working-level meetings between Dairyland Power Cooperative (DPC) and NRC representatives. The enclosed draft Safety Evaluation Report (SER) identifies many open items concerning the completed analyses and that your evaluations of LACBWR are incomplete. Therefore, the conclusions presented in the evaluation may be revised should new information be presented in later Dairyland Power Cooperative seismic reports.

Based upon the NRC staff and its consultants review of the analyses and criteria supplied by the licensee for structures, piping, equipment and components, we cannot conclude that these analyses are adequate. Further, as indicated in the enclosure, many analyses for piping, equipment and components are yet to be performed. Schedules for the completion of all required analyses and implementation of modifications shown to be necessary based upon your completed analyses have not been provided.

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ADD:

6. Staley

We acknowledge that NRR management personnel met with DPC representatives on several occasions and permitted the SEP seismic analysis schedule to be deferred pending results of utility risk assessment studies. However, recent conversations with DPC indicate that no formal relief requests are planned until completion of the SEP Integrated Assessment.

This evaluation will be a basic input to the integrated safety assessment for your facility unless you identify changes needed to reflect the as-built conditions at your facility. With respect to the potential modifications and open items outlined in this report, a determination of the need to actually implement these or other changes will be made during the same integrated assessment. This topic assessment may be revised in

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DATE ▶							

the future if your facility design is changed or if NRC criteria relating to this topic are modified before the integrated assessment is completed.

Sincerely,

Original signed by:

Dennis M. Crutchfield, Chief
Operating Reactors Branch No. 5
Division of Licensing

Enclosure:
As stated

cc w/enclosure:
See next page

OFFICE	SEPB	SEPB	SEPB	SEPB	ORB#5	ORB#5	AD:SA:DL
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DATE	10/27/82	10/27/82	10/3/82	10/4/82	10/ /82	11/16/82	10/ /82

Docket No. 50-409
LaCrosse
Revised 8/82

Mr. Frank Linder

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SYSTEMATIC EVALUATION PROGRAM

TOPICS III-6 AND III-11

LACROSSE BOILING WATER REACTOR

TOPICS: III-6, SEISMIC DESIGN CONSIDERATIONS
III-11, COMPONENT INTEGRITY

I. INTRODUCTION

The eleven nuclear power plant facilities under review in the SEP received construction permits between 1956 and 1967. Seismic design procedures evolved significantly during and after this period. The Standard Review Plan (SRP), first issued in 1975; along with the Regulations 10 CFR Part 50, Appendix A; and 10 CFR Part 100, Appendix A constitute current licensing criteria for seismic design reviews. As a result, the original seismic designs of the SEP facilities vary in degree from the Uniform Building Code up through and approaching current standards. Recognizing this evolution, the staff found that it is necessary to make a reassessment of the seismic safety of these plants.

Under the SEP seismic reevaluation, these eleven plants were categorized into two groups based upon the original seismic design and the availability of seismic design documentation. Different approaches were used to review the plant facilities in each group. The approaches were:

Group I: Detailed NRC review of existing seismic design documents with limited reevaluation of the existing facility to confirm judgments on the adequacy of the original design with respect to current requirements.

Group II: Licensees were required to reanalyze their facilities and upgrade, if necessary, the seismic capacity of their facility. The staff reviews the licensee's reanalysis methods, scope and results. Limited independent NRC analysis performed to confirm the adequacy of of the licensee's method and results.

Based on the staff's assessment of the original seismic design, the LaCrosse Boiling Water Reactor (LACBWR) was placed in Group II for review.

The LaCrosse Boiling Water Reactor is a 165 MW thermal boiling-water reactor. It is located about 1 mile south of Genoa, Wisconsin on the east bank of the Mississippi River. Allis-Chalmers had the responsibility for the design, fabrication, construction, and startup of the reactor plant. Allis-Chalmers retained Sargent & Lundy Engineers as architect-engineers for the project and the Maxon Construction Company as constructors. The Construction Authorization (CAPR-5) was issued in March 1963. Allis-Chalmers operated the plant until November 1, 1969, under Operating Authorization DPRA-5, issued in July 1967. Since that time, the plant has been operated by Dairyland Power Cooperative under Operating Authorization DPRA-6, issued in November 1969, and Provisional Operating License DPR-45, issued in August 1973. No seismic loads were considered in the initial design of LACBWR. A seismic assessment of certain structures and systems was begun in 1974 by Gulf Atomic. Seismic analyses were continued to the present time, with the latest evaluations being performed by Nuclear Energy Services (NES). The details of the earlier seismic evaluations are described in the draft summary report "Seismic Review of LaCrosse Boiling Water Reactor (LACBWR) Phase 1 Report - Review and Document Existing Seismic Analysis and Design" (Attachment 1).

The SEP seismic review of the LACBWR facility addressed only the Safe Shutdown Earthquake (SSE), since it represents the most severe seismic event that must be considered in the plant design. The scope of the review included three major areas: (1) the integrity of the reactor coolant pressure boundary; (2) the integrity of fluid and electrical distribution systems related to safe shutdown; and (3) the integrity of mechanical and electrical equipment designed as engineered safety feature systems (including containment).

By letters dated August 4, 1980 and April 24, 1981 (References 1 and 2), the licensee, Dairyland Power Cooperative, was requested in accordance with 10 CRF 50.54(f) to seismically reevaluate and upgrade, if necessary, all safety-related structures, systems and components to a level of seismic resistance consistant with ground motion associated with the site specific spectra.

II. REVIEW CRITERIA

Since the SEP Group II plants were not designed to current codes, standards, and NRC requirements, it was necessary to perform "more realistic" or "best estimate" assessments of the seismic capacity of the facility. A set of review criteria and guidelines was developed for the SEP plants. These review criteria and guidelines are described in the following documents:

1. NUREG/CR-0098, "Development of Criteria for Seismic Review of Selected Nuclear Power Plants," by N.M. Newmark and W.J. Hall, dated May 1978.

2. "SEP Guidelines for Soil-Structure Interaction Review," by SEP Senior Seismic Review Team, dated December 8, 1980.

3. Letter to F. Linder (Dairyland Power), from D.M. Crutchfield (NRC), "Systematic Evaluation Program Position Re: Consideration of Inelastic Response Using the NUREG/CR-0098 Ductility Factor Approach," dated June 23, 1982.

4. Letter to F. Linder (Dairyland Power), from D.M. Crutchfield (NRC), "SEP Topic III-6, Seismic Design Considerations, Staff Guidelines for Seismic Evaluation Criteria for the SEP Group II Plants," dated July 26, 1982.

5. Letter to F. Linder (Dairyland Power), from D.M. Crutchfield (NRC), "SEP Topic III-6, Seismic Design Considerations, Staff Guidelines for Seismic Evaluation Criteria for the SEP Group II Plants - Revision 1," dated September 20, 1982.

For the cases that are not covered by the criteria stated above, the following SRPs and Regulatory Guides were used for the review:

1. Standard Review Plan, Sections 2.5, 3.7, 3.8, 3.9, and 3.10.

2. Regulatory Guides 1.26, 1.29, 1.60, 1.61, 1.92, 1.100 and 1.122.

Any deviations from the criteria or guidelines were to be justified by the licensee on a case-by-case basis.

III. RELATED TOPICS AND INTERFACES

The SEP topics related to the review of Seismic Design Considerations and Component Integrity are II-4, II-4.A, II-4.B, and II-4.C. These topics relate to specification of seismic hazard at the site, namely, the site specific free-field ground response spectra for the LACBWR site.

IV. EVALUATION

A. General Approach

The seismic reevaluation of the LACBWR was initiated by conducting a detailed review of the docketed plant seismic related design criteria. The results of this review are summarized in Attachment 1. Based on the findings of this docket review, two NRC 10 CFR 50.54(f) letters were issued to require the licensee to complete a seismic reevaluation program. This program scope included: (1) providing a justification to demonstrate that the plant can continue to operate in the interim until the program is complete; (2) proposing a program plan which addresses the scope, criteria, and schedule for completion of the program; and (3) performing seismic analysis after staff acceptance of the proposed program, and providing the final results to the staff for review. The results of the staff review of this program would provide the basis for seismic safety assessment of the facility.

The licensee has not completed the requisite seismic analyses. As discussed later, several outstanding open issues remain to be resolved regarding the structural, piping and mechanical equipment analyses presented by the licensee. Evaluation of the safety-related electrical equipment have not been performed nor has a plan for such evaluations been provided.

The review approach which was followed on LACBWR was to first perform a review of their program plan. This plan consisted of a presentation of general methodologies and criteria to be used by the licensee. Next, a working-level meeting was conducted among the licensee's consultants, and a review team consisting of the NRC staff and its consultants. At this meeting, the licensee's analyses and calculations for structures, piping, equipment, and components were reviewed on an audit basis.

When structures are evaluated, they are judged to be adequately designed if:

1. The analyses are sufficient to adequately determine structural responses consisting of member loads, and floor response spectra piping, equipment and components evaluations; and

2. The loads generated from the analyses are less than original loads; or

3. The seismic stresses from the analyses are low compared to reasonable estimates of the maximum strengths of the steel and concrete; or

4. The seismic stresses from the analyses exceed reasonable estimates of the steel or concrete maximum strengths, but estimated reserve capacity (or ductility) of the structure is such that inelastic deformation would be expected without structural failure or adverse impacts on piping, equipment or component responses.

If the above criteria are not satisfied, more comprehensive reanalyses are required to demonstrate design adequacy. Section II criteria 1 through 3 provide the basic guidelines for all evaluations, in conjunction with the previously referenced SRP and Regulatory Guide guidelines.

Piping and mechanical equipment evaluations were presented in a working-level meeting with the licensee's consultant. Acceptance criteria contained in Reference 5 were used as review guidelines. Piping is judged to be adequate if:

1. The analyses are sufficient to adequately determine piping system responses; and

2. The piping response stresses are in conformance with the criteria contained in References 4 and 5; or

3. The piping responses exceed the criteria referenced above, but estimated ductility is such that inelastic deformation is expected without loss of integrity or adverse impacts on the response of attached piping, equipment or components.

If the above criteria are not satisfied, more comprehensive reanalyses are required to demonstrate design adequacy. Section II criteria 1 through 5 provide the basic guidelines for all evaluations, in conjunction with the previously referenced SRP and Regulatory Guide guidelines.

No program has been described or implemented by the licensee for the evaluation of the structural integrity of electrical cabinets. The adequacy of electrical equipment and components would be judged using criteria similar to that outlined above for the structures and piping.

B. Detailed Evaluation

1. Seismic Input

The site specific ground response spectra, which are acceptable to the staff as input for the seismic reevaluation of the LACBWR plant, were provided to the licensee by NRC letters dated August 4, 1980 and June 17, 1981 (References 1 and 7). These spectra are based on the results of the NRC Seismic Hazards Analysis Program (Ref. 6) conducted by the staff and its consultant, Lawrence Livermore National Laboratory (LLNL).

2. Justification for Continued Operation

The licensee provided information supporting continued operation by letters dated October 14, 1980 and June 12, 1981, and the letter from, Craig Finnan (NES) to R.E. Shimshak (Dairyland Power), dated April 21, 1981. In addition, the staff and its consultant (Professor W.J. Hall of the University of Illinois) visited the site on May 22, 1981, to evaluate the seismic resistance of the facility. The NRC Safety Evaluation Report (SER) to support continued operation of the LACBWR plant until completion of the seismic reevaluation program, was issued on September 4, 1981 (Ref. 8).

The conditions imposed in the September 4, 1981 SER were that:

(1) results of seismic analysis are submitted for NRC review on the schedule specified in a June 12, 1981 letter; and

(2) any modifications shown to be necessary as a result of the seismic analysis which are not implemented by January 1, 1983, were justified on a case-by-case basis with a schedule for implementation.

The justification for continued operation was based upon analyses being performed in a timely manner such that any necessary modifications would be identified, the upgrading of the high pressure core spray system, the proper anchorage and support of safety-related electrical equipment, the addition of redundant cooling water supplies, and the inherent capacity of the remaining plant structures and systems coupled with the low seismic hazard associated with the LACBWR site.

3. Review of the Seismic Reevaluation Program Plan

Descriptions of seismic criteria, scope, analytical procedures, and modeling techniques are described in The Full Term License Application for LaCrosse, Attachment 2. The results of our review were documented in a January 19, 1982 NRC letter to the licensee. As discussed later, this and subsequent reviews have identified many open issues which must be resolved. No schedule for the resolution of these issues and the implementation of modifications has been provided by the licensee. Therefore, we find that the program plan is not in conformance with the SEP requirements.

4. Review Scope

The scope of the reevaluations was specified in the August 4, 1980 and April 24, 1981 NRC 10 CFR 50.54(f) letters (Refs. 1 and 2) to include those structures, systems and components necessary to assure, both during and after a postulated seismic event:

1. The integrity of the reactor coolant pressure boundary,

2. The integrity of fluid and electrical distribution systems related to safe shutdown and engineered safety features; and

3. The integrity and functionability of mechanical and electrical equipment and engineered safety feature systems (including containment).

The resolution of issues related to the functionability of mechanical and electrical equipment was later deferred to the Unresolved Safety Issue (USI) A-46.

5. Review of Reevaluation Criteria and Scope Proposed by the Licensee

The scope of the licensee's evaluations are defined by the analyses described in the July 22, and August 2, 1982 licensee letters (Refs. 9 and 10). The analyses attached to these letters also provide the details of the criteria employed in the licensee's evaluations. These analyses were reviewed by the staff and its consultants. Attachment 2, the Lawrence Livermore National Laboratory (LLNL) report "Review of the Seismic Re-evaluation of the LaCrosse Boiling Water Reactor Facilities," dated September 1982, describes in detail the results of our review of the licensee's structural evaluations Attachment 3, the Structural Mechanics Associates (SMA) report "Structural Review of the LaCrosse Boiling Water Reactor Under Seismic Loads for the Systematic Evaluation

Program," dated September 1982, describes an NRC sponsored independent seismic analysis of the LACBWR containment structure and forms the basis for certain conclusions drawn in Attachment 2. Attachment 4, the EG&G report "Technical Evaluation of LaCrosse Boiling Water Reactor Power Station Seismic Design," dated September 1982, describes in detail the results of our review of the licensee's piping, equipment and component evaluations.

The licensee has evaluated the Reactor Containment Building, the Turbine Building, the 1-B Diesel Generator Building, and the LACBWR and Genoa 3 stacks. These structures are appropriate to meet the scope of required structural evaluations provided that the licensee either confirms that there are not safety-related tunnels or, if there are, evaluates them. This conclusion is based upon the licensee's installation of the Emergency Service Water System.

The following piping and equipment have been evaluated by the licensee: the shutdown condenser and platform, the feedwater piping system, the main steam piping system, the recirculation piping systems, the high pressure core spray suction and discharge piping system, and the 14" shutdown condenser vent piping system. This is not in conformance with the minimum required scope (Ref. 2), for example, the Control Rod Drive (CRD) system and reactor vessel internals were not evaluated. For the piping system analyses, in many cases modifications were assumed but they have not been implemented and may not be possible to implement due to physical contraints or geometry. The licensee should evaluate the "as-built" plant to determine the ability to install the assumed modifications. Analyses to demonstrate structural integrity of electrical equipment, including their anchorages, were not provided, although, the licensee indicated in a July 28, 1980 letter to the NRC on the subject that the anchorage portion had been completed. Pumps and valves were not included in the licensee's evaluations. Additional deficiencies in the licensee's scope of seismic review are listed in Attachment 4. Therefore, the scope of the licensee's analyses is not in conformance with that specified in References 1 and 2. No licensee justification for the deviations from the SEP scope of seismic review have been provided.

For those analyses which have been provided by the licensee for review, many deficiencies and open items have been identified by the NRC staff and its consultants. These are identified in Attachment 2 and 4. These identified deficiencies and open items are of sufficient magnitude to preclude a finding by the staff that the facility would resist an earthquake as defined by the Site Specific Spectra. Insufficient information is available to enable us to quantify the level of seismic resistance that may exist for the LACBWR. In addition, we cannot conclude that the implemented modifications are adequate. Substantial additional analyses and studies are required to provide for quantification of the facility's seismic resistance.

V. SUMMARY AND CONCLUSIONS

The scope of the licensee's evaluations is not in conformance with that required by the SEP criteria. Several deficiencies and open items have been identified (see Attachment 2 and 4) in those analyses which have been performed.

No schedule for resolution of these open issues or for the implementation of any required modifications has been provided by the licensee. It is the staff's judgement that such schedules would extend beyond January 1, 1983. No justification for such an extension has been provided by the licensee.

The licensee has indicated that they do not intend to expend further resources on the seismic evaluation of the LACBWR, since this may not be economically justified. The staff does believe that the facility possesses inherent seismic resistance. However, insufficient analyses have been performed and implementation of corresponding required modifications, to allow the staff to conclude that there is assurance that the facility can safely withstand the occurrence of an SSE as defined by the site specific spectra. Based upon the low probability of an earthquake with significant ground motion at the site and the potentially low radiological consequences of an accident, the staff concludes that continued operation is acceptable pending completion of the integrated assessment.

VI. REFERENCES

1. Letter to F. Linder (Dairyland Power), from D.G. Eisenhut (NRC), "RE: LaCrosse," dated August 4, 1980.

2. Letter to F. Linder (Dairyland Power), from D.M. Crutchfield (NRC), "SEP Topic III-6, Seismic Design Considerations - LaCrosse," dated April 24, 1981.

3. Letter to F. Linder (Dairyland Power), from D.M. Crutchfield (NRC), "Systematic Evaluation Program Position Re: Consideration of Inelastic Response Using the NUREG/CR-0098 Ductility Factor Approach," dated June 23, 1982.

4. Letter to F. Linder (Dairyland Power), from D.M. Crutchfield (NRC), "SEP Topic III-6, Seismic Design Considerations, Staff Guidelines for Seismic Evaluation Criteria for the SEP Group II Plants," dated July 26, 1982.

5. Letter to F. Linder (Dairyland Power), from D.M. Crutchfield (NRC), "SEP Topic III-6, Seismic Design Considerations, Staff Guidelines for Seismic Evaluation Criteria for the SEP Group II Plants - Revision 1," dated September 20, 1982.

6. NUREG/CR-1582, "Seismic Hazard Analysis," Volumes 2 through 5, dated October 1981.

7. Letter to all SEP Owners (Except San Onofre Unit 1), from D.M. Crutchfield (NRC), "Site Specific Group Spectra for SEP Plants Located in the Eastern United States," dated June 17, 1981.

8. Letter to F. Linder (Dairyland Power), from D.M. Crutchfield (NRC), "SEP Topic III-6, Seismic Design Considerations - LaCrosse," dated September 4, 1981.

9. Letter to R. Dudley (NRC), from R.M. Brimer (Dairyland Power), "Dairyland Power Cooperative, LaCrosse Boiling Water Reactor (LACBWR, Provisional Operating License No. DPR-45, Seismic Analysis Reports," dated July 22, 1982.

10. Letter to R. Dudley (NRC), from R.M. Brimer (Dairyland Power), "Dairyland Power Cooperative, LaCrosse Boiling Water Reactor (LACBWR, Provisional Operating License No. DPR-45, Seismic Analysis Reports," dated August 2, 1982.

Attachment 1

SEISMIC REVIEW OF LA CROSSE
BOILING WATER REACTOR (LACBWR)
PHASE 1 REPORT
SUBJECT: REVIEW AND DOCUMENT EXISTING
SEISMIC ANALYSIS AND DESIGN

Prepared by: C. Y. Liaw

Checked by:

Approved by:

TABLE OF CONTENTS

1. SUMMARY OF FINDINGS

The seismic analysis of La Crosse Boiling Water Reactor (LACBWR) Plant was based upon a seismic event with a ground surface level peak acceleration of 0.12 g for the Safe Shutdown Earthquake (SSE) and 0.06 g for the Operating Basis Earthquake (OBE or ½SSE).

The horizontal ground motion for the SSE event was specified in the form of response spectrum curves conforming with the Regulatory Guide 1.60. A ground time history, which envelopes the specified 2% damping response spectrum, was generated using the Fourier decomposition method with the Taft 1952 record chosen as the initial accelerogram. The seismic input at different soil levels of the pile foundation systems were generated using a deconvolution process in the frequency domain. The soil layers were modeled by a "shear beam" model. Iterations were performed to converge the root-mean-square values of soil shear strain to the levels compatible to the soil properties.

The time history method was used in the seismic analysis of major structures on pile foundation including the Reactor Containment Building, LACBWR stack and Genoa stack. The soil-structure interaction effects were modeled by soil springs and dashpots attached at different levels to the beam elements which represent the pile group. The structures were also modeled by beam elements and lumped masses. The SSE or ½SSE criteria pulse was input at all pile nodes with the magnitude of the pulse scaled with depth according to the "shear beam" study to account for soil amplification effects.

The response spectrum method was used in the seismic analysis of most major piping systems and equipment. The horizontal floor spectra were generated from the time history analysis of the structure. The vertical spectrum was taken as two-thirds of the horizontal ground acceleration (no amplification).

The horizontal and vertical acceleration were applied simultaneously. The modal responses were combined by the square-root of the sum of square method.

The turbine building was analyzed using the equivalent static method. Each column line was modeled by a single-degree-of-freedom system.

The damping values assumed for concrete structures, piping systems and equipment are equivalent to those suggested in Regulatory Guide 1.61.

According to the analysis report of Gulf United Services, Inc., 1974, all major structural components in the Reactor Containment Building are adequate for the SSE event, the LACBWR and Genoa III stacks would collapse under SSE condition, additional lateral bracing could be required in the turbine building steel framework, and piping systems could require additional seismic restraints. In the subsequent seismic analysis of piping systems performed by Nuclear Energy Services, additional seismic restraints were also recommended for most piping systems.

2. INTRODUCTION

2.1 GENERAL

NRC Systematic Evaluation Program (SEP) has selected LACBWR as one of the operating plants currently considered for the seismic review. The prime concern is the plant's existing capability to ensure safe shutdown in the event of a SSE. As the first step, existing seismic design and analysis information of these plants are being collected and documented. This report is to present the findings on LACBWR from available documents in the NRC docket (see Section 2.2). In the future, additional information may appear and will be accordingly incorporated into this report.

The topics covered in this report include those listed in NRC Standard Review Plant (SRP) Section 3.7.1 to 3.7.3. These are the topics which will be emphasized upon.

The data presented in this report will be used later for comparison with current standards, criteria, and procedures in Phase 2 of the seismic review. All major systems are divided into the following three groups respectively considered in Section 4, 5, and 6 of the report.

1. Major Structural Systems and their foundations
2. Major Piping and their Supports
3. Major Mechanical and Electrical Equipment and their Supports

Summary tables of seismic design and analysis method of the above three groups are given for quick reference.

It is acknowledged that errors due to misinterpretation of data or the use of obsolete data may occur. Therefore, care was taken to provide sufficient reference to the material presented.

2.2 AVAILABLE DOCUMENTS

The documents related to license application of all U.S. Nuclear Power plants are maintained on active files in the NRC docket room, Bethesda, Maryland. The docket number for Palisades Plant is 50409. The compiled seismic information in this report is taken mainly from Docket 50409 which is comprised of the following documents:

1. LACBWR Safety Analysis Report - Draft (July 1978)
2. Seismic Analysis Report by Gulf United Services, 1974
3. Seismic Analysis Reports by Nuclear Energy Services, Inc., 1975-1978
4. Letters and correspondence among utility, NRC and other agencies
5. Technical reports on various subjects, i.e., technical specification, operation, abnormal occurrence, inspections, modifications in design, application for conversion from POL to FTOL, etc.
6. LACBWR Safeguards Report for Operating Authorization, Allis-Chalmers, 1965

2.3 STATUS OF SEISMIC DATA

1. Most of plant descriptions and design criteria are found in Safety Analysis Report.
2. Most of seismic analysis data of structural systems are found in Gulf United Services' report.
3. Most of seismic analysis data of piping systems and equipment are found in Nuclear Energy Services' reports.

3. GENERAL INFORMATION

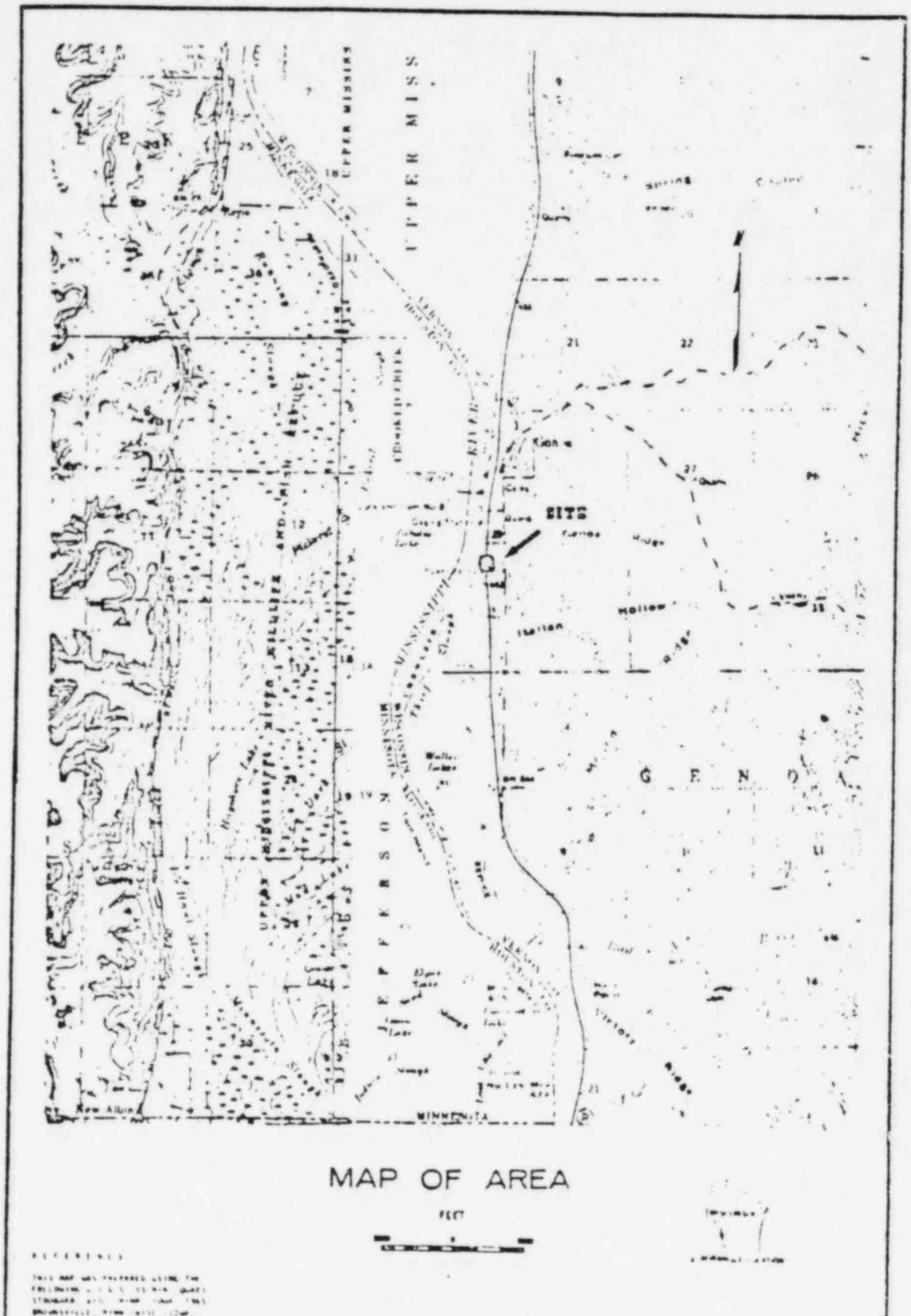
3.1 PRINCIPAL DESIGN FEATURES

(Reference 1 and 2)

The LACBWR is a nuclear power plant of nominal 50 Mw electrical output, which utilizes a forced-circulation, direct-cycle boiling-water reactor as its heat source. The plant is on the east bank of the Mississippi River in Vernon County, Wisconsin, approximately one mile south of the village of Genoa, Wisconsin, and approximately 19 miles south of the city of La Crosse, Wisconsin (PLATE 1, Reference 2).

The reactor and its auxiliary systems are within a steel containment building. The turbine-generator and associated equipment, the control room for both turbine and reactor controls, and plant shops and offices are in a conventional building adjacent to the containment building. Waste-handling facilities, including facilities for processing liquid wastes and for packaging, decontaminating, and temporarily storing solid wastes, are in a separate building (PLATE 9, Reference 2).

The turbine building contains a major part of the power plant equipment. The turbine-generator is on the main floor. Other equipment is located below the main floor. This equipment includes the feedwater heaters, reactor feedwater pumps, air ejector, vacuum pump, full flow demineralizers, off-gas compressor and cooler, condensate pumps, air compressors, air dryer, oil purifier, service and component cooling water coolers and pumps, make-up water demineralizer system, domestic water heater, turbine oil reservoir, oil tanks and pumps, turbine condenser, unit auxiliary transformer, 2400 volt and 480 volt batteries, inverter set and other electrical, pneumatic, mechanical and hydraulic systems and equipment required for a complete power plant. A 30/5-ton capacity, pendant-operated overhead electric traveling crane spans the turbine building. The crane has access to major equipment items located below the floor through numerous



(Reference 2)

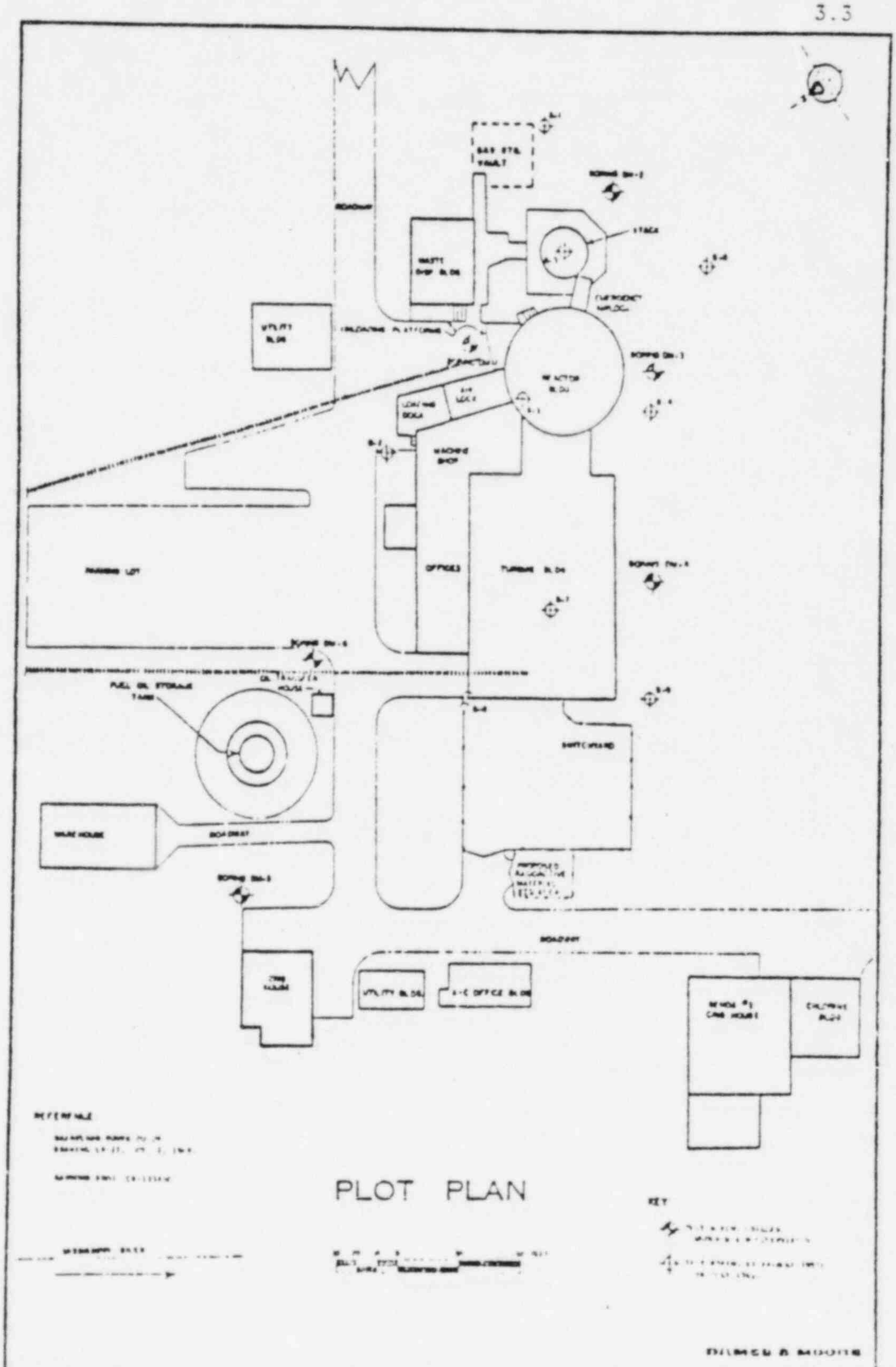


PLATE 3 (Reference 3)

hatches in the main floor.

The turbine building also contains the main offices, the control room (for both turbine-generator and reactor),locker room facilities, laboratory, shops, counting room, personnel change and decontamination facilities, heating, ventilating and air conditioning equipment, rest rooms, storeroom, and space for other plant services. In general, these areas are separated from power plant equipment spaces. The control room is on the main floor on the side of the turbine building that is adjacent to the containment building.

Miscellaneous structures which are associated with the power plant and are located adjacent to the turbine building include the electrical switchyard, crib hourse, oil pump house, warehouse, construction and contractor office buildings, outdoor fuel oil and acid storage tanks, underground septic tanks and condenser circulating water discharge seal well.

3.2 INDENTIFICATION OF AGENTS AND CONTRACTORS

(Reference 1)

LACBWR was designed and constructed by Allis-Chalmers Manufacturing Company and is operating by Dairyland Power Cooperative of Wisconsin.

The Architect-Engineer was Sargent and Lundy Engineers and the General Contractor was Maxon Construction Company.

3.3 LICENSING STATUS

The reactor was operated under Provisional Operating Authorization No. DPRA-5, issued July 3, 1967, which authorized Allis-Chalmers to use and operate the reactor up to 165 Mw. In August, 1969, full power operation was achieved. On November 1, 1969, the Atomic Energy Commission accepted LACBWR from Allis-Chalmers and Operating Authorization No. DPRA-6 was issued to Diarylan Power Cooperative. On October 9, 1974, an application for conversion of Provisional Operating License No. DPR-45 for LACBWR to a full-term operating license was filed by the Dairyland Power Cooperative to the U. S. AEC.

3.4 SITE GEOLOGY AND SEISMOLOGY

(Reference 2)

LACBWR is situated within the Central Stable Region of the North American continent. This region includes the dense igneous and metamorphic rocks of the Canadian Shield and adjacent early Paleozoic sedimentary strata. The geological structure of the Central Stable Region is relatively simple. Very little structural activity other than uplift and subsidence has occurred in this quiescent area since Proterozoic time.

The LACBWR facilities are situated on about 15 feet of hydraulic fill overlying approximately 100-130 feet of glacial outwash and fluvial deposits on the east flood plain of the Mississippi River Valley. The surface configuration of the underlying bedrock is unknown because of the relative paucity of bore hole data. The bedrock below the site consists of nearly flat-lying sandstones and shales of the Dresbach Group (Upper Cambrian). Dense Precambrian crystalline rock underlying these sedimentary rocks is estimated to be at a depth of approximately 650 feet.

The Safe Shutdown Earthquake was considered as the occurrence of a MM Intensity VI shock with its epicenter close to the site. The estimated maximum horizontal ground acceleration induced by this event would be less than 12 percent of gravity at the foundation level of the existing structures.

The liquefaction potential of the granular soils underlying the existing plant was analyzed by comparing the anticipated shear stresses due to the Safe Shutdown Earthquake with the shear stresses required to produce liquefaction at various depths.

The factor of safety with regard to liquefaction is defined as the ratio of the cyclic shear stress required to produce liquefaction to the average cyclic shear stress induced by the earthquake. The calculations were based on ten significant stress cycles. The results of the analysis indicate that the factor of safety with respect to liquefaction for the Safe Shutdown Earthquake is in excess of 1.47.

3.5 SEISMIC CLASSIFICATION

No specific seismic classification was mentioned in the available documents. However, seismic analyses have been performed in the 1974 analysis (Reference 2) for the following structures and components:

Reactor Containment Building (RCB)

RCB Sybsystems - water storage tank

- core support and primary reactor internals
- main steam piping system

LACBWR Stack

Genoa Stack

Waste Disposal Building

Turbine Building

Diesel Generator Building

Diesel Generator Piping System

Spent Fuel Storage Racks

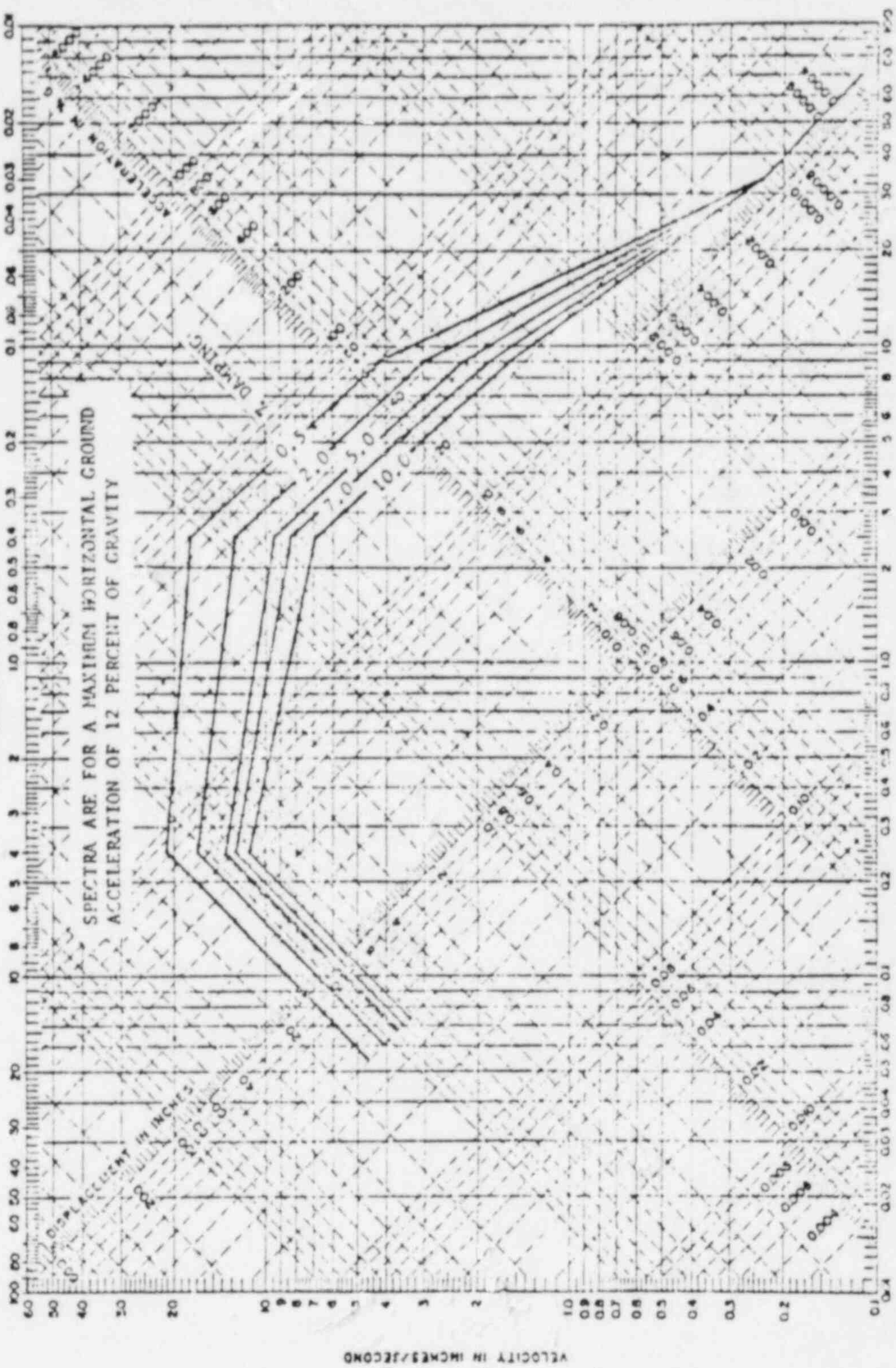
In the re-analysis of 1975 to 1977, main steam piping system (Reference 3), feedwater piping system (Reference 8) and recirculation piping system (reference 5) were analyzed for both the Safe Shutdown Earthquake (SSE) and the Operating Basis Earthquake (OBE). The stresses due to seismic dead weight, pressure and thermal expansion loadings, were combined according to the ASME code rules for Class 2 components. High pressure core spray suction line piping system (Reference 6) and discharge line piping system (Reference 7) were analyzed for SSE and OBE, and the stresses were combined according to the ASME code rules for Class 1 components.

The high density spent fuel storage racks (Reference 8) were designed to meet the requirements for Seismic Category I structures. The Spent fuel pool structure (Reference 9) and spent fuel storage pool drain line (Reference 10) were analyzed according to ASME, Class 1 components code design requirements.

3.6 BASIC SEISMIC DESIGN CRITERIA

(References 2, 3, 4, 5, 6, and 7)

In Section 2.4 of Part 1, Reference 2, it stated "In our selection of the Safe Shutdown Earthquake, we make the very conservative hypothesis that a shock similar in intensity to the 1934 earthquake at Rock Island, Illinois - MM Intensity VI - could occur at the site. We estimate that the maximum horizontal seismically-induced ground motion at the foundation level of the site resulting from such an event would be less than 12 percent of gravity. This value [illegible]es the amplification of bedrock acceleration through the natural overburden and hydraulic fill at the site". The response spectrum curves of the selected horizontal ground motion at the foundation level for SSE, with the peak acceleration normalized to 0.12g, is given in PLATE 8, reproduced from Part 1 of Reference 2. The report did not specify the origin of the spectral curves, however, these smoothed response spectra, apparently, agree with those recommended in U. S. AEC Regulatory Guide 1.60. No vertical ground motion was discussed or recommended in Dames & Moore study (Part 1, Reference 2). In the re-analysis of main steam, feedwater, HPCS piping systems, the vertical response spectrum was taken as 2/3 of the horizontal ground response spectrum assuming no amplification of vertical response in the structure.



RESPONSE SPECTRA
LACBWR
GULF UNITED

DAMES & MOORE

PLATE 8

Damping factors are listed in the table below.

	SSE	½ SSE
Reactor Containment Building	7% and up*	3% and up*
Turbine Building	7%	
STACKS	7% and up*	
New Diesel Generator Building	7%	4%
Piping	2%	1%

*Proportional damping

3.7 GENERAL DESIGN CRITERIA

3.7.1 Containment and Other Buildings (Reference 2)

Since seismic load was not included in the original design criteria at the time of plant construction, the seismic effect was not combined with other load cases in the design consideration. At the subsequent seismic re-analysis (Reference 2) only the seismic effect was considered. However, in computing the load capacity of pile foundation of the reactor containment building, the dead weight loading was included in the consideration (P. 4-10, Part 1, Reference 2).

The allowable structural capacities for RCB, two stacks, the turbine building and the waste disposal building, are summarized in the following chart:

	SSE	½ SSE
Concrete:		
Moment	Mu	0.63 Mu
Shear	Vu	0.60 Vu
Steel:		
Moment	My	0.66 My
Shear	0.53V	0.40V

where Mu is the ultimate moment capacity for the fully plastic section

My is the yield moment calculated at the bending moment required to just product a yield stress at the extreme fibers of the cross-section

Vu is the ultimate section shear capacity of the effective shear area of the reinforcement and the concrete

V is the yield capacity of the gross steel area

The structural capacities of subsystems and components were determined by the maximum elastic stress in each subsystem or components.

For the New Emergency Diesel Generator Building the proposed design criteria and load combination were summarized in two tables (Reference 11) which are reproduced in the following pages.

3.7.2 Piping Systems

(References 3, 4, 5, 6, and 7)

The requirements for acceptability of Class 1 piping systems (Reference 6 and 7) are those given in AEC Regulatory Position 1 and Subsections NB-3600 of Section III of the ASME Boiler and Pressure Vessel Code. Calculated stresses resulting from the design and operating loading conditions given in Subsection NB-3110 and NB-3620 must meet the stress limits of equations 9 through 14 of Subsection NB-3650 of the ASME Code.

Class 1 piping stress criteria are listed in the table below.

DESIGN CONDITIONS (Primary)	$Po + DL + E < 1.5\ Sm$ (as Eq. 9)
NORMAL CONDITIONS (Primary and Secondary)	$T^1 + Pt + SA + TA + E < 3\ Sm$ (as Eq. 10)
UPSET CONDITIONS	Same as for Normal Conditions
EMERGENCY CONDITIONS	Primary stress $< 2.25\ Sm$ (as Eq. 9)
FAULTED CONDITIONS	$Po + DL + E^1 < 3\ Sm$ (as Eq. 9)

TABLE 4.5-1 LOAD FACTORS–STRUCTURAL STEEL–ELASTIC DESIGN

Load Condition		Load		Thermal	Normal pipe reaction		OBE	SSE	Design Stresses
		D	L	T_o	R_o	W	E	E'	
Construction	1	1.0	1.0	1.0		1.0			1.33 AISC
Test	2	1.0	1.0	1.0	1.0				1.33 AISC
Normal	3	1.0	1.0	1.0	1.0				AISC
Severe	4	1.0	1.0	1.0	1.0	1.0			AISC
Environmental	5	1.0	1.0	1.0	1.0		1.0		AISC
Extreme Environmental	6	1.0	1.0	1.0	1.0			1.0	1.6 AISC ≤ 0.95 Fy

(1) If for any load combination, the effect of any load other than dead load reduces the stresses or reduces the net uplift when checking for the uplift condition, it shall be deleted from the load combination.

(2) Loads not applicable to a particular system may be deleted.

(3) For both E and E', the resultant effects (resultant stresses) for both horizontal and vertical earthquake forces shall be determined by combining the individual effects by the square root of the sum of the squares method.

(4) AISC allowables shall be in accordance with Part I of the 1969 AISC specifications, excluding section 1.5.6.

15.

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TABLE 4.5-2 LOAD FACTORS - REINFORCED CONCRETE-STRENGTH DESIGN

Load Condition		Load							Design Strength
		D	L	T_o	R_c	W	E	E'	
Construction	1	1.1	1.3	1.3		1.3			ACI 318-71
Test	2	1.1	1.3	1.3	1.3				ACI 318-71
Normal	3	1.4	1.7	1.3	1.3				ACI 318-71
Severe Environmental	4	1.4	1.7	1.3	1.3	1.3			ACI 318-71
	5	.9		1.3	1.3	1.3			ACI 318-71
	6	1.4	1.7	1.3	1.3		1.4		ACI 318-71
	7	.9		1.3	1.3		1.4		ACI 318-71
Extreme Environmental	8	1.0	1.0	1.0	1.0			1.0	ACI 318-71

(1) If for any load combination, the effect of any load other than dead load reduces the stresses or reduces the net uplift when checking for the uplift condition, it shall be deleted from the load combination.

(2) Loads not applicable to a particular system may be deleted.

(3) For both E and E', the resultant effects (resultant stresses) for both horizontal and vertical earthquake forces shall be determined by combining the individual effects by the square root of the sum of the squares method.

where P = Design pressure

P = Operating pressure

DL = Dead weight and other sustained mechanical loads

SA = Seismic anchor movements

TA = Thermal anchor movements

E = Operating basis earthquake (OBE or ½ SSE)

E = Safe Shutdown Earthquake (SSE)

T = Thermal loads maximum credible accident

Sm = Allowable design stress intensity at maximum temperature

The requirements for acceptability of Class 2 piping systems (Reference 3,4, and 5) are given in AEC regulatory position 8 and Section NC-3611 of Section III of the ASME Boiler and Pressure Vessel Code. Calculated stresses resulting from specified load combinations must meet the stress limits of equations 8 through 11 of the ASME code.

Class 2 piping stress criteria are listed in the table below.

NORMAL OPERATING CONDITIONS:		Po + DL < S_h (as Eq. 9)
		stress range due to T and SA < SA (as Eq. 10)
	or	Po + DL + stress range due to T < SA + S_h (as Eq. 11)
UPSET CONDITIONS:		In addition to the normal operating conditions
		P max + DL + E < 1.2 S_h (as Eq. 9)
FAULTED CONDITIONS:		P max + DL + E' + SA < 1.8 S_h (as Eq. 9)

where P max = Peak pressure

S_h = Basic material allowable stress at maximum temperature

SA = Allowable expansion stress range

3.7.3 Fuel Storage Pool Structure

(Reference 12)

The following design codes, regulatory guides and references have been used in the structural analysis of the fuel storage pool structure.

1. ACI 318-71 - "Building Code Requirements for Reinforced Concrete" American Concrete Institute.
2. Uniform Building Code, 1973 Edition.
3. USNRC Standard Review Plan, Section 3.8.4.
4. "USNRC Proposed Position for Review and Acceptance of Spent Fuel Storage and Handling Application."
5. Nuclear Energy Services, Inc. document NES 81A0544, Rev. 0. "Quality Assurance Program Plan for the LaCrosse Boiling Water Reactor Spent Fuel Storage Rack Design Program", March 1978.
6. George Winter, et al - "Design of Concrete Structures", McGraw Hill Book Company, 1964.

The allowable stress/load limits constitute the structural acceptance criteria used for each load combination are based on the ultimate strength design methods described in ACI-318-71.

3.7.4 Fuel Storage Racks

(Reference 8)

The design/analysis of spent fuel storage racks is based on the following design codes and regulatory guides.

1. A.I.S.C. Manual of Steel Construction, Seventh Edition, 1970.
2. USNRC Regulatory Guide 1.61, "Damping Values for Seismic Design of Nuclear Power Plants," October, 1973.
3. USNRC Regulatory Guide 1.92, "Combination of Modes and Spatial Components in Seismic Response Analysis, Rev. 1, February, 1976.

4. USNRC Standard Review Plan, Section 3.8.4.

5. USNRC Proposed Position for Review and Acceptance of Spent Fuel Storage and Handling Applications.

The allowable stress/load limits constitute the structural acceptance criteria used for each load combination are based on the elastic working stress design method of AISC.

4. SEISMIC ANALYSIS OF MAJOR STRUCTURAL SYSTEMS

4.1 REACTOR CONTAINMENT BUILDING

4.1.1 Description

(Reference 17, Sec. 6)

The containment building (Figs. 6.1 to 6.5 of Ref. 17) is a right circular cylinder with a hemispherical dome and semi-ellipsoidal bottom. It has an overall internal height of 144 ft. and an inside diameter of 60 ft., and it extends 26 ft. 6 in. below grade level. The shell thickness is 1.16 inc., except for the upper hemispherical dome, which is 0.60 in. thick.

The building contains most of the equipment associated with the nuclear steam supply system, including the reactor vessel and biological shielding, the fuel element storage well, the forced-circulation pumps, the shutdown condenser, and process equipment of the reactor water purification system, decay heat cooling system, shield cooling system, seal injection system, emergency core spray system, boron injection system, and storage well cooling system.

The containment building is designed to withstand the instantaneous release of all the energy of the primary system to the containment atmosphere at an initial temperature of 80°F, neglecting the heat losses from the building and heat absorption by internal structures.

The interior of the shell is lined with a 9-in. thick layer of concrete, to an elevation of 727 ft. 10 in., to limit direct radiation doses in the event of fission-product release within the containment building.

The containment building is supported on a foundation consisting of concrete-steel piles and a pile capping of concrete approximately 3 ft. thick. This support runs from the bottom of the semi-ellipsoidal head at about

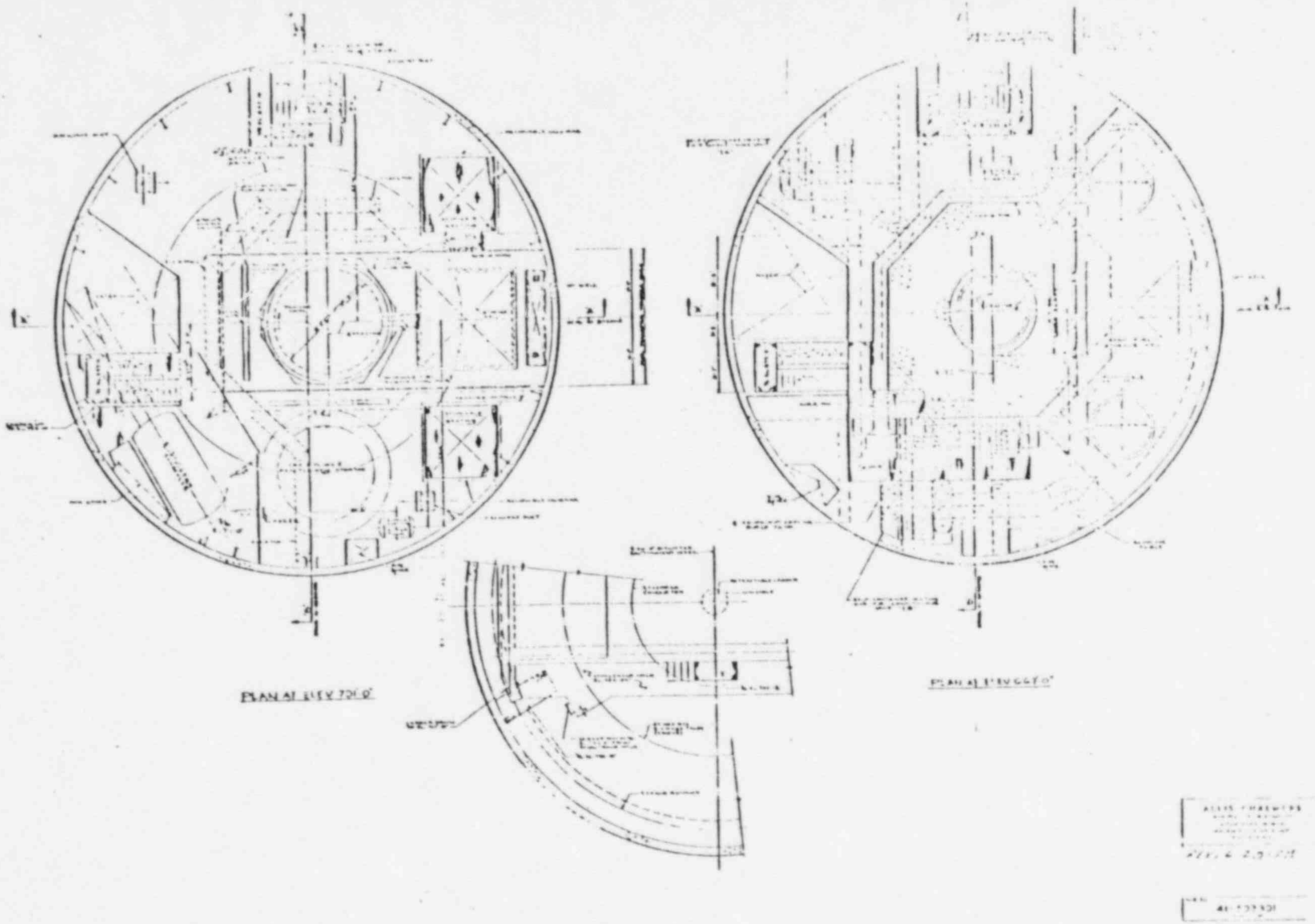
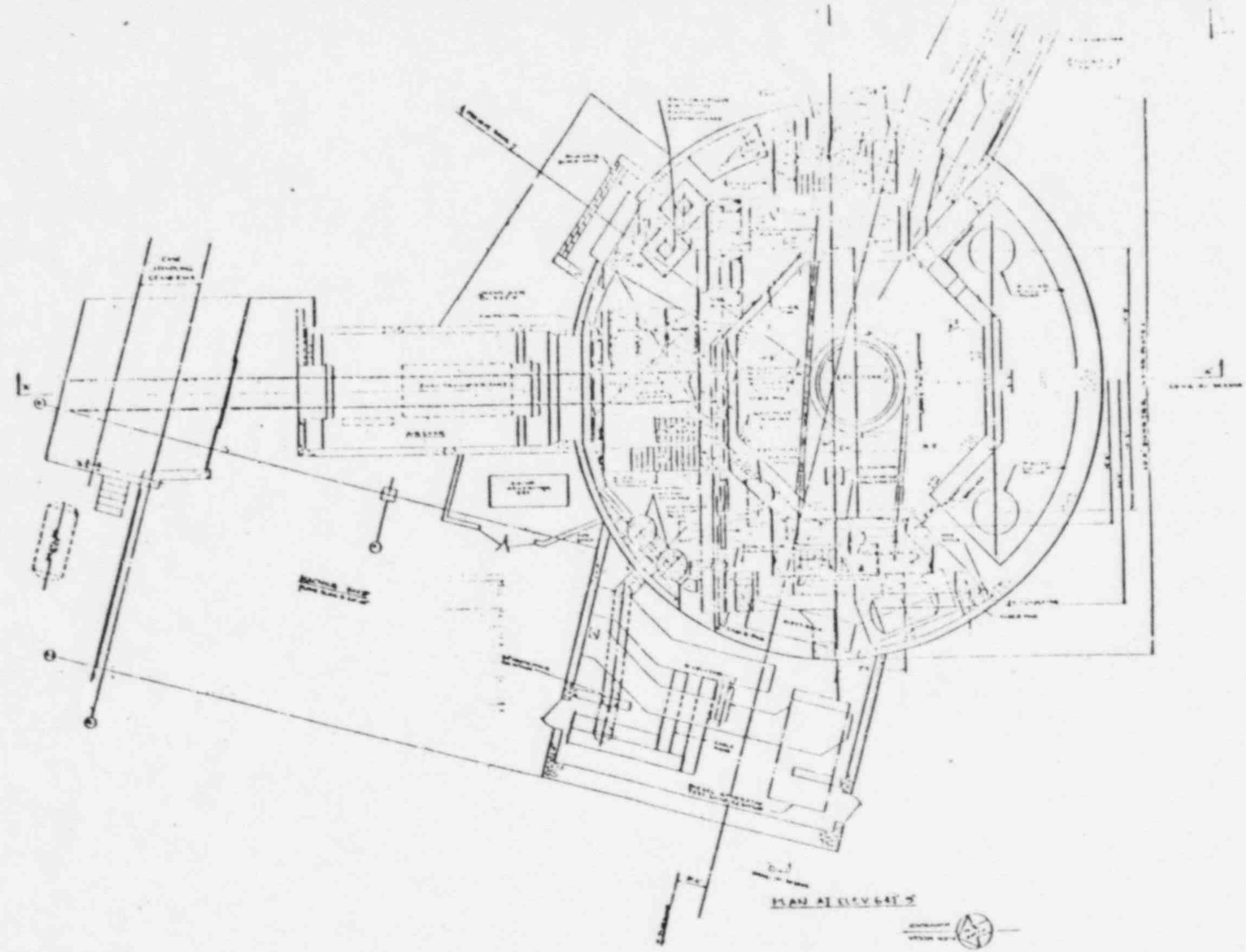
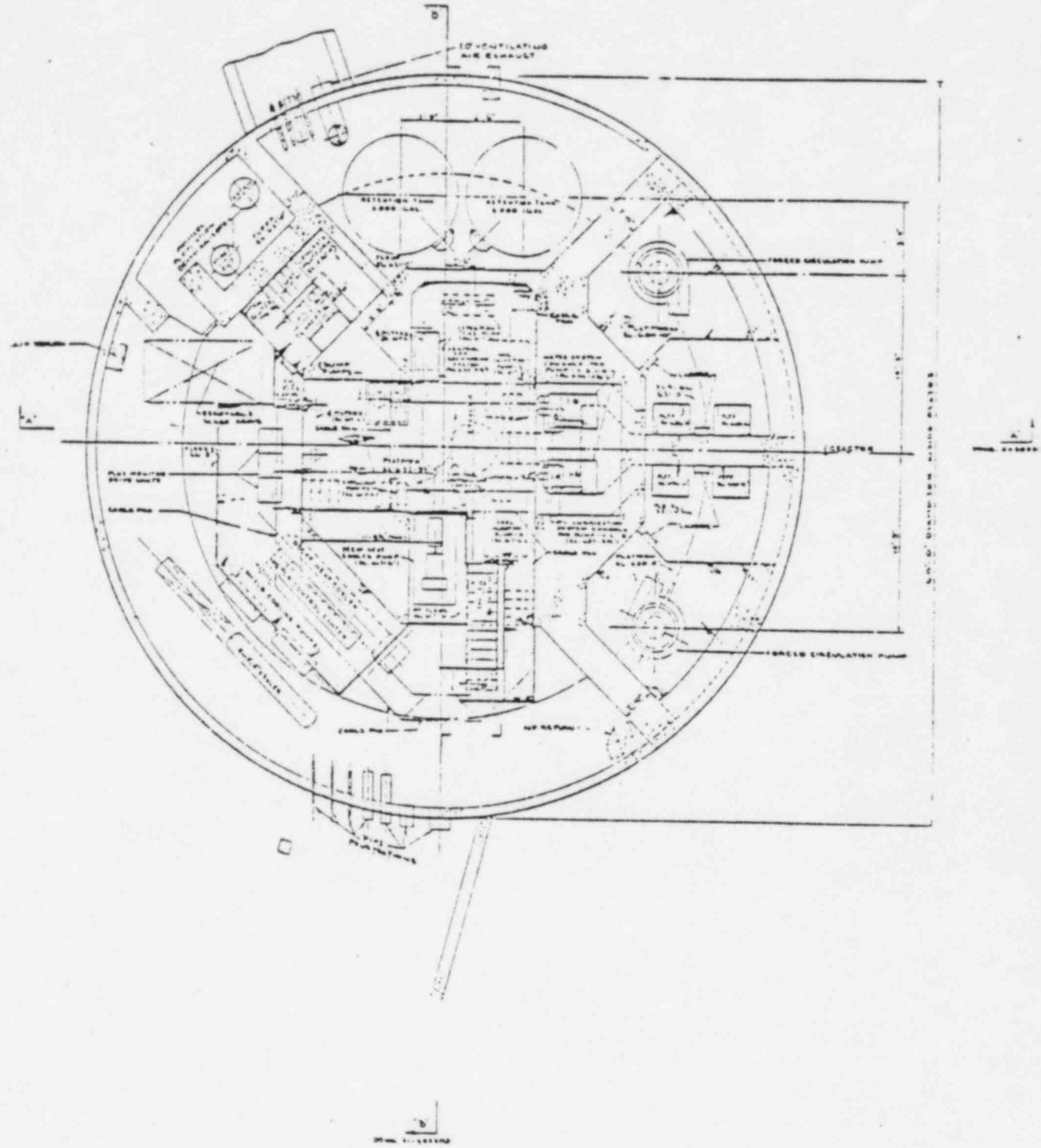


FIG. 6-1



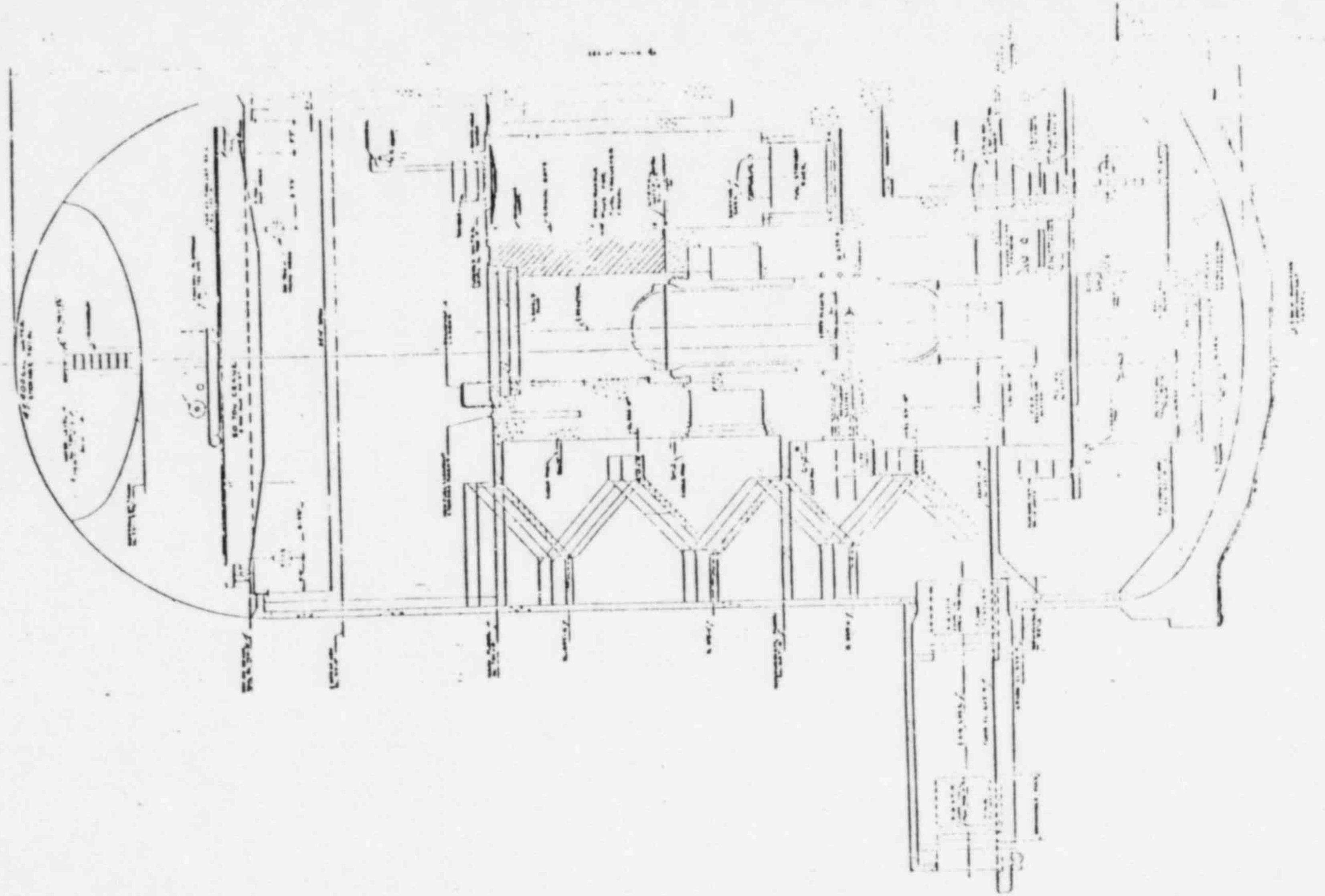
GENERAL ARRANGEMENT, GRADE FLOOR

FIG. 6-2



GENERAL ARRANGEMENT, BASEMENT FLOOR

FIG. 6.3



GENERAL ARRANGEMENT, SECTION A-A

REV. 6-28-78

FIG. 6.4

GENERAL ARRANGEMENT, SECTION B-B
REV. 6 [illegible]-78

FIG. [illegible]

el. 612 ft. 4 in. to an el. of 621 ft. 6 in. The 232 piles that support the containment structure are driven deep enough to support over 50 tons per pile.

The containment bottom head above el. 621 ft. 6 in. and the shell cylinder from the bottom head to approximately 9 in. above grade elevation (639 ft. 9 in.) are enveloped by reinforced concrete laid over a 1/2 in. thickness of premolded expansion joint filler. The reinforced concrete consists of a lower ring, mating with the pile capping concrete. The ring is ~ 4-1/2 ft. thick at its bottom and 2-1/2 ft. thick at a point 1-1/2 ft. below its top (owing to inner surface concavity). The ring then tapers externally to a thickness of 9 in. at the top (el. 627 ft. 6 in.), and the 9 in. thickness of concrete extends up the wall of the shell cylinder to el. 639 ft. 9 in. The filler and concrete are not used, however, where cavities containing piping and process equipment are immediately adjacent to the shell.

The shell includes two airlocks. The principal access to the shell will be through the personnel airlock that connects the containment building to the turbine building. The airlock is 21 ft. 6 in. long between its two doors, which are 5 ft. 6 in. by 7 ft. and are large enough to permit passage of a spent fuel element shipping cask. The containment building can also be evacuated, if necessary, through the emergency airlock which is 7 ft. long and 5 ft. in diameter, with two circular doors of 32-1/2 in. diameter (with a 30-in. opening). Both airlocks are at el. 642 ft. 9 in. and lead to platform structures from which descent to grade level can be made.

An 8 ft. by 10 ft. freight door opening in the containment building accommodates large pieces of equipment.

A 45,400 gal. storage tank in the dome of the containment building supplies water for the emergency core spray system and the building spray system. The piping connection to the emergency core spray system is on the bottom of the tank. The connection to the spray headers of the building spray system is a standpipe within the tank; the top of the standpipe is sufficiently above the bottom of the tank to leave 15,000 gal. of water for use in the emergency core spray system. The storage tank also provides water for use during refueling and during loading of the fuel element shipping cask.

A 50-ton traveling bridge crane with a 5-ton auxiliary hoist is located in the upper part of the containment building. The bridge completely spans the building and travels on circular tracks supported by a ring of concrete around the inside of the building just below the hemispherical upper head.

4.1.2 Loading Condition

(Reference 2, Part 2)

In the seismic evaluation of the LACBWR reactor containment building, the seismic event was not coupled with other loading conditions.

The seismic data was specified in the form of response spectra for the horizontal seismic inputs (see Section 3.6). This event is considered as the free-field motion at the ground surface of the site. A time history record was generated to envelop the two percent damped free-field response spectrum. The fitting procedure was conducted by employing mode suppression and raising techniques, with the Taft 1952 record as the chosen initial accelerogram. After several fitting trails, a final pulse was generated and the response spectra compared with the design criteria. The acceleration time history of the pulse is shown in Fig. 2.2 of Ref. 2. A comparison of the fits of the spectra of the generated accelerogram with the criteria response spectra were made, the two

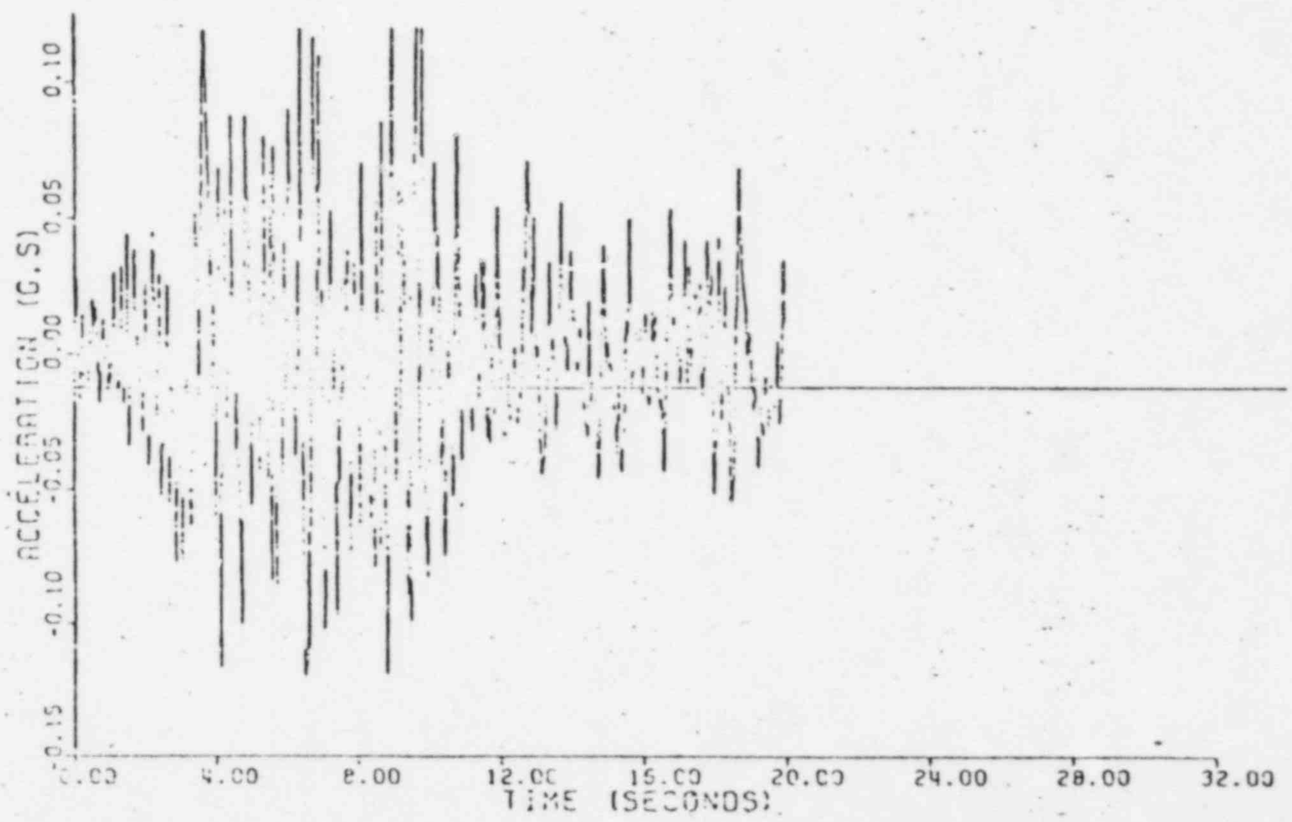


FIG. 2.2 — LACBWR CRITERIA HORIZONTAL ACCELEROGRAM, 0.12 G PEAK ACCELERATION

percent damped spectrum case is shown in the attached Figure 2.5 of Ref. 2. With the free-field time-history record, time-history records at different soil layer and bedrock were generated, using a shear beam model to model the coil layer system (see Fig. 2.14 of Ref. 2). This deconvolution process was performed basing on compatible root-mean-square values of the soil shear strain. Interation process was carried out in the frequency domain. Fig. 2.15 of Ref. 2 shows the response spectra at different soil layers for the two percent damping case. The motions so generated have different time histories at each soil layers. However, it seems from the report that the actual excitations used in the structural analysis of the RCB and the two stacks were the same motion as the surfact motion only the magnitudes had been scaled to match the peak accelerations found in the deconvolution study. This may be due to the limitation of the computer code, SIM, utilized in this study.

4.1.3 Structural Capacities

In checking for the adequacy of the primary structural system to the seismic inputs, the dynamic response of the system was computed and peak loads determined in each structural element. For the bending elements, the peak loads of interest are the peak moments and shears developed in the section while for the compression elements (three floor levels between the biological shield and the outer containment shell), peak compressive forces in the elements were computed. These peak values were then compared with the allowable values to ascertain if structural damage will occur.

The required capacities of the section were computed using the criteria described in Section 3.7.1.

For the pile foundation system, the yield moment capacity was defined as the moment which would develop the nominal pile bearing capacity (100 kips) at the outermost pile of the cluster. The loads on the interior piles then vary

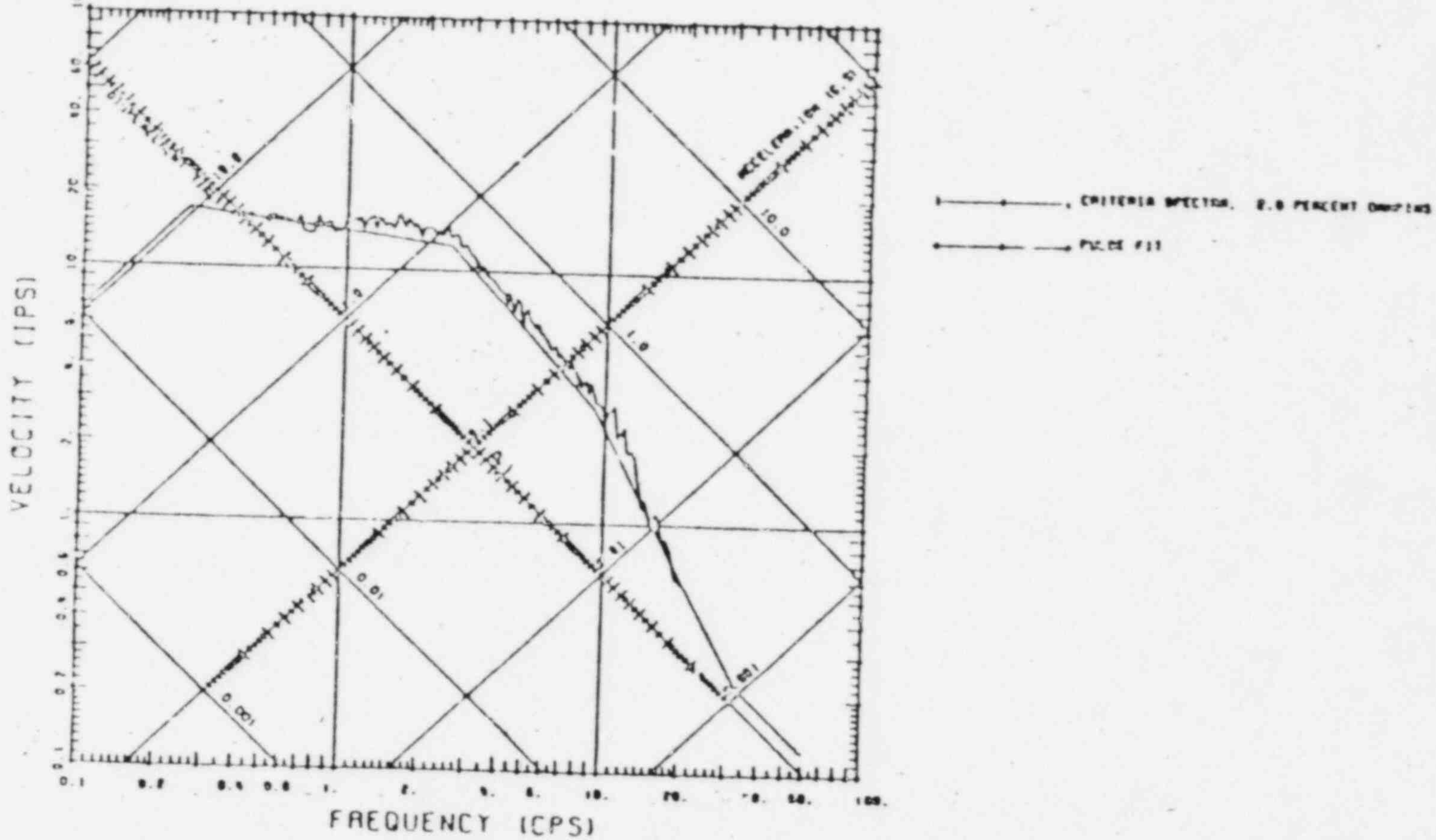


Fig. 2.5 — Response Spectra, LACBWR Horizontal Criteria Pulse, 2% Damping

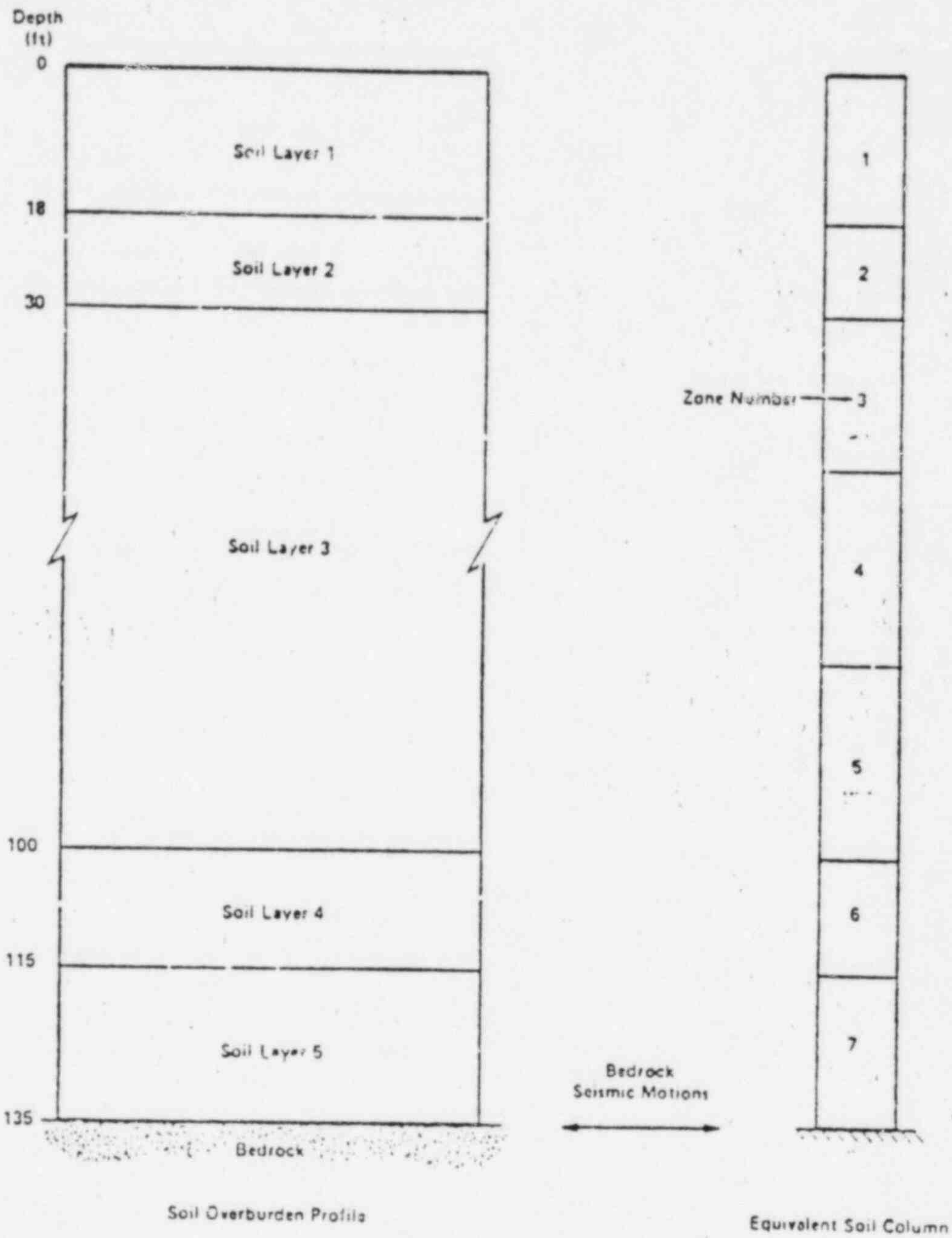


Fig. 2.14 – Overburden Configuration and Soil Shear Beam Model

2-25

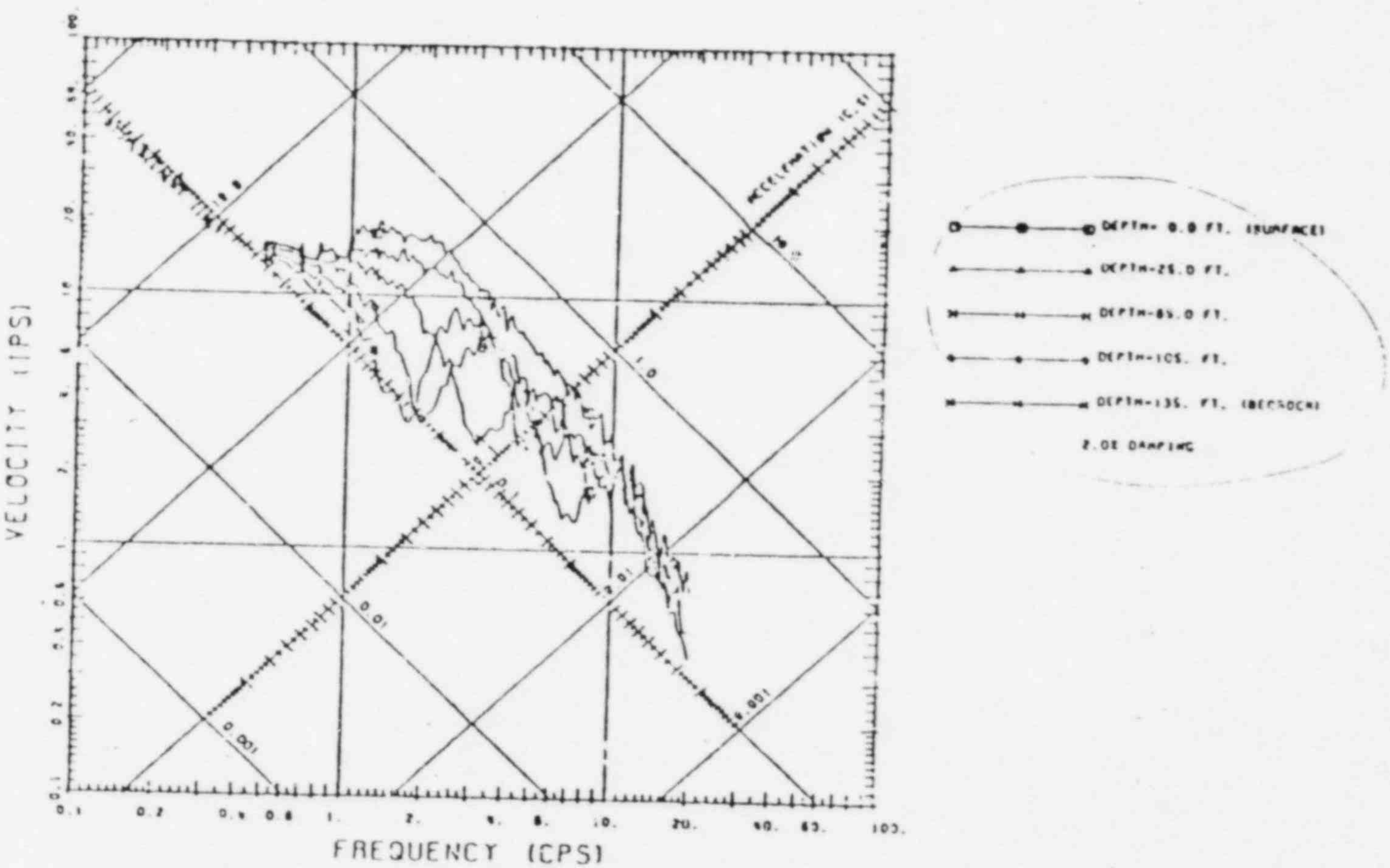


FIG. 2.15 — SPECTRA OF ASSOCIATED MOTIONS FOR SSE CRITERIA AT SURFACE

linearly with the distance from the center line of the pile cluster. The ultimate moment capacity was defined as the moment which would develop the ultimate pull-out capacity (estimated as 400 kips) in the outermost pile with the loads on the other piles scaled linearly as before. This pullout capacity also agrees with the approximate value of the ultimate bearing capacity of the pile.

The shear capacities indicated for all sections are the maximum allowable shears for the SSE seismic input condition. For the pile sections, the peak allowable shear is the shear strength of the individual pile times the number of piles.

For the concrete floor-sections, the peak capacities were determined by calculating the peak elastic shear stresses in the plate. This was then compared to the ultimate stress allowed for reinforced concrete sections. For the reactor skirt and biological shield supports, the capacities were determined from the criteria specified above for concrete and steel bending sections.

4.1.4 Method of Analysis

The containment building and its pile foundation were modeled by 36 lumped masses and a number of beams and inter-connecting elements (see Figure 4.1 of Ref. 2).

The soil structure interation effect was accounted for by including springs and dashpots at the nodes of the embedded pile system. The coefficients of the springs and dashpots were estimated from either half-space theory or a embedded cylinder study. The effect of the soil enclosed in the pile cluster was considered as added mass to the structure.

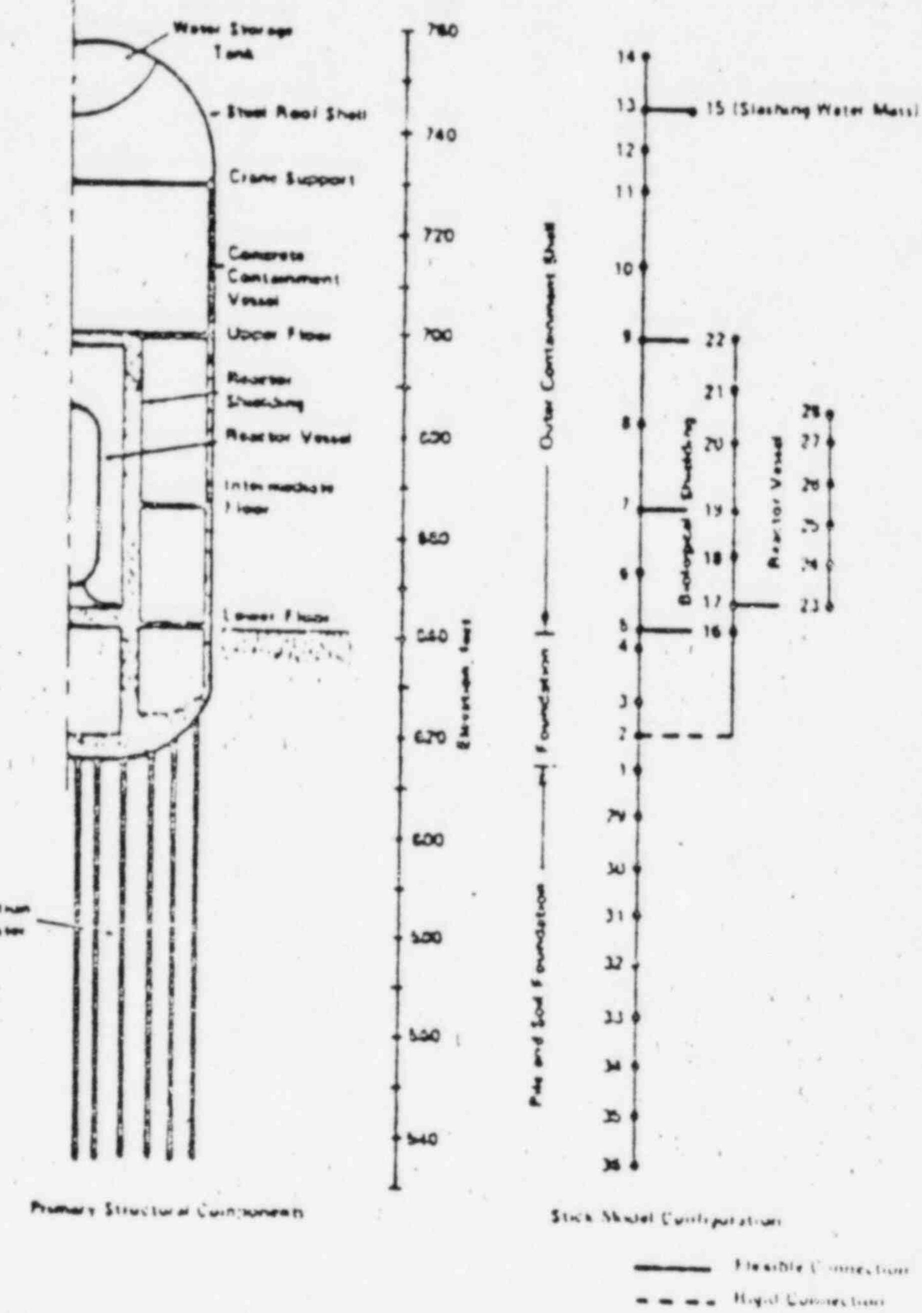


Fig. 4.1 – Model Used for Dynamic Analysis of Reactor Containment Building

4-32

The structural model includes:

1. the outer containment shell and its foundation
2. the biological shielding around the reactor
3. the reactor vessel
4. the upper water storage tank, and
5. the pile foundation system

The effects of the internals were included in this model only to the extent of adding mass to the primary structural system.

In addition to the flexible shear beam elements, six other interconnecting elements are represented in the model, these being:

1. the three floors (lower, intermediate and upper) connecting the biological shielding to the outer containment structure
2. The vertical support (4 concrete columns) of the biological shielding below the first floor
3. the circular steel skirt supporting the reactor vessel, and
4. a fictitious spring representing sloshing effects of the water mass in the upper storage tank

The floor stiffnesses (Connecting Elements No. 1, No. 2, and No. 3) were computed by determining the elastic response of a flat circular plate with a centered hole subjected to a lateral displacement of the hole. The details of this computation are presented in Appendix E of Ref. 2. The stiffness parameters of the biological shield column support (Connecting Element No. 4) were determined from standard beam theory, while the stiffness of the reactor skirt support (Connecting Element No. 5) was derived from shear beam solutions. The determination of the sloshing parameters (Connecting Element No. 6) is presented in Appendix F or Ref. 2.

The soil within the pile cluster was assumed to move with the pile cluster so that the pile soil system is taken as a circular structure to which the interaction loadings are applied. At the base of the pile, a base translation spring and damper was included to account for relative displacement between the pile tip and soil foundation below the pile tip. The moment spring and dashpot at the pile tip, however, were eliminated and it was assumed that the piles transfer their load primarily through pile friction.

To eliminate possibilities of numerical instabilities in the dynamic response analyses, the effects of the sloshing water mass were eliminated and the entire water mass was considered as a rigid mass (node 15 of Fig. 4.1 was lumped into node 13).

Two separate dynamic analyses were performed for the RCB, one using SSE criteria motion as input and the second one using 1/2 SSE criteria motion as input (the SSE criteria motion scaled by 0.5). Both analyses included only one horizontal component of time history excitation. The structural damping was represented by mass and stiffness proportional damping, the first mode had 7% damping for the SSE event and 3% for the 1/2 SSE event. The structural responses were solved by mode superposition and time integration technique.

For each problem, the integration was carried out for the full 20 seconds of the input pulse and node point motion-time histories generated. From the results, the associated node point response spectra were computed. These spectra being computed for 2% equipment damping for the SSE, and 1% equipment damping for the 1/2 SSE input condition.

4.1.5 Results of Analysis

The frequencies included in the dynamic analysis vary from 3.87 cps to 38.13 cps with frequencies above this range neglected for the structural computations. From the mode shape data it was noted that the first or lowest mode is associated with the rocking of the reactor vessel about its base support (reactor skirt support), while the second mode involves coupling of the reactor vessel motion and the other components of the structural system. The third mode involves primarily the outer containment shell motion while the higher modes indicate coupling between all the primary structural elements.

The dynamic stress results were checked with the ultimate strength design criteria in the SSE case and the working stress design criteria in the 1/2 SSE case.

On the basis of the results generated, the following conclusion was reached.

All the primary structural components of the RCB are adequately designed to withstand the specified SSE and 1/2 SSE input conditions. These components include the outer containment shell, biological shielding, reactor vessel and the pile support structure.

4.2 STACKS

(Reference 2)

Two stacks, the LACBWR stack and the Genoa III stack, were analyzed for seismic evaluation.

4.2.1 Loading Condition

The stacks were analyzed using the same SSE as input with a peak acceleration 0.12 g's, as described in Sec. 4.1.2 for the reactor containment building. A 7% structural damping valve was used for the structure.

4.2.2 Structural Capacities

In checking for the adequacy of the structure to the SSE seismic inputs, the dynamic response of the system was computed and peak moments and shears developed in each bending element. These peak values were then compared with the allowable capacities to ascertain if structural damage will occur. The required capacities of the sections were computed using the criteria described in Section 3.7.1.

For the pile system, the yield moment capacity was computed by assuming that the outermost pile developed its nominal capacity (100 kips), with the remaining piles sustaining loads proportional to the distance of the pile from the center-line. The ultimate moment capacity was computed by assuming that the outermost pile sustained a load equal to its tension pull-out capacity.

4.2.3 Method of Analysis

The structural model used to analyze the response of the LACBWR Stack is shown in Fig. 5.1 of Ref. 2. This model includes both the 350 foot stack and the 80 foot pile foundation system. The lumped mass model of the system is also shown in Fig. 5.1 and is composed of 23 lumped mass points connected by means of flexible shear beam elements.

For the pile soil foundation system, the soil within the pile cluster was assumed to move with the pile system, but provide no additional stiffness. Thus the soil within the pile group was assumed to add mass to the pile nodes only. For this stack, the pile cluster was composed of 78 piles, 80 feet long, each with a nominal pile capacity of 50 tons.

The system damping was taken as proportional to the mass and stiffness matrices of the structural model which for the higher modes yields increasing values of damping.

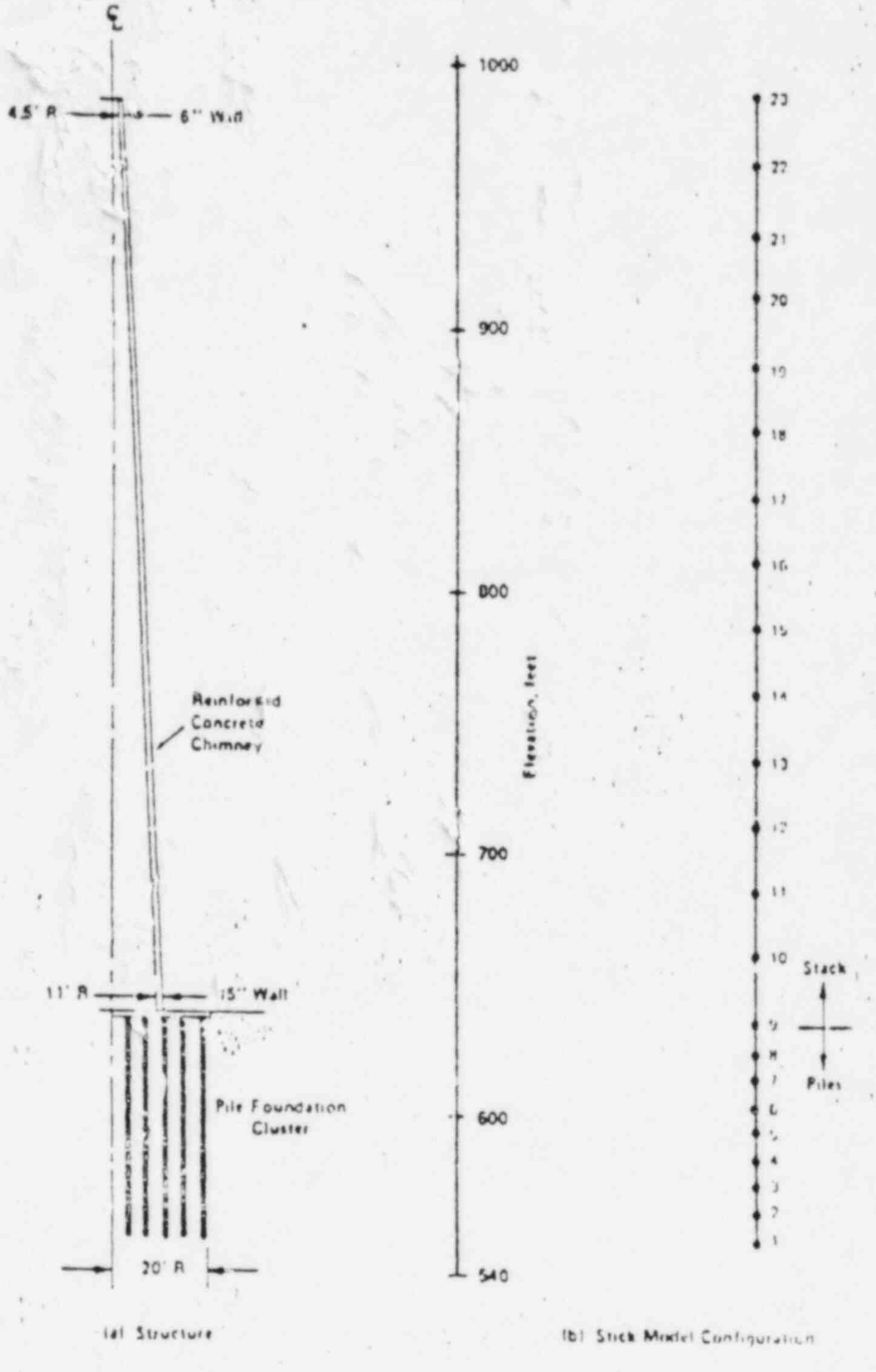


Fig. 5.1 – Model for Dynamic Analysis of LACBWR Stack

The soil within the pile cluster was assumed to move with the pile cluster so that the pile soil system is taken as a circular structure to which the interaction loadings are applied.

4.2.4 Structural Response

The frequencies and modal damping are tabulated in Table 5.3 of Ref. 2.

The peak shear and moment ratios developed for the SSE seismic input condition are tabulated in Table 5.5 of Ref. 2. As may be noted, the developed shear forces in the stack are all significantly less than the ultimate shear strengths. However, the peak moments developed throughout the stack are all significantly larger than the ultimate moment capacities of the sections clearly indicating that the stack cannot sustain the specified SSE criteria seismic inputs. Results in the piling system indicate no particular difficulties. Peak moments developed are less than the yield moment, or the moment which would just develop the yield capacity of the outermost pile.

A seismic analysis of the Genoa III Stack was also conducted for the SSE seismic input condition, using the SIM Code soil/structure interaction analysis. The analysis performed was similar to that conducted for the LACBWR Stack. The structural model used for the dynamic analysis is shown in Fig. 6.1 of Ref. 2 with the lumped mass model for consisting of 29 mass points. As can be noted from the free-field beam frequencies listed in Table 6.3, the taller Genoa III Stack is a lower frequency system than the LACBWR Stack.

Using the same procedures as described for the LACBWR Stack, the moment and shear capacities for the Genoa III Stack were computed and are shown in Table 6.4. The results for peak moment and shear ratios developed for the SSE input condition are tabulated in Table 6.5 and, as for the LACBWR Stack, indicate that the stack cannot sustain the seismic event due to the low moment capacities of the reinforced concrete sections.

TABLE 5.3 – FREE-FREE BEAM MODAL DATA, LACBWR STACK

Mode No.	Frequency (cps)	Mode Mass (k, ft, sec, units)	Modal Damping for SSE Input (%)
1	1.00	4.503	7.00
2	2.36	5.381	7.00
3	4.29	4.328	10.078
4	6.95	5.374	15.156
5	10.12	6.103	21.518
6	12.71	23.480	26.798
7	15.42	9.497	32.362
8	19.29	11.940	40.334
9	21.96	34.892	45.853
10	24.92	11.752	51.969

TABLE 5.5 — PEAK MOMENT AND SHEAR RATIOS, LACBWR STACK, SSE SEISMIC INPUT

Structural Unit	Element No.	Connecting Nodes	Max. Moment (k-ft)	Max. Shear (kips)	Moment Ratio M_{max}/M_u	Shear Ratio V_{max}/V_u
Pile	8	8-9	48,400	341	0.346	0.015
Stack	9	9-10	48,467	309	1.790	0.129
	10	10-11	48,467	377	1.763	0.172
	11	11-12	40,163	369	1.475	0.198
	12	12-13	32,425	347	1.289	0.205
	13	13-14	28,708	319	1.318	0.208
	14	14-15	24,478	289	1.313	0.209
	15	15-16	19,828	252	1.282	0.205
	16	16-17	15,883	208	1.317	0.190
	17	17-18	12,431	169	1.414	0.176
	18	18-19	9,743	145	1.659	0.174
	19	19-20	7,162	134	2.895	0.186
	20	20-21	4,598	105	3.273	0.177
	21	21-22	2,435	91	2.218	0.181
	22	22-23	760	67	0.917	0.104

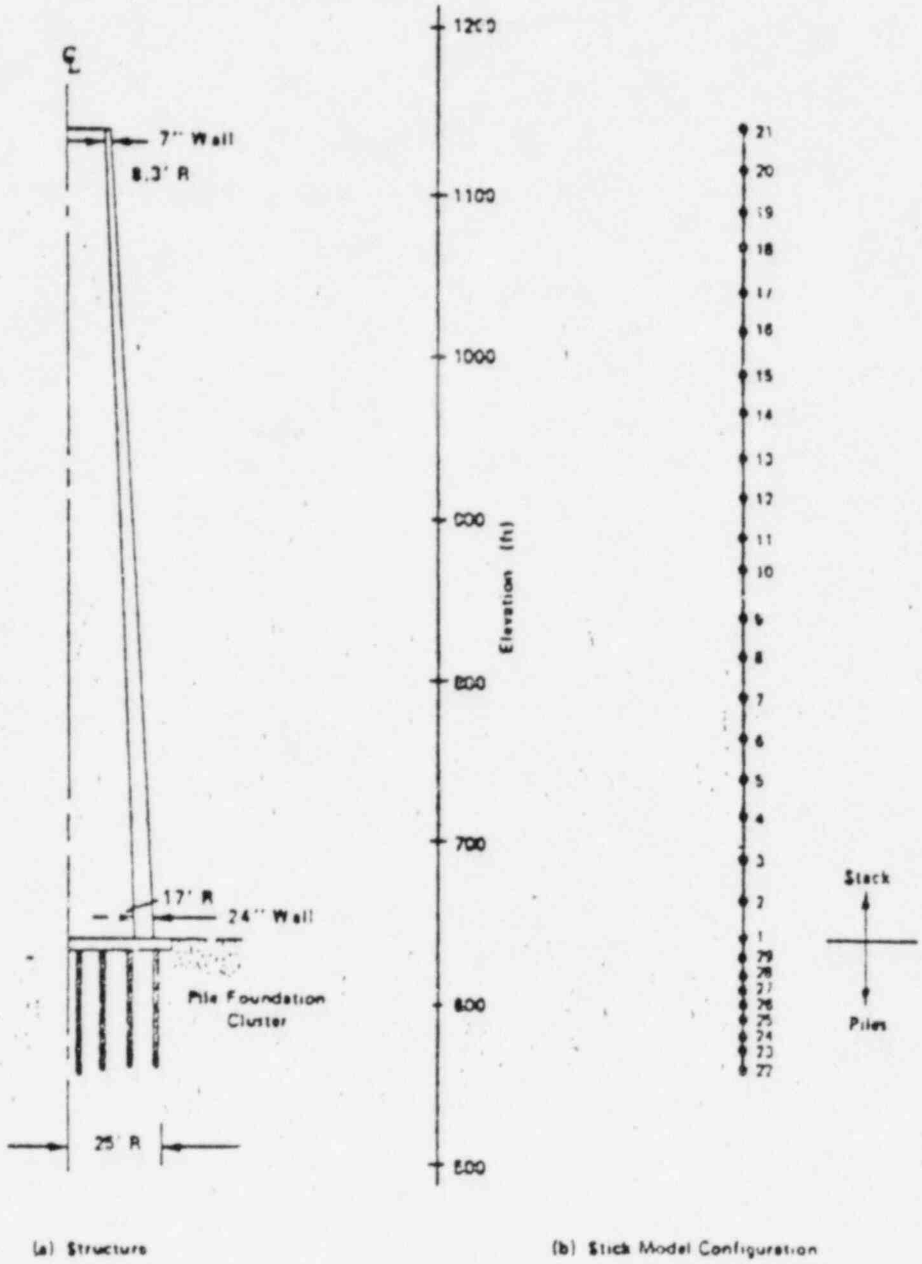


Fig. 6.1 — Model for Dynamic Analysis of Genoa Stack

TABLE 6.3 – FREE-FREE BEAM MODAL DATA, GENOA STACK

Mode No.	Frequency (cps)	Mode Mass (k, ft, sec. units)	Modal Damping for SSE Input (%)
1	0.79	17.289	7.00
2	2.26	20.495	7.00
3	4.26	23.987	10.734
4	6.58	27.060	15.713
5	9.26	25.905	21.686
6	12.31	29.032	28.578
7	15.23	52.500	35.194
8	17.61	47.758	40.628
9	20.84	36.451	47.993
10	24.00	75.006	55.209

TABLE 6.5 — PEAK MOMENT AND SHEAR RATIOS, GENOA STACK, SSE SEISMIC INPUT

Structural Unit	Element No.	Connecting Nodes	Max. Moment M_{max} (k-ft)	Max. Shear V_{max} (kips)	Moment Ratio M_{max}/M_u	Shear Ratio V_{max}/V_u
Stack	1	1-2				
	2	2-3	167,060	775	1.471	0.115
	3	3-4	151,493	878	1.385	0.111
	4	4-5	134,671	697	1.[illegible]48	0.175
	5	5-6	117,237	705	1.318	0.200
	6	6-7	101,829	609	1.257	0.209
	7	7-8	92,808	648	1.263	0.214
	8	8-9	83,820	581	1.303	0.208
	9	9-10	74,965	503	1.559	0.271
	10	10-11	66,209	411	1.738	0.240
	11	11-12	58,121	367	2.139	0.230
	12	12-13	50,105	346	2.802	0.225
	13	13-14	41,953	331	3.299	0.223
	14	14-15	33,789	317	4.109	0.226
	15	15-16	25,950	2[illegible]3	3.462	0.217
	16	16-17	19,219	257	2.744	0.198
	17	17-18	13,643	208	2.122	0.167
	18	18-19	8,821	172	1.479	0.144
	19	19-20	4,546	127	0.825	0.111
	20	20-21	1,364	64	0.273	0.050
Pile, Foundation	21	21-1				

6-6

4.3 TURBINE BUILDING

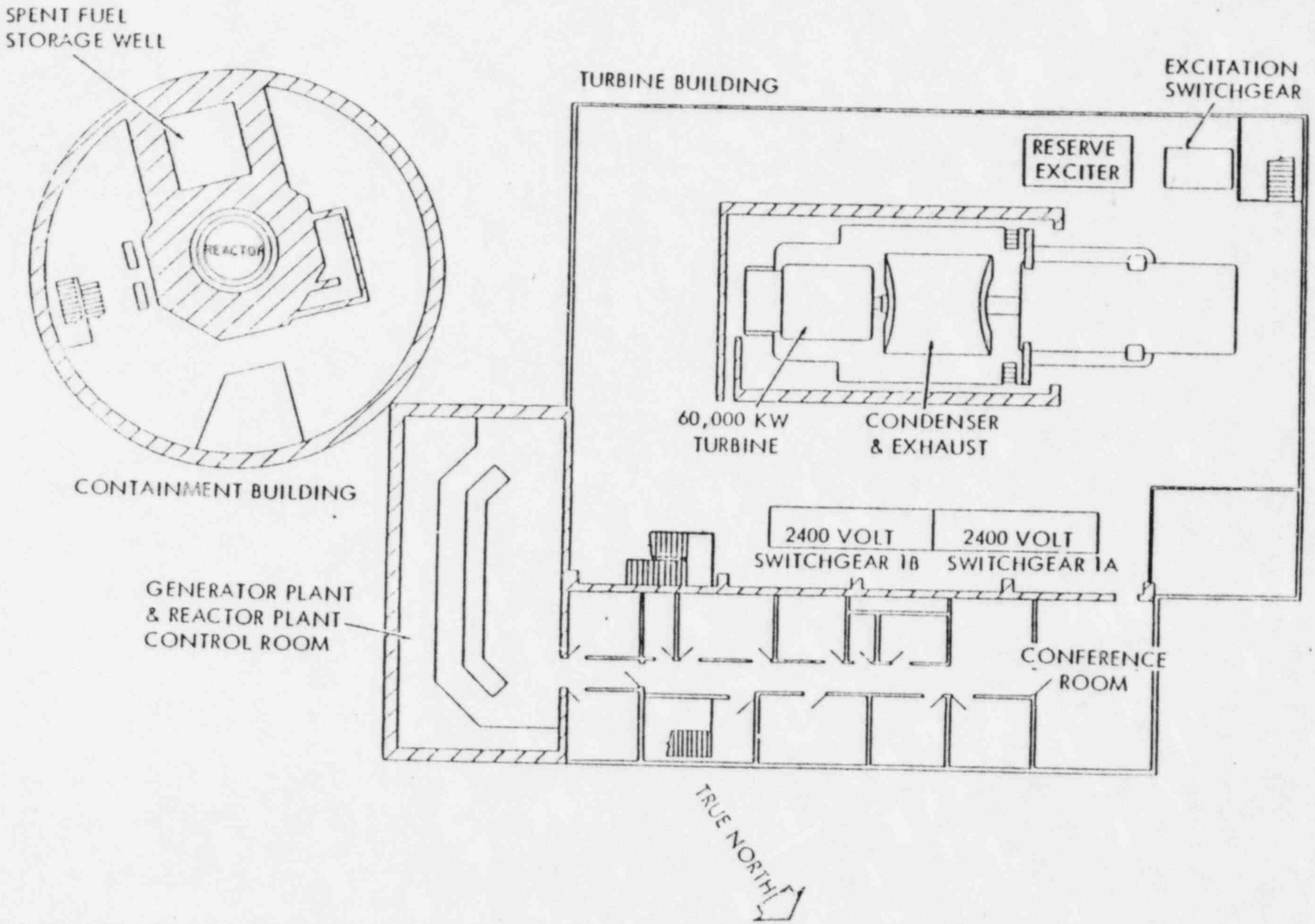
(Reference 17, Sections 7 & 10, Reference 2)

4.3.1 Description

The Turbine Building contains a major part of the power plant equipment (Fig. 7.1, Fig. 10.1 to 10.3 of Ref. 17). The turbine-generator is on the main floor. Other equipment is located below the main floor. This equipment includes the feedwater heaters, reactor feedwater pumps, air ejector, vacuum pump, full flow demineralizers, off-gas compressor and cooler, condensate pumps, air compressors, air dryer, oil purifier, service and component cooling water coolers and pumps, make-up water demineralizer system, domestic water heater, turbine oil reservoir, oil tanks and pumps, turbine condenser, unit auxiliary transformer, 2400-volt and 480-volt switchgear, motor control centers, diesel engine-generator set, emergency storage batteries, inverter set and other electrical, pneumatic, mechanical and hydraulic systems and equipment required for a complete power plant. A 30/5-ton capacity, pendant-operated overhead electric traveling crane spans the Turbine Building. The crane has access to major equipment items located below the floor through numerous hatches in the main floor.

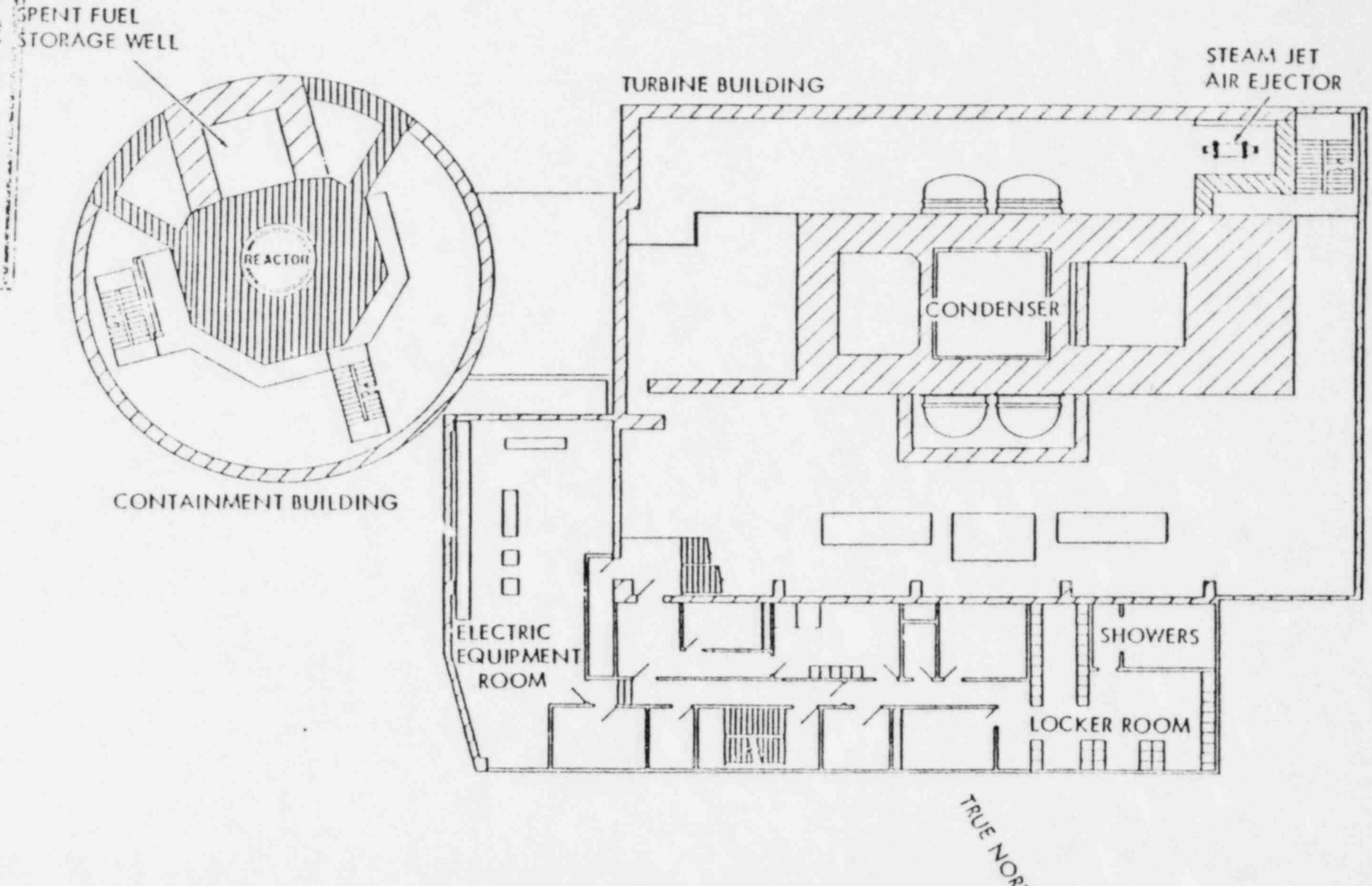
The Turbine Building also contains the main offices, the Control Room (for both turbine-generator and reactor), locker room facilities, laboratory, shops, counting room, personnel change, and decontamination facilities, heating, ventilating and air conditioning equipment, rest rooms, storeroom, and space for other plant services. In general, these areas are separated from power plant equipment spaces. The Control Room is on the main floor on the side of the Turbine Building that is adjacent to the Containment Building.

FIG 7.1



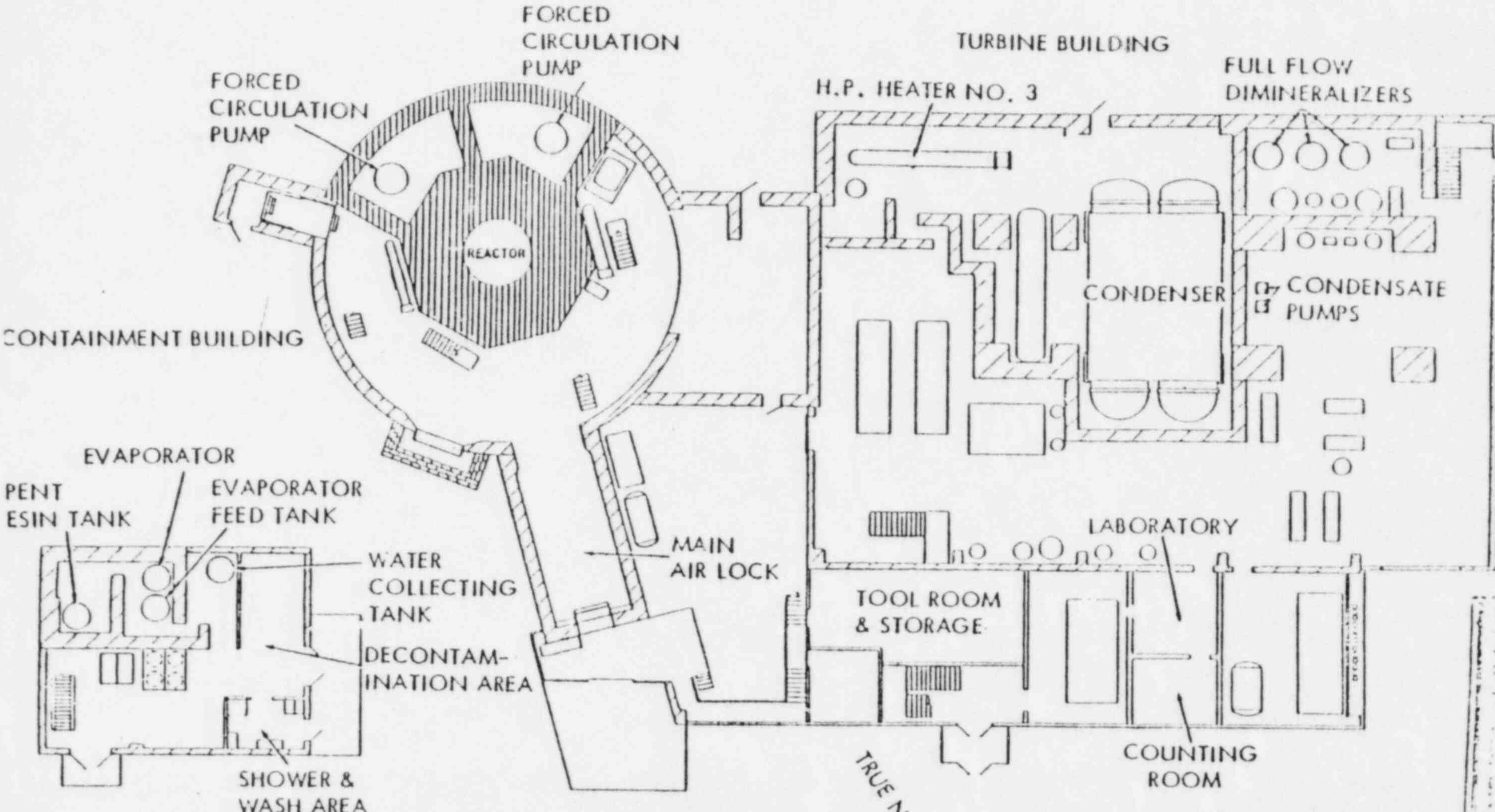
MAIN FLOOR OF TURBINE BLDG, EL. 668' -0"

FIG. 10.1



MEZZANINE FLOOR OF TURBINE BLDG, EL. 654' -0"

FIG. 10.2



GRADE FLOOR OF TURBINE BLDG, EL. 640' -0"

FIG. 10.3

A schematic plan of the Turbine Building is shown in Fig. 8.1 of Ref. 2. The support structure consists of a reinforced concrete floor/framing system supporting the turbine/generator unit together with a steel frame structure supporting the surrounding floor area as well as the demineralized water tank. The column lines for the framework are indicated in the Fig. 8.1. Both the turbine unit as well as the water tank are located at elevation 686 ft. which from Fig. 8.2 is the top floor of the four story structure.

A schematic diagram of the turbine/generator support structure is shown in Fig. 8.4 of Ref. 2. This structure is a reinforced concrete floor and column framing system.

4.3.2 Loading Condition

Response spectra methods of analyses were utilized to estimate the peak loads that had to be sustained, using the SSE criteria horizontal response spectra as input to the base of the building. Peak loads carried by both the steel and concrete framework were analyzed separately.

4.3.3 Method of Analysis

To estimate the peak loads carried by the outer steel framework, several different column lines were analyzed separately. In each case, the column line was simplified into an equivalent single degree of freedom system with the primary stiffness determined by the lateral bracing system. Computations indicated that the lateral stiffness of the bracing system was significantly larger than that of the main frame itself. The steel framing plan of Column Line 1 (Fig. 8.1) is shown in Fig. 8.2. Besides the floor loads carried at each level, this column line supports the demineralized water tank. The SDOF model for this column line is shown in Fig. 8.2, and indicates a fundamental

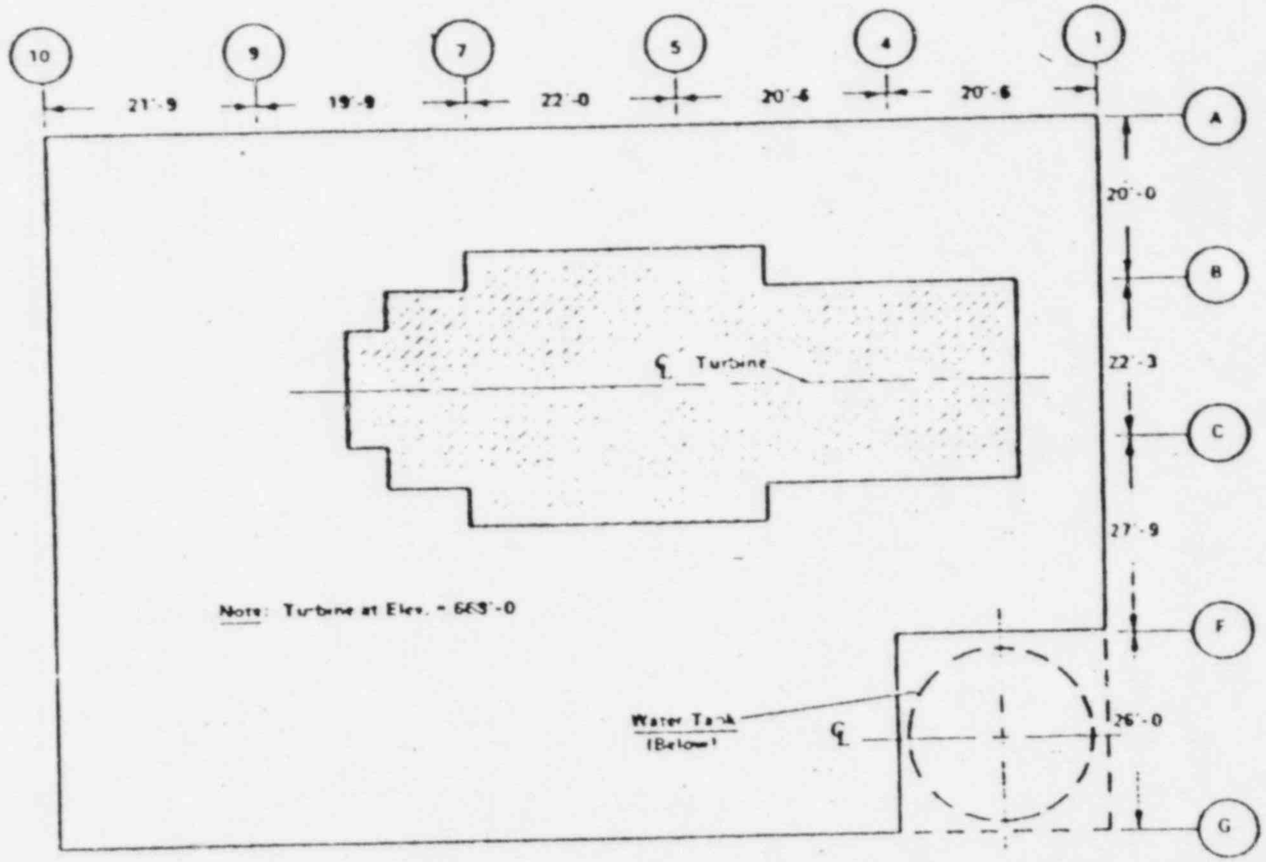
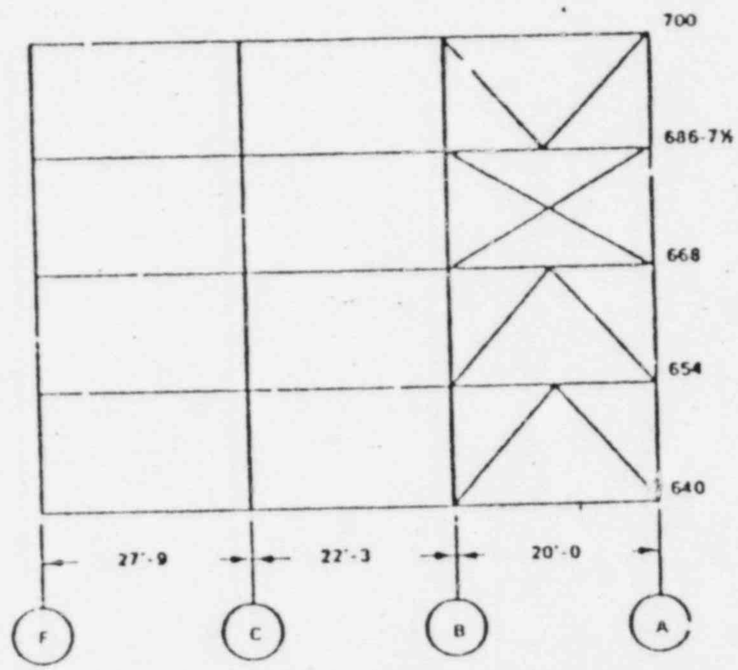
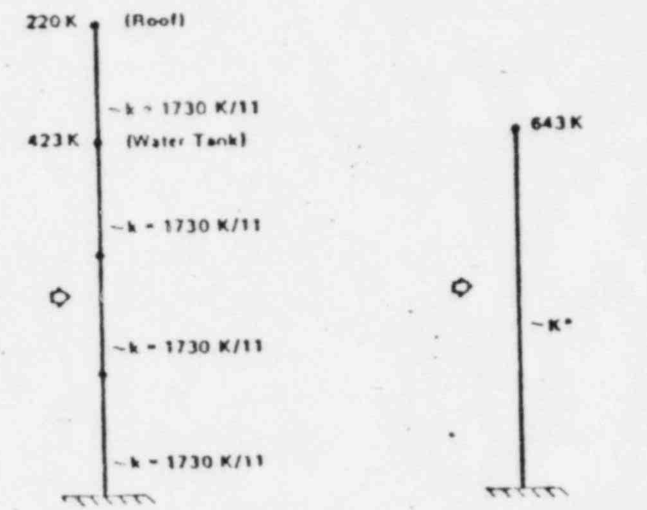


Fig. 8.1 — Plan for LACBWR Generating Plant





$$\frac{1}{k^*} = \Sigma\frac{1}{k}$$

$$\therefore k^* = \frac{1730}{3} = 577\ K/11$$

$$W^2 = \frac{k^*}{m_{total}} = 328$$

$$f = \frac{W}{2\pi} = \underline{2.9}\ \text{cps}$$

Fig. 8.2 — Dynamic Model of Column Line 1

frequency of 2.9 cps. The masses indicated were obtained by considering the weights of the roof together with that of the full water tank.

From the criteria spectra, the peak acceleration of the mass is found to be 0.3 g's using the SSE criteria spectra for 7% structural damping.

A similar model for Column Line 10 at the other end of the building is shown in Fig. 8.3. The two story framework is again simplified into a SDOF system as shown, with a fundamental period of 6.2 cps. Again the stiffness is primarily determined from the properties of the bracing system, while the mass is determined from the floor weights only.

The equivalent SDOF system of the turbine/generator support structure is also shown in Fig. 8.4, with the mass representing the turbine/generator weight only.

4.3.4 Results of Analysis

The computation showed that the turbine support structure was adequate for the SSE event but the steel frame structure needed additional lateral bracing system.

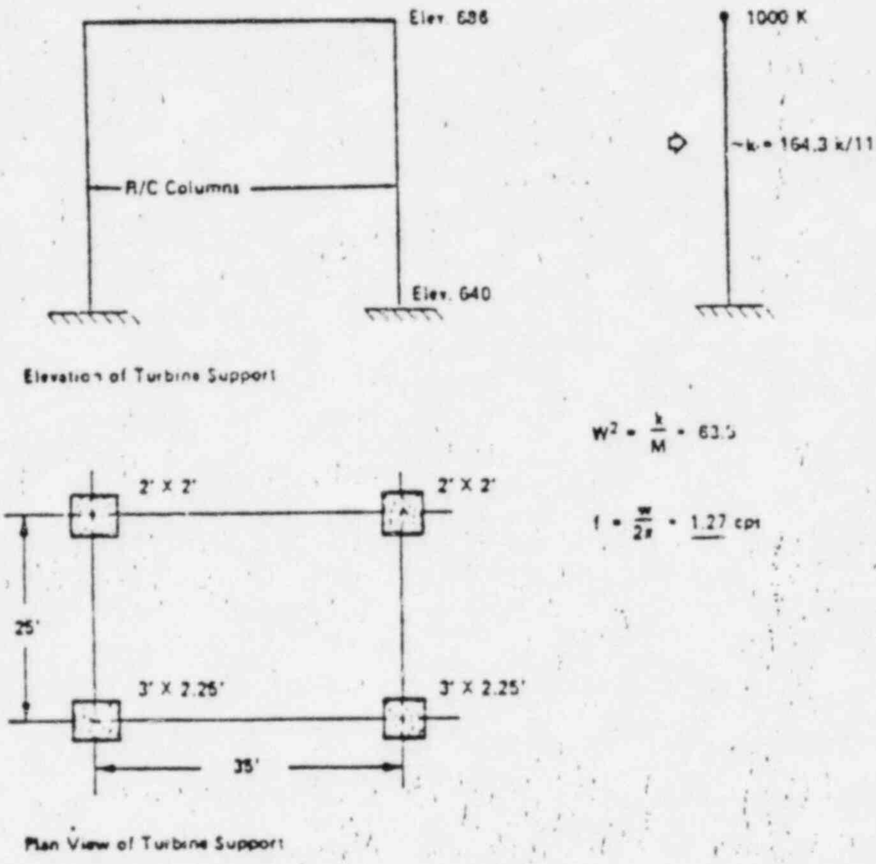


Fig. 8.4 — Dynamic Model of Turbine Support

8-C

5. SEISMIC ANALYSIS OF PIPING SYSTEMS AND THEIR SUPPORTS

5.1 MAIN STEAM LINE

(Reference 3)

5.1.1 Description

The main steam piping system within the containment shell carries steam from the reactor vessel to either the turbine building or the shutdown condenser. Steam is withdrawn from the reactor vessel through two 8-inch steam lines leading to a single 10-inch line. The steam passes out of the biological shield and through a rotoport steam isolation valve in the 10-inch line before leaving the containment shell for the turbine building. Within the biological shield, the 10-inch steam line branches upward and out of the biological shield to the main steam safety valves. The line then continues upward in the form of a 6-inch line to the shutdown condenser via a redundant system of control valves.

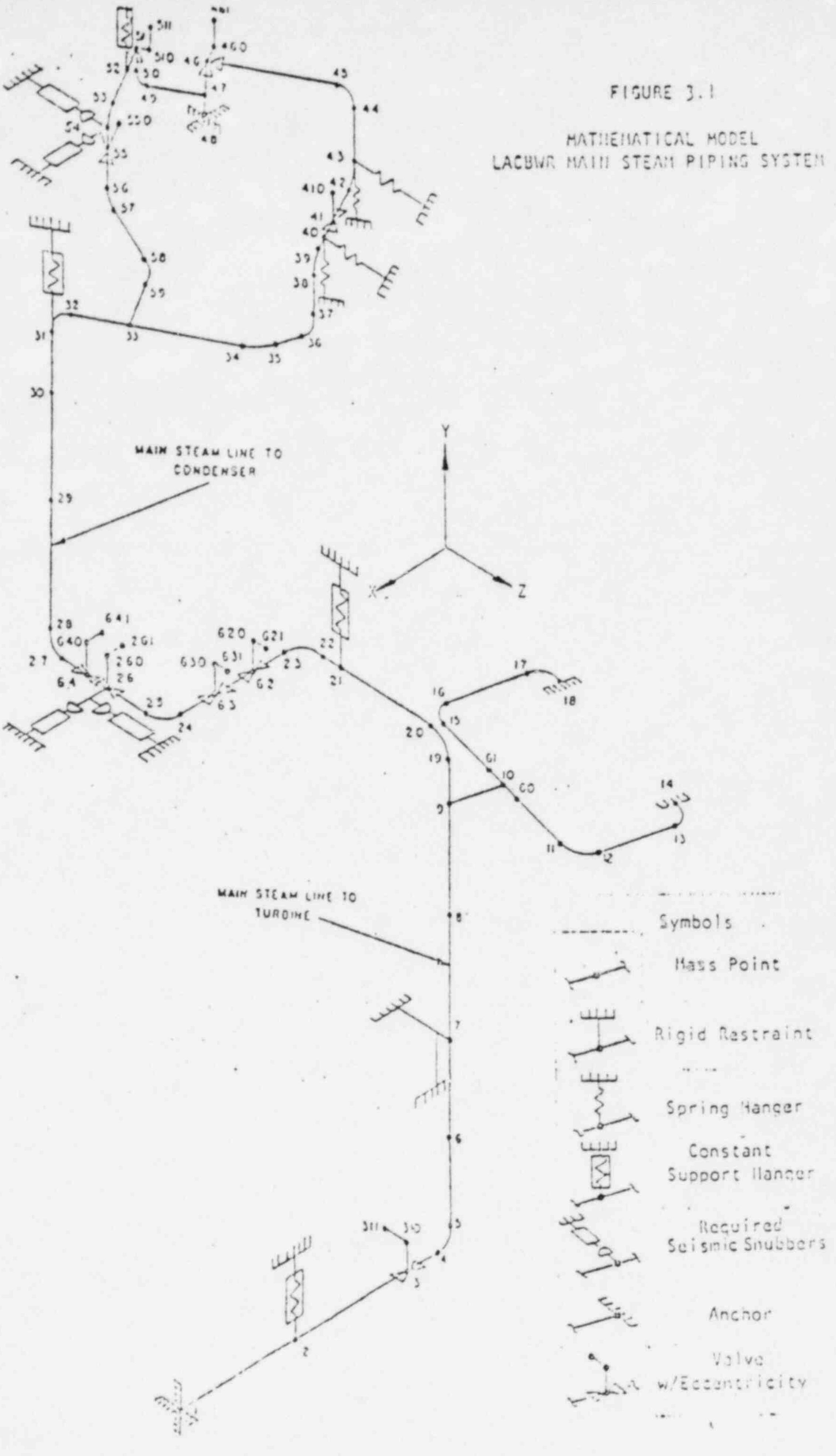
An isometric drawing showing the main steam piping system as analyzed, including the suspension system and recommended seismic snubbers, is given in the attached Figure 3.1 of Reference 3.

In order to verify that the seismic stresses are acceptable, it is necessary to show that the combined stresses in the piping system are within ASME Boiler and Pressure Vessel Code allowable values. This requires that the seismic stresses be combined with the stresses due to deadweight, pressure and thermal loadings in accordance with the ASME Code Section III rules.

The rules for a Class 1 (Section III) analysis require that thermal stress and fatigue due to thermal cycling be considered. A review of the available mainsteam piping system flexibility and stress analyses indicated that only thermal expansion was considered together with the pressure and deadweight loads in the original design. Consequently, it is not possible to perform a Class 1 analysis with the existing analytical data.

FIGURE 3.1

MATHEMATICAL MODEL
LACBWR MAIN STEAM PIPING SYSTEM



The existing analytical data, however, is sufficient to perform a Class 2 (Section III) analysis. Therefore, in the subject analysis, the adequacy of the main steam piping system to withstand an earthquake event is evaluated by combining the stresses due to deadweight, pressure, thermal and seismic loadings in accordance with ASME Code requirements for the design of Calss 2 components.

5.1.2 Loading Criteria

The load cases which must be considered in performing a Class 2 stress analysis include: dead loads and sustained mechanical loads, internal pressure, thermal expansion loading, seismic inertia loads and seismic anchor movement loading.

The dead weight of the piping system is calculated assuming the system to be insulated and filled with water. The weight of valves, valve operators, and branch piping are included in the analysis. Sustained loads imposed on the piping system by constant load hangers are also considered in the dead weight analysis. A value of 1300 psia for operating pressure and 1415 psia for peak pressure are used for most of the piping system. A peak pressure of 1300 psia is used in the section between the containment vessel and the rotoport isolation valve (Nodes 1 through 3).

The thermal expansion stresses are based on the thermal loading resulting from the normal operating temperature of 577.5°F.

Thermal anchor movements at the nozzle connections to the pressure vessel are calculated based on the thermal expansion of the pressure vessel at the design temperature of 577.5°F.

Two seismic loading events are considered: the safe shutdown earthquake (SSE), and the operating basis earthquake (OBE). The established design criteria Reg. Guide 1.48, May 1973) for Class 2 analysis considers the OBE (or ½ SSE) to be the normal and upset condition while the SSE is considered the faulted condition.

Seismic inertia loading is imposed on the piping system in the form of seismic acceleration spectra which were derived for the LACBWR plant (Reference 2). The horizontal acceleration spectrum used for the main steam line is that corresponding to the reactor vessel at an elevation of 664.5 feet. The vertical response spectrum for the SSE loading is taken as 2/3 of the horizontal SSE ground response spectrum assuming no amplification of vertical response in the structure. For the operating basis earthquake the vertical piping response spectrum is taken as ½ of the SSE vertical response spectrum. Damping values used are one percent for the OBE and two percent for the SSE.

The horizontal spectra in either the global X- direction or the global Z- direction are applied simultaneously with the vertical spectra in the global Y- direction.

Seismically induced anchor movements for points 7, 14, 18, 40, 42, and 48 were estimated by calculating low frequency displacements from the containment vessel response spectra at the different anchor point elevations.

5.1.3 Stress Acceptance Criteria

a. Normal Operating Conditions

Under normal operating conditions, the combined stresses due to design pressure, weight, and other sustained loads must not exceed the basic material allowable stress at maximum temperature, S_h and the requirements of Equation 8, ASME CODE. Additionally, either the stress range due to thermal expansion and seismic anchor movements as calculated by Equation 10, ASME CODE, must not exceed the allowable expansion stress range S_A or the combined stresses due to design pressure, weight, other sustained loads and the stress range due to thermal expansion must not exceed the sum of S_A and S_h as required by Equation 11, ASME CODE.

b. Upset Conditions

The requirements for operation under upset conditions include compliance with the requirements of Equations 8, 10 and 11 as described above as well as Equation 9. Equation 9 requires that the combined stresses produced by peak pressure, live and dead loads, and those produced by occasional loads - in this analysis defined as the OBE earthquake - must not be greater than 1.2 times the allowable stress value S_h.

c. Faulted Conditions

During faulted conditions, the requirements of Equation 9 must be met using a stress limit of 1.8 S_h. For the purpose of satisfying this criteria, the faulted conditions are specified as peak pressure loads, live and dead loads, the SSE seismic inertia loadings and the seismic anchor movement loads associated with the SSE.

5.1.4 Methods of Analysis

In order to perform static, dynamic and stress analyses, the continuous piping system is mathematically modeled as an assembly of elastic structural elements interconnected at discrete nodal points (Figure 3.1). Nodal points are located at all points of interest in the piping system such as elbows, valves, anchorages, hangers, tee intersections, load points, all structural and material discontinuities, etc. This three dimensional multidegree-of-freedom model of the piping system is attached to the "ground" (structure) by means of rigid hangers, support springs, hydraulic snubbers and anchors. Stiffness characteristics of structural elements are related to the moment of inertia and the axial and effective shear area of the pipe cross section. The stiffness characteristics of the elbows and tee connections are modified to account for local deformation by using the flexibility factors given in the ASME CODE.

For the seismic analysis the distributed mass of the piping system is lumped at the system nodal points. Masses are lumped so that the lumped mass, multi-degree-of-freedom model represents the dynamic characteristics of the piping system. In order to reduce the number of dynamic degrees-of-freedom, only translational degrees-of-freedom are considered at each mass point (the masses associated with the rotational degrees-of-freedom are set to zero).

Special items such as valves and actuators are modeled by lumping their masses at an appropriate offset from the centerline of the piping system.

The response spectrum method was used to obtain the modal responses of all modes having natural frequency under 30 cycles per second. The total system response is then obtained by combining individual modal response valves by the square root of the sum of the squares method.

The above mentioned static, dynamic and stress analyses are carried out using the PIPESD computer code.

5.1.5 Results

1. The existing support system of the LACBWR main steam piping system is not adequate to withstand the specified seismic events.
2. The results of the subject analysis, which includes the effects of four additional seismic restraints, indicate that the deflections of the main steam piping system, due to dead weight, thermal expansion and seismic loading are nominal. In addition, the stresses resulting from these loadings, as calculated and combined in accordance with the rules given in Subarticle NC-3652 of Section III of the ASME Code satisfy the design requirements of Class 2 piping systems.
3. It was recommended that the main steam piping system be provided with four seismic restraints at the locations indicated in Figure 3.1.

5.2 FEED WATER PIPING SYSTEM

(Reference 4)

5.2.1 Description

The feedwater piping system returns condensate from the turbine building and feeds it directly to the force circulation suction header where the condensate is mixed with the recirculating coolant. Feedwater enters the containment building through an 8" line, passes through an 8" check valve and an 8" gate valve and flows into a manifold section. Two 6" lines connected to this manifold enter the biological shield and feed the water directly to the 16" forced-circulation suction header through four 4" nozzles. The condensate return line from the shutdown condenser is included in the analysis in order to account for its effects on the feedwater line. Condensate water from the shutdown condenser flows by gravity from a 6" to a 4" line and then through a parallel system of 4" control, check, and gate valves before entering the 8" feedwater line through a branch connection.

An isometric drawing showing the feedwater piping system as analyzed, including the suspension system and recommended seismic snubbers, is given in the attached Figure 3.1 of reference 4.

The rules for a Class I (Section III) analysis require that thermal stress and fatigue due to thermal cycling be considered. A review of the available feedwater piping system flexibility and stress analyses indicated that only thermal expansion was considered together with the pressure and deadweight loads in the original design. Consequently, it is not possible to perform a Class 1 analysis with the existing analytical data.

The existing analytical data, however, is sufficient to perform a Class 2 (Section III) analysis. Therefore, in the subject analysis, the addquacy of the feedwater piping system to withstand an earthquake event is evaluated by combining the stresses due to deadweight, pressure, thermal and seismic loadings in accordance with ASME Code requirements for the design of Class 2 components.

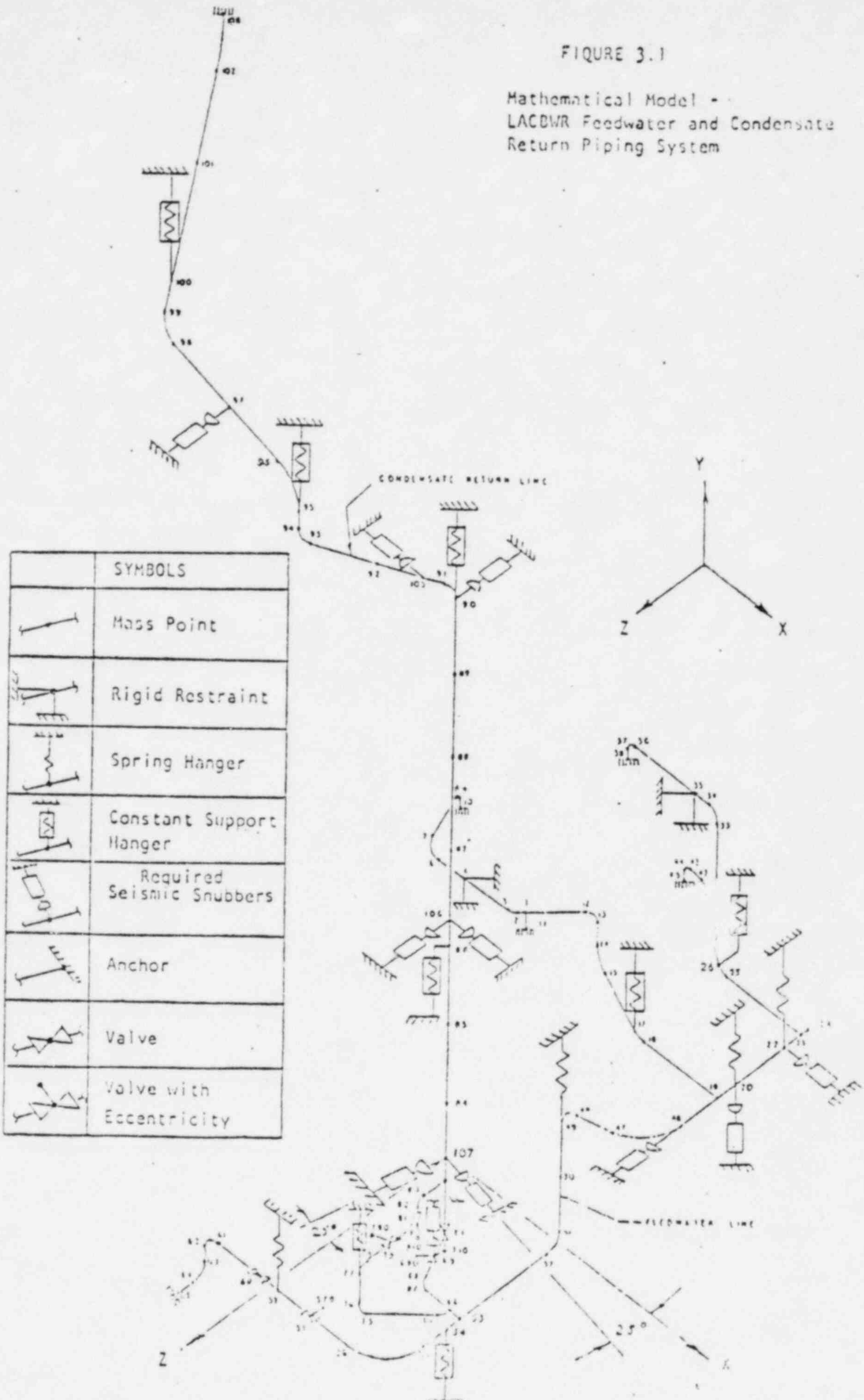


FIGURE 3.1

Mathematical Model - LACBWR Feedwater and Condensate Return Piping System

5.2.2 Loading Criteria

The load cases which must be considered in performing a Class 2 stress analysis include: dead loads and sustained mechanical loads, internal pressure, thermal expansion loading, seismic inertia loads and seismic anchor movement loading.

The deadweight of the piping system is calculated assuming the system to be insulated and filled with water. The weight of valves, valve operators, and branch piping are included in the analysis. Sustained loads imposed on the piping system by constant load hangers are also considered in the deadweight analysis.

A value of 1300 psia for operating pressure and 1415 psia for peak pressure are used for the condensate return and main feedwater system. An operating pressure of 1350 psia and a peak pressure of 1615 psia are used for the feedwater piping between the containment vessel and the 8" gate valve.

The thermal expansion stresses are based on the thermal loading for the normal operating condition. A normal operating temperature of 547°F is used for the condensate return and main feedwater piping, while a temperature of 295°F is used for the feedwater piping between the containment vessel and the 8" gate valve.

Thermal anchor movements at the nozzle connections to the recirculation suction line manifold are taken from the NES recirculation line thermal analysis (Ref. 5). Thermal anchor movement at the shutdown condenser connection is taken from Ref. 3.

Two seismic loading events are considered: the safe shutdown earthquake (SSE), and the operating basis earthquake (OBE). The established design criteria, (Reg. Guide 1.48, May 1973) for Class 2 analysis considers the OBE (or 1/2 SSE) to be the normal and upset condition while the SSE is considered the faulted condition.

Seismic inertia loading is imposed on the piping system in the form of seismic acceleration spectra which were derived fro the LACBWR plant (Ref. 2). The horiziontal acceleration spectrum used for the feedwater line is that corresponding to the reactor vessel at an elevation of 664.5 ft. The vertical response spectrum for the SSE loading is taken as 2/3 of the horizontal SSE ground response spectrum assuming no amplification of vertical response in the structure. For the operating basis earthquake the vertical piping response spectrum is taken as 1/2 of the SSE vertical response spectrum. Damping values used are 1% for the OBE and 2% for the SSE.

The horizontal spectra in either the global X- direction or the global Z- direction are applied simultaneously with the vertical spectra in the global Y- direction.

Seismically induced anchor movements for points 10, 11, 38, 45 and 104 were estimated by calculating low frequency displacements from the containment vessel response spectra at the different anchor point elevations.

5.2.3 Stress Acceptance Criteria

The same criteria (see Section 5.1.3) as those of the main steam line were also applied to the feedwater line.

5.2.4 Methods of Analysis

The same methods of analysis (see Section 5.1.4) as those of the main steam line were also applied to the feedwater line.

5.2.5 Results

1. The existing support system of the LACBWR feedwater and condensate return piping system is not adequate to withstand the specified seismic events.
2. The results of the subject analysis, which includes effects of 11 additional seismic restraints, indicate that the

deflections of the feedwater and condensate return piping system, due to dead weight, thermal expansion and seismic loading are nominal. In addition, the stresses resulting from these loadings, as calculated and combined in accordance with the rules given in Subarticle NC-3652 of Section III of the ASME Code satisfy the design requirements for Class 2 piping systems.

3. It was recommended that the feedwater and condensate return piping system be provided with eleven seismic restraints at the locations indicated in Figure 3.1.

5.3 RECIRCULATION PIPING SYSTEM

(Reference 5)

5.3.1 Description

The recirculation piping system provides forced-circulation for the reactor core. The system includes the 16-inch diameter forced-circulation suction manifold, four 16-inch diameter nozzles, two 20-inch suction lines, two variable speed pumps, the 20-inch diameter pump discharge lines and the 16-inch diameter forced-circulation discharge manifold and the four equally-speced 16-inch reactor inlet nozzles.

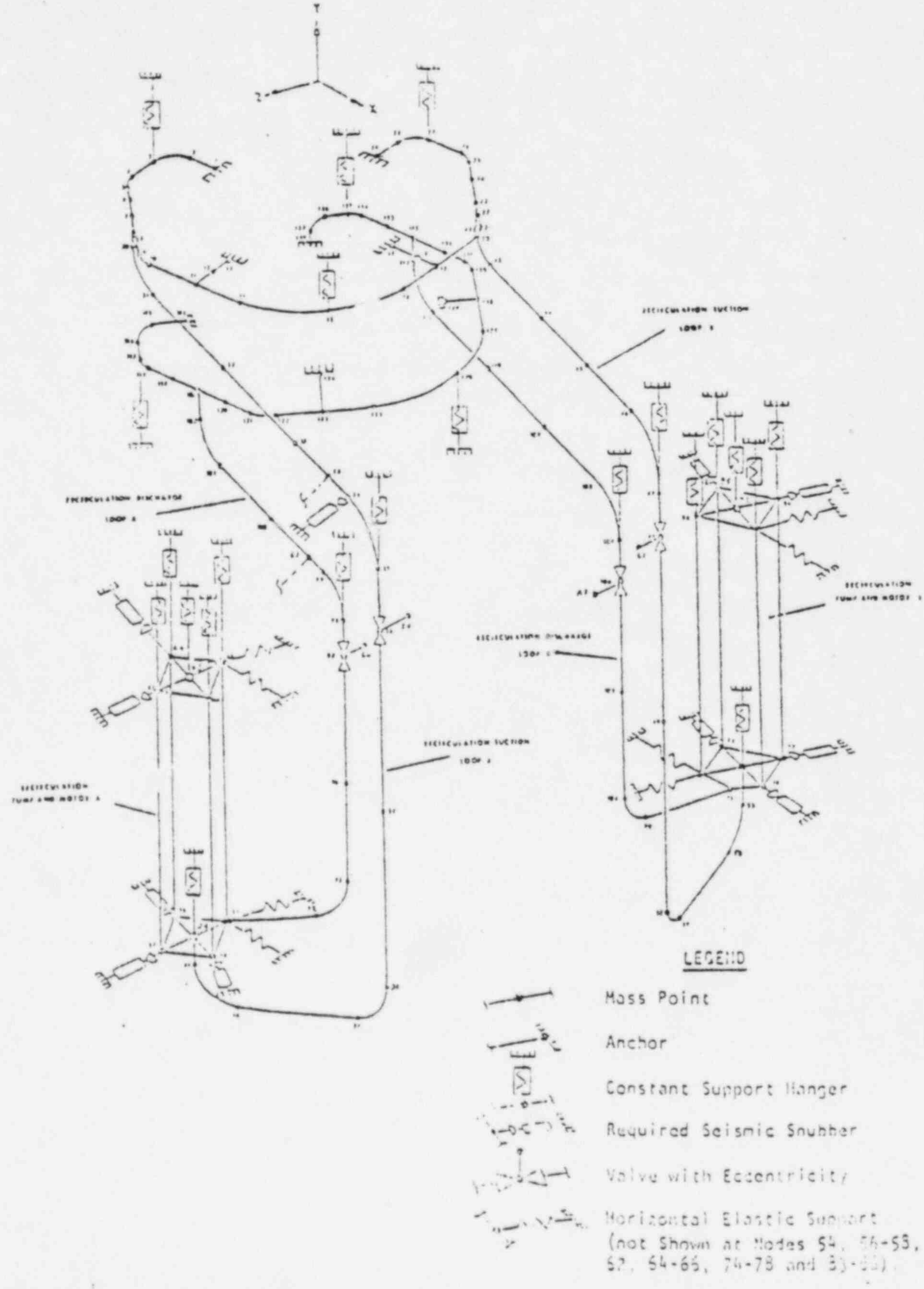
An isometric drawing showing the recirculation piping system as analyzed, including the suspension-system and recommended seismic snubbers, is given in the attached Figure 3.1 of Reference 5.

For the same reason of insufficient data as the case of main steam and feedwater piping systems, Class 2, not Class 1, analysis was performed for the recirculation piping system.

Figure 3.1

Mathematical Model

LACBWR Recirculation Piping System



5.3.2 <u>Loading Criteria</u>

The load cases which must be considered in performing a Class 2 stress analysis include: dead loads and sustained mechanical loads, internal pressure, thermal expansion loading, seismic inertia loads and seismic anchor movement loading.

The dead weight of the piping system is calculated considering the piping system to be insulated and filled with water. The weight of valves, valve oprators, pumps, pump motors, and branch piping as well as the effects of constant weight hangers, etc. are included in the analysis. The seismic inertia loadings on the piping system are imposed in the form of seismic response spectra.

The response spectrum values for the safe shutdown earthquake (SSE, Figure G.34 of Reference 2) are only 20% greater than the response spectrum for the operating basis earthquake (OBE Figure G.36 of Reference 2). However, the allowable stress values for the SSE are 50% greater than the allowable stress values for the OBE. Therefore, only OBE inertia loadings were considered in the subject analysis.

The horizontal spectra in either the global X-direction or the global Z-direction are applied simultaneously with the vertical spectra in the global Y-direction.

Thermal loading for normal operation is produced by the overall thermal expansion of the piping system and the thermal movement of the piping/reactor vessel anchors. The overall thermal expansion of the piping system is that which results from the plant heating up from ambient temperature of 70°F to the normal plant operating temperature of 577°F. The thermal movement of the piping/reactor vessel anchors are taken from Reference 13.

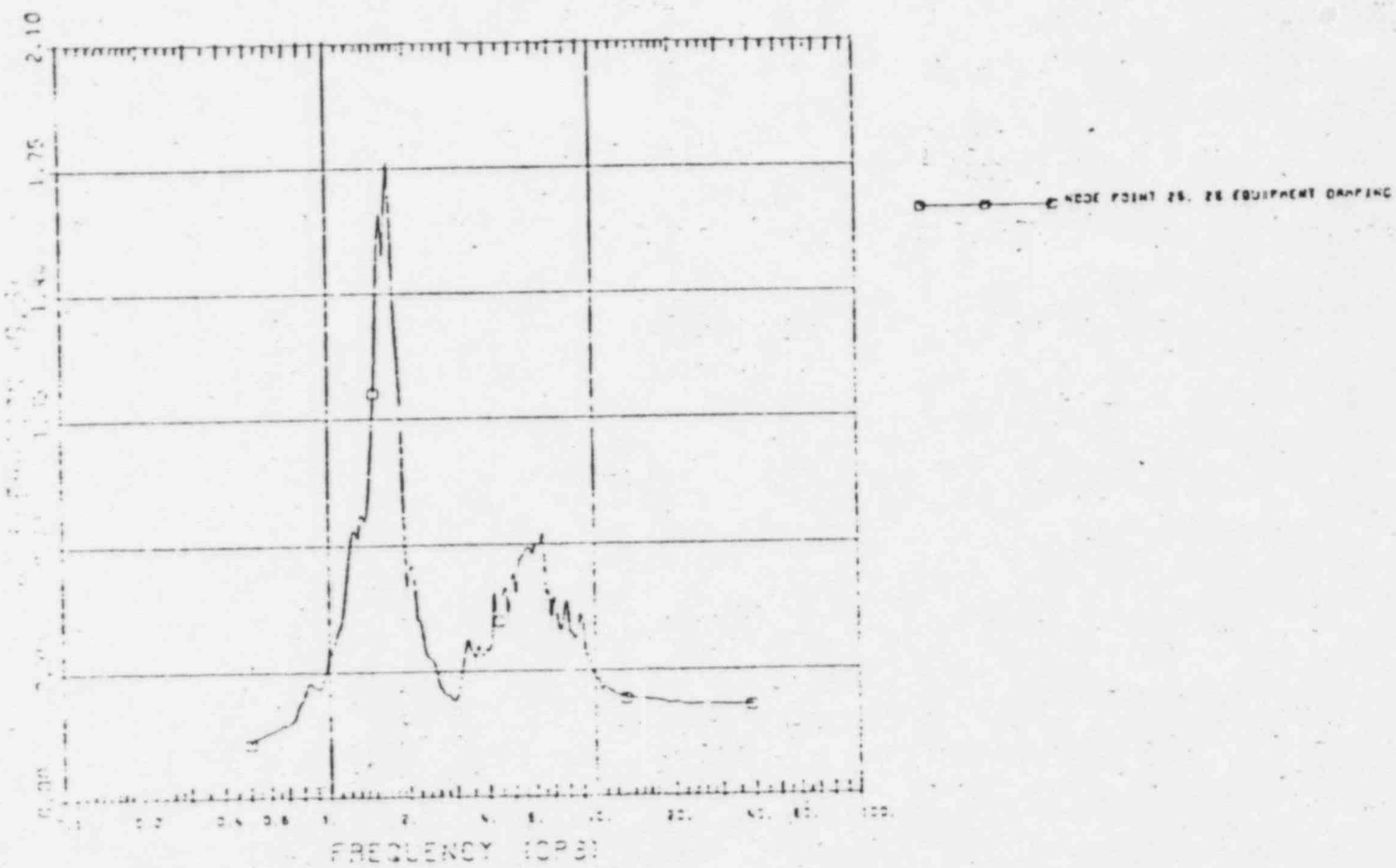


FIG.3.34 — REACTOR MIDDLE SPECTRA, SSE SEISMIC INPUT

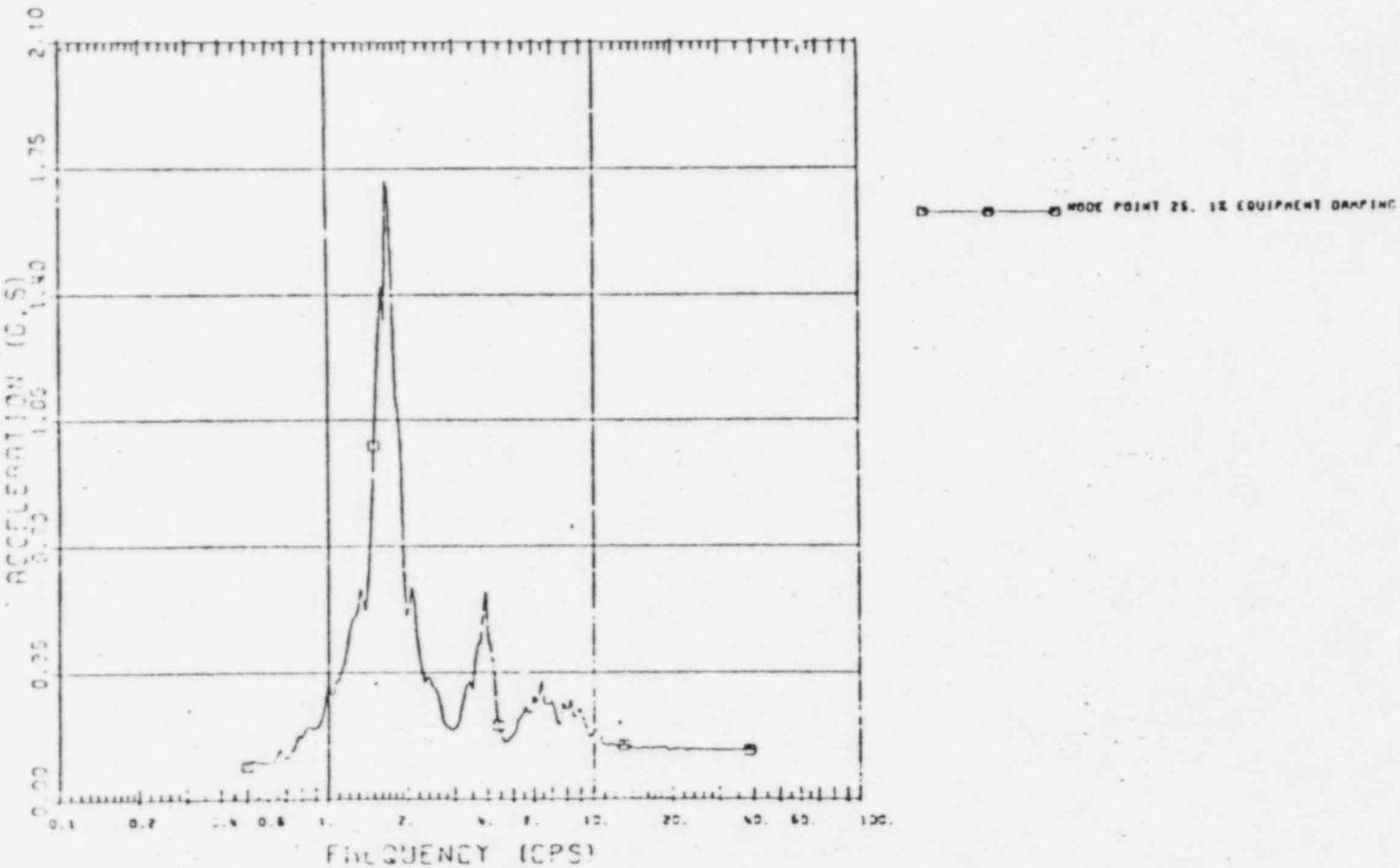


FIG.G.36 — REACTOR MIDDLE SPECTRA. 1/2 SSE SEISMIC INPUT

The seismic anchor movement are estimated using data given in Reference 2. 0.5 inch displacement in the X and Z-direction of the recirculation pumps support points relative to the piping/reactor vessel anchor points are considered.

The normal operating internal pressure load imposed on the piping system is 1300 psi and the peak plant pressure taken as the recirculation piping design pressure is 1450 psi.

5.3.3 Stress Acceptance Criteria

The stress accpetance criteria used in this analysis are the ASME Section III, Class 2 component stress design rules which define the stress limits for the vaarious load combinations under normal operating, upset and faulted conditions as interpreted by the AEC in Regulatory Guide 1.48.

a. Normal Operating Conditions

The combined stresses for the dead weight, sustained mechanical loads and the normal oeprating pressure loads should be less than S_h as calculated by equation 8 of Section III, ASME Code.

The combined stresses for the normal thermal loads and seismic movement should be less than S_a as calcualted by equation 10 of Subsection NC-3650 of ASME Code or the combined stresses for the dead weight, sustained mechanical loads, thermal loads and seismic movement should be less than S_a and S_h as calculated by equation 11.

b. Upset Conditions

The combined stresses due to dead weight, sustained mechanical loads, OBE seismic loadings and peak pressure loads should be less than 1.2 S_h as calcualted by equation 9.

c. Faulted Conditions

Combined stresses due to the safe shutdown earthquake (SSE), peak pressure, dead weight and other sustained mechanical loads as calculated by equation 9 of subsection NC-3650 of the ASME Code should be less than 1.8 S_h. This requirement will be met if the requirements of upset conditions are met since the SSE magnitude is only 20% greater than the OBE while the allowable faulted stress is 50% greater than the allowable upset stress.

5.3.4 Method of Analysis

The same methods of analysis as discussed in Section 5.1.4 were also applied.

5.3.5 Results

By providing adequate seismic restraints (hydraulic snubbers) at the locations shown in Figure 3.1 the deflections and stresses in the piping due to a seismic event can be reduced to acceptable values.

The results of the recirculation piping system stress analysis indicate that deflections of the recirculations piping system due to dead weight, thermal expansion and seismic loadings are nominal. The stresses in the piping system due to dead weight, pressure, thermal expansion and seismic loads as calculated and combined in accordance with the rules given in Subarticle NC 3652 of Section III of the ASME Code, satisfy the design requirements for Class 2 piping when the effects of seismic restraints are included in the analysis.

5.4 HIGH PRESSURE CORE SPRAY (HPCS) SYSTEM SUCTION LINE (Reference 6)

5.4.1 Description

The High Pressure Core Spray (HPCS) System of the LACBWR power plant is designed to provide an emergency coolant spray to the reactor core in the event that reactor water level drops accidentally. This is done by either direct gravity feed of water from an overhead storage tank to the core spray header under low reactor pressure conditions, or by means of high pressure water injection under high reactor pressure conditions.

In order to simplify the piping system analysis, the long and complex HPCS piping system was divided into two sections: the first consisting generally of the suction piping which runs from the overhead storage tank to the high pressure core spray pumps and the second consiting of the discharge piping which runs from the high pressure core spray pumps to the core spray header inlet. The HPCS discharge piping analysis is presented in a separate report. The subject of this section is the HPCS suction line (Reference 7).

To further simplify the analysis the suction line was divided into two subsections: Line 1 as shown in Figure 3.1-1 of Reference 6 and Line 2 as shown in Figure 3.1-2 of Reference 6. Line 1 consists of the 4" Schedule 40S stainless steel pipe line leading from the 42,000 gallon overhead water storage tank to a 4" x 4" reducer at node point 19. A section of the 4" fuel storage well flooding line connecting at node point 18 is included in the analysis of Line 1. Line 2 begins at node 19, Figure 3.1-2 and consists mostly of 3" schedule 40S stainless steel piping up to the two ECCS high pressure pumps. Rigid anchors located at points of expected large seismic deflections, serve to isolate the suction lines for analytical purposes.

The HPCS system is the principal emergency core cooling system. Class 1 component analysis was performed for this system.

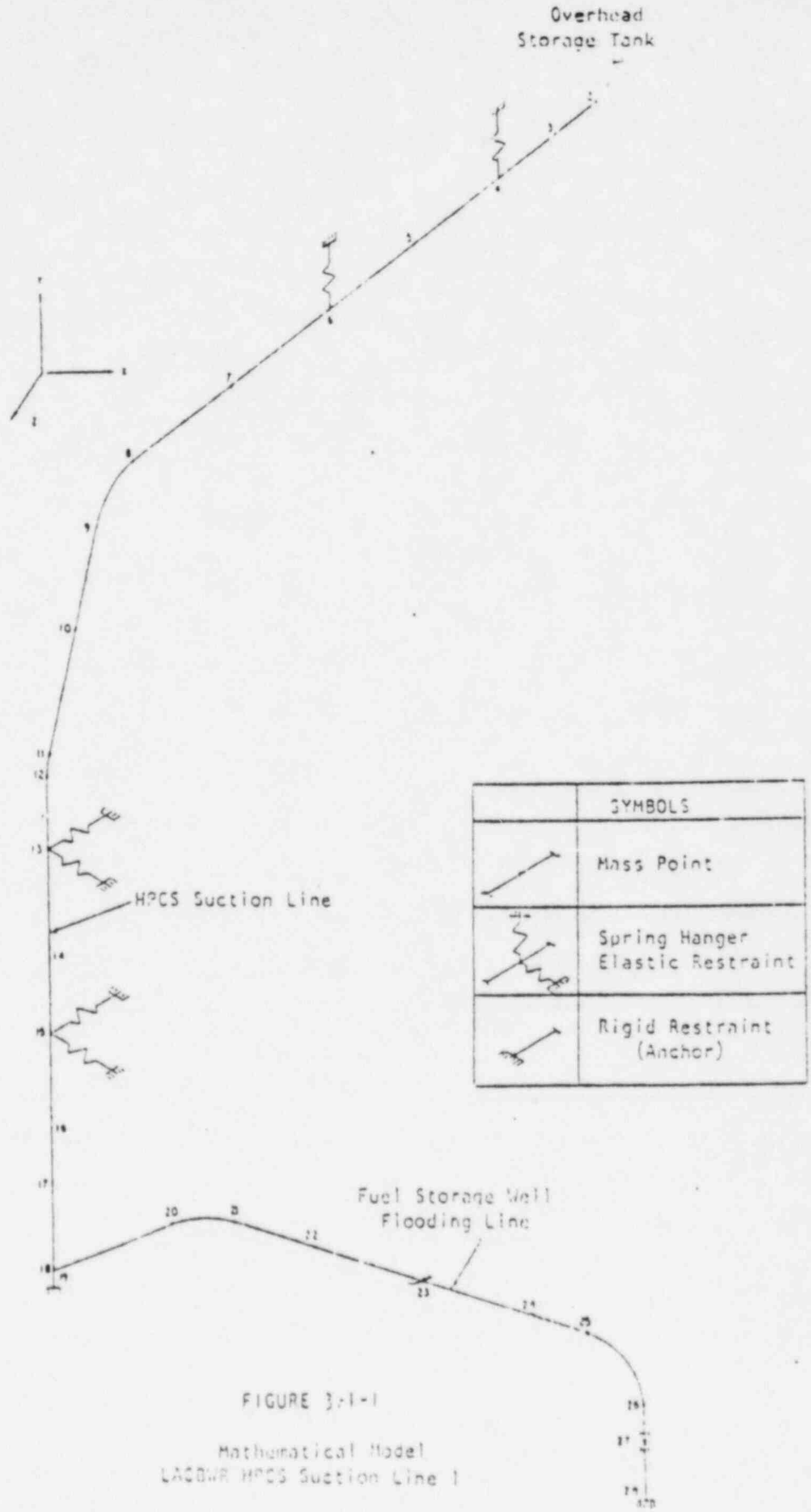
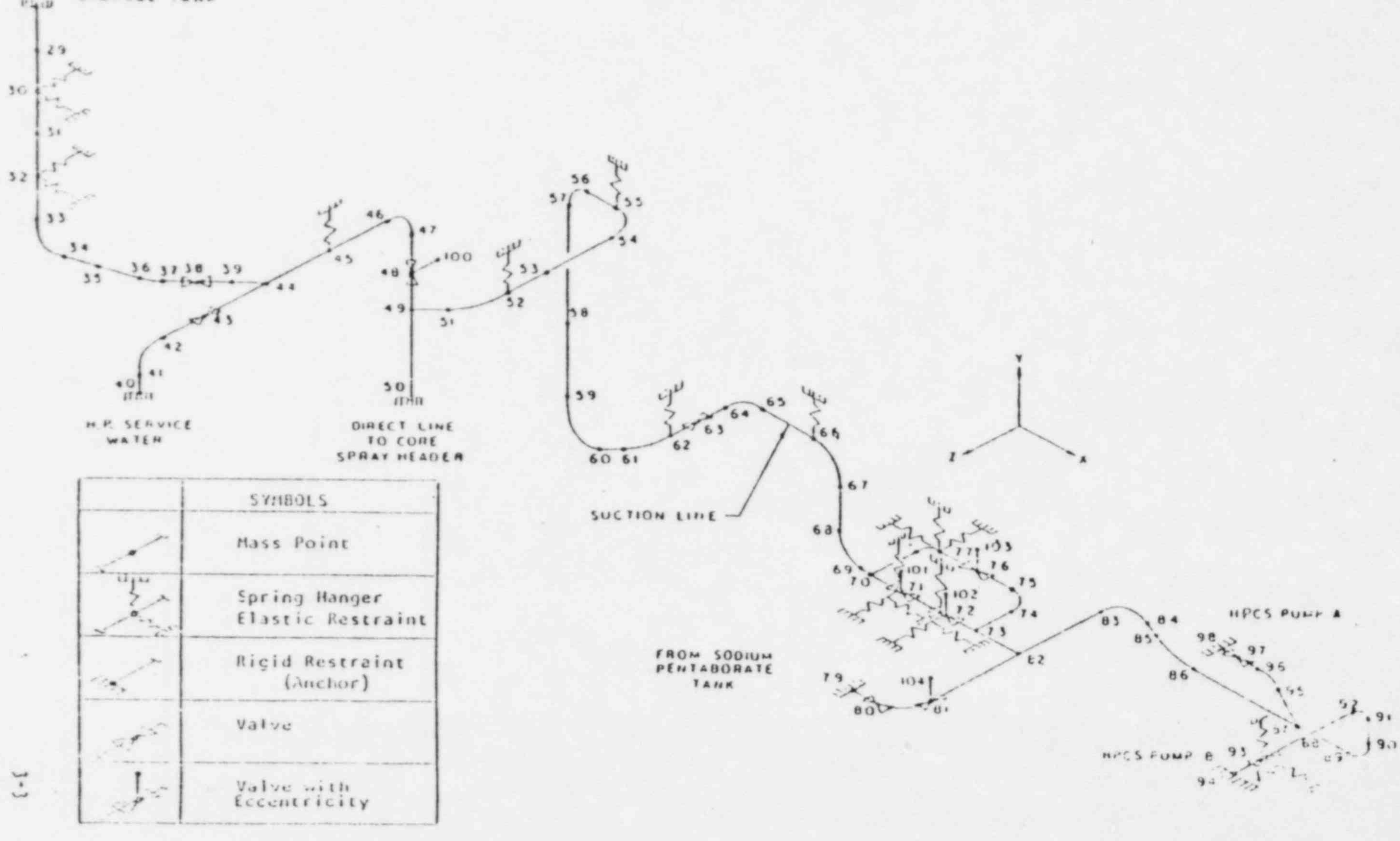


FIGURE 3.1-1

Mathematical Model
LACBWR HPCS Suction Line 1

FIGURE 3.1-2
MATHEMATICAL MODEL
LACBWR HPCS SUCTION LINE 2



5.4.2 Loading Criteria

The loading conditions which must be taken into account in performing a Class 1 analysis of a piping system are specified in Subsection NB-3110 of Section III, ASME Code. These include dead weight, internal pressure, thermal effects; and earthquake loads.

The dead weight of the piping system is calculated assuming the system to be insulated and filled with water. The weights of valves and valve operators, with appropriate eccentricities are included in the analysis.

The HPCS suction line is basically a cold line containing room temperature water from the overhead storage tank. Thermal expansion stresses are calcualted assuming the design temperature of 120°F to be the normal operating condition. Thermal discontinuity and thermal gradient secondary bending stresses are negligible at this temperature and are, therefore, no considered in the analysis.

The normal operating pressure for the HPCS system is the static head resulting from the overhead water storage tank. Constant internal operating pressures of 20 psi and 50 psi are assumed for suction lines 1 and 2 respectively. A pressure of 100 psig is used as the design condition for the complete HPCS Suction Line.

Two seismic loading events are considered: the safe shutdown earthquake (SSE), and the operating basis earthquake (OBE). The established design criteria Regulatory Guide 1.48, May, 1973) for Class 1 analysis specifies that thr OBE (or 1/2 SSE) must be considered in conjunction with the normal and upset plant condition while the SSE must be considered in conjunction with the faulted plant condition.

Seismic inertia loading is imposed on the piping system in the form of seismic acceleration spectra which were derived for the LACBWR plant (Reference 2). The horizontal acceleration spectra used for the HPCS lines

1 and 2 are those corresponding respectively to the subsystem support points on the reactor containment shell at elevations of 745 feet (Water Sotrage Tank) and 700 feet (upper floor). The vertical response spectrum for the SSE loading is taken as 2/3 of teh horizontal SSE ground response spectrum assuming no amplification of vertical response in the structure. For the Operating Basis Earthquake the vertical piping response spectrum is taken as 1/2 of the SSE vertical response spectrum. Damping values used are 1 percent for the OBE and 2 percent for the SSE.

The horizontal spectra in either the global X-direction or the global Z-direction are applied simultaneously with the vertical spectra in the global Y-direction.

Seismically induced anchor movements for the OBE were estimated by calculating low frequency displacements from the containment vessel response spectra at the different anchor point elevations.

5.4.3 Stress Acceptance Criteria

The requirements for acceptability of a Class 1 piping system are given in AEC Regulatory Position 1 and Subsections NB 3600 of Section III of the ASME Boiler and Pressure Vessel Code, Reference 2. Calculated stresses resulting from the design and operating loading conditions given in Subsection NB-3110 and NB-3620 must meet the stress limits of equations 9 through 14 of Subsection NB-3650 of the ASME Code.

a. Design Conditions

The primary stress intensity, resulting from the combined effects of the design pressure and the resultant moment loading due to loads caused by dead weight and the Operating Basis Earthquake and calculated in accordance with equation 9 of Subsection NB-352 of the Code must be less than 1.5 times the allowable design stress intensity, S_m, at maximum temperature.

b. Normal Conditions

The primary plus secondary stress intensity range resulting from the combined effects of thermal expansion, linear thermal gradient and discontinuity, operatin g pressure, anchor movements and earthquake effects, calculated in accordance with equation 10 of the Code must be less than 3 times S_m. In the event that the above requirement is not met the piping product may still be acceptable provided the requirements of a simplified Elastic-plastic discontinuity analysis are met. This requirement is met if 1) the nominal expansion stress resulting from thermal expansion and thermal anchor movements, calculated in accordance with equation 12 of the Code is less than 3 S_m and 2) if the range of primary plus secondary membrane plus bending stress intensity, resulting from the combined loading of operating pressure, dead weight, one-half the range of the earthquake and thermal discontinuity stresses, calcualted according to equation 13 of the code is less than 3 S_m.

The requirements for acceptability under cyclic loading conditions are met by first calculating the peak stress intensity by means of equation 11 of the Code, resulting from the loadings specified for equation 10 plus the loadings resulting from the non-linear portion of the thermal gradient through the wall thickness (considered negligible in this analysis) and then calculating the alternating stress intensity in accordance with equation 14 of the Code. The total number of operating stress cycles must then be less than those determined from the fatigue curves fro Appendix 1-9 of the Code for the calculated alternating stress intensity in accordance with the requirements of paragraphs NB 3653.4 and NB 3653.5 of the Code.

c. Upset Conditions

The requirements for acceptability under upset conditions (not specified in this analysis) are the same as for Normal Conditions.

d. Emergency Conditions

The requirement for accep.ability under emergency conditions (not specified in this analysis) is that the primary stress intensity, as calculated by equation 9 of the Code, must be lwss than 2.25 S_m.

e. Faulted Conditions

Under faulted conditions the primary stress intensity resulting from the combined effects of design pressure, dead weight, and the vibratory motion of the full Safe Shutdown Earthquake as calculated by equation 9 of the Code must be less than 3 S_m.

5.4.4 Method of Analysis

For the static and dynamic analysis, the High Pressure Core Spray suction line has been mathematically modeled as a finite element model. The static response of the HPCS Suction Line to the dead weight, thermal expansion and anchor movement loadings have been calculated using direct stiffness displacement methods of structural analysis. The seismic response of the HPCS Suction Line to the operating basis earthquake (OBE) and safe shutdown earthquakequake (SSE) have been determined using response spectrum, model superposition methods. The modal responses were combined by the square-foot of the sum of the squares method. Stresses due to various loadings have been calculated and combined in accordance with the ASME Code Section III, Subsection NB rules.

In order to perform static, dynamic and stress analyses, the continuous piping system is mathematically modeled as an assembly of elastic structural elements interconnected at discrete nodal points (Figure 3.1). Nodal points are located at all points of interest in the piping system such as elbows, valves, anchorages, hangers, tee intersections, load points, all structural and material discontinuities, etc. This three dimensional multi-degree-of-freedom model of the piping system is attached to the "ground" (structure) by means of rigid hangers, support springs, hydraulic snubbers and anchors. Stiffness characteristics of structural elements are related to the moment of inertia and the axial and effective shear area of the pipe cross section. The stiffness characteristics of the elbows and tee connections are modified to account for local deformation by using the flexibility factors given in the ASME Code.

For the seismic analysis the distributed mass of the piping system is lumped at the system nodal points. Masses are lumped so that the lumped mass, multidegree-of-freedom model represents the dynamic characteristics of the piping system. In order to reduce the number of dynamic degrees-of-freedom, only translational degrees-of-freedom are considered at each mass point (the masses associated with the rotational degrees-of-freedom are set to zero).

5.4.5 Results

By providing rigid seismic restraints at the locations shown in Figure 3.1 the deflections and stresses is the HPCS suction piping due to a seismic event can be reduced to acceptable values.

The results of the subject analysis, which includes effects of five additional rigid restraints indicate that the deflections of the HPCS suction piping system, due to deadweight, thermal expansion and seismic loading are nominal. In addition, the stresses resulting from these loadings as calculated and combined in accordance with the rules given in Subarticle

NB-3650 of Section III of the ASME Code, satisfy the design requirements for Class 1 piping systems.

5.5 HPCS SYSTEM DISCHARGE LINE

(Reference 7)

5.5.1 Description

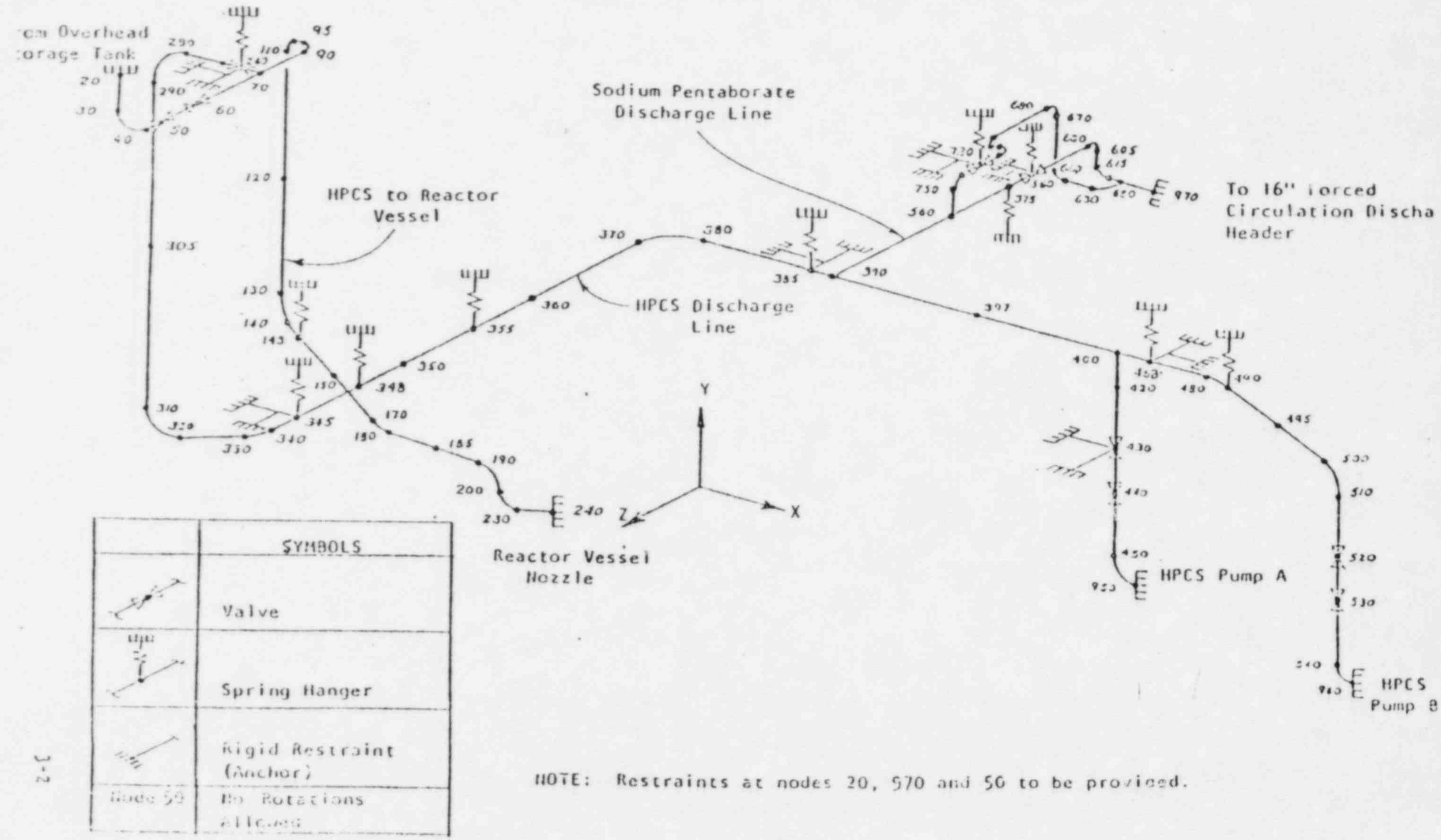
The HPCS piping system includes two sections. The first section, suction piping, has been discussed in the last section. The second consists of the discharge piping which runs from the high pressure core spray pumps to the core spray header inlet.

The HPCS discharge line consists of stainless steel pipe line leading from the two high pressure core spray pumps to the core spray header inside the reactor vessel. The pumps are used for core spray when the reactor remains pressurized, as in the case of a small leak below the ocre. When the reactor and containment building pressures are equalized, as after a major system leak or rupture, a low pressure supply line bypassing the emergency core spray pumps allows water to flow directly from the overhead storage tank (or service water line) to the core spray header. The high pressure core spray pumps are also used in the boron injection system. Redundant control valves are provided for this purpose in the core spray pumps suction and discharge lines.

Rigid anchors located at points of expected large seismic deflections serve the purpose of isolating the discharge lines from the interconnecting piping systems. Figure 3.1 of Reference 7 shows the routing of the discharge line and the extent of suction line and sodium pentaborate lines considered in the subject analysis.

FIGURE 3.1

PIPESD MATHEMATICAL MODEL
LACBWR HPCS DISCHARGE LINE



NOTE: Restraints at nodes 20, 570 and 50 to be provided.

Anticipating the possibility of a seismically induced loss of coolant accident, it was concluded that analyses of the major Class 1 piping systems should be performed to evaluate their structural integrity.

5.5.2 Loading Criteria

The loading conditions which must be taken into account in performing a Class 1 analysis of a piping system are specified in Subsection NB-3110 of Section III, ASME Code. These inlcude dead weight, internal pressure, thermal effects; and earthquake loads.

Piping design pressures are taken from the LACBWR piping specification (Reference 14) and are 100 psig for the piping between node points 20 and 50 and 1400 psig elsewhere in the system.

Operating pressures for the HPCS system are based on the LACBWR Safeguards Report (Reference 5). These are 100 psig up to node point 50, 1340 psig from node 50 to the reactor vessel nozzle and 1400 psig for the remainder of the system.

The dead weight of the piping system is calculated considering the piping to be insulated and filled with water. Other sustained loads included in the analysis are valve weights, valve operator weights, and the tributary weights from branch piping.

Seismically induced anchor movements for the OBE earthquake were estimated by calculating low frequency displacements from the containment vessel response spectra (from Reference 2) at different elevations.

Thermal expansion or contraction of the reactor vessel during start-up and shutdown reuslts in maximum displacements of the piping system anchor at the reactor vessel nozzle (node point 240).

Two seismic loading events are considered: the safe shutdown earthquake (SSE), and the operating basis earthquake (OBE).

Seismic inertia loading is imposed on the piping system in the form of seismic acceleration spectra which were derived for the LACBWR plant (Reference 2). The horizontal acceleration spectra used for the HPCS discharge line is that corresponding to the subsystem support points on the reactor containment shell at an elevation of 695 feet. The vertical response spectrum for the SSE loading is taken as 2/3 of the horizontal SSE ground response spectrum assuming no amplification of vertical response in the structure. For the Operating Basis Earthquake the vertical piping response spectrum is taken as 1/2 of the SSE vertical response spectrum. Damping values used are 1 percent for the OBE and 2 percent for the SSE.

The horizontal spectra in either the global X-direction or the global Z-direction are applied simultaneously with the vertical spectra in the global Y-direction.

The sudden introduction of cold water from the HPCS system piping into the hot pressure vessel nozzle, due to a LOCA or other low water level condition results in a transient thermal condition in the NOZZLE region. This temperature transient generates stresses in the pipe due to the large temperature gradients across the pipe wall and due to any material discontinuities present. These thermal loads which are applied at node points 230 and 240 have been calculated by means of a transient thermal analysis with the Lion Computer Code (Appendix B and E of Reference 7). These loads are considered in conjunction with the upset plant condition.

During normal start-up and shutdown a temperature change of 344°F is assumed in the pipi· in the region of the reactor vessel HPCS discharge nozzle.

5.5.3 Stress Acceptance Criteria

The same stress acceptance criteria as those discussed in Section 5.4.3 were also applied to the HPCS discharge line.

5.5.4 Method of Analysis

Similar three dimensional model as described in Section 5.4.4 was used for the HPCS discharge line.

For determining the peak stresses resulting from the thermal transient produced by HPCS initiation, and ANSYS finite element model (see the attached Figure 6.1 of Reference 7) of the socket weld coupling/reactor nozzle region was used.

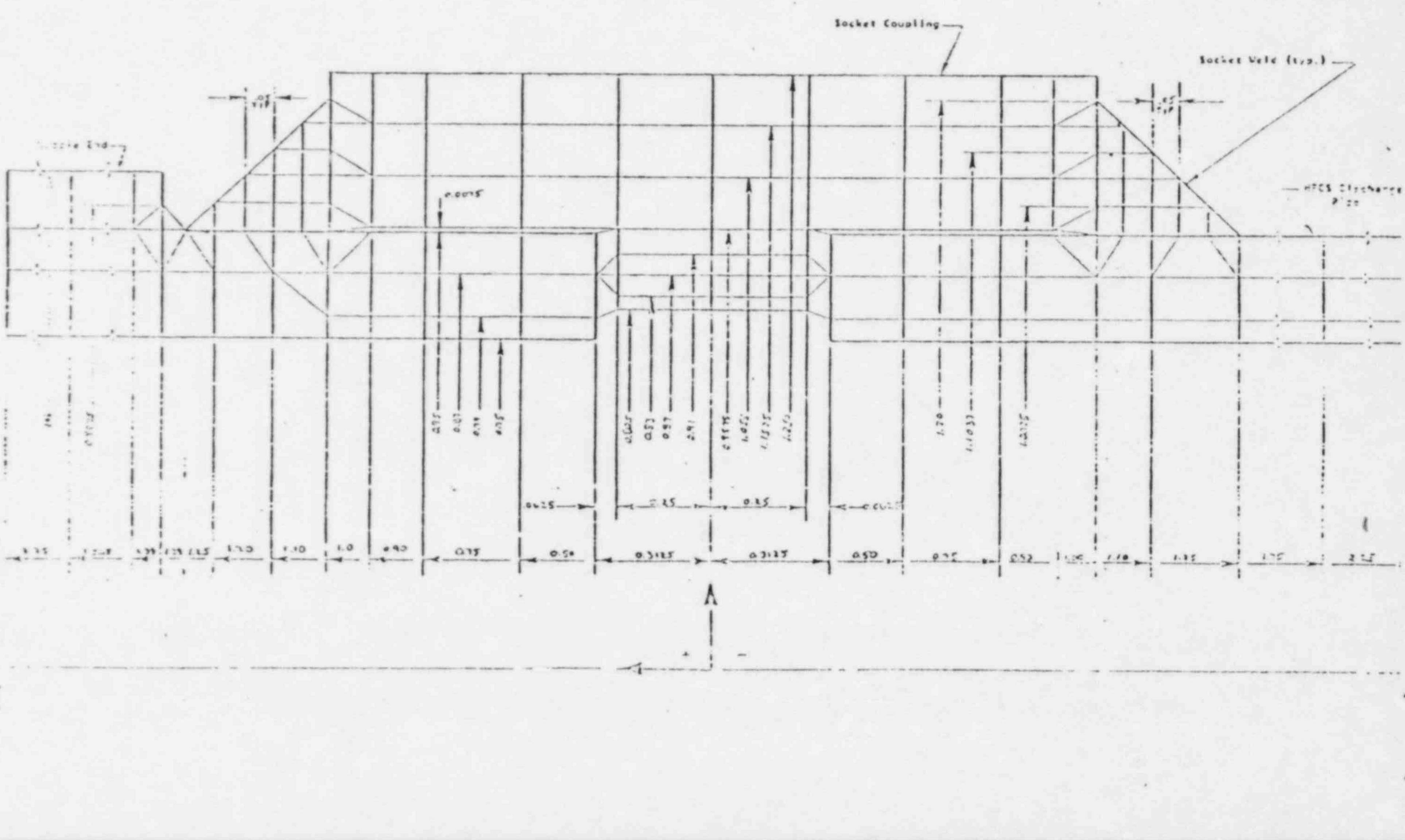
5:5.5 Results

The results of the analysis indicate that the deflection of the HPCS discharge piping system due to dead weight, thermal loads and the specified seismic events are acceptable and that the stresses resulting from these loads, as calculated and combined in accordance with the rules given in Subarticle NB-3650 of Section III of the ASME Code, satisfy the design requirements for Class 1 piping systems provided that:

1. rigid anchors are provided as indicated by node points 20 and 970 of Figure 3.1,
2. the rotation of the eccentric accutator of the control valve CSV 204 (node point 50, Figure 3.1) is restrained by means of appropriate bars or struts,
3. the restraints are designed using the support reaction forces given in Table B-11 of Appendix B, and
4. the total number of HPCS initiations is limited to 2900 cycles.

FIGURE 6.1

HPCS DISCHARGE NOZZLE SOCKET WELD
MATHEMATICAL MODEL GEOMETRY



NOTE: The HPCS system has operated (or cycled) a total of 220 times during all phases of plant testing and operation. However, only 25 operations or cycles have occurred with the LACBWR plant at or near operating temperature during its 7 year operating history. Considering a 40 year plant life, the total number of HPCS system operations with the plant at temperature is expected to be less than 150 cycles. Clearly this number is well below the maximum allowable number of cycles at operating temperature (2900 cycles).

5.6 SPENT FUEL STORAGE POOL DRAIN LINE

(Reference 10)

5.6.1 Description

The fuel storage pool drain line serves as the discharge line for the storage pool filter, pumps and cooler recirculation loop. The line also provides a method of draining the pool should that requirement ever exist.

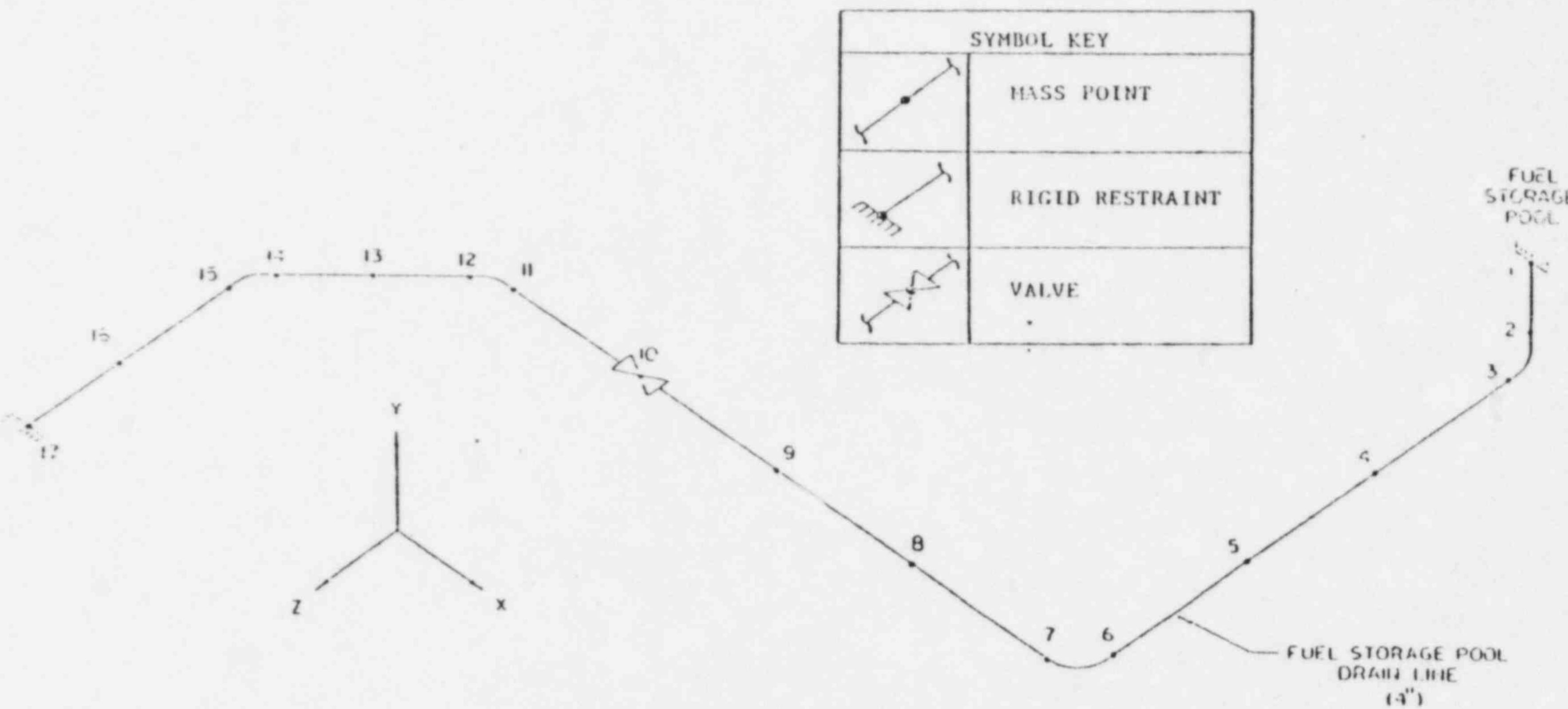
A check valve installed approximately 12 feet downstream of the storage pool drain connection simplifies the analysis by isolating the storage pool and drain line from any possible adverse effects downstream of this valve.

The drain line between the pool drain connection and the concrete wall penetration including the isolation check valve was analyzed as a Seismic Category 1 system.

The subject piping, shown in the attached Figure 3-1 of Reference 10, consist of a four-inch schedule 40, stainless steel line, from the spent fuel storage pool to a concrete wall (Mass Point 17) just downstream of the isolation check valve. Rigid anchors at the concrete penetrations serve to isolate the line from the remainder of the piping system for analytical purposes.

FIGURE 3-1

MATHEMATICAL MODEL FOR
LACBWR SPENT FUEL STORAGE POOL DRAIN LINE



There are no intermediate hangers or restraints.

5.6.2 Loading Criteria

The loading conditions taken into account in the Class 1 analysis include dead weight, internal pressure, thermal effects and earthquake loads.

The dead weight of the piping system is calculated assuming the system to be non-insualted and filled with water. The weight of the valve, estimated from vendor data to be 150 lbs., is included in the analysis.

The drain line is basically a cold line containing room temperature water from the fuel storage pool cooling system. Thermal expansion stresses are calculated assuming the design temperature of 120°F to be the normal operating condition. Thermal discontinuity and thermal gradient secondary bending stresses are negligible at this temperature and are, therefore, not considered in the analysis.

The normal operating pressure for the drain system is the static head resulting from the fuel storage well pool drain. A constant internal operating pressure of 31 psi is assumed. A pressure of 100 psig is used as the design condition.

Seismic inertia loading is imposed on the piping system in the form of seismic acceleration spectra which were derived for the LACBWR plant (Reference 2). The horizontal acceleration spectra used for the analysis are those corresponding to an elevation 667' (upper floor). The vertical response spectrum for the SSE loading is taken as 2/3 of the horizontal SSE ground response spectrum assuming no amplification of vertical response in the structure. For the Operating Basis Earthquake the vertical piping response spectrum is taken as 1/2 of the SSE vertical response spectrum. Damping values used are 1 percent for the OBE and 2 percent for the SSE.

The horizontal spectra in either the global X-direction or the global Z-direction are applied simultaneously with the vertical spectra in the global Y-direction.

Seismically induced anchor movements for th- OBE were estimated by calculating low frequency displacements from the containment vessel response spectra at the different anchor point elevations.

5.6.3 Stress Acceptance Criteria

The same stress acceptance criteria as those discussed in Section 5.4.3 were also applied to the spent fuel storage pool drain line.

5.6.4 Method Of Analysis

Similar three dimensional model as described in Section 5.4.4 was used for the pool drain line.

5.6.5 Results

The results of this analysis indicate that the deflections of the spent fuel storage pool drain line due to deadweight, thermal expansion and seismic loadings are nominal. Further, the resulting maximum stresses as calculated and combined in accordance with the rules given in Subarticle NB-3650 of Section III of the ASME Code, satisfy the design requirements for Class 1 piping systems. In addition, it can be concluded that the maximum cycling Code stress requirements are met for the specified loading conditions.

6. SEISMIC ANALYSIS OF EQUIPMENT AND THEIR SUPPORTS

6.1 REACTOR CORE SUPPORT SYSTEM

(Reference 2)

6.1.1 Description

The dynamic analysis of the reactor internal system was initiated by first considering the behavior of the core support structure.

The structural configuration of the core support system is shown in Figure 4.2 of Reference 2. It consists of an upper lateral support structure (about 9 feet long) and a lower cylindrical support skirt (about 3 feet long). The cross-section of the upper support structure is shown in Figure 4.3 of Reference 2 and is a thin-walled section (0.063 inches thick) with upper and lower flanges. Twenty long bolts (and sleeves) are used to connect the support structure to the skirt.

The upper skirt is supported at the top against lateral motion by means of three brackets while the skirt is supported at the base by means of the circular plenum plate (Figure 4.2). The core houses eighty (80) fuel elements each weighing approximately 400 pounds.

6.1.2 Method of Analysis

A lumped mass model of this structural system was generated and is shown in Figure H.1 of Reference 2. It consists of 13 mass points with 12 interconnecting shear beam elements. The boundary conditions imposed on the model are restraint against lateral motion at the top and both lateral and rotational restraint at the bottom. The dynamic input to the model is the motion-time histories at the top and bottom generated for the reactor vessel from the primary structural response model presented in Section 4.

In computing the mass data for the node points, the weights of the support structure and fuel elements were used. In computing the section stiffness properties, the stiffness of the outer support structure was considered

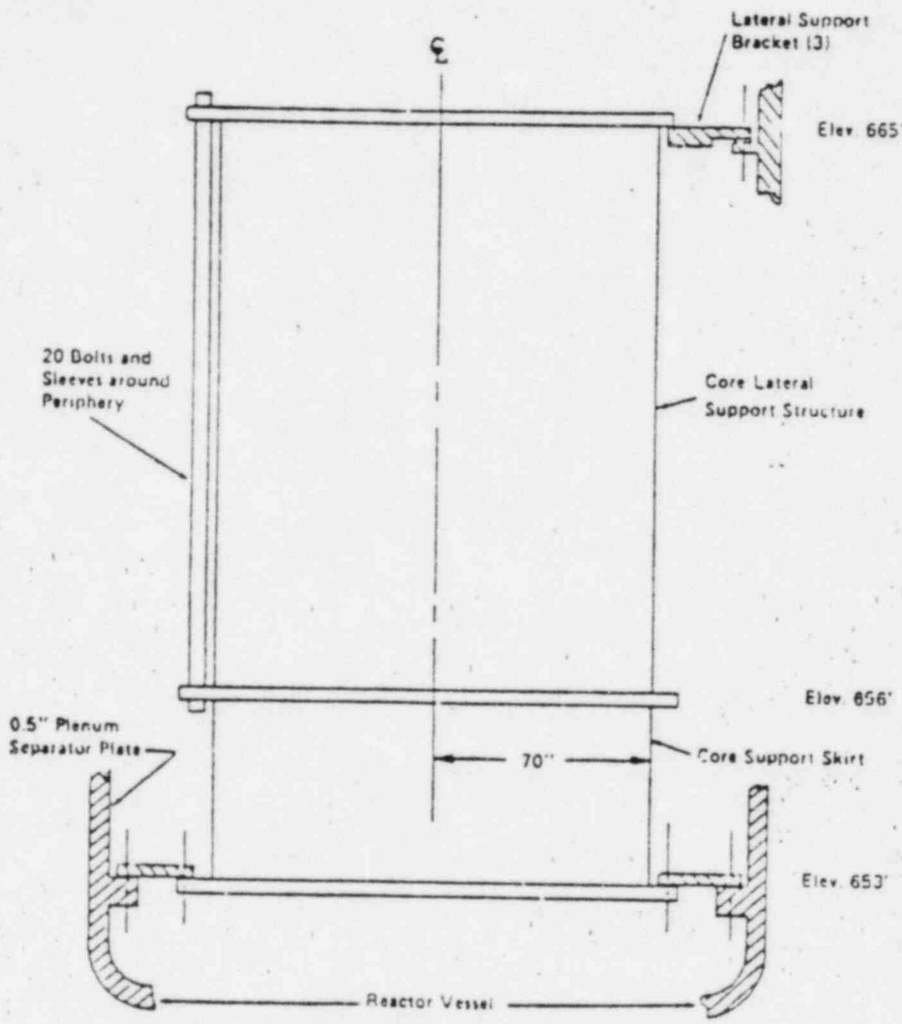


Fig. 4.2 – Configuration of Core Support System

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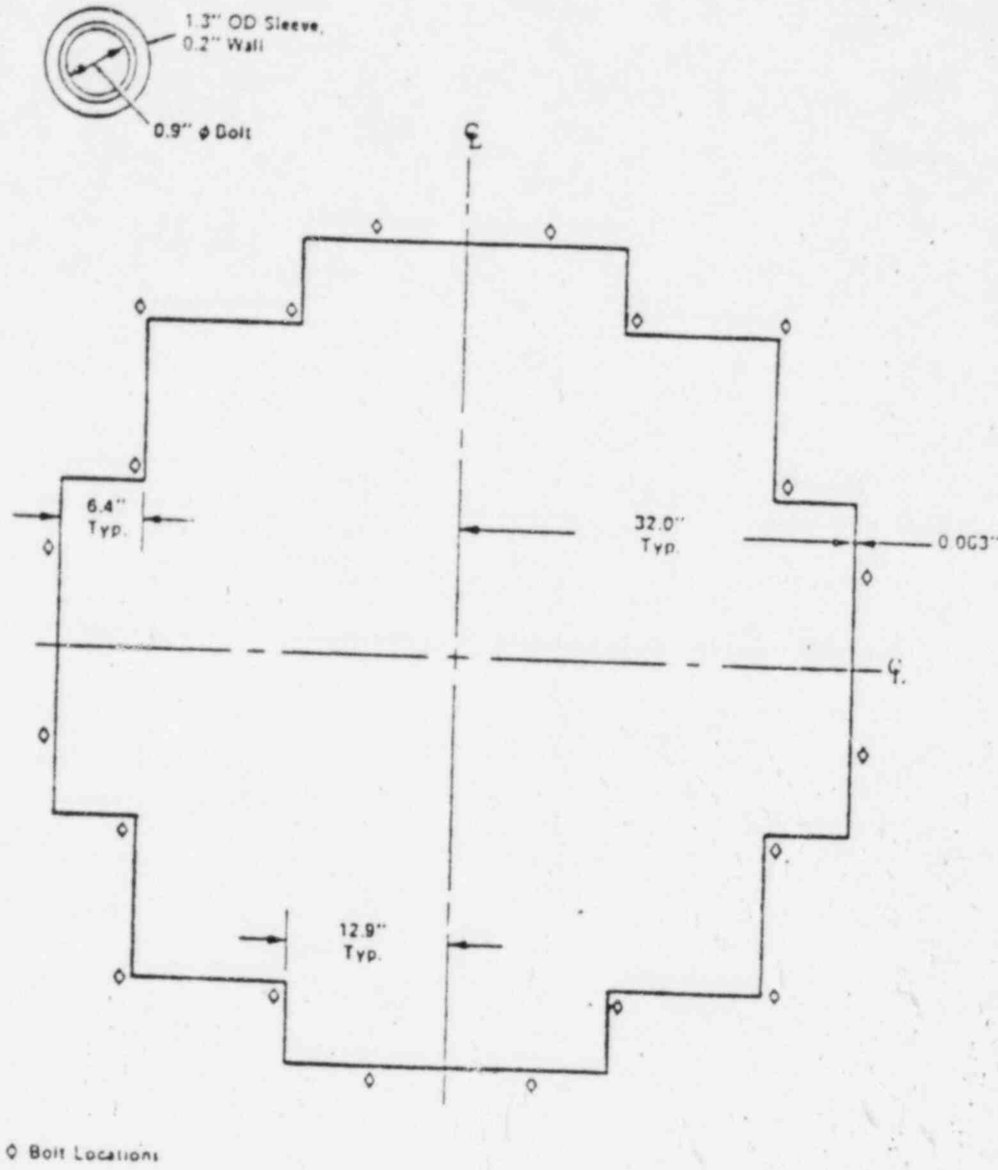


Fig. 4.3 — Core Lateral Support Structure

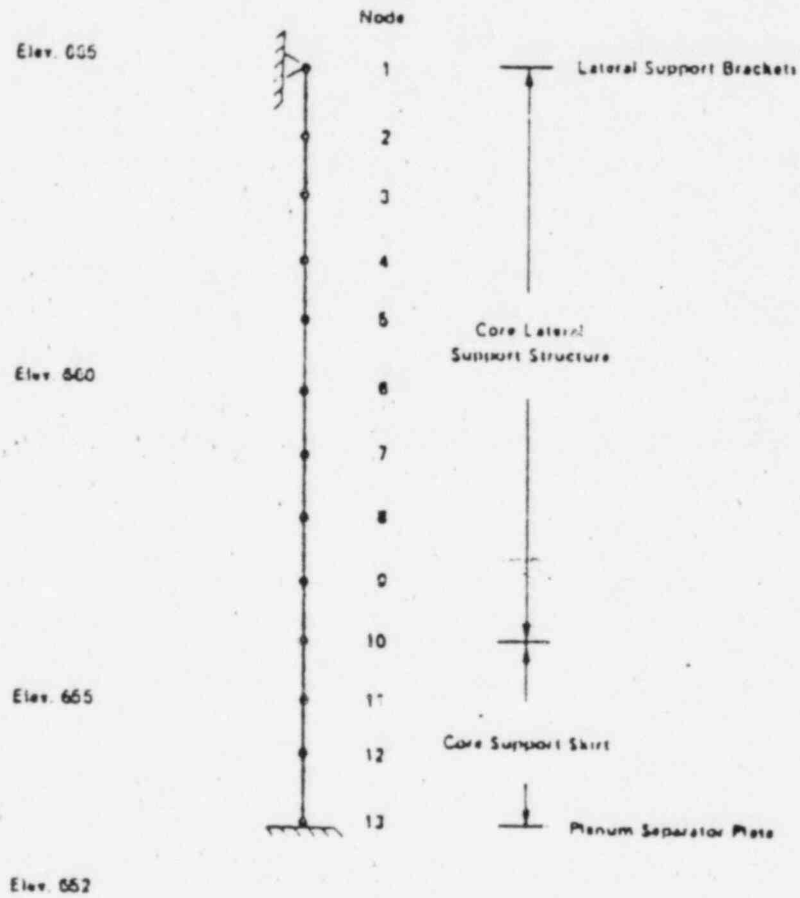


Fig. H.1 — Lumped Mass Model of Core Support System but the stiffness of the fuel elements were neglected. The stiffnesses of the lower three elements located with the support skirt were computed from the properties of a thin-walled circular cross-section. The inside diameter of this member is 69.75 inches with a wall thickness of 1 inch. In computing the bending properties of the upper lateral support structure, the stiffness properties of the bolts and sleeves were included.

From this analysis, structural frequencies of the core support structure were determined. Since this support system has frequencies well above 33 cps, an equivalent static analysis was performed to determine peak stress levels developed in the support structure.

From the computed lateral response spectra for the support points the peak accelerations at the high frequency end of the spectra is relatively constant and equal to 0.25 g's for the SSE input condition. A static lateral load of 0.25 times the weight per foot of beam was used.

6.1.3 Results

From this equivalent static analysis, the peak support point reactions were found to be:

Reaction at top support brackets	1.2 kips
Reaction at plenum separator plate	7.6 kips
Moment at plenum separator plate	49.5 kip-feet

From these values, stress levels in the various components of the support system (skirt, upper support structures, brackets, various connecting bolts, etc.) were computed and in all cases were found to be negligibly small. Thus, it was concluded that the strength of the core support structure is adequate for the SSE input condition. The peak lateral displacement of the system was computed to be 9.7×10^{-6} ft.

6.2 CONTROL ROD DRIVE MECHANISM

(Reference 2)

An analysis was performed to estiamte the peak force that may be required to drive the control rods during the seismic event. The details of the analysis are also presented in Appendix H. The approach used was to allow the core structure to take its peak deflected shape as determined from the equivalent static analysis described above. This deflected pattern was then imposed upon the control rods and the lateral loads required to generate this deflected shape for the control eods computed. Assuming a friction coefficient of unity, a peak drive force was determined to be less than 10 pounds. From this analysis, it is concluded that the control rod drive force required during the seismic event is negligibly small.

6.3 SLOSHING EFFECTS OF WATER STORAGE TANK

(Reference 2)

As mentioned in Section 4, the sloshing water mass was eliminated from the primary structural analysis to ensure that numerical difficulties would not occur. The sloshing fluid mass is considered as a separate subsystem and the effects on the structure analyzed by response spectra methods. These calculations are presented in Appendix F of Reference 2 and indicated that the average stresses developed in the support shell are extremely low. Neglecting shell bending effects, peak vertical stresses are computed conservatively to be about 300 psi while peak average shear stresses are computed to be about 80 psi. With these nominal stress values, it is concluded that any higher order analysis is unnecessary and that the effects of water sloshing are negligible.

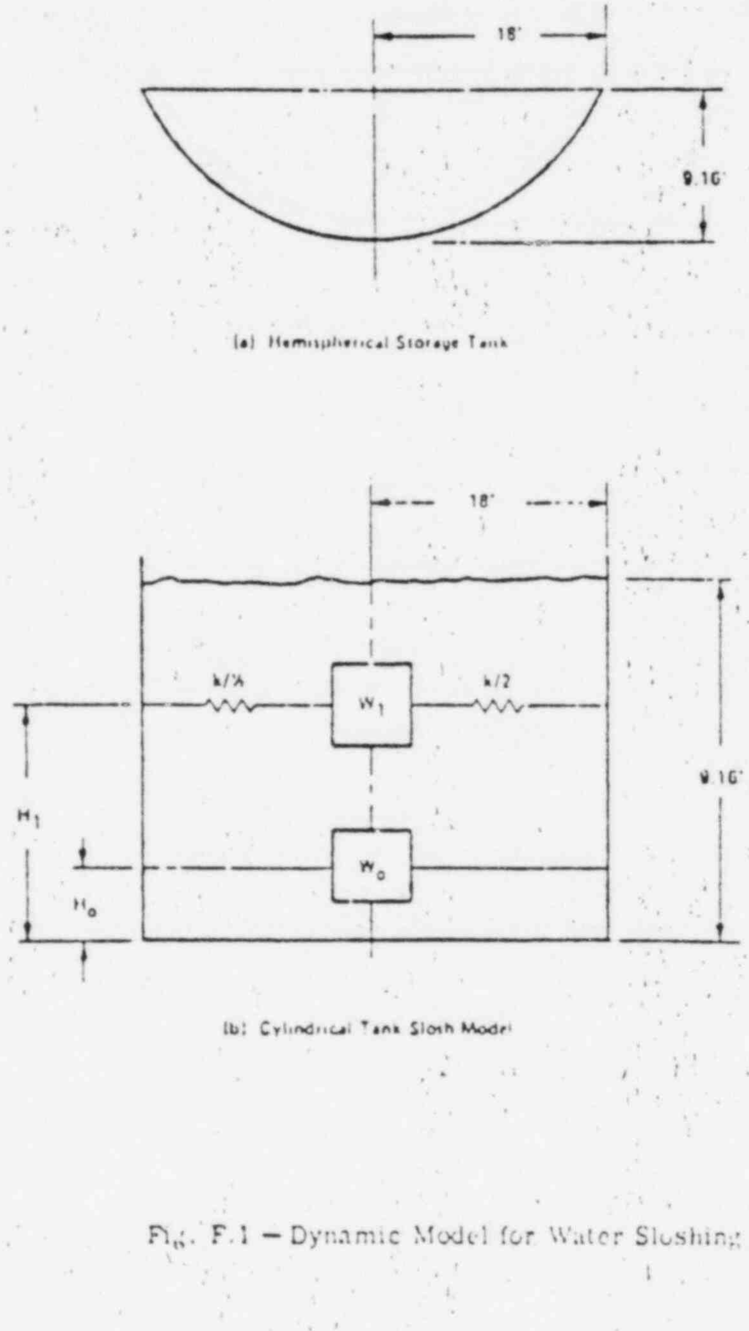


Fig. F.1 — Dynamic Model for Water Sloshing

6.4 THE EXISTING AND THE PROPOSED NEW (1975) SPENT FUEL STORAGE RACKS (Reference 16)

6.4.1 Description

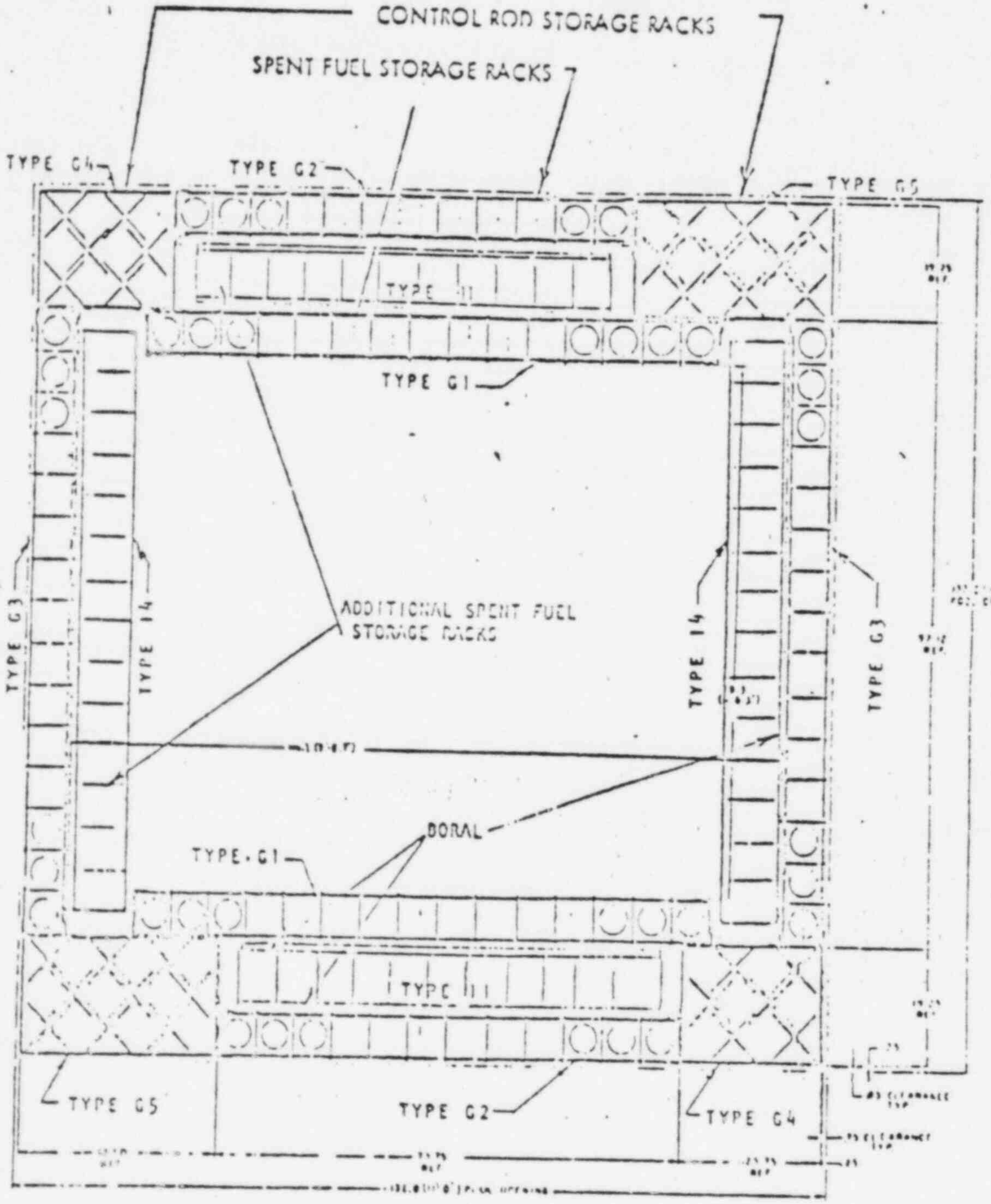
A plan of the spent fuel and control rod storage racks including the new NEX racks is shown in Figure 1 of Reference 16. The existing fuel racks (rack types G1, G2 and G3) are box-shaped structures fabricated from stainless steel type 304 plate which are bolted to the four free-standing control rod storage rakcs (rack types G4 and G5). Each fuel storage rack is attached to the control rod storage racks by means of nine bolts at each side. The four new NES fuel storage racks (rack types 11 and 14) are box-sh fabricated from stainless steel type 304 vertical and horizontal angle members. The four new NES racks are not laterally attached to any support structure but are arranged so that they are completely trapped by the existing racks.

Reference 16 presents the seismic and structural analysis existing and new fuel storage racks and their attachments to withstand the loadings associated with a seismic event. The two cases that could result in the structural failure of the existing and new fuel storage racks during a seismic event are discussed in detail. These cases are (1) the possible toppling over of the existing rack G1 (Figure 1) due to failure of the attachment bolts or failure of the control rod storage racks to which the existing rack G1 is attached and (2) the possible toppling over of the new NES rack Type 14 (Figure 1) due to its buckling and subsequent loss of edge supports.

6.4.2 Loading Conditions

1 (Collapse of existing racks Type G1)

The seismic lateral inertia loading on the coupled model of the existing fuel and control rod storage racks is in the form of the applicable



SPENT FUEL AND CONTROL ROD STORAGE, PLAN

FIGURE 1

-4-

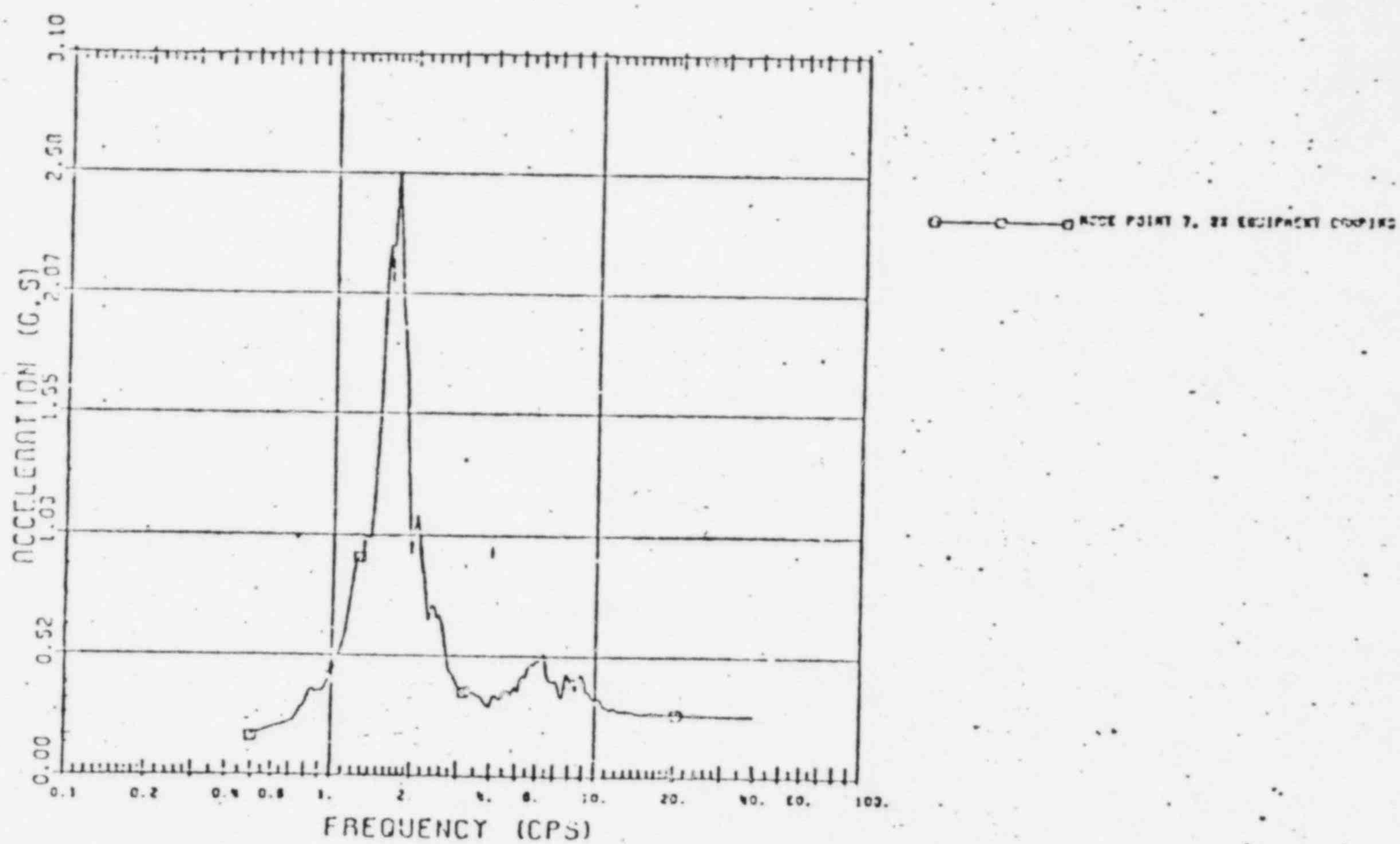


FIGURE 2 - INTERMEDIATE FLOOR SPECTRA, SSE SEISMIC INPUT

acceleration response spectrum given in Reference 2. The acceleration response spectrum (Figure 2) for the safe shutdown earthquake (SSE) at the intermediate floor level of the LACBWR containment building and equipment damping is used in the seismic analysis.

In addition to the seismic inertia loadings, the coupled model of the existing fuel and control rod storage racks will be subjected to the potential impacting load of the adjacent NES rack (type 11). The NES rack is assumed to topple over and hit rack G1 with the intermediate floor and wall seismic g-loading of 0.28g multiplied by an impact factor of 2.

Case 2 (Collapse of new NES rack Type 14)

The inertia loadings imposed on the new NES rack during a seismic event are calculated by multiplying the distributed mass of the rack and its contents by an equivalent static seismic acceleration value. The equivalent static seismic acceleration value corresponding to the fundamental frequency of vibration of the rack is obtained from the applicable acceleration response spectrum curve case.

6.4.3 <u>Design Criteria</u>

The acceptable maximum stresses in the fuel storage racks and their attachments are established based on the guidelines given in USAEC Document (B) "Structural Design Cirteria for Evaluating the Effects of High Energy Pipe Breaks on Category 1 Structures Outside the Containment" prepared by the Structural Engineering Brach; Directorate of Licensing. The acceptable stress values are given below:

Allowable stress for axial compression or tension plus bending		= 0.9fy
Allowable shear stress value		= 1.6x0.4fy = 0.64fy
where	fy	= yield stress value for stainless steel
		= 1.15x34 - 39.1 Ksi

The factor of 1.15 accounts for the increase in yield stress under dynamic loadings.

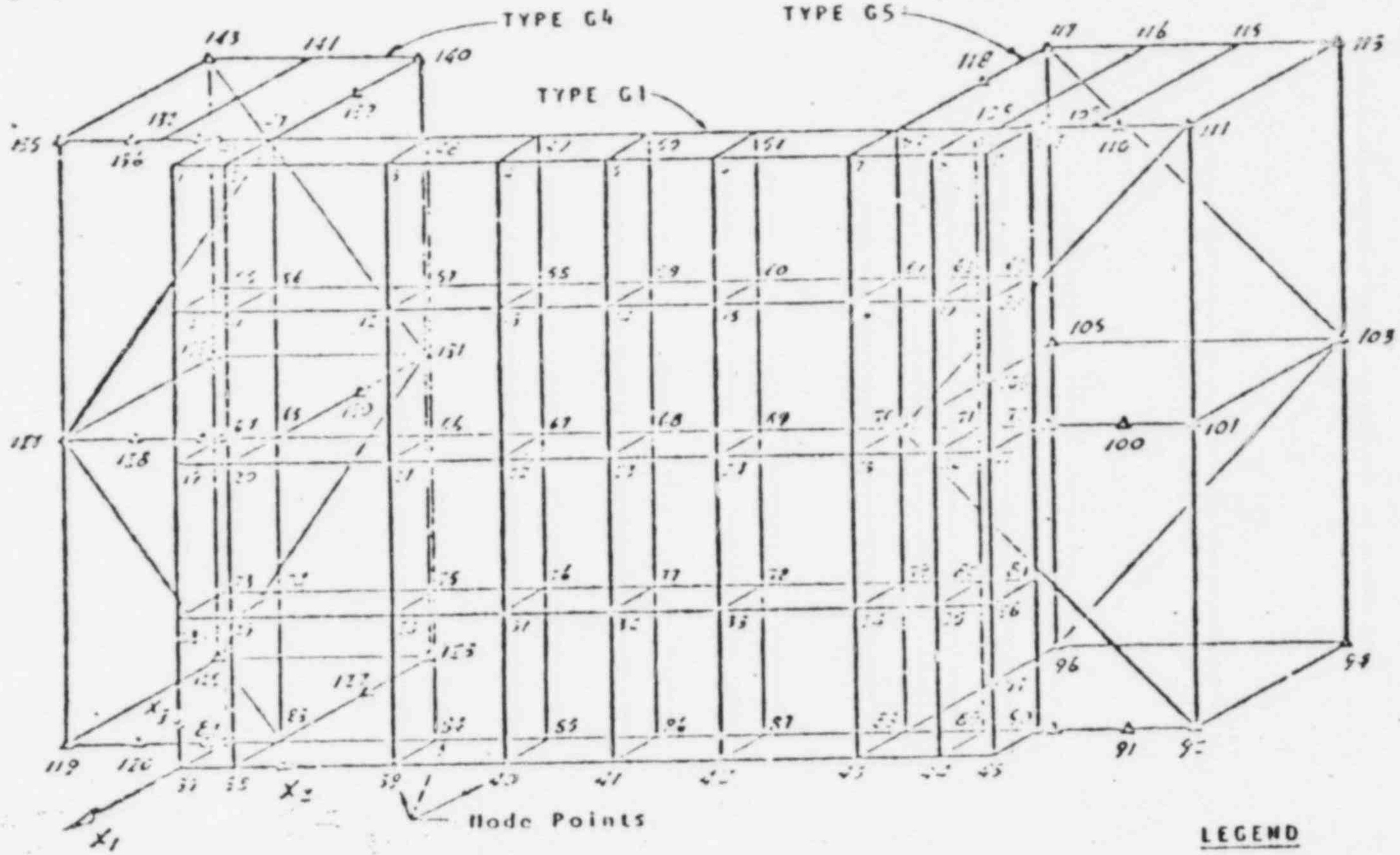
6.4.4 Methods of Analysis

Case 1

The existing fuel storage rack and the control rod storage racks are mathematically modeled as an assemblage of finite element plate and beam elements as shown in Figure 3 of Reference 16. For the seismic analysis, the tributory weights of the structural elements, fuel assemblies, control rods, and water are lumped at the appropriate node points. The stiffness characteristics of the model are calculated considering axial, flexural and shear deformations of each structural element. From the mass and stiffness matrices, the eigen-values and eigen-vectors (frequencies and mode shapes of vibration) are calculated using the Householder-QR technique. The seismic response of the coupled model is then calculated using the response spectrum, modal super-position method of dynamic analysis.

Case 2

NES rack type 14 has been analyzed using standard methods of seismic and structural analysis for simple structural systems. The fundamental frequency of vibration of the most flexible structural member (vertical angles) is first calculated considering its own flexibility characteristics as well as the flexibility characteristics of the horizontal support angles. Corresponding to this frequency, the equivalent static seismic accelerations are obtained from the SSE response spectrum curve. Seismic lateral inertia loadings, equal to the tributory mass times the equivalent static seismic accelerations are then applied to various structural members and the state of maximum stress and deflections are calculated using standard methods of structural analysis.



LEGEND

o Attachment Point
Δ Support Point

FIGURE 3 - MATHEMATICAL MODEL
SPENT FUEL STORAGE RACK TYPE G1
AND CONTROL ROD STORAGE RACKS TYPES G4 and G5

It should be noted that in these calculations, two models have been analyzed. In model 1, it has been conservatively assumed that the horizontal angles at the front and back of the NES racks act independently of each other. Since these horizontal angles at the front and back are tied together by spacer plates, a second model has been analyzed which assumes that these two horizontal angles act together. The structural stiffness of the NES rack will lie between these models.

6.4.5 Results of Analysis

The natural freuqencies of vibration for the first five modes of the coupled model of the fuel and control rod storage racks range from 11.76 cps to 38.94 cps.

The fundamental frequency of vibration of the NES rack type 14 for the two models analyzed are given below:

	Natural Frequency of Vibration (cps)
Model 1 (Horizontal angles act independently)	3.99
Model 2 (Horizontal angles act together)	4.02

The results of the subject analysis indicate the following:

1. The design of the existing fuel storage racks G1, control rod storage racks G4, G5 and their attachment bolts are adequate to withstand lateral inertia loadings associated with safe shutdown earthquake.
2. The existing design of the NES rack type 14 is adequate to withstand lateral inertia loadings associated with a seismic event. The factor of safety against the collapse during a seismic event is of the order of 3.3 if the horizontal angles at the front and back of the racks are assumed to act independently:

if these horizontal angles act together, the factor of safety against the collapse of these horizontal members during a seismic event is of the order of 27.

3. In order to provide backup to the attaching bolts between the existing racks G2 and the control rod storage racks G4 and G5 it is recommended that structural members be added between the NES type 14 racks and the existing type G2 racks to prevent their toppling during a seismic event.

6.5 HIGH DENSITY SPENT FUEL STORAGE RACKS (1978) (Reference 8)

6.5.1 Description

The LaCrosse Boiling Water Reactor high density spent fuel storage racks have been designed to meet the requirements for Seismic Category I structures.

The arrangement of the storage racks in the LACBWR fuel storage well is shown in Figure 3-1 of Reference 8. From this figure it can be seen that the fuel storage well has two (2) storage racks with a 9x8 array of fuel storage locations, one (1) storage rack with a 4x10 array of fuel storage locations, plus a special storage array similar to a 4x10 array except for a region allocated for control rod storage. Each storage location is capable of storing two (2) fuel assemblies in a two-tier cc figuration (i.e. one assembly positioned above the other). Fuel assemblies tored in the lower tier are accessible (e.g. for periodic surveillance). Floor area is provided at the South end of the fuel storage well for the spent fuel shipping cask. This area is also used to store the core spray bundle during refueling operations.

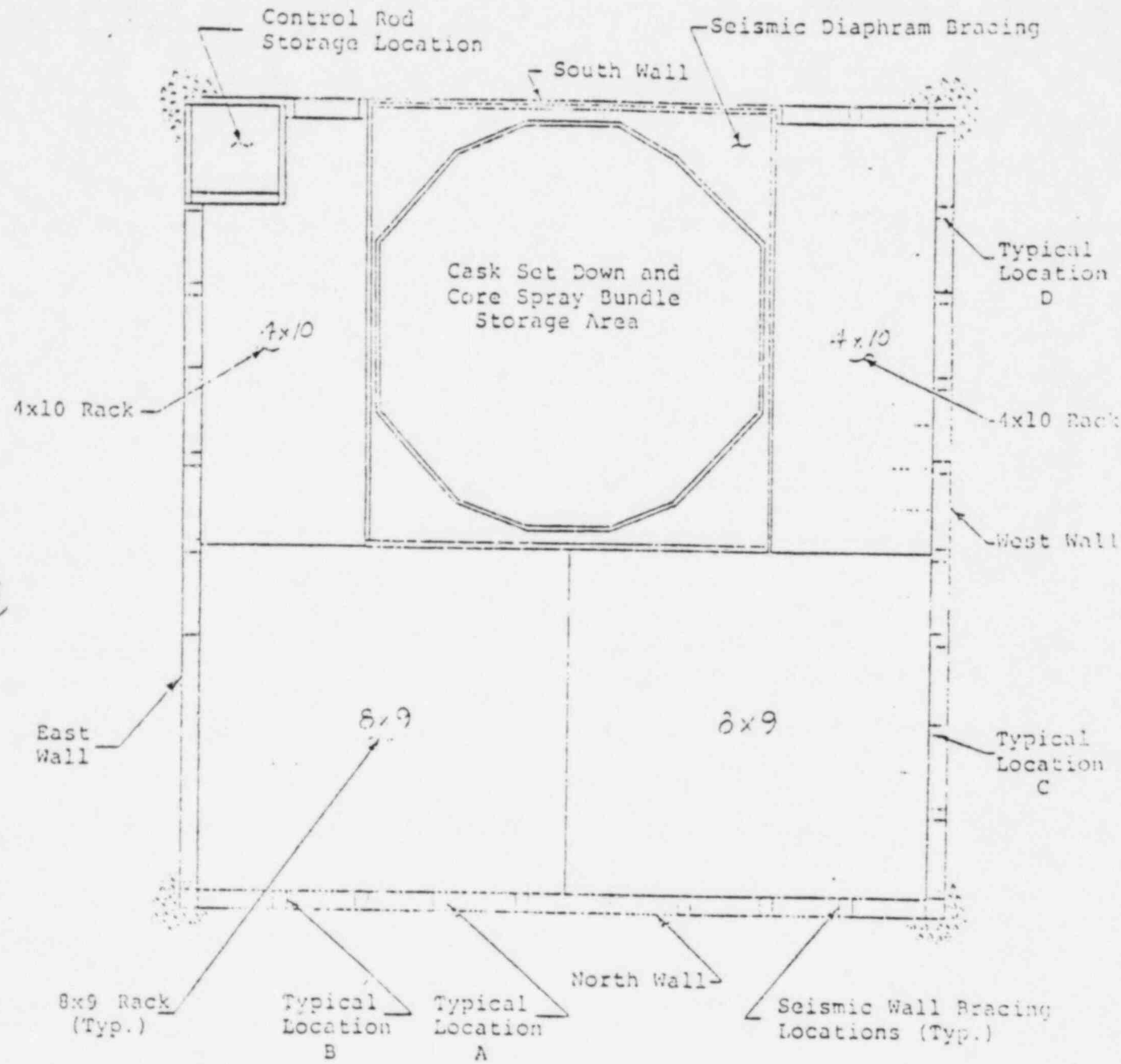


FIGURE 3.1

SPENT FUEL STORAGE RACK ARRANGEMENT PLAN

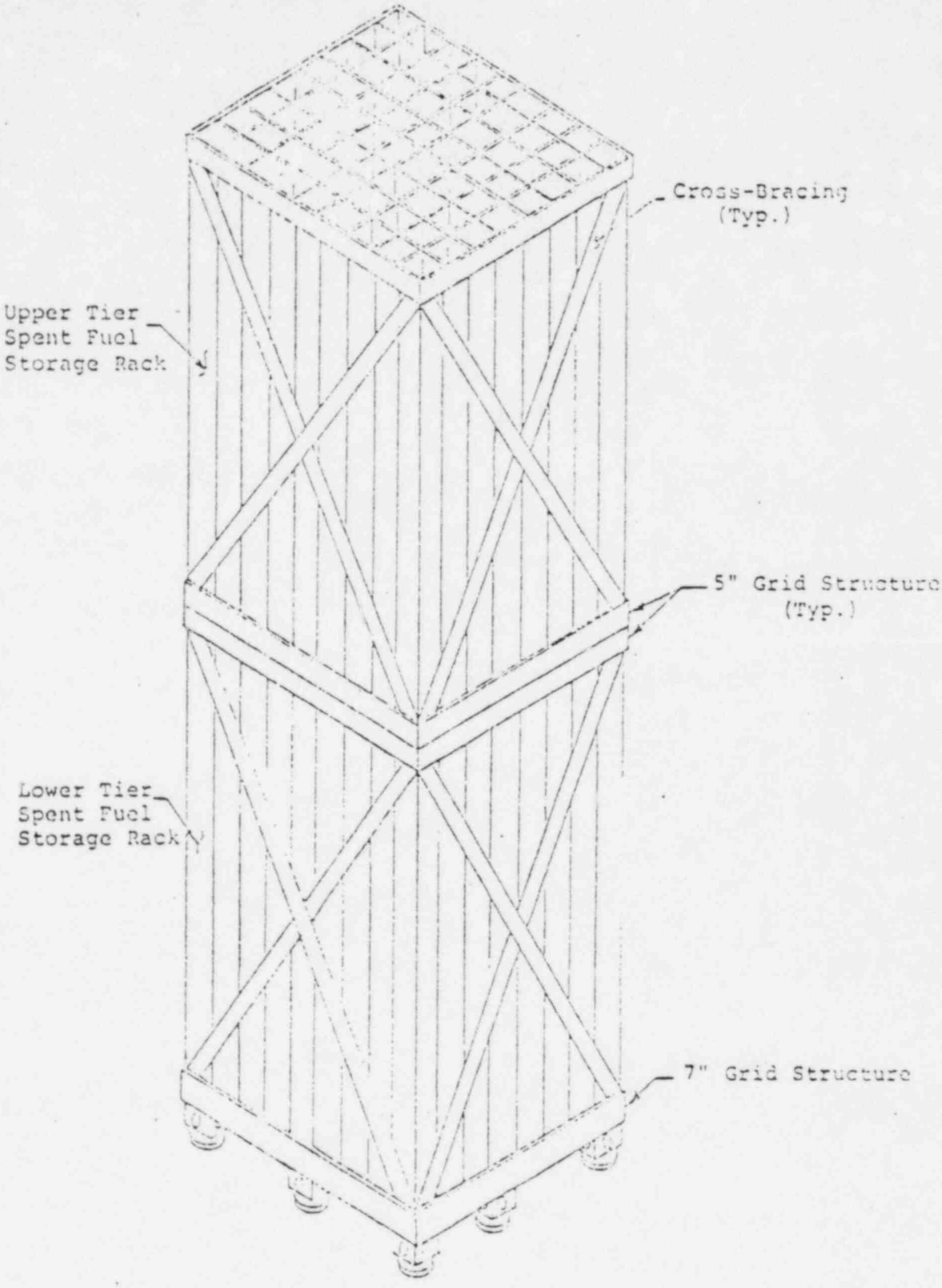


FIGURE 3.2

8x9 SPENT FUEL STORAGE RACK

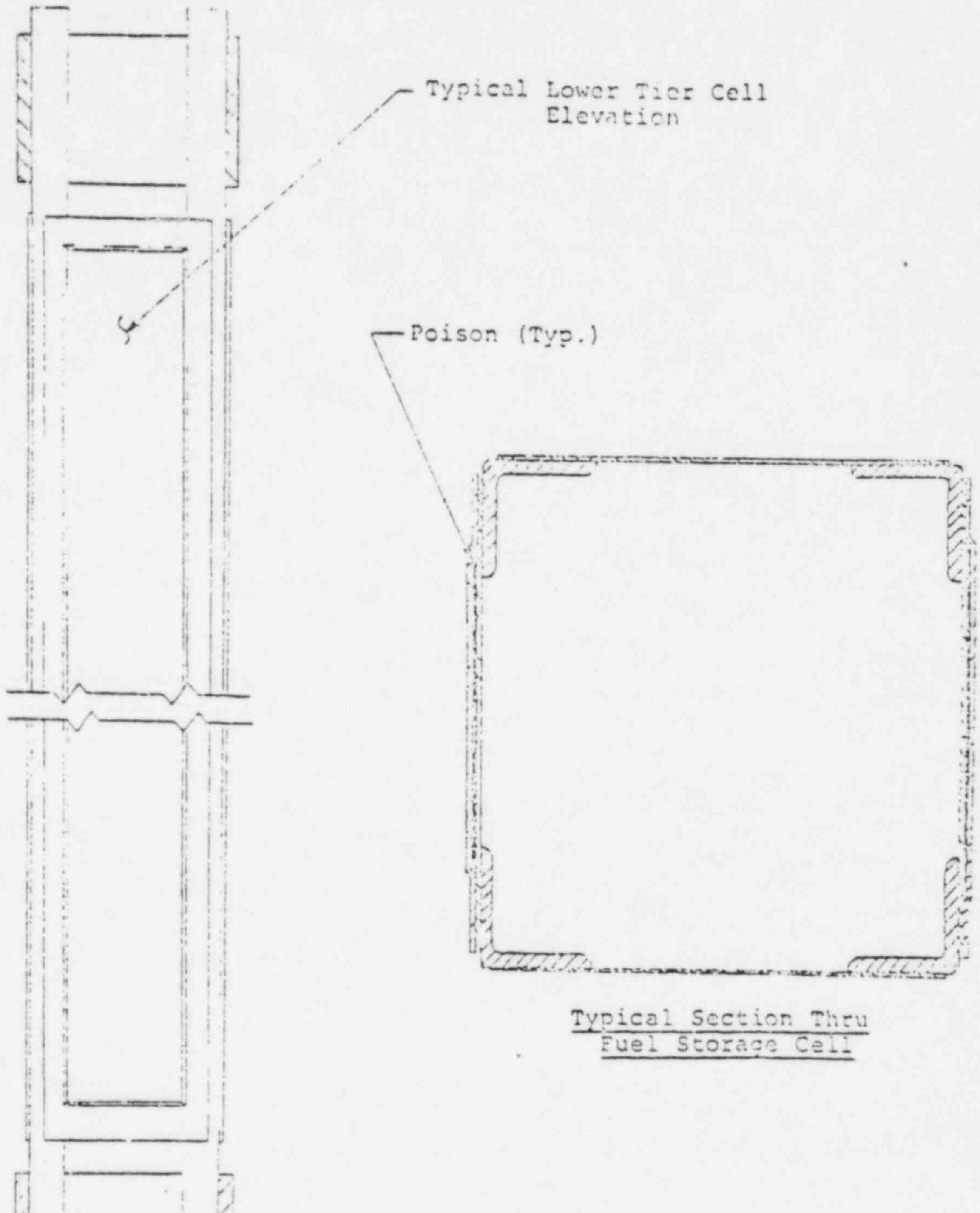


FIGURE 3.3

SPENT FUEL STORAGE CELL

Each storage rack consists of a welded assembly of fuel storage cells spaced 7 inches on center. Each rack, however, is fabricated in two sections designated upper tier rack and lower tier rack sections respectively. After the upper tier and lower tier rack sections are brought up to the operating floor, the sections are assembled to each other to complete each rack structure. The completed structure is shown in Figure 3.2 of Reference 8.

The upper tier rack section consists of two "egg-crate" grid structures which position and secure the fuel storage cells. A typical cross-section for the fuel storage cell is shown in Figure 3-3 of Reference 8.

The horizontal seismic loads are transmitted from the rack structures to the fuel storage well walls at three elevations (the top grid of the upper tier rack section, centerline of the inter-section of upper and lower rack tiers, and the bottom grid of the lower tier rack section) through adjustable pads attached to the rack structures. The thickness of these pads are adjusted as required to accommodate variations in the storage well walls and to provide the small gaps needed for thermal expansion. Lateral diaphram bracing are provided around the periphery of the cask setdown/core spray bundle storage area to ensure proper transfer of the seismic loads across and/or around this area at the three rack elevations. The vertical dead-weight and seismic loads are transmitted to the storage well floor by the rack support feet.

The fuel storage racks and associated seismic bracing are fabricated from Type 304 stainless steel.

6.5.2 Loading Conditions

Load Case 1 - Dead Weight of Rack, D + L (Normal Load)

Load Case 2 - Dead Weight of Rack Plus 1 G. Vertical Installation Load, D + I.L. (Normal Load)

Load Case 3 - Dead Weight of Rack Plus Uplifting Load, D + U.L.
(Abnormal Load)

Load Case 4 - ½ Safe Shutdown Earthquake, E (Severe Environmental Load)

The storage rack structural components are subjected to the seismic inertia loading of the fuel assembly, storage cell structure, trapped and hydrodynamic mass and the fuel assembly impact loads. The seismic loads are based on the simultaneous application of the horizontal and vertical components of the seismic response acceleration spectra specified for the ½ Safe Shutdown Earthquake.

Load Case 5 - Safe Shutdown Earthquake, E' (Extreme Environmental Load)

Same as Load Case 4 except that the seismic response acceleration spectra corresponding to the Safe Shutdown Earthquake was used in the analysis.

Load Case 6 - Assembly Drop Impact Load, (Abnormal Load)

Thermal Loading, T (Normal Load)

The stresses and reaction loads due to thermal loadings are insignificant since clearances are provided between racks to allow unrestrained growth of the racks for the maximum expected temperature differential based on a maximum pool temperature of 150°F.

Load Combinations

a. For service load conditions, the following load combinations are considered using elastic working stress design methods of AISC:

(1) D + L	(1a) D + L + T
(2) D + I.L.	
(3) D + L + E	(3a) D + L + T + E

b. For factored load conditions, the following load combinations are considered using elastic working stress design methods of AISC:

(4) D + L + T + E'

(5) D + T + U.L.

6.5.3 Design Criteria

The following design codes and regulatory guides have been used in the design/analysis of spent fuel storage racks.

1. A.I.S.C. Manual of Steel Construction, Seventh Edition, 1970.
2. USNRC Regulatory Guide 1.61, "Damping Values for Seismic Design of Nuclear Power Plants", October 1973.
3. USNRC Regulatory Guide 1.92, "Combination of Modes and Spatial Components in Seismic Response Analysis", Rev. 1, February 1976.
4. USNRC Standard Review Plan, Section 3.8.4.
5. Nuclear Energy Services Inc. Document NES81A0544, Rev. 0, "Quality Assurance Program Plan for the LaCrosse Boiling Water Reactor Spent Fuel Storage Rack Design Program", March 1978.
6. USNRC "Position Paper for Review and Acceptance of Spent Fuel Storage and Handling Applications".

The following allowable stress limits constitute the structural acceptance criteria used for each of the loading combinations presented in Section 6.5.2

Load Combinations	Limit
1, 2, 3	S
1a, 3a	1.5S
4, 5	1.6S or 0.5Fy (shear stress)
	0.9Fy (Tensile or compressive stress)

Where S is the required section strength based on the elastic design methods and the allowable stresses defined in Part 1 of the AISC "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings", February 12, 1969. The yield stress value Fy for stainless steel is taken as 30.0 ksi.

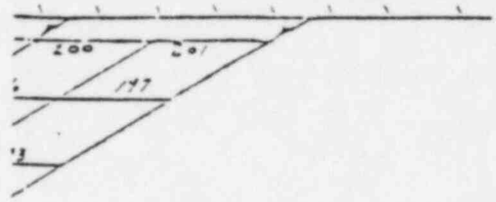
6.5.4 Methods of Analysis

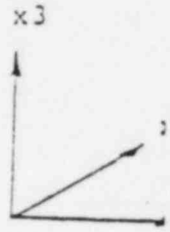
The response of the rack structure to the specified static and dynamic loading conditions have been evaluated by means of linear elastic analysis using the finite element method. The seismic response of the rack structure has been determined using response spectrum modal superposition methods of dynamic analysis.

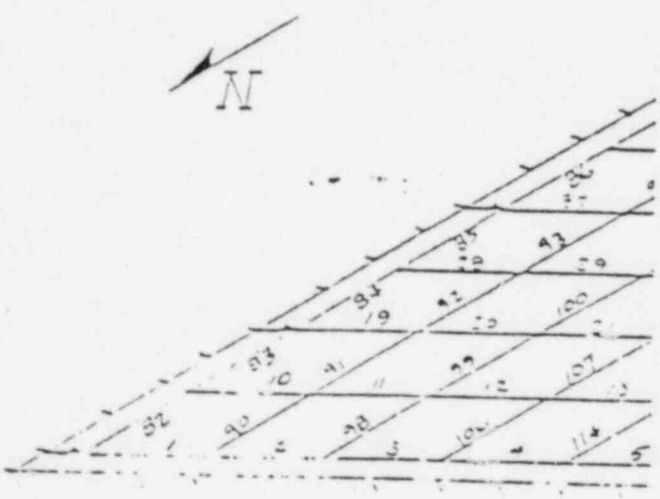
The following mathematical models have been developed to perform a static, dynamic and stress analysis of the spent fuel storage rack structure.

8x9 - 4x10 RACK COUPLED MODEL

The first model, shown in Figures 7.1.a and 7.1.b of Reference 8, is the coupled model of the grid structures of an 8x9 and 4x10 arrays located in the east side of the spent fuel pool (Figure 3.1). This coupled model of the structural grid arrays including the region for control rod storage represent the controlling structural case having higher flexibility characteristics (therefore, lower frequencies of vibration and high spectra accelerations). The grid structure models are detailed three dimentional finite element models consisting of discrete beam elements interconnected at a finite number of nodal points.




x3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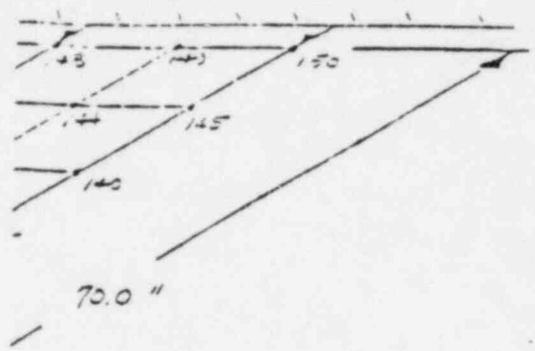
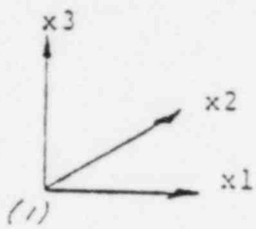
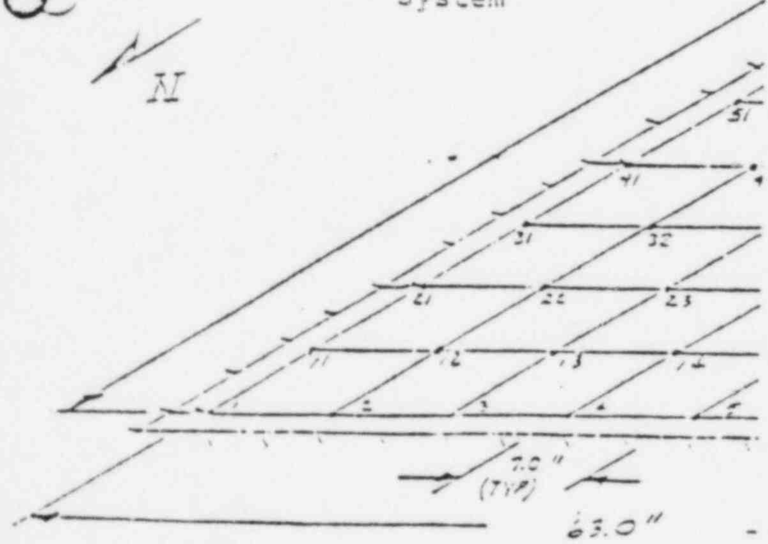


RACK GRID
TURE



Global Coordinate
System



NOTE: Both the lower 7" grid and the 5" grid structure having similar dimensions and nc numbers.

Boundary conditions are assumed at pinned connections at the appropriate vertical and horizontal locations for both the static and dynamic analysis. For the static dead weight analysis, the distributed masses of the structural elements and stored fuel elements of both tiers are lumped at the system nodal points of the lower grid structure model.

For the horizontal, seismic analysis, the tributory weight of the rack structure, individual stored cells, fuel assemblies, contained and hydrodynamic water masses are lumped at the appropriate nodal points. The horizontal seismic analysis is performed on both the lower grid and intermediate structural grid models. For the vertical seismic analysis, the distributed weight of the rack structures at the three grid elevations, individual storage cells, and stored fuel assemblies of both tiers are lumped at the appropriate nodal points of the lower grid model. The effects of the adjacent racks and seismic bracing are accounted for by means of developing equivalent spring/mass systems rpresenting their lateral dynamic characteristics and attaching these systems to appropriate nodal points around the periphery of the design model. The horizontal and vertical weights are distributed such that the resulting lumped mass multi-degree-of-freedom model best represents the dynamic characteristics of the fuel storage racks.

The effects of the storage cell lateral frequency are considered by combining the structural grids and storage cell frequencies for the first mode of vibration. The storage rack dynamic response is then calculated by applying the spectral acceleration value for the combined first mode frequency to all modes. Total system response is then obtained by combining the individual modal response values in accordance with Regulatory Guide 1.92; lower modes having large contribution to the response are considered and higher modes with negligible participation are neglected.

The combined seismic response of the three spatial components of the earthquake has been obtained by taking the square root of the sum of the squares of the corresponding maximum response values due to the three components calculated independently (Regulatory Guide 1.92).

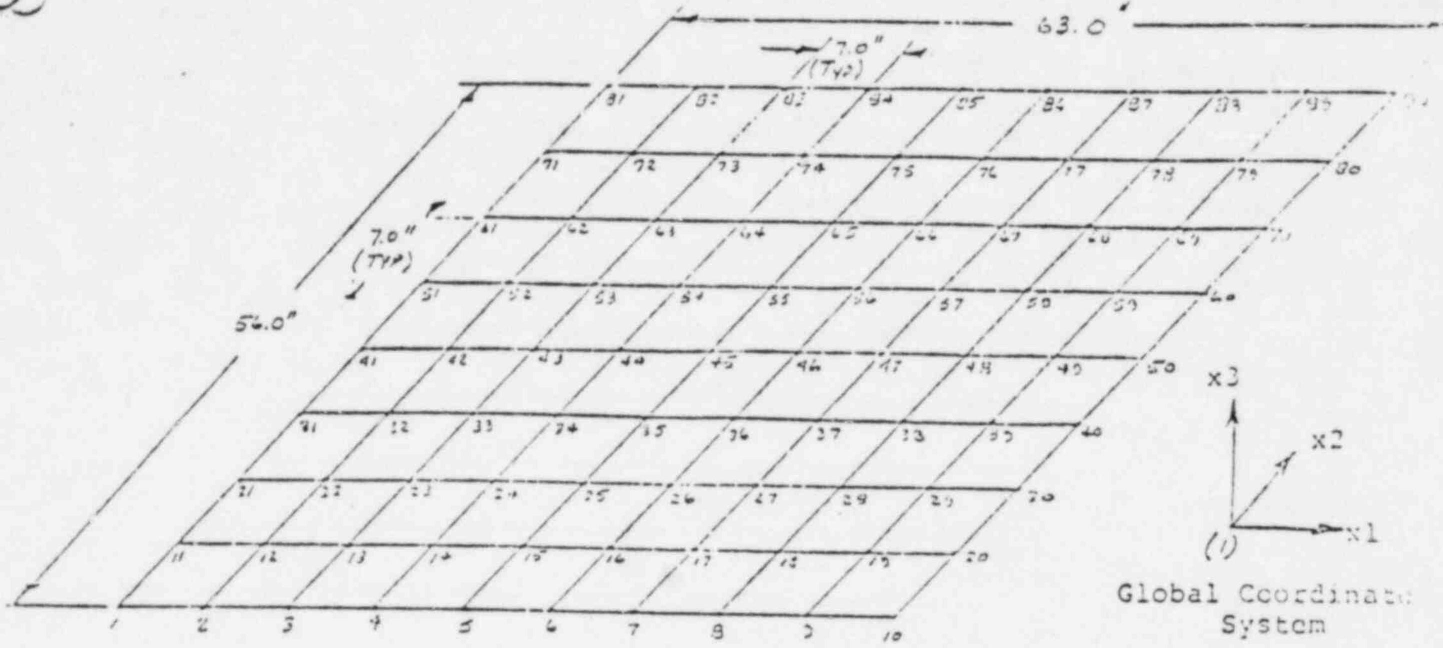
8x9 AND 4x10 RACK - INDIVIDUAL GRID MODELS

The second and third models shown in Figures 7.1.c and 7.1.d of Reference 8 are detailed three-dimensional finite element modes of individual 8x9 adn 4x10 rack intermediate grid structures. These models consist of discrete beam elements interconnected at a finite number of nodal points, and are used for the installation load analysis.

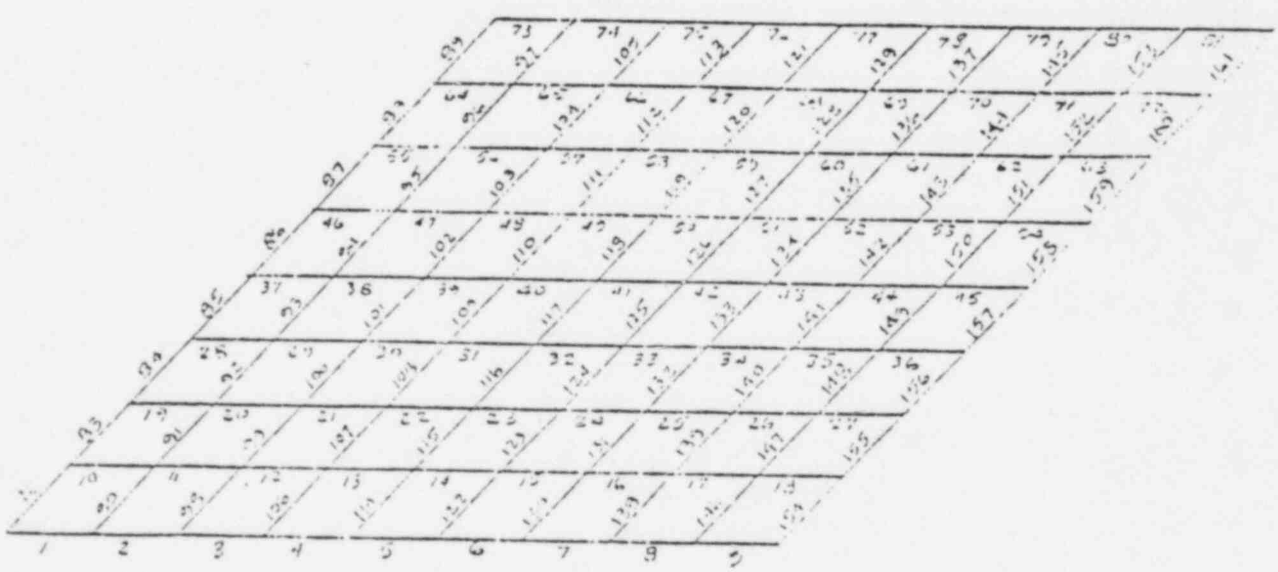
6.5.5 Results of Analysis

The static and seismic structural stress analysis of LACBWR high density fuel storage racks were performed with the STARDYNE computer code. The fundamental frequency of 14.65 cps for the lower tier and 11.81 cps for the upper tier represent the first mode frequency of the upper and lower grids (including the flexibility effects of seismic bracings) combined with the first mode frequency of the storage cells.

The results of the rack structural/stress analysis, which includes fuel assembly impact, are summarized in Table 8.2 of Reference 8. This table presents the maximum stresses and deflections in each type of rack structural member for the various load combinations developed in accordance with the NRC Standard Review Plan, Section 3.8.4 and compares them with the allowable values as specified in the accpetance criteria of Section 6.5.3. From this table, it can be seen that the maximum stresses and deflections are nominal and well within the allowable limits.



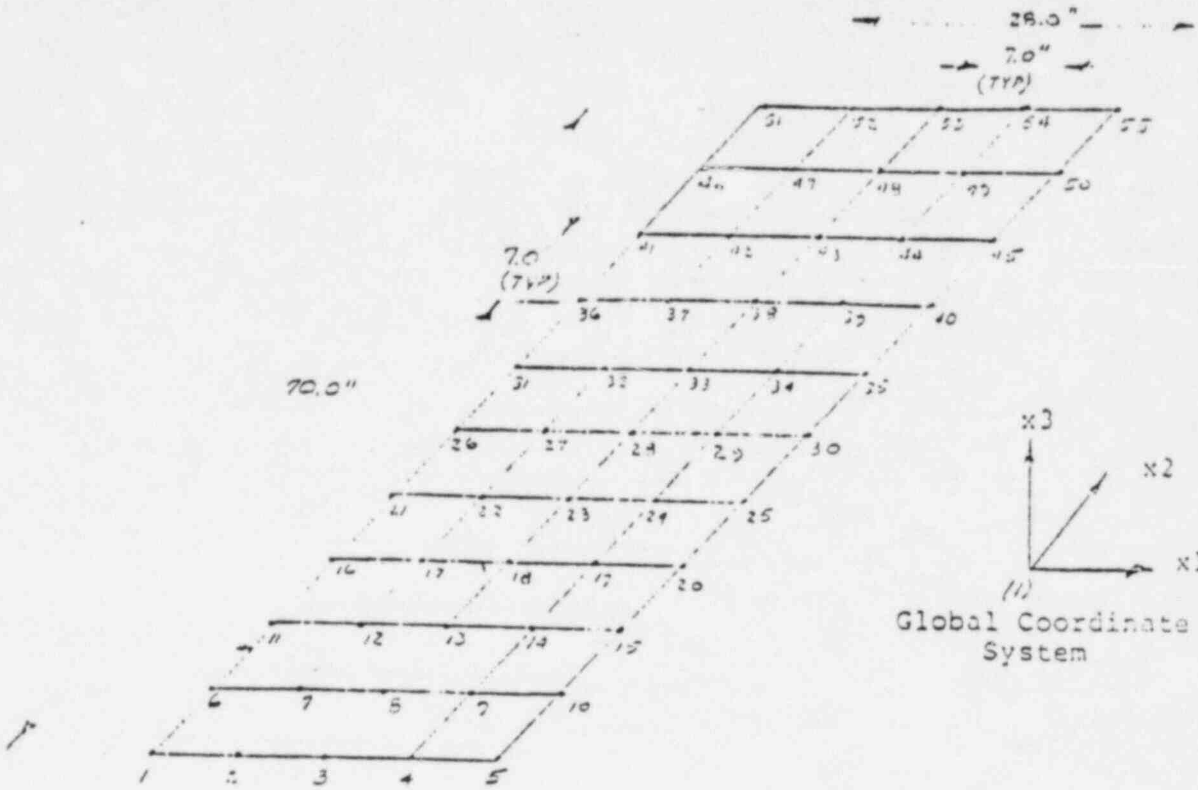
Dimensions and Node Numbers



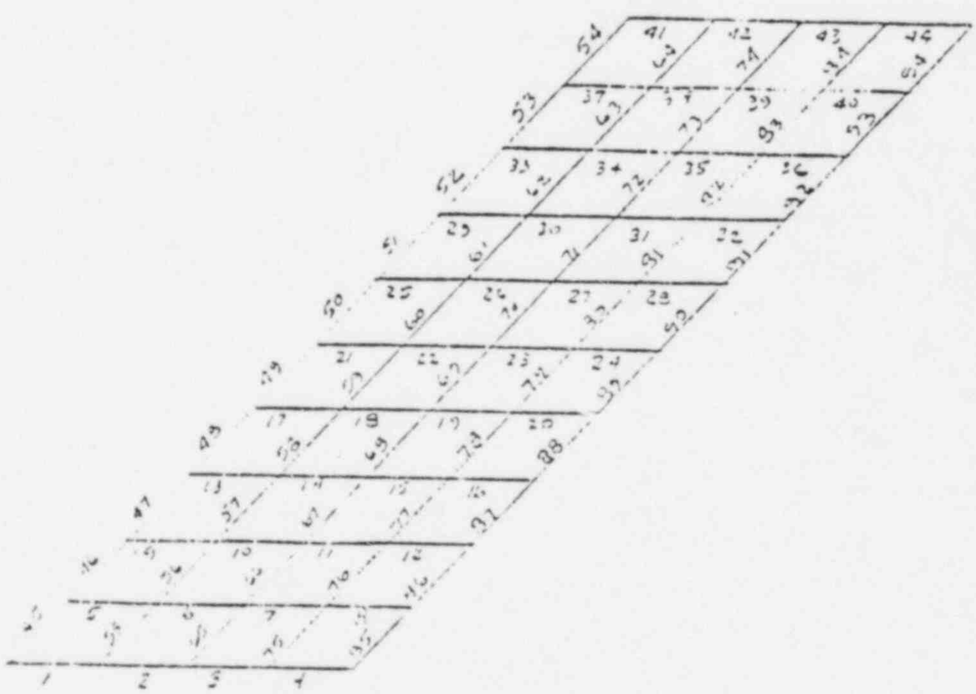
Beam Element Numbers

FIGURE 7.1.c

9x9 RACK GRID STRUCTURE
FINITE ELEMENT MODEL



Dimensions and Node Numbers



Beam Element Numbers

FIGURE 7.1.d

4x10 RACK GRID STRUCTURE
FINITE ELEMENT MODEL

The maximum reaction loads transmitted to the pool wall and floor resulting from the dead weight, live loads, thermal effects and seismic loadings (including fuel assembly impact effects) are presented in Table 8.3 of Reference 8. These maximum reaction loads are calculated ocnisdering the pool to be fully-loaded with spent fuel storage racks and the full compliment of spent fuel assemblies including stainless steel shrouds (heaviest shrouds).

Detail calculations to evaluate the effects of sloshing water on the fuel storage racks have been performed for two cases. In the first case, operating water level in the pool is considered to be at an elevation 697'-11.625" (typical water level required for storing the spent fuel in the upper tier of the storage racks). In the second case, water level in the pool is considered to be at the same elevation as top of the storage racks (approximate water level required for storing the spent fuel in the lower tier of the storage racks only). It has been concluded that sloshing of the pool water during a seismic event will have insignificant effects on the fuel storage racks.

The structural design and stress analysis of the seismic bracing around the rack periphery and seismic diaphram bracing for the three elevations of grid structures and their components are summarized in Table 8.4 of Reference 8. The stresses in these structures, as well as the developed concrete bearing stresses, are within the allowable limits of the AISC code and the ACI code as modified in accordance with NRC standard review plan Section 3.8.4.

In order to verify that the results of the detailed seismic analysis are suitably conservative, additional analyses were performed using a coupled model representing the storage cells and upper, intermediate, and lower grid structural systems.

Load Combination	Structural Element Description	ksi ing Allowable F_b	Combined Stress Ratio*
3. D+L+E - 7" Grid	8 x 9 Rack	18.0	0.200
(3a. D+L+T+E)	E-W Edge Mem	18.0	0.291
Dead weight of rack,	E-W Interior	18.0	0.204
fuel assembly weight	N-S Edge Mem	18.0	0.266
plus ½ SSE seismic	N-S Interior		
event applied to a 7"	4 x 10 Rack	18.0	0.126
rack grid structure	E-W Edge Mem	18.0	0.235
(negligible thermal	E-W Interior	18.0	0.362
load included)	N-S Edge Mem	18.0	0.347
	N-S Interior	18.0	
	Storage Cell		
4. D+L+E' - 7" Grid	8 x 9 Rack	27.0	0.178
(3a. D+L+T+E)	E-W Edge Mem	27.0	0.262
Dead weight of rack,	E-W Interior	27.0	0.214
fuel assembly wieght	N-S Edge Mem	27.0	0.242
plus SSE seismic event	N-S Interior		
applied to a 7" rack grid	4 x 10 Rack	27.0	0.114
structure (negligible	E-W Edge Mem	27.0	0.213
thermal load included)	E-W Interior	27.0	0.328
	N-S Edge Mem	27.0	0.315
	N-S Interior	27.0	
	Storage Cell		
5. D+U.L.	4 x 10 Rack	27.0	0.088
Dead Weight of rack	E-W Edge Mem	27.0	0.015
plus 4000# uplift load	E-W Interior	27.0	0.522
(negligible thermal	N-S Edge Mem	27.0	0.284
load included)	N-S Interior	27.0	
	Storage Cell		

* Must be ≤ 1.0 for component acceptanc

Load Combination	Structuring Element Description	ksi Allowable F_b	Combined Stress Ratio*
1. D+L (1a. D+L+T) Dead weight of 8 x 9 and 4 x 10 racks (plus negligible thermal load)	8 x 9 Rack		
	E-W Edge Mem	18.0	0.09
	E-W Interior	18.0	0.242
	N-S Edge Mem	18.0	0.082
	N-S Interior	18.0	0.250
	4 x 10 Rack		
	E-W Edge Mem	18.0	0.085
	E-W Interior	18.0	0.192
	N-S Edge Mem	18.0	0.240
	N-S Interior	18.0	0.227
	Storage Cell		
2. D+I.L Dead weight of 8 x 9 rack plus 1.0G vertical installation load	8 x 9 Rack		
	E-W Edge Mem	18.0	0.823
	E-W Interior	18.0	0.679
	N-S Edge Mem	18.0	0.7[illegible]9
	N-S Interior	18.0	0.617
	Storage Cell		
3. D+L+E - 5" Grid (3a D+L+E+T) Dead weight of rack fuel assembly weight plus ½ SSE seismic event applied to a 5" rack grid structure (negligible thermal loads included)	8 x 9 Rack		
	E-W Edge Mem	18.0	0.306
	E-W Interior	18.0	0.364
	N-S Edge Mem	18.0	0.584
	N-S Interior	18.0	0.623
	4 x 10 Rack		
	E-W Edge Mem	18.0	0.306
	E-W Interior	18.0	0.289
	N-S Edge Mem	18.0	0.507
	N-S Interior	18.0	0.547
	Storage Cell	18.0	
4. D+L+E' - 5" Grid (4a D+L+T+E') Dead weight of rack, fuel assembly weight, plus SSE seismic event applied to a 5" rack grid structure (negligible thermal load included)	8 x 9 Rack		
	E-W Edge Mem	27.0	0.274
	E-W Interior	27.0	0.323
	N-S Edge Mem	27.0	0.530
	N-S Interior	27.0	0.566
	4 x 10 Rack		
	E-W Edge Mem	27.0	0.274
	E-W Interior	27.0	0.262
	N-S Edge Mem	27.0	0.460
	N-S Interior	27.0	0.495
	Storage Cell	27.0	

* Must be ≤ 1.0 for component acceptance

TABLE 8.3

SPENT FUEL POOL WALL AND FLOOR LOADS

Pool Wall Loading Summary

Load Location (See Figure 8.1)	Maximum Wall Loading For Each Seismic Bracing (KIPS)					
	Lower Grid Elevation		Intermediate Grid Elevation		Upper Grid Elevation	
	½ SSE	SSE	½ SSE	SSE	½ SSE	SSE
A	3.09	4.22	4.46	6.08	2.23	3.04
B	5.87	8.01	10.04	13.70	5.02	6.85
C	6.80	9.28	15.74	21.48	7.87	10.74
D	3.55	4.84	3.98	5.40	1.98	2.70

Wall Designation	Total Wall Load (KIPS)					
	Lower Grid Elevation		Intermediate Grid Elevation		Upper Grid Elevation	
	½ SSE	SSE	½ SSE	SSE	½ SSE	SSE
North Wall	38.2	49.5	57.5	78.4	28.8	39.2
South Wall	38.2	49.5	57.5	78.4	28.8	39.2
East Wall	38.6	52.6	68.5	93.5	34.3	46.8
West Wall	38.6	52.6	68.5	93.5	34.3	46.8

Load Description	Spent Fuel Storage Rack Array Size	Maximum Floor Loads (KIPS)	
		D. + L. + ½ SSE 1% Equip. Damping	D. + L. + SSE 2% Equip. Damping
Max. Load For Each of the Corner Feet	8 x 9	10.6	11.37
	4 x 10	11.8	12.7
Max. Load For Each of the Center Feet	8 x 9	23.5	25.1
Max. Load for Each of the Mid-edge Feet	8 x 9	16.8	18.1
	4 x 10	18.67	21.4
Total 2 Tier Floor [illegible]	8 x 9	115.5	124.1
	4 x 10	57.7	79.0
Total Pool Floor Load		346.4	40[illegible].2

TABLE 8.4

SEISMIC BRACING ANALYSIS RESULTS

Location	Structural Element Description	Maximum Beam Stresses (ksi): Axial, Calculated fa, ½SSE	Axial, Calculated fa, SSE	Axial, Allowable Fa *, ½SSE	Axial, Allowable Fa *, SSE	Bending, Calculated Fb2+Fb3, ½SSE	Bending, Calculated Fb2+Fb3, SSE	Bending, Allowable Fb **, ½SSE	Bending, Allowable Fb **, SSE	Combined Stress Ratio, ½SSE	Combined Stress Ratio, SSE	Concrete Bearing Stresses (ksi): Calculated, ½SSE	Calculated, SSE	Allowable, ½SSE	Allowable, SSE
Upper Grid Seismic Diaphram Bracing	Beam Element	1.26	1.72	14.52	23.2	1.99	2.72	18.0	27.0	0.197	0.175				
	Plate Element	1.80	3.48	18.0	27.0	1.32	1.81	12.0	19.2						
Intermediate Grid Seismic Diaphram Bracing	Beam Element	2.52	3.44	14.52	23.2	3.98	5.44	18.0	27.0	0.376	0.329				
	Plate Element	3.60	6.96	18.0	27.0	2.65	3.61	12.0	19.2						
Lower Grid Cask Area Seismic Bracing	Edge Member	0.69	0.94	16.02	25.63	4.58	6.23	18.0	27.0	0.298	0.267				
	Interior Member	1.37	1.88	12.83	20.54	4.41	6.02	18.0	27.0	0.352	0.314				
Rack Periphery Seismic Bracing	Lower Grid														
	Location B #	0.49	0.67	18.0	27.0							0.49	0.67	1.05	1.785
	Location C	0.57	0.77	18.0	27.0							0.57	0.77	1.05	1.785
	Intermediate Grid														
	Location B	0.42	0.57	18.0	27.0							0.42	0.5.	1.05	1.785
	Location C	0.66	0.90	18.0	27.0							0.66	0.90	1.05	1.785
	Upper Grid														
	Location B	0.42	0.57	18.0	27.0							0.42	0.57	1.05	1.785
	Location C	0.48	0.66	18.0	27.0							0.48	0.66	1.05	1.785

* For Plate Results, Stresses are Maximum Von Misses Stresses.

** For Plate Results, Stresses are Maximum Shearing Stresses.

\# See Figure 3.1 for Locations.

The details of the model and the analysis are given in Appendix G of Reference 8. The results of these analyses are summarized in Table 8.6. From Table 8.6 it can be seen that the fundamental frequency and the overall seismic response results of both analyses (detailed and verification) are directly comparable.

The following conclusions were reached in Reference 8.

1. The results of the seismic and structural analysis indicate that the deflections and/or stresses in the rack structure resulting from the loadings associated with the normal and abnormal conditions are within allowable deflection and stress limits for Seismic Category I structures.
2. Sloshing of pool waters in a seismic event will have insignificant effects on the fuel storage racks.
3. The earthquake generated stresses in the seismic wall bracing and control rod rack are within the specified allowable values.
4. The analysis of the accidental fuel assembly drop condition indicates acceptable local structural damage to the storage cells with no buckling or collapse, no crumbing of the pool concrete floor and no puncturing of the stainless steel liner. Therefore, no significant changes in the value of k_{eff} will occur and the leak tightness of the fuel pool will be maintained.
5. It is concuded that the designs of the LaCrosse Boiling Water Reactor high density fuel storage racks, and the associated seismic bracing are adequate to withstand the loadings of normal and abnormal conditions.

TABLE 8.6

TWO TIER 54 MASS COUPLED MODEL SEISMIC ANALYSIS RESULTS

	Detailed Analysis Uncoupled Model	Verification Analysis Coupled Model
Overall System Fundamental Frequency (min. cps)	11.81	13.32
Spectral Acceleration Value (G)	0.45	0.45
Maximum Storage Cell Acceleration Value (G)	1.20	0.791
Maximum Storage Cell Seismic Stress (ksi)	6.18	3.84
Maximum Reaction Loads* (k)		
Lower Grid	23.16	12.97
Intermediate Grid	51.36	50.4
Upper Grid	25.66	29.24

*Reaction loads without fuel assembly impact effects.

6.6 SPENT FUEL POOL STRUCTURE

(Reference 12)

6.6.1 <u>Description</u>

The fuel storage pool is located inside the reactor containment building (south of the reactor pressure vessel) between elevation 659'-5-5/8" and 701'-3". The fuel storage pool is a 11' x 11' x 40' deep reinforced concrete structure lined with AISI Type 316 stainless steel plate. The 56 inch thick storage pool floor is lined with 3/8 inch thick stainless steel plate and is supported along its perimeter by the four pool walls and along its mid-span by a 29 inch thick wall. The pool walls, which vary in thickness, are lined with a 1/16 inch thick stainless steel sheet.

Elevation sections of the pool floor, the north, south, east and west walls including their detailed reinforcement patterns, changes in wall thickness and pool floor support walls are indicated in Figures 3.1 and 3.2 of Reference 12.

The horizontal s ismic loads are transmitted from the rack structures to the fuel storage pool walls at three elevations (the top grid of the upper tier rack section, centerline of the inter-section of upper and lower rack tiers, and the bottom grid of the lower tier rack section) through adjustable pads attached to the rack structures. The vertical dead-weight and seismic loads are transmitted to the storage pool floor by the rack support feet. The impact loads associated with the cask drop event are transmitted to the pool floor by the crash pad.

6.6.2 Loading Conditions

Load Case 1 - Dead Weight D (Normal Load)

Load Case 2 - Live Load, L (Normal Load)

Load Cases 3 to 6 - 1/2 Safe Shutdown Earthquake,

E (Severe Environmental Load)

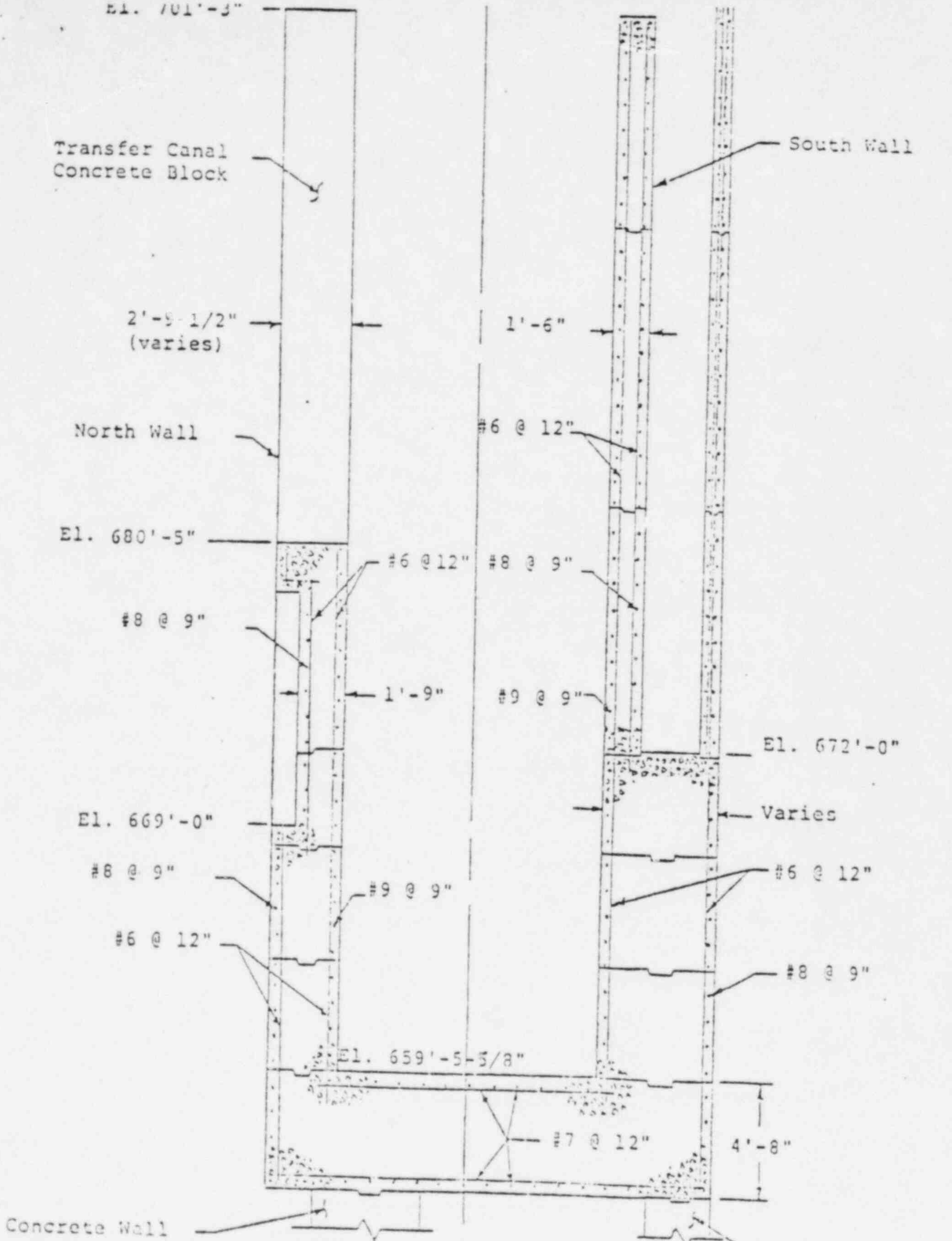


FIGURE 3.1

FUEL STORAGE POOL ELEVATION - NORTH AND SOUTH WALLS

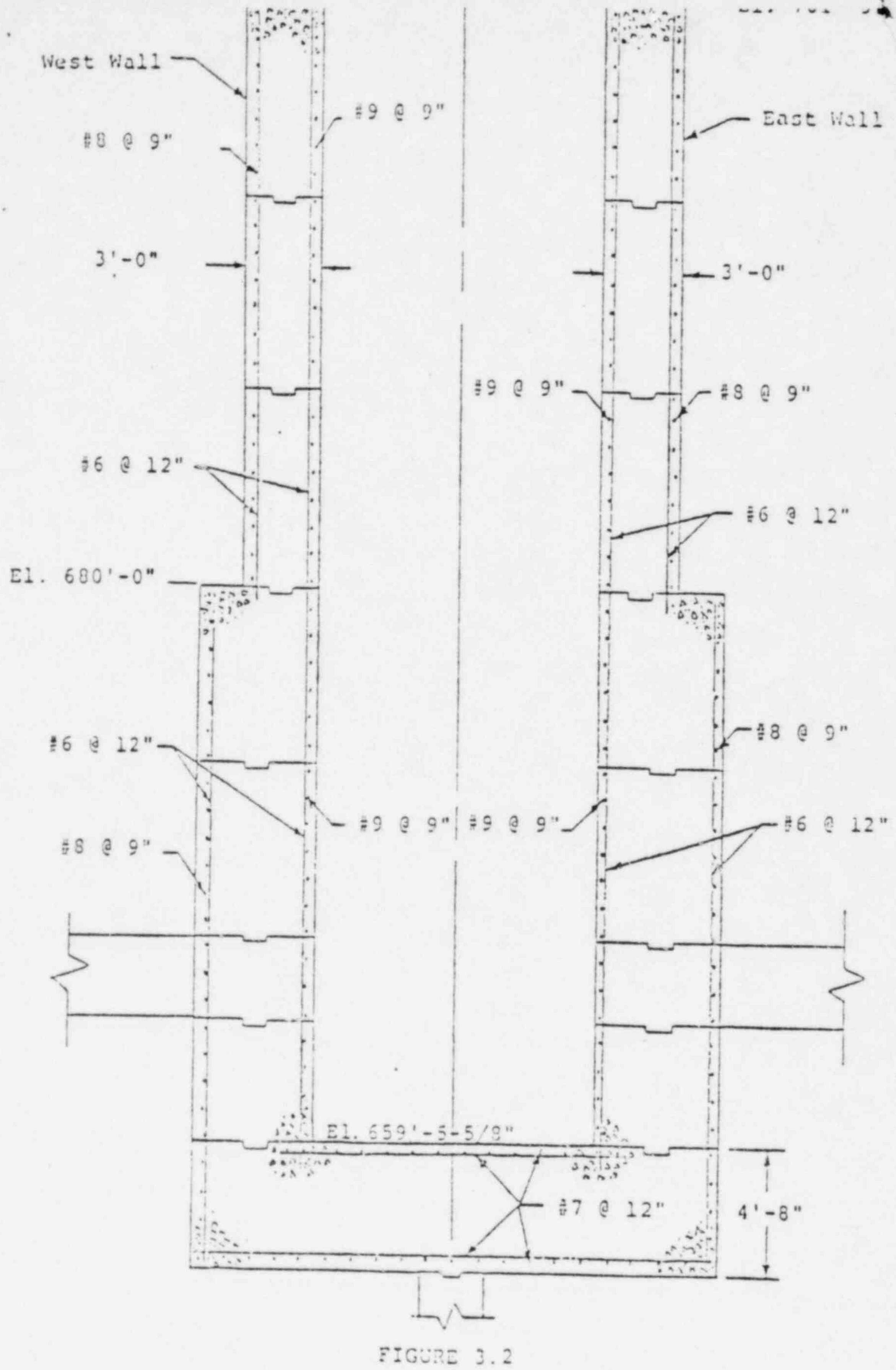


FIGURE 3.2

FUEL STORAGE POOL ELEVATION - EAST AND WEST WALL

The fuel storage pool walls are individually subjected to the seismic inertia loading of the concrete walls, pool water mass, and the maximum seismic reaction loads of the fuel storage racks (Section G.5) for the 1/2 Safe Shutdown Earthquake event.

The load combinations involving the Safe Shutdown Earthquake (E') are less severe than those involving the 1/2 Safe Shutdown Earthquake (E) while the acceptance criteria for these load combinations are same. Therefore, the analyses have been performed for the 1/2 Safe Shutdown Earthquake loading condition only.

Load Case 7 - Thermal Loading, T_o (Normal Load)

The pool floor and walls are analyzed for a linear thermal gradient of 80°F across the thickness of concrete elements.

Load Case 8 - Spent Fuel Shipping Cask Drop Impact

Load I.L. (Abnormal Load)

Load Combinations

a. For service load conditions, the following load combinations are considered using the ultimate strength design methods of ACI-318-71.

(1) 1.4 D + 1.7 L

(2) 1.4 D + 1.7 L + 1.9 E

(3) 0.75 (1.4 D + 1.7 L + 1.7 T_o)

(4) 0.75 (1.4 D + 1.7 L + 1.9 E + 1.7 T_o)

b. For factored load conditions, the following load combinations are considered using the ultimate strength design methods of ACI-318-71.

(2) 1.4 D + 1.7 L + 1.9 E > D + L + E'*

(5) 1.4 D + 1.7 L + I.L.

6.6.3 Design Criteria

The following design codes, regulatory guides and references have been used in the structural analysis of the fuel storage pool structure.

1. ACI 318-71 - "Building Code Requriements for Reinforced Concrete" American Concrete Institute.
2. Uniform Building Code, 1973 Edition.
3. USNRC Standard Review Plan, Section 3.8.4.
4. "USNRC Proposed Position for Review and Acceptance of Spent Fuel Storage and Handling Application."
5. Nuclear Energy Services, Inc. document NES 81A0544, Rev. 0. "Quality Assurance Program Plan for the LaCrosse Boiling Water Reactor Spent Fuel Storage Rack Design Program", March 1978.
6. George Winter, et al - "Design of Concrete Structures", McGraw Hill Book Company, 1964.

The following allowable stress/load limits constitute the structural acceptance criteria used for each of teh loading combinations discussed in Section 6.6.2.

Load Combinations	Limit
1, 2, 3, 4, 5	U

Where U is the required section strength based on the ultimate strength design methods described in ACI-318-71. The compressive strength of concrete at 28 days is taken at 3500 psi (Reference 10).

6.6.4 Methods of Analysis

The fuel storage pool floor and walls have been mathematically represented by a three dimensional finite element model (Figure 7.1.a of Reference 12) consisting of plate elements and having appropriate boundary conditions. The response of the finite element model of the storage pool structures to the applicable loads have been determined using linear static analysis methods. The computer code STARDYNE was used.

6.6.5 Results of Analysis

Table 8.2 of Reference 12 presents the results for load combination No. 2. From this table it can be seen that the maximum shear stress, compressive stress, critical (horizontal and vertical reinforcements) design moment values of 0.075 ksi, 0.167 ksi, 695.3 K. in/ft and 77.8 K.in/ft respectively are lower than the corresponding allowable values of 0.20 ksi and 2.082 ksi, 2142.0 K in/ft and 528.0 K in/ft respectively.

The results of the storage pool structural analysis for load combinations which includes the effects of dead, live earthquake and thermal loadings are summarized in Table 8.4 of Reference 12. It shows that in the critical section (pool floor) the maximum moment of 702.9 K in/ft for load combination 4 is lower than the allowable value of 1200 K in/ft.

The effects of additional loadings from the adjacent building structures on the pool structures are evaluated in Appendix D of Reference 12. The sum of the ratios of maximum shear stress to allowable shear stress for the pool structure and for the overall building structure is 0.479. Similarly, the sum of the ratios for the maximum moment to allowable moment is 0.432. Since these two ratios are less than 1, it can be concluded that the storage pool structures are adequate to withstand its own internal loadings as well as those from the adjacent building structures.

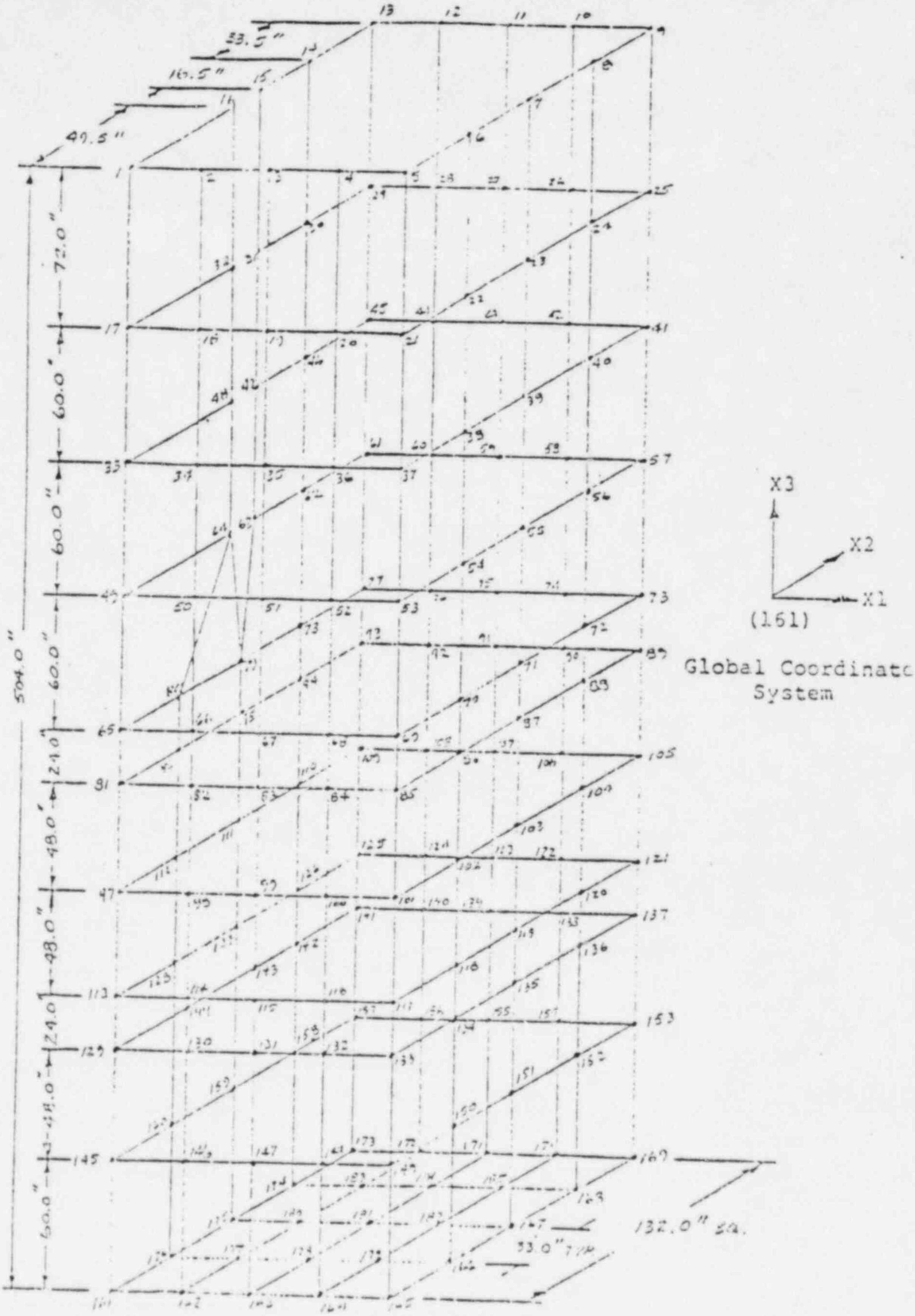


FIGURE 7.1.a
SPENT FUEL STORAGE POOL
FINITE ELEMENT MODEL NODE NUMBERS AND DIMENSIONS

TABLE B.2

RESULTS OF THE STORAGE POOL ANALYSIS LOAD COMBINATION #2
1.4D + 1.7 L + 1.9 E - OBE SEISMIC EVENT

STRUCTURAL ELEMENT DESCRIPTION	MAXIMUM * SHEAR STRESS		MAXIMUM ** COMPRESSIVE STRESS		MAXIMUM TENSILE STRESS				MAXIMUM DESIGN MOMENT		ALLOWABLE MOMENT		DESIGN/ALLOWABLE MOMENT RATIO	
	Element No.	Stress (Ksi)	Element No.	Stress (Ksi)	Element No.	Horizontal Reinf. (Psi)	Element No.	Vertical Reinf. (Psi)	Horizontal Reinf. (K-in/ft)	Vertical Reinf. (K-in/ft)	Horizontal Reinf. (K-in/ft)	Vertical Reinf. (K-in/ft)	Horizontal Reinf.	Vertical Reinf.
Pool Floor	171	0.066	163	0.136	162	0.069								
North Wall														
El. 680'-5" to 701'-3" (36" Elements)	61	0.037	61	0.066	61	0.06	61	0.030	155.5	77.8	1260.0	528.0	0.123	0.143
El. 680'-5" to 701'-3" (21" Elements)	TRPT #1	0.027	1	0.056	1	0.041	1	0.02	36.2	17.6	714.0	299.2	0.051	0.059
El. 678'-5" to 650'-5" (23.5" Elements)	78	0.045	78	0.104	78	0.088	77	0.018	197.5	40.4	1239.0	519.6	0.159	0.078
El. 659'-5.625 to 678'-5" (36" Elements)	144	0.075	128	0.167	128	0.147	125	0.011	381.0	28.5	1260.0	528.0	0.302	0.054
El. 659'-5.625" to 679'-5" (21" Elements)	143	0.046	127	0.131	126	0.102	126	0.026	90.0	22.93	714.0	299.2	0.126	0.077
South Wall														
El. 672'-0" to 701'-3" (14" Elements)	86	0.027	102	0.083	70	0.016	70	0.008	10.4	5.2	504.0	211.2	0.021	0.025
El. 659'-5.625" to 672'-0" (57.6 Elements)	119	0.041	134	0.078	152	0.010	134	0.009	65.0	58.5	2142.0	897.6	0.030	0.065
East Wall														
El. 680' to 701'-3" (36.0" Elements)	55	0.041	42	0.071	42	0.092	42	0.011	238.5	28.5	1260.0	528.0	0.189	0.054
El. 659'-5.625" to 680' (57.0" Elements)	74	0.054	74	0.084	74	0.106	155	0.018	688.8	117.0	2142.0	897.6	0.322	0.130
West Wall														
El. 680'-0" to 701'-3" (36.0" Element)	51	0.041	35	0.071	35	0.092	35	0.011	238.5	28.5	1260.0	528.0	0.189	0.054
El. 659'-5.625" to 680'-0" (57" Elements)	67	0.056	67	0.084	67	0.107	146	0.018	695.3	117.0	2142.0	697.6	0.325	0.130

* Allowable Shear Stress = 0.201 ksi
** Allowable Compressive Stress = 2.012 ksi

TABLE 8.4

RESULTS OF THE STORAGE POOL STRUCTURAL ANALYSIS
LOAD COMBINATION #4, 0.75(1.4D + 1.7L + 1.9E + 1.7T_o)

STRUCTURAL ELEMENT DESCRIPTION	MAXIMUM DESIGN MOMENT	ALLOWABLE MOMENT	DESIGN/ALLOWABLE MOMENT RATIO
	HORIZONTAL REINFORCEMENT (K-in/ft)	HORIZONTAL REINFORCEMENT (K-in/ft)	
Pool Floor (56" Element)	702.9	1200.0	0.586
North Wall			
El. 680'-5" to 701'-3" (36" Elements)	538.9	1260.0	0.428
El. 680'-5" to 701'-3" (21" Elements)	210.8	714.0	0.294
El. 678'-5" to 680'-5" (33.5" Elements)	505.3	1239.0	0.408
El. 659'5.625" to 678'-5" (36" Elements)	708.0	1260.0	0.562
El. 659'-5.625" to 678'-5" (21" Elements)	251.1	714.0	0.352
South Wall			
El. 672'-0" to 701'-3" (18" Elements)	149.1	504.0	0.296
El. 659'-5.625" to 672'-0" (57" Elements)	779.1	2142.0	0.364
East Wall			
El. 680'-5" to 701'-3" (36" Elements)	601.2	1260.0	0.477
El. 659'-5.625" to 680'-5" (57" Elements)	1246.9	2142.0	0.582
West Wall			
El. 680'-5" to 701'-3" (36" Elements)	601.2	1260.0	0.477
El. 659'-5.625" to 680'-5" (57" Elements)	1246.9	2142.0	0.582

6.7 STORAGE TANKS OF THE WASTE DISPOSAL BUILDING

(Reference 2. Part 2. Section 7)

The tanks considered were the 1000 gallon Evaporation Feed Tank, Spent Resin Tank, Evaporator and Concentrated Waste Tank. The first four of these tanks are supported on four legs and the fifth on saddle plants. The area of primary concern was the stability of the storage tanks to overturning due to the SSE seismic input condition.

Dynamic analyses were conducted to determine the peak stresses that would develop in the leg supports. These analyses were response spectra methods of analyses and yielded conservative estimates of peak stresses which were well within the allowable stresses. It is concluded that no special problems exist in this area.

REFERENCES

1. "La Crosse Boiling-Water Reactor SAFEGUARDS REPORT for Operating Authorization", Allis-Chalmers AEC Division, July 1965.
2. "Seismic Evaluation of the La Crosse Boiling Water Reactor", Gulf United Services, January 1974, Docket 50409-172.
3. "Seismic and Stress Analysis of LACBWR, Main Steam Piping System", Nuclear Energy Services, Inc., 1975.
4. "Seismic and Stress Analysis of LACBWR Feedwater Piping System", Nuclear Energy Services, Inc., 1975.
5. "Seismic and Stress Analysis of LACBWR Recirculation Piping System", Nuclear Energy Services, Inc., 1975.
6. "Seismic and Stress Anslysis of the LACBWR High Pressure Core Spray Suction Line Piping System", Nuclear Energy Services, Inc., 1976.
7. "Seismic and Stress Analysis of the LACBWR High Pressure Core Spray Discharge Line Piping System", Nuclear Energy Service, Inc., 1977.
8. "Structural Analysis Design Report for the LACBWR High Desnity Spent Fuel Storage Racks", Nuclear Energy Services, Inc., 1978.
9. "Structural Analysis Report for the LACBWR Spend Fuel Pool Structure", Nuclear Energy Services, Inc., 1978.
10. "Seismic and Stress Analysis of LACBWR Spent Fuel Storage Pool Drain Line", Nuclear Energy Services, Inc., 1978.
11. "Description and Design Criteria for Diesel Generator Building and Second Emergency On-Site Electrical Power Systems, LACBWR", Sargent & Lundy, 1974.
12. "Structural Analysis Report for the La Crosse Boiling Water Reactor Spent Fuel Pool Structure", Nuclear Energy Services, Inc., 1978.

13. "Piping Flexibility Analysis for LACBWR Project", Sargent & Lundy Engineers, October 1965.

14. "Specification for Piping System - La Crosse Boiling Water Reactor", Sargent & Luncy, LACBWR #256.

15. "La Crosse Boiling Water Reactor Safeguards Report", Volume I and II, LACBWR #283, August 1967.

16. "Seismic and Structural Analysis of Spent Fuel Storage Racks for LACBWR Nuclear Power Plant", Nuclear Energy Services, Inc., Docket 50409-278, July 1975.

17. "Safety Analysis Report, LACBWR", July 1978.

APPENDIX

SUMMARY TABLE, SEISMIC ANALYSIS OF

STRUCTURAL SYSTEMS - LACBWR

STRUCTURE	DOCUMENT	INVESTIGATOR/DATE	SUPPORTING MEDIA	FOUND. TYPE	CASE	INPUT DESCRIPT.	MAX. GROUND ACCEL	COMPUTER PROGRAM	METHOD OF ANALYSIS
REACTOR CONTAINMENT BUILDING	REF. 2 (5-369-172)	DES CONSTRUCTION & MILLER (CONY), GOLF UNITED / 1974	SOIL	CONCRETE - STEEL PILES	½ SSE	MULTIPLE SUPPORT TIME HISTORY	½ of SSE	SIM	TIME-HISTORY MULT SUP.
					SSE		0.12g AT SURFACE 0.11g AT -135'		MULTI-SUPPORT
TURBINE BUILDING	"	"	SOIL		SSE	0.3g FROM GROUND SPECTRA	[illegible]	N/A	EQUIVALENT STATIC
LACBWR STACK	"	"	SOIL	PILES	SSE	MULTIPLE SUPPORT TIME HISTORY	0.12g AT SURFACE	SIM	TIME-HISTORY MULT SUP.
GENOA II STACK	"	"	"	"	"	"	"	"	"

SUMMARY TABLE, SEISMIC ANALYSIS OF

STRUCTURAL [illegible] - LACBWR

[illegible] INTERACTION ANALYSIS			[illegible] OF COMBINING [illegible] DUE TO		[illegible] & ACCEPTANCE CRIT	DYNAMIC ANALYSIS		
FOUNDATION MODEL	DOF & FOUND	INPUT LOC	3 COMP [illegible]	SEISMIC & OTHER LOADS		MODEL	MODAL COMB	DAMPING
[illegible] SPRINGS [illegible] ON PILES	ONE/PILE NODE	EACH PILE NODE	1 HORIZ COMP ONLY	[illegible] ADDED TO [illegible] LOAD	$M < 0.63 M_U$ $V < 0.6 V_U$ / $M < M_U$ $V < V_U$	LUMPED MASS STICK	VA	[illegible] / 7% [illegible]
[illegible]	NA	GROUND SURFACE	1 HORIZ. COMP. ONLY	SEISMIC ONLY	ALLOWABLE STRESS AISC CODE	SDOF MODEL /[illegible]	VA	7%
SOIL [illegible] / [illegible]	ONE/PILE NODE	[illegible] PILE NODE	1 HORIZ COMP ONLY	SEISMIC ONLY?	$M < M_U$ $V < V_U$	[illegible]	VA	7% & UP
"	"	"	"	"	"	"	"	"

3

DYNAMIC ANALYSIS NO. [illegible]	DOF/NODE	EL. TYPE	MATERIAL PROP.	SUPPLEMENTARY REMARKS
10	2?	BEAM	ELASTIC	
1	1	[illegible] FRAME	ELASTIC	
10	2?	BEAM	ELASTIC	
	"	"	"	

SUMMARY TABLE, SEISMIC ANALYSIS OF MAJOR PIPING - LACBWR

ITEM	DOCUMENT	INVESTIGATOR /DATE	METHOD OF ANALYSIS	INPUT DESCRIPTION	COMPUTER PROGRAM	MODAL COMBINATION	DAMPING	NO OF MODES	[illegible] /MODE
[illegible]	REF 3	NUCLEAR ENERGY SERVICES/1975	RESPONSE SPECTRUM METHOD	FLOOR [illegible] VERT. [illegible]	PIPESD	SRSS	1% OBE 2% SSE	ALL [illegible] < 30 HZ INCLUDED	6 [illegible] D.O.F)
FEEDWATER PIPING	REF 4	"	"	"	"	"	"	"	"
RECIRCULATION PIPING	REF 5	"	"	"	"	"	1% OBE	"	"
HPCS SUCTION LINE	REF 6	" /1976	"	"	"	"	1% OBE 2% SSE	"	"
HPCS DISCHARGE LINE	REF 7	" /1977	"	"	"	"	"	"	"
SPENT FUEL STORAGE POOL [illegible] LINE	REF 10	" /1978	"	"	"	"	"	ALL [illegible] < 35 HZ INCLUDED	"

5.

SUMMARY TABLE [illegible] ...MIC ANALYSIS OF

MAJOR PIPING - LACBWR

[illegible] MODEL	[illegible] TYPE	[illegible]	FLEX. FACTOR	METHOD OF COMBINING RESPONSES DUE TO 3 COMP MOTION	INERTIA & B.L.MOV	LOAD COMBINATION	SUPPLEMENTARY REMARKS
LUMPED MASS	ELASTIC PIPE	ASME CODE CLASS 2	ASME CODE	MAX. HOR + VERT.	ASME CODE CLASS 2	EQ. 8, 9, 10 & 11 OF ASME CODE	4 ADDITIONAL SEISMIC RESTRAINTS REQUIRED
"	"	"	"	"	"	"	11 "
"	"	"	"	"	"	"	11 "
"	"	ASME CODE CLASS 1	"	"	ASME CODE CLASS 1	EQ 9 TO 14 OF ASME CODE	5 "
"	"	"	"	"	"	"	REQUIRE ADDITIONAL SEISMIC RESTRAINTS
"	"	"	"	"	"	"	

SUMMARY TABLE SEISMIC ANALYSIS OF MAJOR EQUIPMENT - LACBWR

ITEM	DOCUMENT	INVESTIGATOR /DATE	METHOD OF ANALYSIS	INPUT DESCRIPTION	COMPUTER PROGRAM	MODAL COMBINATION	DAMPING	NO OF MODES	REF [illegible]
REACTOR CORE SUPPORT	REF 2	GULF UNITED SERVICE/1974	EQUIVALENT STATIC	STATIC LATERAL LOAD		NA		NA	
[illegible]	REF. 2	"	SPECTRUM METHOD	FLOOR SPECTRUM	—	—		1	—
CONTROL ROD DRIVE	"	"	RELATIVE DEFORMATION	DISPLACEMENT	—	—	—	—	
SPENT FUEL STORAGE RACKS	REF. 16	NUCLEAR ENERGY SERVICES/1975	RESPONSE SPECTRUM METHOD	FLOOR SPECTRUM	STARDYNE		2%	5	6
HIGH DENSITY SPENT FUEL STORAGE [illegible]	REF. 8	" /1978	RESPONSE SPECTRUM METHOD	½ SSE & SSE SPECTRA	STARDYNE	SRSS			6
SPENT FUEL POOL	REF. 12	" /1978	"	"	"	"			

SUMMARY TABLE [illegible] SEISMIC ANALYSIS OF MAJOR EQUIPMENT — LACBWR

MODE/TYPE [illegible]	[illegible]	ELEM TYPE	MATERIAL PROP.	STATIC ANALYSIS: METHOD	STATIC ANALYSIS: LOAD [illegible]	STATIC ANALYSIS: [illegible]	METHOD OF COMB. RESP. DUE TO: 3 COMP. MOTION	METHOD OF COMB. RESP. DUE TO: [illegible]	LOAD COMB. METHOD	[illegible]	REMARKS
VA	VA	SHEAR BEAM	ELASTIC		UNIFORM	0.25g	1 HORIZ ONLY				
HOUSING [illegible] / [illegible]		—							SLOSHING ONLY		
									[illegible] LOAD ONLY		
[illegible] ER-OR	11.76 HZ TO [illegible] HZ	BEAM	ELASTIC		MASS × ACCEL				SEISMIC + IMPACT		
"		BEAM	ELASTIC				1 HORIZ + VERTICAL		D, L, E, To		
"		PLATE	ELASTIC				1 HORIZ + VERTICAL		"		

Attachment 2

REVIEW OF THE SEISMIC RE-EVALUATION PROGRAM OF THE LACROSSE BOILING WATER REACTOR FACILITIES

September, 1982

Lawrence Livermore National Laboratory

TABLE OF CONTENTS

1 INTRODUCTION

The U.S. Nuclear Regulatory Commission (NRC) is conducting the Systematic Evaluation Program (SEP) which consists of a plant-by-plant safety reassessment of a few older operating plants. Lawrence Livermore National Laboratory (LLNL) has been providing technical assistance to the NRC staff in performing SEP seismic reviews.

As part of the SEP, the Dairyland Power Cooperative (DPC) was requested to perform a seismic re-evaluation of the LaCrosse Boiling Water Reactor (LACBWR) facilities. LLNL and its consultant, EG&G/San Ramon Operations, reviewed the licensee's seismic re-evaluation program plan and submitted a summary letter report to NRC on December 7, 1981 (Ref. 1). The program plan review concentrated primarily on the methodology and criteria the licensee is committed to follow in their seismic re-evaluation. The structures portion of the review summary table submitted to the NRC has been updated and is included in this report as Appendix A.

A meeting was held to review the results of the seismic re-evaluation program at the office of DPC's consultant, Nuclear Energy Services, Inc. (NES), in Danbury, Connecticut, on August 10 & 11, 1982. During the meeting, a total of five reports in the structures area (Ref. 2 to 6) were handed over to the NRC Seismic Review Team which, in the structures area, includes personnel from LLNL, EG&G/SRO and NRC.

A list of questions or comments was given to NES at the end of the review meeting. Some additions were made to the list and attempt was made to rank the questions and list them in priority order. The new list was transmitted to the NRC on August 16, 1982 (Ref. 7). This new list was again updated and included in Section 2.2 of this report.

The following documents formed the basis of our review: NUREG/CR-0098 (Ref. 8), the SSRT guideline for SEP soil-structure interaction review (Ref. 9), the Standard Review Plan (SRP), and the pertinent NRC regulatory guides. The first two documents prevail wherever they contradict the SRP and the NRC regulatory guides. This is in recognition of the fact that the LACBWR facilities were designed and built prior to the publication of the current design methodology and criteria. In addition, the seismic re-evaluation is deemed adequate when it reasonably meets the intent of the above documents.

The intent of this report is to document the results of this review. The review covers the following structures:

1) Reactor containment building
2) Turbine building
3) 1-B Diesel generator building
4) LACBWR stack and Genoa 3 stack

A general layout of the structures included in this review is shown in Fig. 1. The stacks are not safety related. However, they were reviewed since the collapse of these stacks might endanger the nearby safety related structures.

Structural Mechanics Associates (SMA), under contract with LLNL, performed an independent seismic analysis of the reactor containment building (Ref. 10). The results of this independent analysis provide a bench mark for the evaluation of the licensee's re-evaluation results and were used in this evaluation effort.

Chapter 3 of this report describes the seismic inputs for structures and subsystems. Chapters 4 through 7 present the review of structures described above. Chapter 8 includes miscellaneous items such as the concrete block walls, roof panels, field erected tanks, and buried piping or tunnels. The summary and conclusions of this review are included in Chapter 2.

2 SUMMARY AND CONCLUSIONS

2.1 Summary and Conclusion

A total of five reports (Ref. 2 through 6) were reviewed. These reports documented the results of a seismic re-evaluation program covering five st.uctures. They are the reactor containment building, the turbine building, the 1-B diesel generator building, the LACBWR Stack and the Genoa 3 stack. The stacks are not safety related. However, the failure of these stacks might endanger the nearby safety related structures. A summary of review evaluation of these structures is provided as follows:

A) Reactor containment building - This building appears to have sufficient capacity to resist seismic excitation based on NES's stress evaluation of the steel containment and the outer shield wall and based on SMA's confirmative type of capacity evaluation of the building without detailed stress calculation. However, two major concerns regarding the stability of the building and the stress and capacity calculation for the lower columns of the inner shield structure need to be addressed as soon as possible.

B) Turbine building - The concrete portion of the building and the turbine pedestal seem to have sufficient seismic capacity in view of the fact that the overall building center of mass is quite low compared to its horizontal dimensions. The steel bracings were found by NES to be overstressed and to need modification.

C) 1-B diesel generator building - the steel framings of this building are believed to be able to withstand the postulated seismic event. However, this building has quite extensive amount of hollow un-reinforced concrete block walls which are believed to have only a limited seismic capacity. The failure of these walls could endanger the equipment housed inside this building.

D) LACBWR stack and Genoa 3 stack - These stacks were found to be overstressed at the top by using ultimate strength design method in accordance with the acceptable stress limits given in ACI-318-77 code. However, the ACI-307-79 design code, which governs chimney design, were not used. ACI-318-77 design code applies to solid member cross-sections. However, it might not be appropriate for hollow circular chimney section. It is necessary to calculate the seismic capacity of these stacks using ACI-307-79 design code with possible higher allowable stress limits permitted by NRC (0.8 fc' for concrete and 0.9 fy for steel) (Ref. 11), and to compare with the existing results.

The calculated shear or moment of the stacks due to seismic could be overconservative by 40% if the conservative SRSS combination method stated by NES personnel in the review meeting was actually applied to these axisymmetric structures.

The above conclusions are, of course, subjected to the satisfactory resolution of the open items identified in Chapters 3 through 8. These open items are summarized in the next section (Section 2.2) and include the open items given in Ref. 7.

The summary table, which was attached to Ref. 1 and was the result of a program plan review of the seismic re-evaluation, was updated and included in this report as Appendix A. The program plan review mainly concentrated on the methodology and criteria that the licensee had committed to follow in executing the seismic re-evaluation program. Most of the comments of the updated program plan review are related to one or more open items identified in the main body of this report. Therefore, they were also merged into Section 2.2.

2.2 Summary of open items.

1) Clarify the stress and capacity calculations for the lower columns of the inner shield structure. Specifically resolve the discrepency between calculations on pages B-208 and B-222 of Ref. 2.

2) Evaluate the stability of the containment against overturning along the interfaces between the internal concrete structure and the steel containment, between the steel containment and the basemat, and between the basemat and the pile group.

3) Justify the use of in-structure response spectra developed from the old 2-D dynamic model for the assessment of the systems and components in the reactor containment building. Submit the in-structure response spectra of the turbine building and the 1-B diesel generator building for review as soon as they become abailable. Evaluate also the effects of floor flexibility on the vertical in-structure response spectra.

4) Document the finite element analysis of the Genoa 3 stack basemat and the assessment of the soil bearing capacity.

5) Provide the stability analyses of the LACBWR stack and document the detailed evaluation of basemat and piles including the basemat-pile connection.

6) Calculate the seismic capacity of the stacks using ACI-307-79 design code with possible higher allowable stress limits permitted by the NRC (0.8 f'c for concrete and 0.9 f_y for steel) (Ref. 11), and to compare with the existing results obtained from the ultimate strength design method.

7) How the G and ν for the soil were calculated? What elevation and shear strain are they corresponding to? References cited in the reports (Refs. 2 and 3) are not the correct references. Dames & Moore report on liquifaction potential (Ref. 12) does not have the cited tables either.

8) Justify ignoring piles and embedment in the soil spring calculation and assess the effect on in-structure response spectra.

9) The ultimate pile capacity of 400 kips is four times the rated capacity. Clarify the methodology that was used to derive this number.

10) Justify using a constant 5% eccentricity in the turbine building, 86 inches eccentricity for the inner shield structure and neglecting torsional soil spring. Address both the center of mass and the center of rigidity. Justify the calculation of torsional rigidity and shear factors. (See for example, page A-64 of Ref. 3). Evaluate the effect on in-structure response spectra.

11) Evaluate the sloshing effects in the overhead water storage tank on the steel containment.

12) Document that the gap between the 1-B diesel generator building and the turbine building is sufficient to preclude interaction between them.

13) Evaluate the connection between the piles in tension and the basemat. What is the maximum number of piles under tension at any one time?

14) Is there any safety related equipment located near concrete block walls. If so, evaluate the walls or consequence of their failure, including the effects of alteration of gross or local structural responses.

15) Evaluate connections of precast roof panels in the turbine building and the 1-B diesel generator building or consequences of the panels falling. Evaluate also the adequacy of the roof panels in the turbine building.

16) Evaluate tunnels housing safety-related piping or equipment, if any.

17) Many bracings in the turbine building are predicted to be over stresses under seismic load. The model used does n' reflect the actual behavior of this type of system. Modification need to be done, or a different model analyzed. If the bracings are not to be modified, discuss the effect of its failure. If ductile response is assumed, verify that the connections have sufficient capacity to allow the development of ultimate member strength.

18) Provide description of the turbine building basemat. Evaluate also the seismic capacity of the basemat. Consider the flexibility of basemat and justify the methods used in calculating the pile axial load.

19) Verify the connection between building basemat and the turbine pedestal basemat. Compare the result of the combined seismic model with those of the separate models and justify which model is correct. Verify also that the connectivity assumed in all LACBWR structural models reflect the actual field conditions, and is consistent with the analytical results.

20) Clarify if the basemat is considered as a lumped mass in the seismic analysis models. Justify if it is not.

21) It is not known if all computer codes have been officially verified. From the review of the reports submitted during the meeting held in NES office on August 10 & 11, 1982, it seems that STARDYNE is the only computer code used. STARDYNE is a public domain program. Licensee is not required to verify and to document this program. Clarification is needed if there are other in-house computer programs used.

22) Verify the adequacy of all structural connections. Wherever the ductile behavior is relied upon for structural integrity, the connections should have the capacity to allow the development of ultimate member strength.

23) Justify that the damping values used are adequate considering both the structural responses and the in-structure response spectra.

3 SEISMIC INPUTS FOR STRUCTURES AND SUBSYSTEMS

3.1 Seismic Input at Free Field

Two seismic spectra at free field (Fig. 2) were used in the seismic re-evaluation of LACBWR facilities. One is the NRC R.G.1.60 design spectrum scaled to 0.12g peak ground acceleration. This design spectrum, together with a 2-D lumped mass model, was used in an original seismic assessment of the reactor containment building by Gulf United Services in 1974. Floor response spectra for the reactor containment building were also developed. The other seismic input is the NRC site specific spectrum (Ref. 13). This site specific spectrum was used later by NES to assess all LACBWR structures including the reactor containment building. The building response results (excluding the in-structure response spectra) obtained previously using the 2-D model were abandoned.

3.2 In-Structure Response Spectra

As stated in the previous section, in-structure response spectra for the Reactor Containment Building were developed from an old 2-D model using 0.12g R.G. 1.60 spectrum. These in-structure response spectra were continued to be used to assess the equipment and piping systems inside the reactor containment building. No effort was made to update these floor response spectra using the new model. NES personnel stated in the review meeting that the input spectrum used in the original analysis is much more conservative than the NRC site specific spectrum that these floor spectra might envelop the floor spectra if the new model and the NRC spectrum were used. While this could very well be the case. No study has been performed to demonstrate that the abandoned 2-D model yields floor spectra that are conservative compared to those using the new model and the NRC site specific spectrum. The floor spectra could be unconservative for certain frequency ranges even though the input spectrum is more conservative. This, of course, mainly depends upon the appropriateness of the abandoned 2-D model for seismic analysis.

There are no in-structure response spectra submitted for the turbine building and the 1-B diesel generator building. This information should be submitted for review as soon as it becomes available.

Based on the above discussion, the following open item has been identified:

1) Justify the use of in-structure response spectra developed from the old 2-D dynamic model for the assessment of the systems and components in the reactor containment building. Submit the in-structure response spectra of the turbine building and the 1-B diesel generator building for review as soon as they become available. Evaluate also the effects of floor flexibility on the vertical in-structure response spectra.

4 REACTOR CONTAINMENT BUILDING

4.1 Description of Structure

The LaCrosse reactor containment building is a welded steel cylinder with reinforced concrete internal structures. The cylinder has a hemispherical upper dome which encloses an integral 42,000 gallon water storage tank. The lower steel head is ellipsoidal and is supported by a reinforced concrete, pile supported, foundation which is approximately three feet thick. Additional concrete is placed above the lower head to form the basement floor and support the concrete internal structures. The overall height of the containment shell is approximately 144 feet and the inside diameter is 60 feet. The cylinder is embedded 26'-6" below grade to the extremity of the lower ellipsoidal head. Major penetrations include the air lock with the fuel transfer equipment, the freight door, and the emergency air lock. A nine inch thick outer shield wall is located inside the steel containment shell and extends up to and supports the main crane girder. The outer shield wall is integral with the concrete internal structures, but is separated from the steel shell by one-half inch of premolded joint filler. The general arrangement of the LACBWR reactor building is shown in Fig. 3.

The steel containment vessel for LACBWR was fabricated and erected by the Chicago Bridge & Iron Company. The vessel was designed to Sections II, VIII, and IX of the ASME Boiler and Pressure Vessel Code with Nuclear Code Cases 1270N, 1271N, and 1272N. All plate parts subject to internal pressure were fabricated from A201B to A300 steel.

The cylinder and lower heads are fabricated from 1.16 inch thick plate. The top head is fabricated from 0.60 inch thick plate for the lower 45° segment and 0.705 inch thick plate for the remainder of the head. The bottom head is supported by the reinforced concrete foundation slab; additional concrete, up to approximately six foot thick, is placed above the bottom head to form the basement and sump floors. No shear ties or other means of positive anchorage exist between the steel shell and concrete.

The LaCrosse Boiling Water Reactor building is supported on a total of 230 steel encased concrete piles. The minimum specified bearing capacity of the cast-in-place piles is 50 tons per pile. Union Metal Company mono-tube, cold rolled, seven gage piles were used. The approximate elevation of bedrock is 507' or an additional 73 feet below the average bottom of the piles.

4.2 Seismic Re-evaluation Performed by NES

The seismic analysis of the reactor containment building was performed using a three-dimensional lumped mass stick model. The model is shown in Fig. 4. At each node, only the translational masses were considered. The masses associated with the rotational degrees of freedom were neglected. The steel containment, the outer concrete shield wall and the inner shield structure were represented by sticks, with nodes on each stick lined up in a vertical axis. The true locations of the mass center and the center of rigidity for the inner shield building were not considered in constructing the model. However, a constant eccentricity of 86 inches was given for all lumped masses of the inner-shield structure.

The soil-structure interaction effects were represented by a set of frequency independent lumped soil springs calculated based on the linear elastic half space theory. The effects of piles and embedment were not considered. The best estimate soil shear modulus of 2400 ksf was multiplied or divided by a factor of 1.5 to yield the upper or lower bound estimates respectively. This calculation was done to account for the possible uncertainties of the soil properties and the soil-structure interaction methodology. It was found that the upper bound estimate of soil shear modulus yields the highest shears and bending moments for steel containment and inner shield structure. The effect of variation of soil properties on the moment responses of the outer shield wall and the vertical or axial responses of all structures were not presented.

Damping values of 4% and 7% were used for steel and reinforced concrete, respectively. These values are consistent with NRC R.G.1.61 values for stresses just below yield. The damping values for soil were not reported. However, judging from the composite modal damping values presented, which fall between 4% to 7%, it appears that additional damping due to soil-structure interaction was not included.

The modal response spectrum method using the STARDYNE computer code was used. U.S. NRC R.G. 1.92 was cited as the guideline for the combination of modes and the three spatial components of a seismic motion. For the steel containment and the outer shield wall, which are symmetric about the vertical axis, the same stress components due to the gross bending effect of the seismic load at two locations 90° apart were further conservatively combined by SRSS. The axial stress due to vertical seismic response was inadvertently omitted in combining the stresses for the steel containment. In general, the stresses of the steel containment and outer shield wall due to seismic load are low compared to the allowable values. The buckling potential of the steel containment was not checked. The load combination involving design basis accidents and seismic load was considered to be beyond the scope of this review.

Excluding the consideration of buckling, the stresses in the steel containment and the outer shield wall were found to be within the limits allowed by the ASME B & PV code (1977) and the ACI-318-77 code. The area of reinforcement in the support columns of the inner shield structure is below the minimum specified by the ACI code (318-77). However, the stress calculation for these columns are not clear.

The piles were evaluated for combined dead load and seismic load. The axial loads on a pile due to bending about two horizontal axes induced by seismic load were combined by the SRSS method. The maximum compressive force on a pile is 192 kips which is substantially higher than the rated capacity of 100 kips. A maximum tensile force of 42 kips was also found to occur under seismic conditions.

4.3 Independent Seismic Analysis Performed By SMA

An analytical, multi-stick model representing the reactor containment building of the LaCrosse Boiling Water Reactor was developed by SMA. The model included the steel containment vessel, the outer shield wall, the inner shield structure, and the reactor vessel. Masses were lumped at nodes, and their geometric eccentricities were taken into account.

Soil flexibility in the present model was accounted for by adding frequency-independent foundation springs at the base of the model. Stiffness and damping of each pile was calculated, and a group effect factor was then applied to the total stiffness to account for the multi-pile group interaction. The effects of containment vessel and foundation slab embedment were also added to the corresponding stiffness and damping impedance terms.

Stiffness proportional composite modal damping in each mode was obtained by assuming 3 percent of critical damping for steel and 4 percent of critical damping for concrete. For the modal analysis, modal damping was restricted to a maximum of 20 percent of critical.

Loads in this analysis were developed using the same NRC site specific horizontal ground response spectra as used in the seismic re-evaluation performed by NES. Vertical spectra were assumed to be two-thirds of the horizontal spectra. A response spectrum analysis was performed and the modal responses were combined using a modified SRSS method. Since no steel reinforcement details were available, concrete wall capacities were assumed to be those reported in the original LaCrosse FSAR seismic analysis. The results show that all peak moments in the outer shield wall and the inner shield wall lie below yield values reported in the LaCrosse FSAR. Also, all computer shear loads were below ultimate shears. Peak axial loads and peak moments were also computed for the piles. Since reinforcement details for the piles were not available, their capacities could not be evaluated. Longitudinal and shear stresses were computed for the steel containment vessel. It was shown that those stresses are relatively low, and the combination of seismic and dead weight compressive membrane stresses is much lower than the code allowable buckling stress.

4.4 Review and Discussion of the Seismic Re-evaluation Results

The methods of calculating the nodal mass and member stiffness for the stick modal appear reasonable. The number of lumped mass points seems to be sufficient to neglect the rotational degrees-of-freedom for each node. However, neglecting the true locations of the mass center and the center of rigidity for the inner shield structure needs further justification.

Soil damping values were conservatively neglected. However, damping values of 4% and 7% appear high for steel and reinforced concrete at stresses substantially lower than yield. They are acceptable for the safety assessment of structures only if brittle type failures are excluded and if the overall stress level consistent with these damping values can be reached before the system dynamic behavior were significantly altered due to yielding of certain major structural elements. Use of these values would be unconservative for development of in-structure response spectra.

Using linear elastic half space theory to calculate the lumped soil spring values seems to be a little oversimplified. The effect of piles and embedment needs to be considered not only for the calculation of structural responses but also for the development of in-structural response spectra.

To assess the results of the seismic re-evaluation, a comparison of the soil-structuare interaction and the natural frequency information between NES and SMA analyses is shown in Table 1. There are signified differences in both the soil spring values and the lowest structural frequencies in three orthogonal directions for each estimated level of soil shear modulus. It is believed that the differences are mainly due to the SSI methodologies used in calculating the soil spring constants, even though some difference in shear moduli exist. Table 2 shows a comparison of the total loads on the pile group and the maximum axial tensile and compressive forces on a single pile. In this table, it is clear that the upper bound estimate case dominates in both NES and SMA results.

Note that while the soil spring values and the natural frequencies analyzed by NES and by SMA differ, the total shears and the total moment at the base of the bulding about N-S axis are very close to each other. It is not clear why there is a significant difference between the moments about the two horizontal axes in the NES results (1.73×10^6 kip-in/rad vs. 2.6×10^6 kip-in/rad for example) while the dominating frequencies for these two directions are very close. Due to the higher total moment about the E-W axis, the maximum compressive and tensile forces on the pile in NES results are significantly higher than those from SMA's analysis. The NES reported maximum compressive force of 191.5 kips in the pile is significantly higher than the rated capacity of 100 kips. NES considered the ultimate capacity of the pile to be 400 kips. However, justification is needed. Since tension in some piles was found in both the NES and SMA studies, it is necessary to study the pile cap connection to determine if it can withstand the maximum tensile force due to seismic load.

There are no studs or shear ties to transfer tangential force between the internal concrete structure and the steel containment, and between steel containment and foundation. It appears that the friction rather than the bond along the surface of the steel containment is the only tangential force transfer mechanism. A stability study is needed to determine if the steel containment will overturn during a seismic event.

The water tank inside the steel containment holds 42,000 gallons of water. It is at the top of the steel containment shell. No study was made to determine the sloshing effect of the water on the shell. Since the tank is at such a high elevation, the seismic effect could be significant.

Based on the confirmative type of capacity evaluation performed by SMA without detailed stress calculation, it appears that the reactor containment building might be able to withstand the SSE described by th NRC site specific spectrum. However, concerns described above exist in NES detailed analysis. A list of open items which need further clarification or justification is provided in the following to summarize the review.

1) Clarify the stress and capacity calculation for the lower columns of the inner shield structure. Specifically resolve the discrepency between calculations on pages B-208 and B-222 of Ref. 2.

2) Evaluate the stability of the containment against overturning along the interfaces between the internal concrete structure and the steel containment, between the steel containment and the basemat, and between the basemat and the pile group.

3) Justify ignoring piles and embedment in the soil spring calculation and assess the effect on in-structure response spectra.

4) The ultimate pile capacity of 400 kips is four times the rated capacity. Clarify the methodology that was used to derive this number.

5) Justify the use of a constant eccentricity of 86 inches for the inner-shield structure. Address both the center of mass and the center of rigidity. Evaluate the effect on in-structure response spectra.

6) Evaluate the sloshing effects in the overhead water storage tank on the steel containment.

7) Evaluate the connection between the piles in tension and the basemat. What is the maximum number of piles under tension at any one time?

8) Justify that the assumed structural damping values are adequate considering both the structural responses and the in-structure response spectra. Please note that some modes of vibration might have little soil-structure interaction phenomena. The neglect of soil damping would not reduce the modal damping for these modes to yield conservative results.

5. TURBINE BUILDING

5.1 Description of Structure

The turbine building contains a major portion of the power plant equipment. The turbine generator and the associated equipment are in the south part of the building. The control room and electric equipment room are in the east and are adjacent to the reactor containment building. The north portion consists mainly of non-safety related facilities, such as the shower and locker room, water tank, conference room and etc. A layout plan of the main floor is shown in Fig. 5 just to give a general ideal about the general arrangement of turbine building.

The turbine building above the main floor outside the control room area is mainly a steel frame structure covered with insulated steel siding. The roof is a structural steel frame supporting precast concrete slabs. The building below the main floor is basically reinforced concrete. It includes the ground floor and a mazzanine floor. The building sits on a group of 311 piles. No description of the basemat was found in the report. It is not clear if the basemat of the building is monlithically connected to the basemat of the turbine pedestal to form a single piece.

5.2 Seismic Re-evaluation

5.2.1 Seismic Analysis Models

There are three lumped mass stick models (Figs. 6, 7, and 8) used in the seismic re-evaluation of the turbine building: 1) building model without the turbine pedestal, 2) turbine pedestal model without the building and 3) combined model of the building and the turbine pedestal. The results of the combined model was used to evaluate the piles only and was not compared with those of the other two models in the report.

The building dynamic analysis model includes four lumped mass nodes representing two roof elevations, main floor and mezzanine floor. Based on the reduced matrix size of 12 degrees-of-freedom in the computer output, it is believed that the basemat was not lumped as another mass point and was ignored. Like the inner shield of the reactor building, the locations of the center of mass and the center of rigidity were not actually modeled. Instead, their effect was represented by assigning an eccentricity equal to 5% of the building dimensions in the corresponding two horizontal axes for the main floor and mezzanine floor. Below the main floor, the member properties of the building model were calculated about the area centroid of the structural elements between two floors. Above the main floor, the structure is considered as symmetric about two horizontal axes and the member properties were calculated based on the stiffness of the bracings and the member properties of the columns. The soil springs were calculated based on elastic half space theory. The effects of piles were not considered. Also, the torsional soil spring was not used.

The turbine pedestal and its basemat was believed to have been modeled by a single lumped mass with three translational degrees-of-freedom only, judging from the computer output which lists only three modes. No description was given regarding the eccentricity of the lumped mass and the mass of the basemat. It is believed that they were treated in the same manner as the building model. The soil springs without the torsional mode were calculated again using the linear elastic half space theory and neglecting the effect of piles. The basement of the turbine foundation is assumed to be separated from that of the building and has a dimension of 26.5' x 71.8'.

The combined model includes the two lumped mass stick models for the building and the turbine foundation. A rigid link at the basemat elevation is provided to connect these two sticks. The relative position of these two sticks is based on the area centroids of the force resisting structural elements above ground floor of these two structures. The soil springs are the same as those of the building model.

There is no description of damping values in the report. However, it is believed that 7% was used for all modes for all three models since the ground spectrum corresponding to 7% only was used.

5.2.2 Stress Analysis

The structure above the main floor is mostly structural steel except for the control room area. It was analyzed by a finite element model. The structural steel members were modeled by beam elements while the reinforced concrete was modeled by plate elements. The envelope acceleration responses from the seismic analysis of the building model were used as the input seismic load. It was found that many steel bracings would be buckled and overstressed under the NRC spectrum.

The lower portion of the turbine building is mainly of reinforced concrete. The stress calculation was based on the results of the building seismic analysis model. The stress level is very low compared to the allowables.

A finite element model employing beam elements was also used to calculate the stresses of the turbine pedestal. The stress level was also found to be very low.

The seismic load on piles were calculated from the combined model for seismic analysis. The maximum force on the piles due to dead load and seismic load was found to be 81.8 kips which is lower than the rated allowable load of 100 kips. No tensile force was found in any of the piles. The method used in calculating the load on piles implies that the basemat of the turbine building was assumed to be completely rigid.

5.3 Review and Discussion

In constructing the models a thorough job was done in computing the mass for each lumped mass point. It is reasonable to believe that most of the structural weight above the basemat was accounted for. It seems that the basemat was not included as a lumped mass point in any of the three dynamic models. No description of the basemat was given in the report. It is also not clear if the building basemat and the basemat of the turbine pedestal are separated or form a monolithic piece. Further clarification or justification is needed.

For the building seismic analysis model, the calculation of torsional rigidity and shear factors needs justification. In all three seismic analysis models, the true locations of mass center and the center of rigidity were not accurately modeled. Their effects were arbitrarily represented by assigning eccentricities equal to 5% of the building dimensions in two horizontal directions. Justification is needed.

Since the turbine building is not massive considering the size of the basemat, the soil-structure interaction effect may not be significant. The use of half space theory neglecting the pile effect is judged to be sufficient. In this case, the 7% concrete damping, if used, could be high for developing in-structure response spectra. This is not only because of low stress level but also due to possible low soil-structure interaction effect. The neglect of soil damping might not necessarily reduce the modal damping.

The basemat is expected to be quite thin and flexible since the building is light and has a large horizontal dimension compared to the vertical. The flexibility of the basemat is further evidenced by the nonuniform arrangement of the piles. Piles are concentrated in areas where heavy dead load is expected, such as the areas under the intersections of column lines and under the columns of the turbine pedestal. The calculation of pile loads based on a rigid basemat assumption needs justification.

The use of envelope peak acceleration response in the stress computation is conservative. Some bracings in the turbine building were found to be overstressed. However, in the static model, the rather light cross bracing was considered to be capable of taking compression. In actuality, only the tension member would be taking significant load. A model considering half of the bracing acting and evaluation of tensile stresses would be more appropriate.

The turbine and turbine pedestal are usually not safety related. The turbine pedestal seismic analysis was reviewed here since there might be some safety related equipment located close by and the failure of the turbine pedestal might endanger the nearby equipment. The stresses in the turbine pedestal were found to be very low. The turbine pedestal stresses are low enough such that it would be adequate even if the soil-structure interaction, the center of mass and the center of rigidity were not accurately accounted for. The stress analysis of the water tank was not reviewed since the tank is not safety related and there is no safety related equipment close by.

To conclude the review of turbine building seismic re-evaluation, a list of open items, which need further clarification and justification, are identified below.

1) Justify using a constant 5% eccentricity and neglecting torsional soil spring. Address both the center of mass and the center of rigidity. Justify the calculation of torsional rigidity and shear factors (see for example, page A-64 of Ref. 3). Evaluate the effect on in-structure response spectra.

2) Provide description of the turbine building basemat. Evaluate also the seismic capacity of the basemat. Consider the flexibility of basemat and justify the methods used in calculating the pile axial load.

3) Verify the connection between building basemat and the turbine pedestal basemat. Compare the results of the combined seismic model with those of the separate models and justify which model is correct. Verify also that the connectivity assumed in structural models reflect the actual field conditions and is consistent with the analytical results. This applies also to other structures.

4) Clarify if the basemat is considered as a lumped mass in the seismic analysis models. Justify if it is not.

5) Many bracings are predicted to be over stressed under seismic load. The model used does not reflect the actual behavior of this type of system. Modifications need to be done, or a different model analyzed. If the bracings are not to be modified, discuss the effect of its failure. If ductile response is assumed, verify that the connections have sufficient capacity to allow the development of ultimate member strength.

6) Justify the damping values used.

6 1-B DIESEL GENERATOR BUILDING

6.1 Description of Structure

The 1-B diesel generator building is a single story structural steel braced frame structure. It is divided into diesel generator room, electrical equipment room and battery room by concrete block walls. The exterior walls are also constructed of concrete blocks. All block walls are hollow and are not reinforced. The roof is a steel frame structure with precast lightweight concrete panels except an 8'-6" x 21'-10" reinforced concrete slab. The basement is a 2-foot thick pile supported reinforced concrete slab. A 2-inch gap is provided between the basement of the 1-B diesel generator building and the turbine building to avoid interference in seismic movements. Nineteen concrete filled piles with a design load capacity of 50 tons each were used. A general layout plan is shown in Fig. 9.

6.2 Seismic Re-evaluation

A dynamic analysis of 1-B diesel generator building was not performed. An equivalent seismic load equal to 1.5 times the peak acceleration of the NRC site specific response spectrum was used. This resulted in a 0.315g uniform building acceleration in the two horizontal directions. The vertical seismic response was taken as 2/3 of the horizontal.

The concrete block walls, the precast concrete roof panels, and the reinforced concrete roof slab were not relied upon to carry the seismic load in the building integrity assessment. They are assumed to be attached to the steel frame structure. A detailed finite element static analysis model of the structural steel was constructed to perform the stress computation. The loads considered are dead load, live load and seismic load. The stresses are all within the allowable values calculated in accordance with the AISC code. The column anchorage details are capable of withstanding the seismic loads.

The precast concrete roof panels and the reinforced concrete roof slab were evaluated separately. The member stresses were found to be within the calculated allowable values based on the ultimate strength design method defined in ACI code. The concrete block walls were not evaluated and included in this analysis. The pile foundation was analyzed. The maximum compressive load of 109 kips is greater than its rated capacity of 100 kips. No tensile force was found in any of the piles. The pile-basement connections were not evaluated.

6.3 Review and Discussion

The methodology and criteria used in the seismic re-evaluation of the 1-B deisel generator building appear reasonable. The equivalent seismic load of 1.5 times the peak acceleration of the ground spectrum is conservative. All the structural elements were evaluated and the stresses were found to be within allowable values except the maximum pile compressive load of 109 kips. However, the pile overstress is not much greater than the rated capacity of 100 kips. This is acceptable considering the fact that the seismic load was generated conservatively.

In summary, the 1-B diesel generator building steel frames are believed to be able to withstand the postulated seismic event. The un-reinforced hollow concrete block walls were not evaluated. The assessment of these walls should be performed and submitted for review if there is any safety related equipment located nearby. The stresses in the concrete precast roof panels are significantly lower than the ultimate strength. However, the connections between the panels and roof steel framing were not evaluated as to the possibility that the panels might fall from the roof. The precast roof panels and the concrete block walls will be discussed further in Section 8.1. The following open item needs further evaluation:

1) Document that the gap between the 1-B diesel generator building and the turbine building is sufficient to preclude interaction between them.

7 LACBWR STACK AND GENOA 3 STACK

7.1 Description of Structures

The LACBWR stack (Fig. 10) is a 350 feet high tapered reinforced concrete chimney. It has an outside diameter of 7.19 feet and a wall thickness of 6 inches at the top. The outside diameter and wall thickness increase from top to bottom and reach 24.72 feet and 15 inches at the bottom. A cluster of 78 piles supports a 4-foot foundation mat. Each pile is 60 feet long with a minimum capacity of 50 tons.

The Genoa 3 stack (Figure 11) is a 500 foot high, tapered reinforced concrete chimney. It has an outside diameter of 17.42 feet and a wall thickness of 7 inches at the top. The outside diameter and wall thickness at the bottom are 38.20 feet and 24 inches, respectively. The stack has an independent steel liner, which is a cylinder of 15.25 feet in diameter for most of its height. The liner bells out at its base and is supported on a concrete pedestal. The basemat is a 75 foot octagon reinforced concrete slab that varies from 3'-6" to 7'-0" in thickness. The basemat is directly supported on soil. No piles were used.

7.2 Seismic Re-evaluation

Lumped mass stick models were used for the seismic analyses of the stacks. These models are shown in Figures 10 and 11. Only translational degrees of freedom are considered at each lumped mass point. It appears that a system damping of 7% was used judging from the fact that ground spectrum with 7% damping only was used in the analysis.

The soil-structure interaction effect was represented by a set of frequency independent soil springs calculated from the linear elastic half space theory. Two soil shear moduli, 1000 ksf and 3000 ksf, were used to account for the possible variation in soil properties.

For each stack, seismic analyses were performed for two shear moduli using the response spectrum method. The closeness of the natural frequencies and the moment responses throughout the height calculated from these two shear moduli for each stack indicates that the soil-structure interaction effect is very small.

The stacks were analyzed using the ultimate strength design method presented by Cannon and Boop (Ref. 14). The acceptable ultimate stress values as given in the ACI 318-77 Design Code were used to calculate the ultimate moment and shear capacities of the stack cross-sections under dead load and seismic load. The ultimate shear capacity of the stacks was found to be considerably greater than the seismic shears. The seismic moment exceeds the ultimate moment capacities of the stacks for the upper portion of the stacks. Figs. 12 and 13 show the seismic moment and ultimate moment capacity along the height of the stacks and indicate the possible failure zones.

The stability of the Genoa 3 stack was evaluated and a 5.9 factor of safety indicates that the stack will not overturn under seismic conditions. No stability analysis was performed for the LACBWR stack, however.

It was concluded that the stacks will experience failure at the top. However the failed section will not hit any safety related structures and equipment. The surviving bottom section will remain upright and attached to its basemat.

7.3 Review and Discussion

The seismic analysis methodology and models for the LACBWR stack and Genoa 3 stack appear reasonable. The conclusion from the study of soil structure interaction effect using different soil shear moduli is that the soil-structure interaction effect is negligible. The use of linear elastic half space theory neglecting the effect of piles is acceptable in this case. The use of 7% damping might be slightly unconservative for the evaluation of basemat and for the consideration of overall stability in view of the fact that while the upper portion of the stack is highly stressed, the lower portion is still far below ultimate strength.

Ultimate strength design methods using ACI-318-77 code were used instead of ACI-307-79, which governs chimney design and does not yet allow the use of the ultimate strength method. ACI-318-77 design code applies to solid member cross-sections. However, it might not be appropriate for hollow circular chimney sections. It is necessary to calculate the seismic capacity of these stacks using ACI-307-79 design code with possible higher allowable stress limits permitted by NRC (0.8 fc' for concrete and 0.9 fy for steel) (Ref. 11).

USNRC R.G. 1.92 was used in the analysis of both stacks. However, it is not clear how it was actually applied regarding the treatment of the three spatial components of an earthquake. If the maximum stresses at two points 90 degrees apart in the hoop direction were combined by SRSS as in the steel containment, the reported results could be conservative. This would provide some relief on the loads contained in the report.

The Genoa 3 stack report indicates that a preliminary analysis of the octagonal basemat of the Genoa 3 stack found it to be slightly overstressed. The report also stated that a detailed finite element model was to be developed for further evaluation. The seismic and dead weight loadings and the soil bearing pressure distributions were sent to Dames & Moore to confirm that the soil could to withstand these loads. However, the detailed information regarding the Genoa 3 basemat finite element model and the soil bearing capacity was not received for review. The detailed information should be reviewed as soon as it becomes available.

The LACBWR stack report states that the basemat was evaluated and will not be overstressed and the piles were found to meet the requirements. However, there is no detailed information given regarding these analyses.

The stability of the LACBWR stack was not analyzed. If the tensile capability of piles is relied upon for stack stability, the basemat, the pile caps and the piles should be evaluated in terms of the tension force that can be carried.

Based on the above review and discussion, the following open items are identified.

1) Calculate the seismic capacity of the stack using ACI-307-79 design code with possible higher allowable stress limits permitted by the NRC (0.8 fc' for concrete and 0.9 fy for steel), and to compare with the existing results obtained from the ultimate strength design method.

2) Provide the stability analyses of the LACBWR stack and document the detailed evaluation of basemat and piles, including the basemat-pile connection.

3) Document the finite element analysis of the Genoa 3 stack basemat and the assessment of the soil bearing capacity.

8. REVIEW OF MISCELLANEOUS ITEMS

8.1 Concrete Block Walls and Concrete Roof Panels

It is not clear if there are any concrete block walls inside the reactor containment building and the turbine building. The concrete block walls in the 1-B diesel generator building were not evaluated. These walls are hollow and are not reinforced. They are expected to have only a limited capacity to carry seismic load.

Precast roof panels were used in the turbine building and the 1-B diesel generator building. The precast roof panels in the 1-B diesel generator building were evaluated for the dead load, the live load and the seismic load. The stresses were found to be substantially lower than the ultimate load capacity. No evaluation of precast roof panels in the turbine building was found in the report. Neither report discussed whether the connections between the roof panel and the steel roof framing would prevent the roof panels from falling during an earthquake.

Two open items are identified below:

1) Is there any safety related equipment located near concrete block walls. If so, evaluate the walls or consequence of their failure, including the effects of alteration of gross or local structural response.

2) Evaluate connections of precast roof panels in the turbine building and the 1-B diesel generator building or consequences of the panels falling. Evaluate also the adequacy of the roof panels in the turbine building.

8.2 Field Erected Tanks and Buried Piping or Tunnels

There is no safety related field erected tanks and buried piping. However, it is not clear if there is a tunnel on site that houses safety related piping or equipment. A response from the licensee on the following open item is needed:

1) Evaluate tunnels housing safety-related piping or equipment, if any.

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13. NRC letter to Dairyland Power Cooperative dated August 4, 1980.
14. Cannon, R. W. and Boop, W. C., "Ultimate Strength Design Charts for Design of Reinforced Concrete Chimneys," Civil Engineering Design Research Report, p. 14, Tennessee Valley Authority, Knoxville, TN (1971).

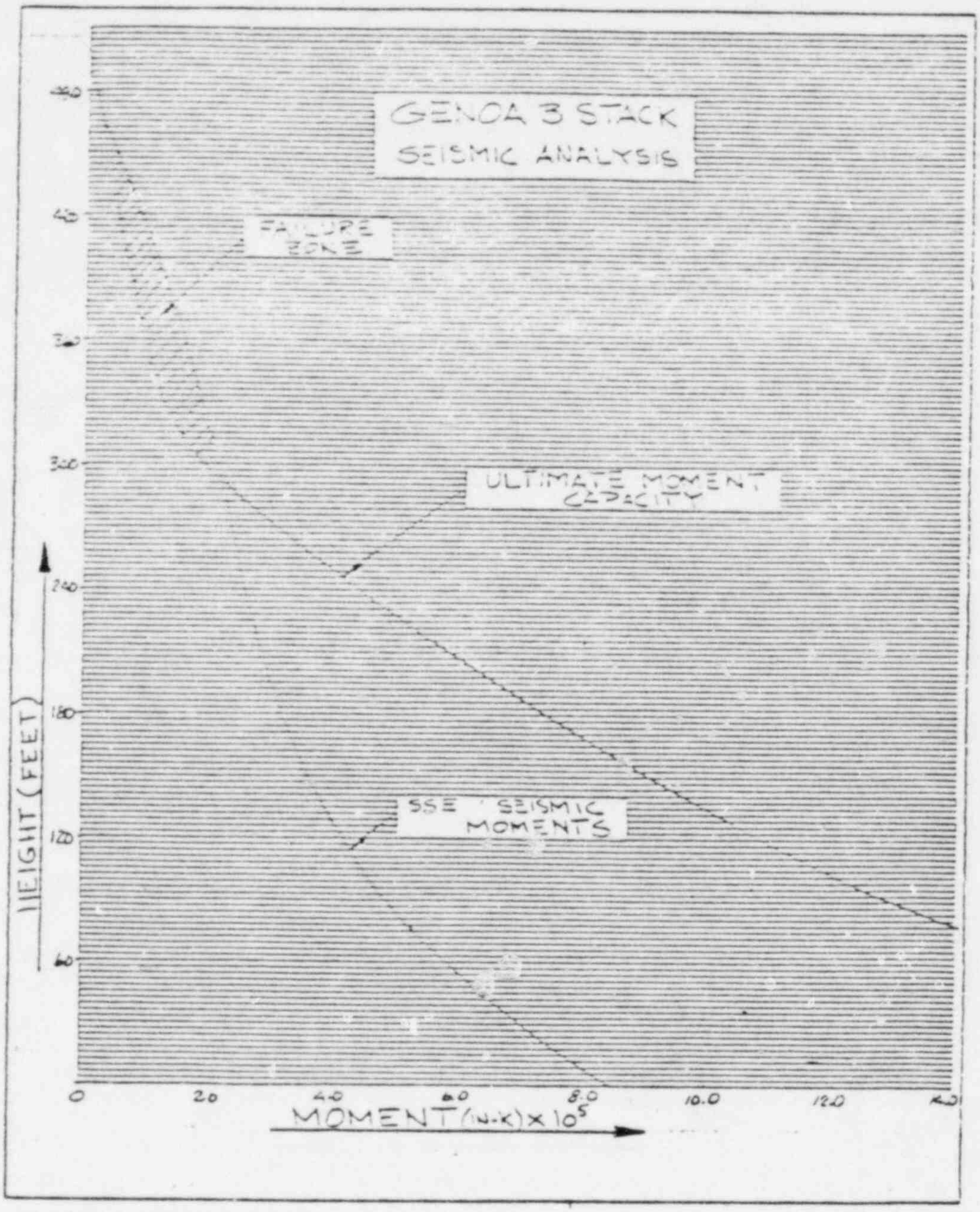

Figure 13: Moment capacity of Genoa 3 stack under SSE.

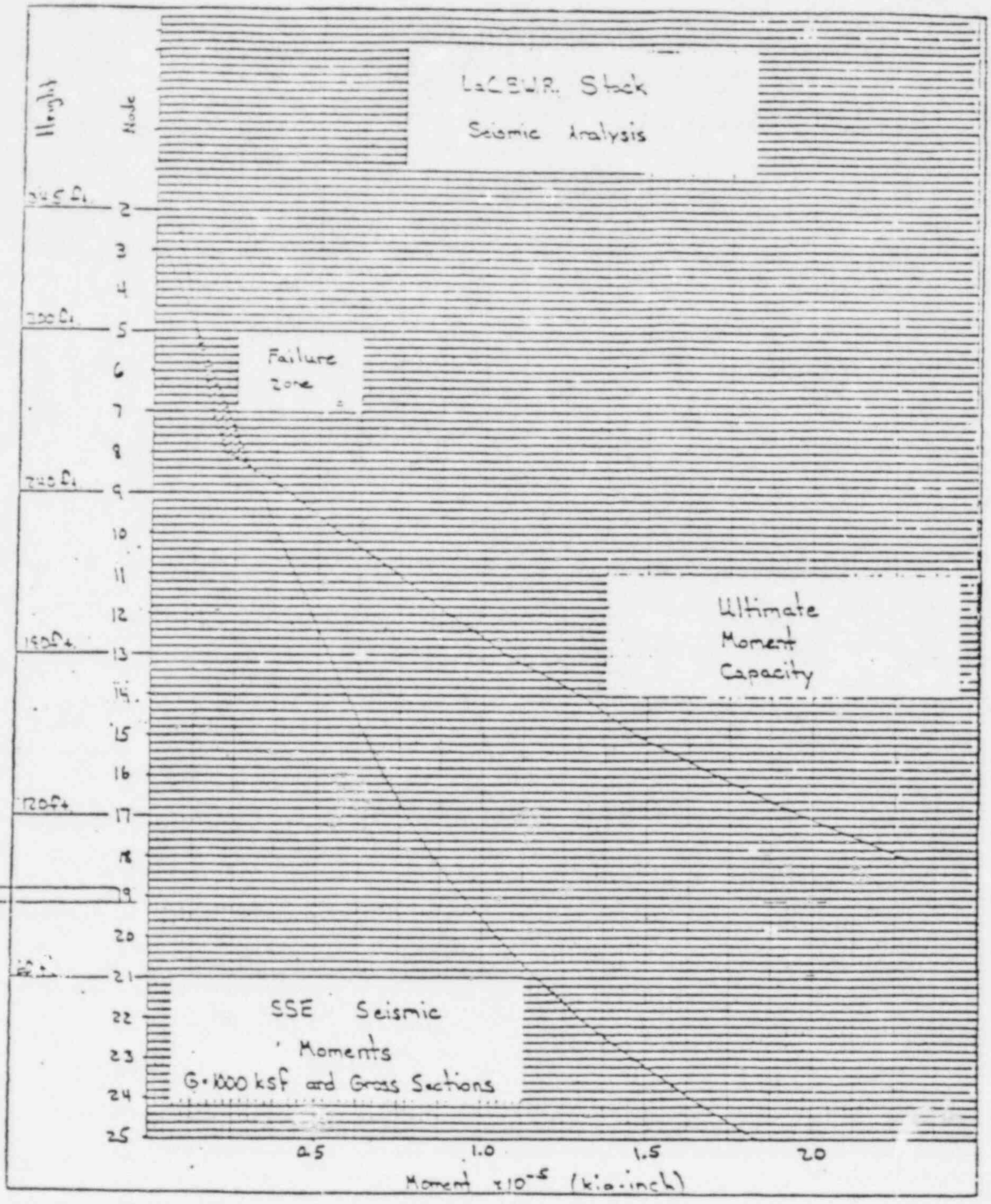

Figure 12: Moment capacity of LACBWR stack under SSE.

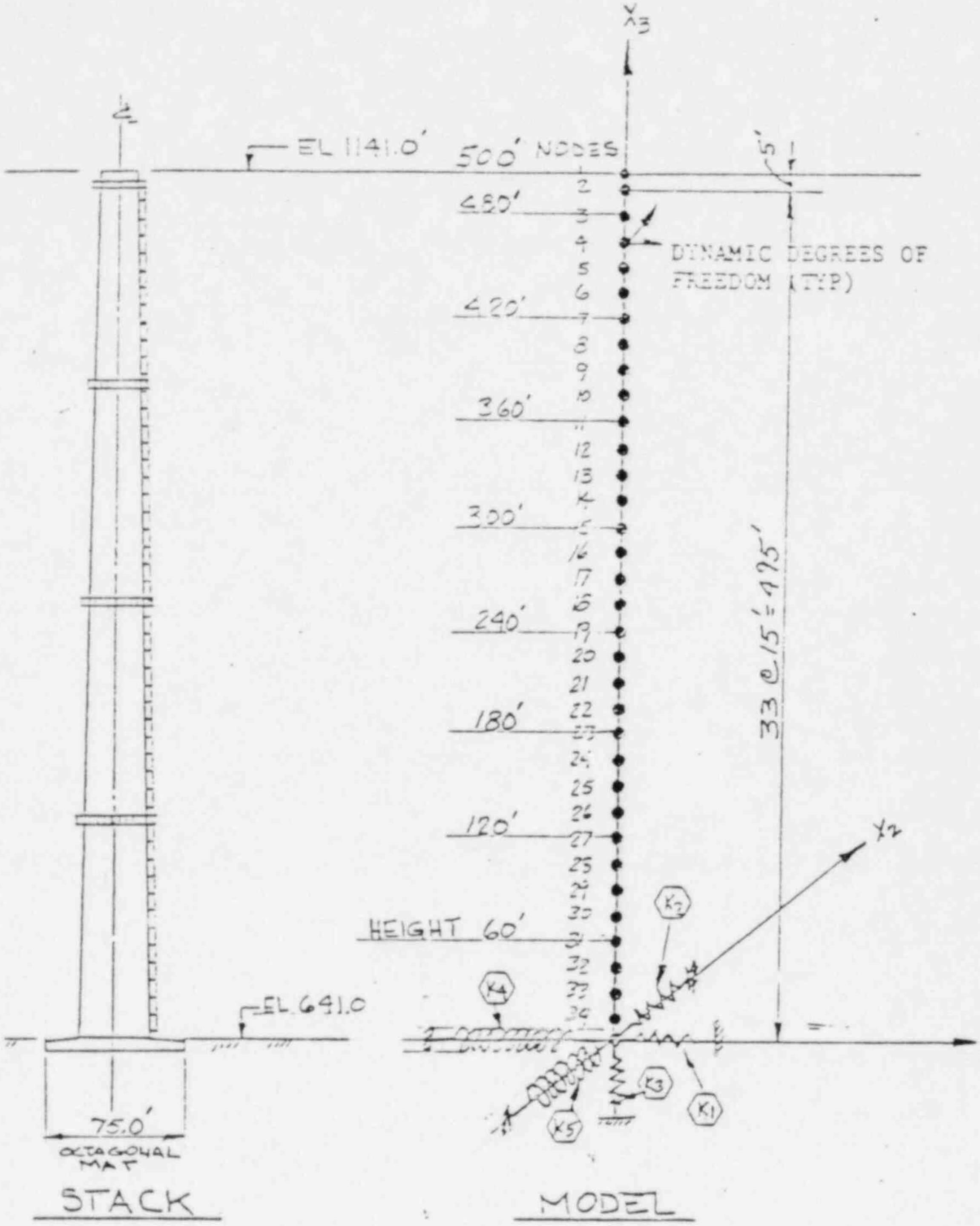

Figure 11: Schematic sketch and dynamic model of the Genoa 3 stack.

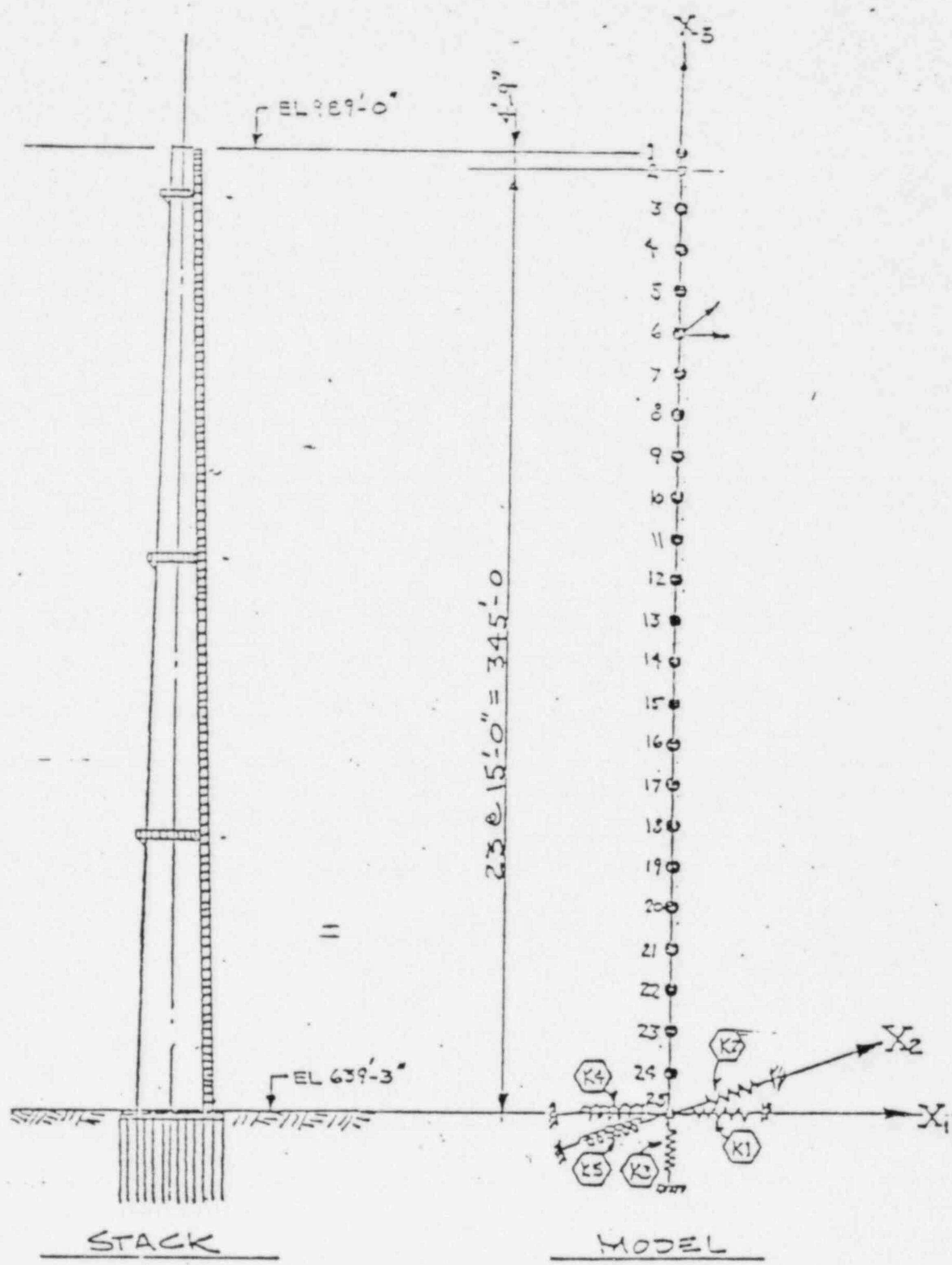

Figure 10: Schematic sketch and dynamic model of the LACBWR stack.

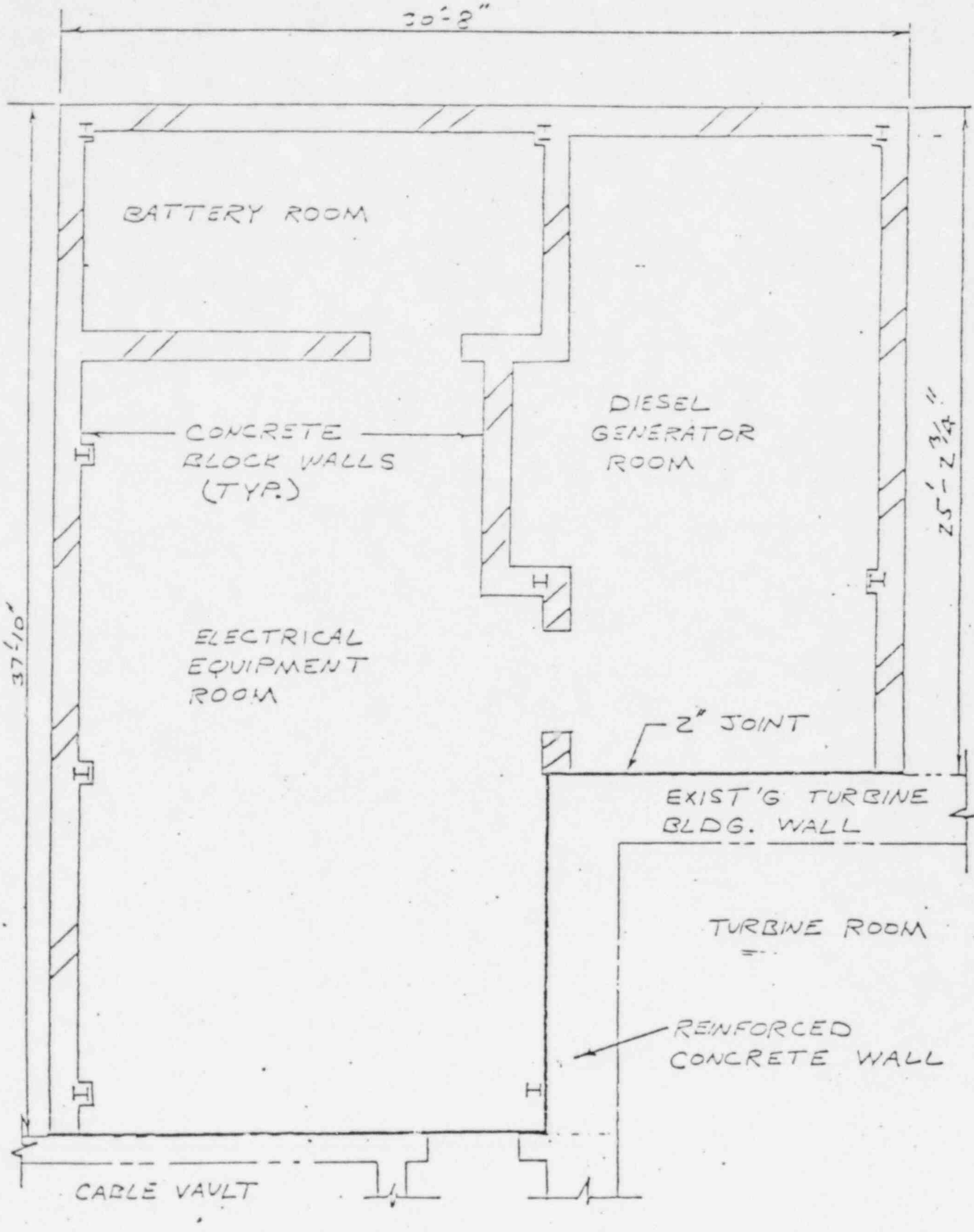

Figure 9: 1-B Diesel generator building

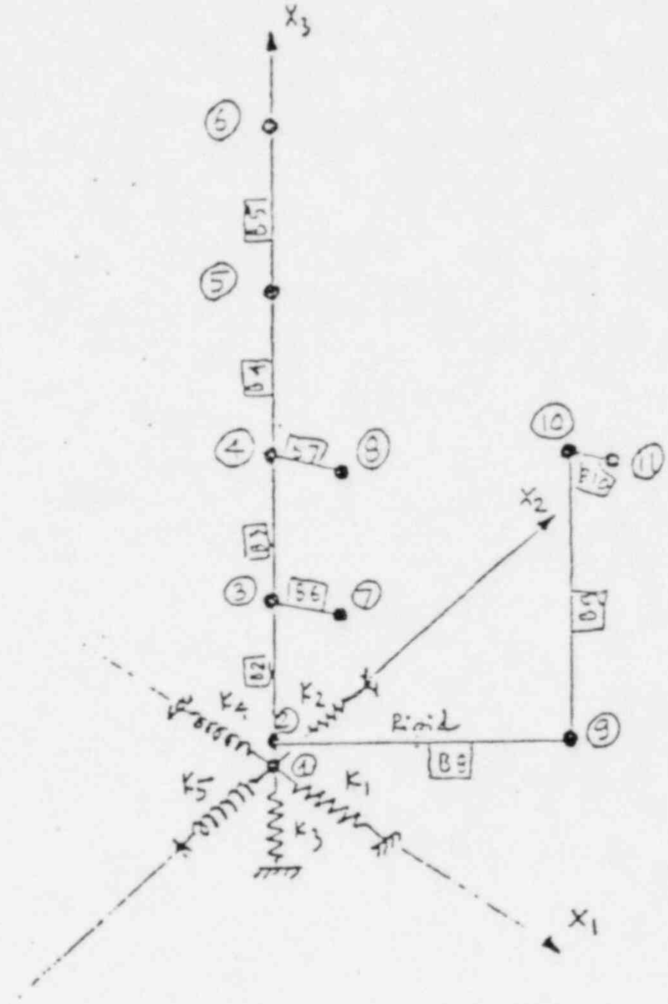

Figure 8: Combined model of Turbine Building and turbine pedestal.

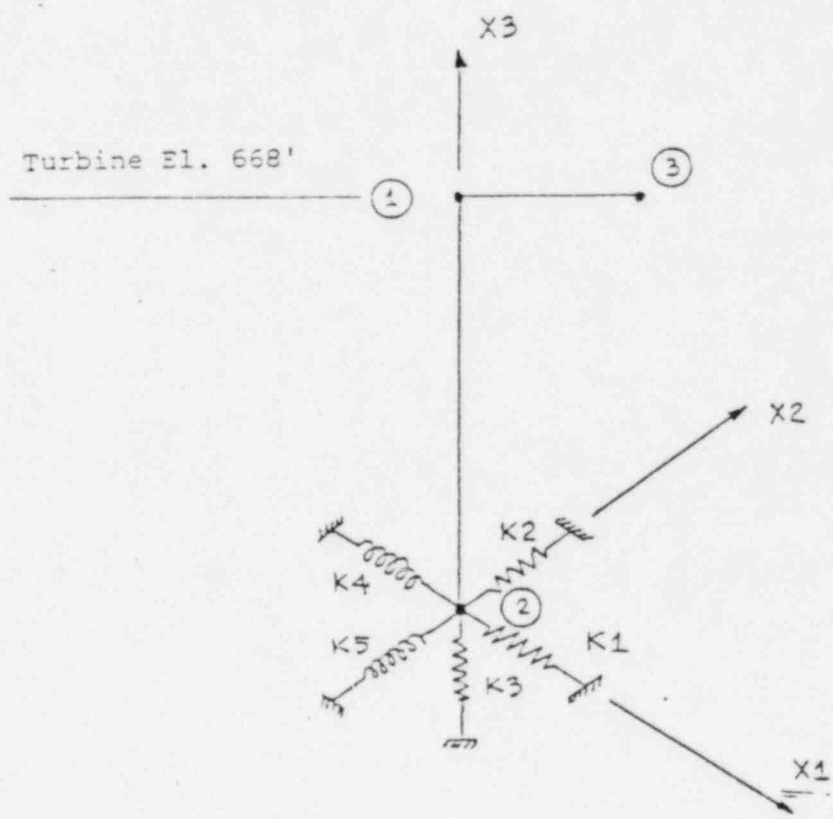

Figure 7: Dynamic model of turbine pedestal

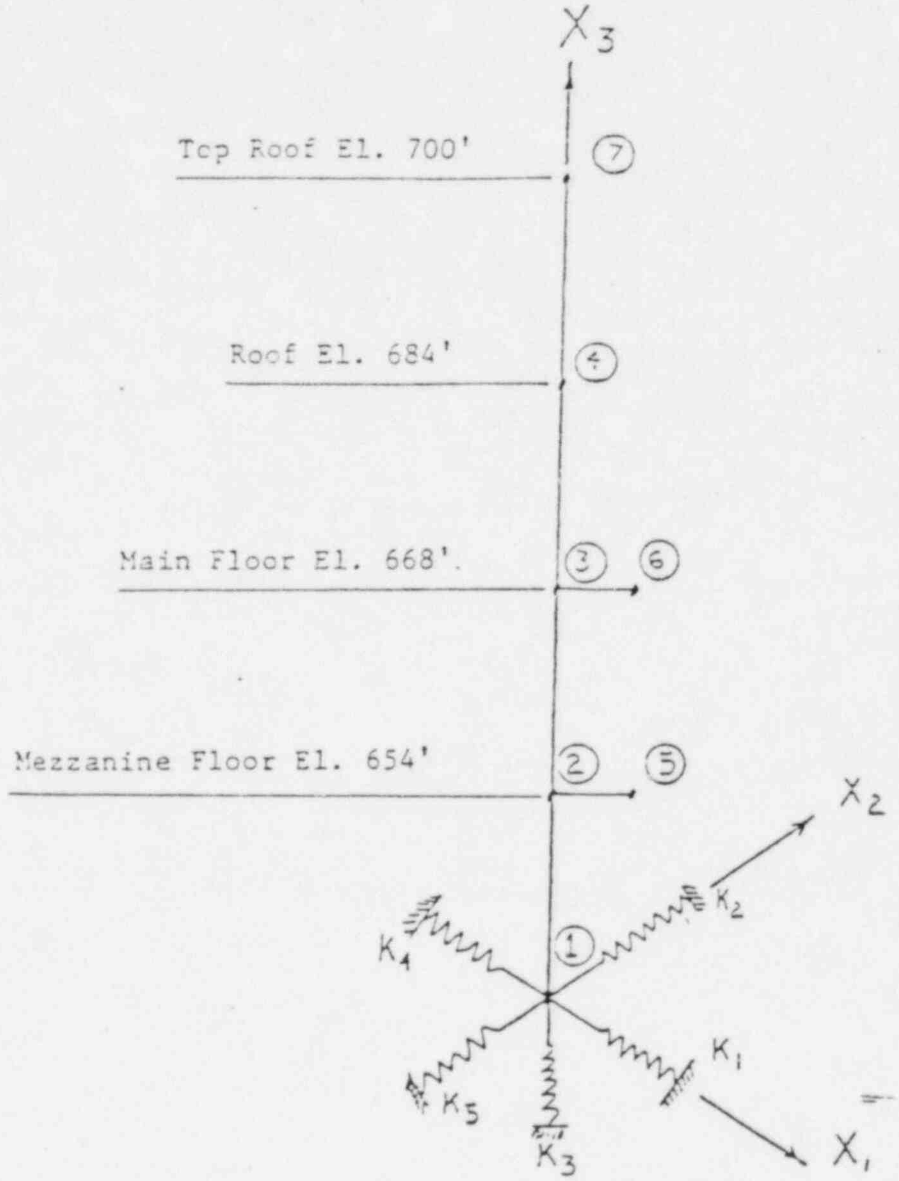

Figure 6: Turbine Building model

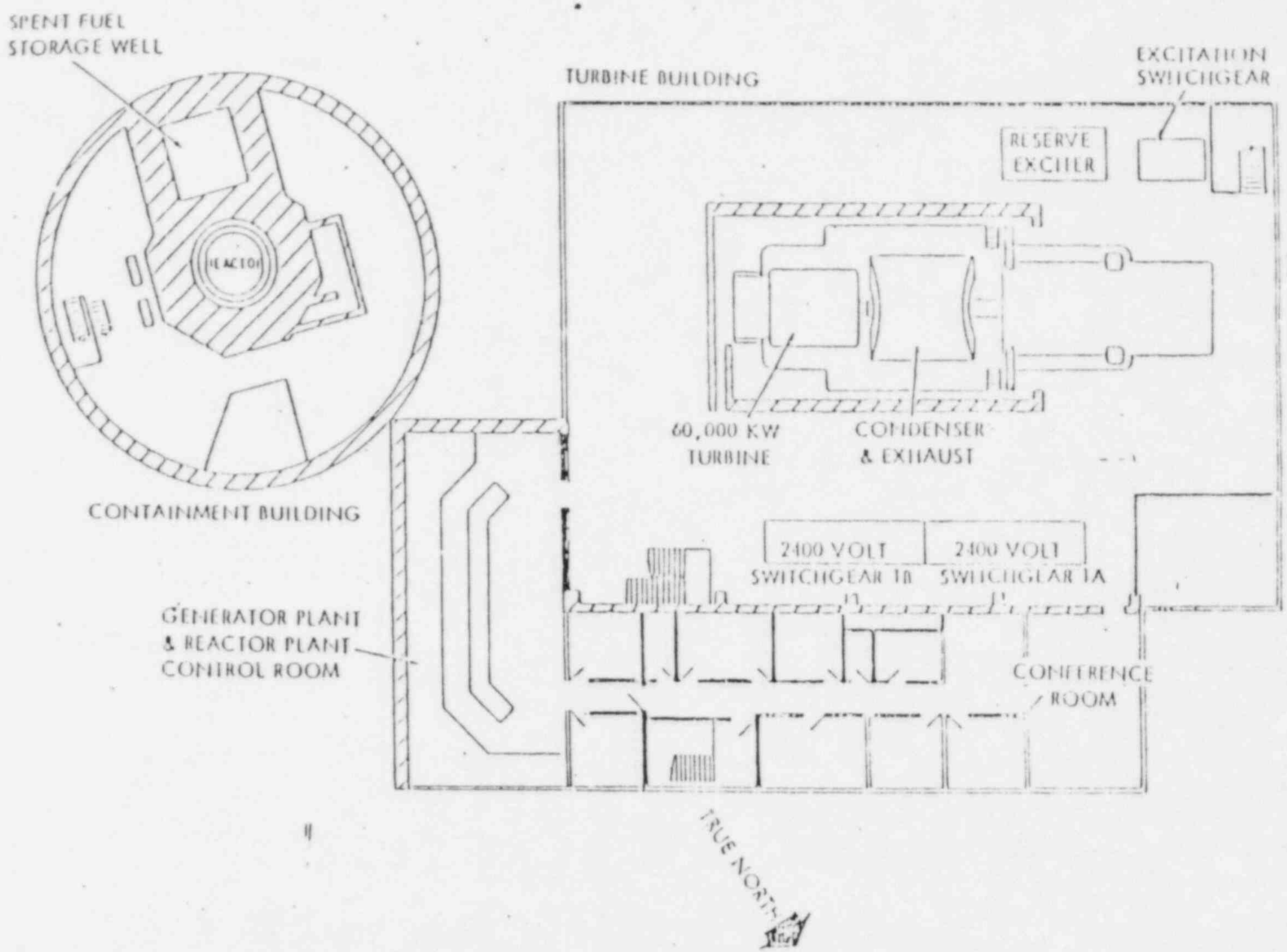

Figure 5: Turbine Building

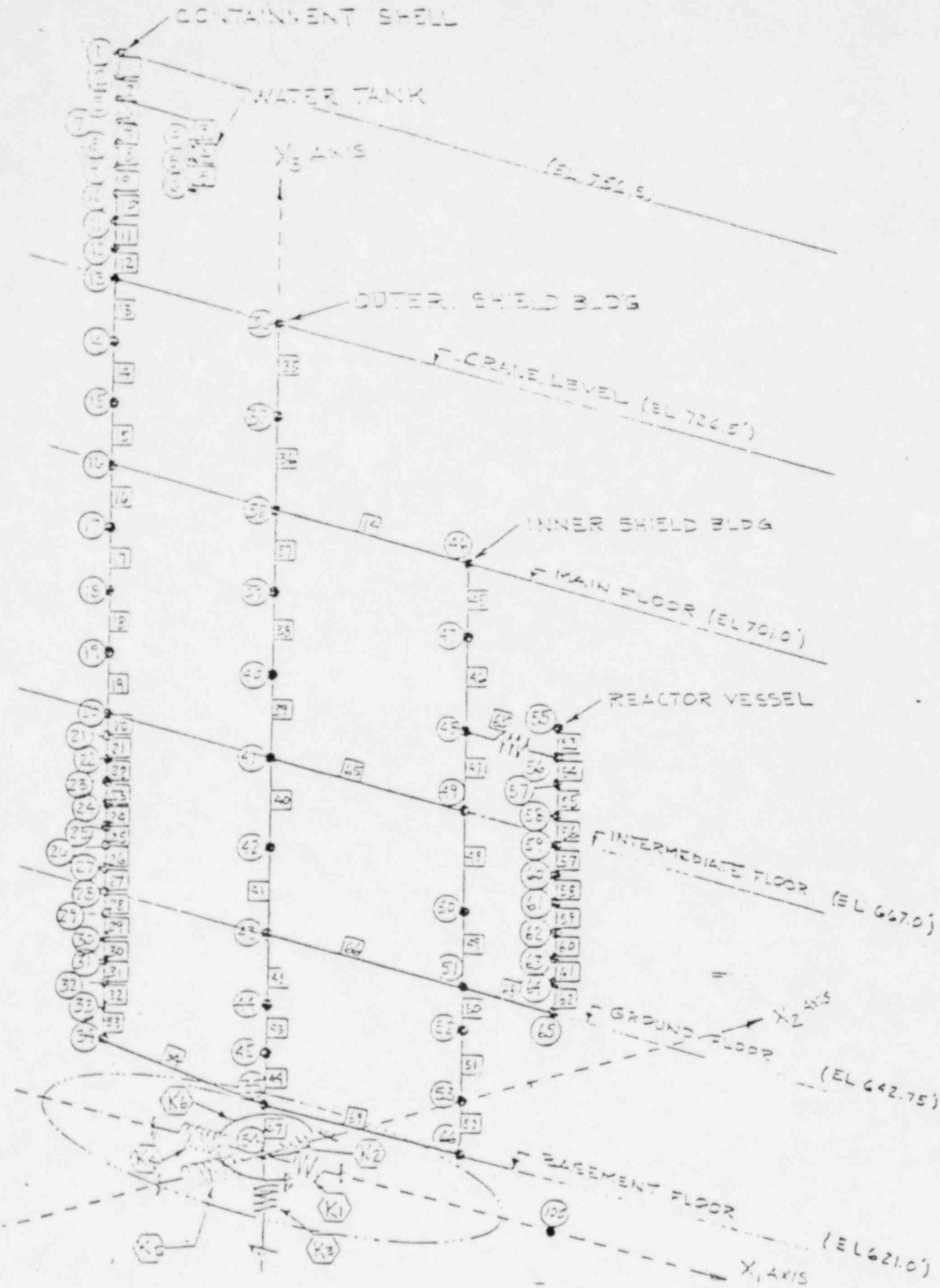

Figure 4: Dynamic model of the reactor containment building.

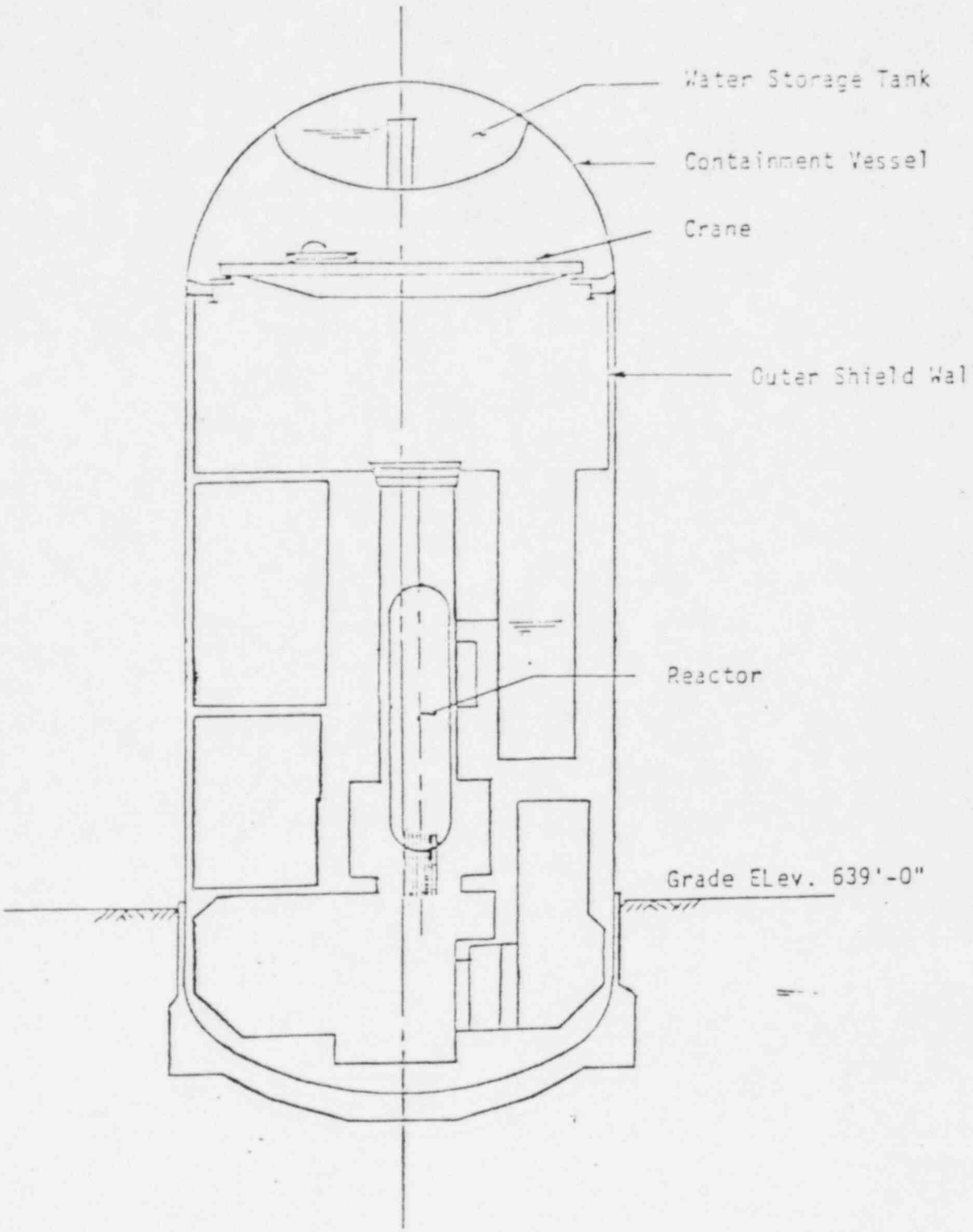

Figure 3: LaCrosse boiling water reactor containment building.

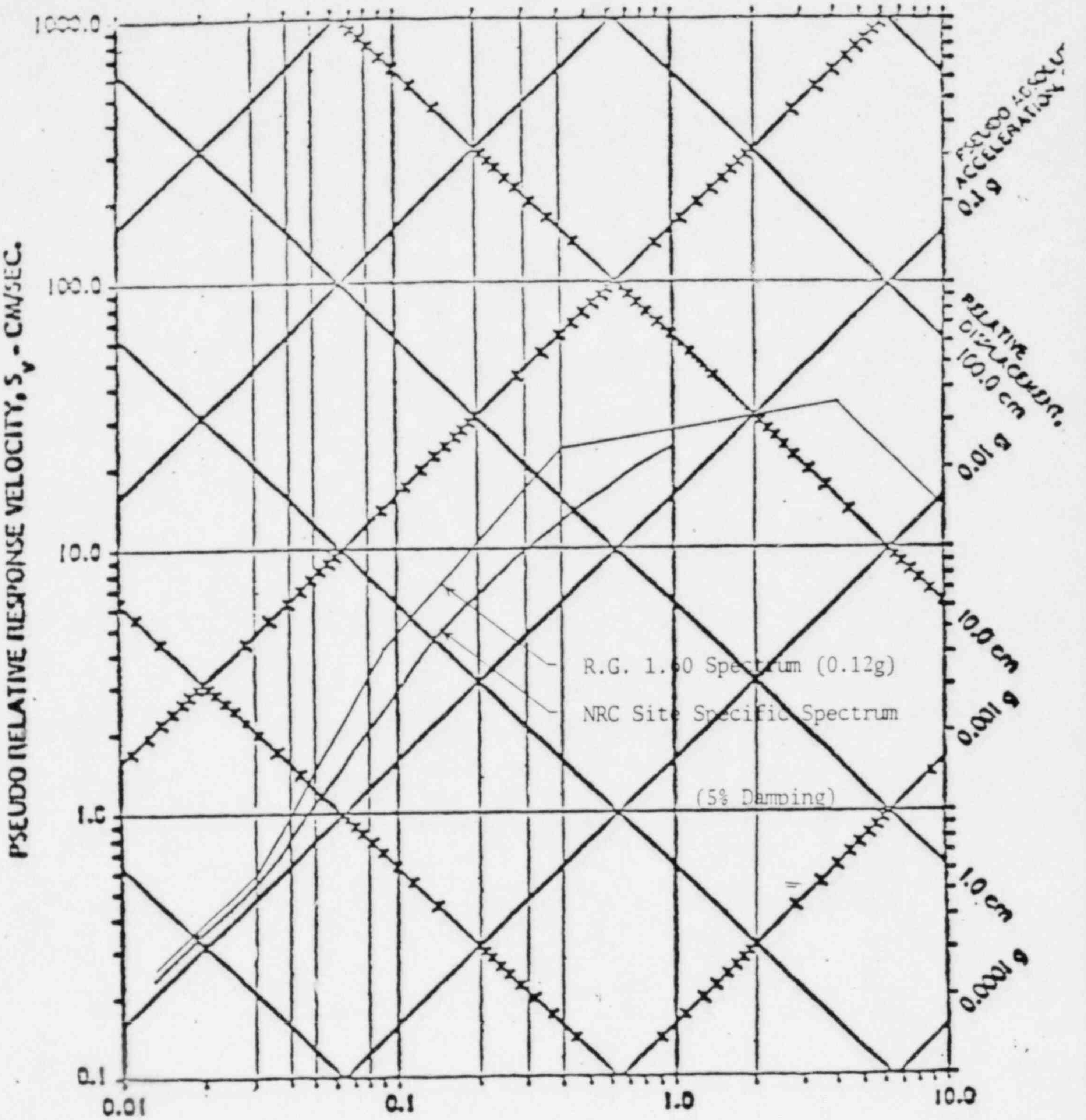

Figure 2: NRC Site specific spectrum and the R.G. 1.60 horizontal design spectrum (0.12g).

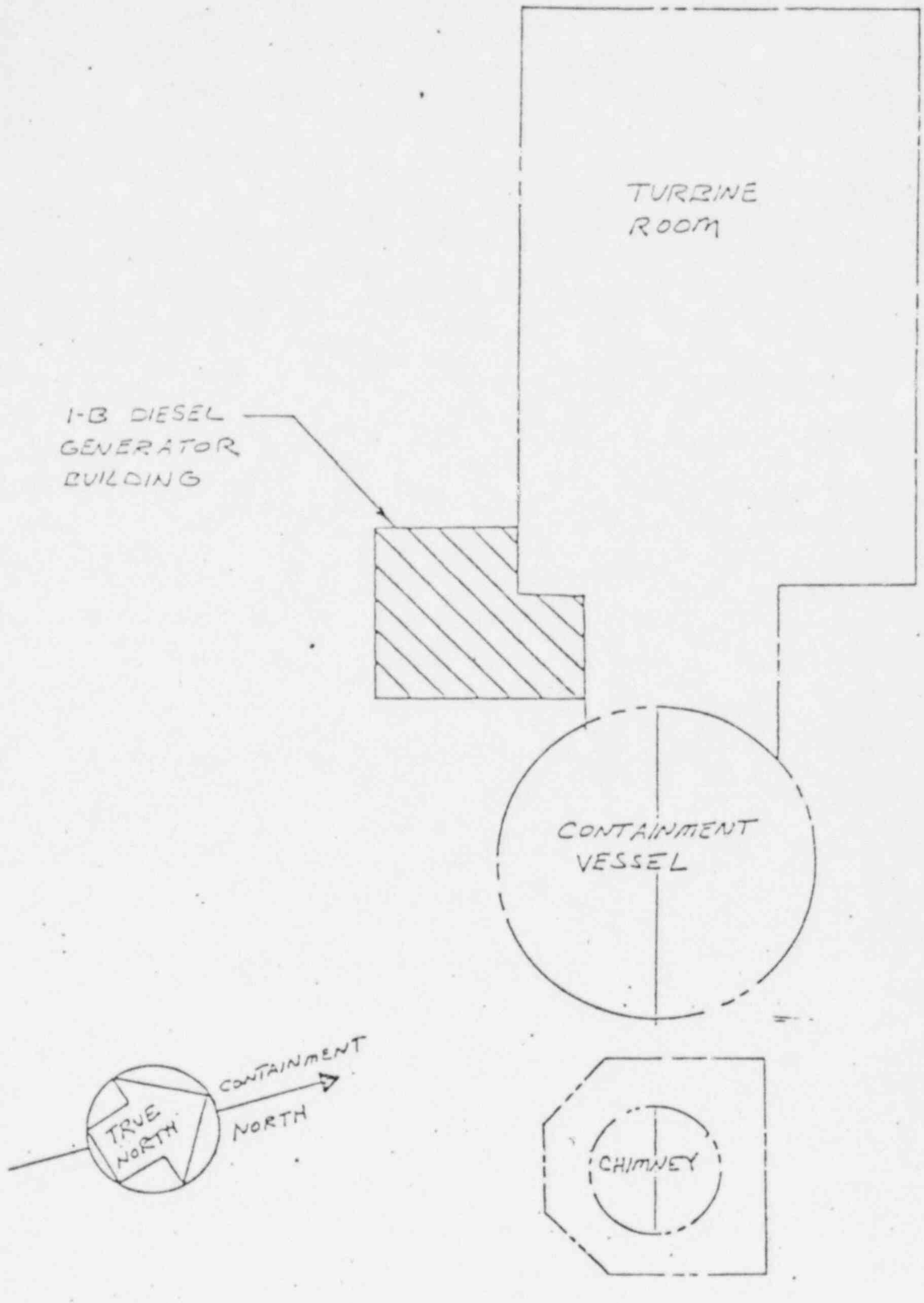

Figure 1: LACBWR Structures

Table 2 Total Shear/Moment on the Pile Group and the Maximum compression and tension on a single pile

			NES			SMA		
			LB	BE	UB	LB	BE	UB
	Shear	N-S dir.	NA	NA	1899	1640	1870	1940
Seismic	(kips)	E-W dir.	NA	NA	1893	1640	1850	1930
Load	Moment	N-S axis	NA	1.50×10^6	1.73×10^6	1.37×10^6	1.60×10^6	1.61×10^6
	(kip-in)	E-W axis	NA	2.38×10^6	2.60×10^6	1.31×10^6	1.60×10^6	1.62×10^6
	Axial (kips)		NA	1573	1511	NA	NA	NA
Dead Load (kips)			17100	17100	17100	20100	20100	20100
Max. Compression (kips)					191.6			105.9
Max. Tension (kips)					41.8			9.9

Table 1 Comparison of the SSI information and the natural frequencies of the dynamic models prepared by NES and SMA

		NES			SMA		
Shear modulus, G(ksf)		LB = BE/1.5 1600	BE 2400	LB = 1.5 BE 3600	LB = 0.5 BE 1120	BE 2230	1.5 BE 3350
SSI	Effect of piles?	No	No	No	Yes	Yes	Yes
	Effect of Embedment?	No	No	No	Yes	Yes	Yes
	Translation(k/in)	21.4×10^3	32.1×10^3	48.2×10^3	24.0×10^3	48.2×10^3	62.3×10^3
	Rocking(k-in/rad)	2.53×10^9	3.80×10^9	5.70×10^9	1.90×10^9	3.00×10^9	4.30×10^9
	Vertical(k/in)	23.5×10^3	35.3×10^3	53.0×10^3	48.1×10^3	73.4×10^3	104.4×10^3
	Torsion(k-in/rad)	3.85×10^9	5.77×10^9	8.66×10^9	1.70×10^9	3.50×10^9	4.50×10^9
Lowest Frequency in	Horiz. N-S	1.55	1.88	2.25	1.27	1.60	1.88
	Horiz. E-W	1.55	1.87	2.24	1.27	1.59	1.87
	Vertical	4.09	4.97	6.03	4.72	5.80	6.84

APPENDIX A

REVIEW SUMMARY OF THE SEISMIC RE-EVALUATION PROGRAM PLAN

ITEM	ADDRESSED?	ADEQUATE?
I. Soil and Foundation		
A. Rock Site	n/a	n/a
B. Soil Site		
o Foundation Input	yes	yes
o Generation of time history	yes	yes (1)
o Modeling technique	yes	yes
o Computer Codes	no	no (7)
C. Description of Foundation	yes	no (2)
D. Free Field Input Spectrum	yes	yes (3)
II. Structural		
A. List and Description of Category I Structures or Structures Affecting Category I systems or Components	yes	(4)
B. Modeling Techniques		
o Damping	yes	(5)
o Stiffness modeling	yes	no (6)
o Mass Modeling	yes	no (6)
o Consideration of 3-D effects	yes	no (6)
C. Seismic Analysis Methods		
o Response Spectrum, time history or equivalent static analysis	yes	yes

	o Selection of significant modes	yes	yes
	o Relative displacements	no	--
	o Modal combinations	no	--
	o Three component input	yes	yes
	o Floor spectra generation	yes	no (8)
	o Peak broadening	no	no (8)
	o Load combination	yes	yes
D.	<u>Analytical Criteria</u>		
	o Codes and criteria, including AISC, ACI and NUREG/CR-0098	yes	yes
E.	<u>Computer Codes</u>		
	o Description and verification	yes	no (9)

Comments

1. A time history whose spectrum envelops the R.G. 1.60 shape at 0.12 g should be adequate for the reactor containment building; however, review will be required. Time histories for other buildings, if used, were not available and were not reviewed.
2. The reactor building and LACBWR stack are supported on pile foundations. No description of the basemat for the turbine building is available.
3. An NRC site specific spectrum was used. The original work done by Gulf United for reactor containment building in-structure response spectra used a 0.12 g R.G. 1.60 spectrum. This envelops the NRC site specific spectrum at 0.105 g.
4. NRC staff will determine the completeness of the list.
5. The damping values used for the seismic safety assessment of buildings and stacks are reasonable. No in-structure response spectra were available for review. It is not clear what structural damping values were used in developing in-structure response spectra. The level of damping used should correspond to the stress level actually predicted for the building structures.
6. It is not clear if the mass of the turbine building basemat has been included in the dynamic analysis models. The treatment of the mass center and the center of rigidity in the reactor containment building and the turbine building needs justification. In addition, the calculation of shear area and torsional rigidity is not clear.
7. The piles and the embedment were ignored in soil spring calculations.

8. If a component support is located away from the center of rigidity, the effect of torsional response of the building should be included in the floor spectrum used to analyze the component.

There are no in-structure response spectra received for review. NES stated that, for the reactor containment building, the floor response spectra developed by Gulf United Services using the old 2-D model were used in the seismic re-evaluation of piping systems and equipment. While the input ground spectra are conservative compared to the NRC site specific spectrum, there is no study performed to demonstrate that the old 2-D model yields conservative floor spectra. Further justification is needed.

9. It is not known if the computer codes mentioned have been officially verified. From the review of the reports submitted during the meeting held in the NES office on August 10 and 11, 1982, it seems that STARDYNE is the only computer code used. STARDYNE is a public domain program. License is not required to verify and to document this program. Clarification is needed if there are other in-house computer programs used.

SMA 12208.21-R1

STRUCTURAL REVIEW OF THE LA CROSSE BOILING WATER REACTOR UNDER SEISMIC LOADS FOR THE SYSTEMATIC EVALUATION PROGRAM

prepared for

LAWRENCE LIVERMORE NATIONAL LABORATORY
Livermore, California

September, 1982

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ABSTRACT

An evaluation of the capacity of the La Crosse containment structure to withstand seismic loads was conducted as part of the Systematic Evaluation Program (SEP). Seismic loads were determined using the site specific ground response spectra with a peak ground acceleration of 0.11g developed by Lawrence Livermore National Laboratory (LLNL). No thermal or pressure loads were considered in the present study.

A lumped-mass stick model was developed for the containment structure. Soil-structure interaction was accounted for by adding springs at the base of the structure. Stiffness and damping properties of the soil springs were evaluated by adding the effect of reactor vessel and foundation slab embedment to the impedances of the pile foundation. Composite modal damping in each mode was limited to 20 percent of critical. A response spectra analysis was carried out, and a modified square-root-of-the-sum-of-the-squares (SRSS) method was used to calculate the response of the containment structure.

A comparison of the results with the original seismic analysis indicates that all moments and shears are less than the ultimate moment and shear capacities reported in the La Crosse FSAR. The analysis also indicates that the steel containment vessel stresses remain relatively low and no damage due to buckling of the shell is expected. Individual pile peak axial loads and peak moments were also computed.

SUMMARY

An analytical, multi-stick model representing the containment structure of the La Crosse nuclear power plant was developed. The model included the steel containment vessel, the outer shield wall, the inner shield wall, and the reactor vessel. Masses of the containment structure were lumped at nodes, and their geometric eccentricities were taken into account.

Soil flexibility in the present model was accounted for by adding frequency-independent foundation springs at the base of the model. Stiffness and damping of each pile was calculated, and a group effect factor was then applied to the total stiffness to account for the multi-pile group interaction. The effects of reactor vessel and foundation slab embedment were also added to the corresponding stiffness and damping impedance terms. Stiffness proportional composite modal damping in each mode was obtained by assuming 3 percent of critical damping for steel and 4 percent of critical damping for concrete. For the modal analysis, modal damping was restricted to a maximum of 20 percent of critical.

Loads in the present analysis were developed using the LLNL 0.11g site specific horizontal ground response spectra scaled. Vertical spectra were assumed to be two-thirds of the horizontal spectra. No effects of LOCA or other external load conditions were combined with the seismic loads. A response spectra analysis was performed and the modal responses were combined using a modified SRSS method. Since no steel reinforcement details were available, concrete wall capacities were assumed to be those reported in the original La Crosse FSAR seismic analysis. The results show that all peak moments in the outer shield wall and the inner shield wall lie below yield values reported in the La Crosse FSAR. Also, all computed shear loads were below ultimate shears. Peak axial loads and peak moments were also computed for the piles. Since reinforcement

details for the piles were not available, their capacities could not be evaluated. Longitudinal and shear stresses were computed for the steel containment vessel. It was shown that those stresses are relatively low, and the combination of seismic and dead weight compressive membrane stresses is much lower than the code allowable buckling stress.

1. INTRODUCTION

The La Crosse Boiling Water Reactor is owned and operated by the Dairyland Power Cooperative of La Crosse, Wisconsin. The site is located on the bank of the Mississippi River approximately one mile south of Genoa, Wisconsin. The plant was designed to produce 48 MW of net electrical power. Allis-Chalmers Manufacturing Company designed and supplied the nuclear steam supply system (NSSS) and Sargent & Lundy Engineers was the architect-engineer. Commercial operation was achieved in 1969. An initial seismic evaluation of the LACBWR containment building was conducted in 1974 by Gulf United Nuclear Fuels Company (Reference 1) and an ongoing study is currently being conducted by Nuclear Energy Services, Inc. (Reference 2).

This report describes the work done to reassess the seismic adequacy of the La Crosse Boiling Water Reactor (LACBWR) reactor building structure. As part of the Systematic Evaluation Program (SEP), Lawrence Livermore National Laboratory (LLNL) is conducting an evaluation of the capacity of a number of operating reactors subjected to combined seismic and Loss-of-Coolant-Accident (LOCA) loads. This work is being performed for the U.S. Nuclear Regulatory Commission (NRC) and is a continuation of an evaluation previously conducted by LLNL to assess the seismic adequacy of a number of these plants. This report describes the work done by Structural Mechanics Associates, Inc. to determine the capacity of the LACBWR containment building and concrete internal structures to withstand the seismic load conditions. No effects of LOCA loads or other extreme load conditions are combined with the seismic and normal operating loads in this report. The results of this work were also used to evaluate the licensee's seismic reevaluation program.

1.1 SCOPE OF EVALUATION

The scope and level of detail of the anaysis for review of the SEP plants are significantly different from those that would be required if the review were being conducted in accordance with the current version of the Standard Review Plan. Also, the assumptions made in modeling are not necessarily as conservative as those used in a design analysis and the acceptance criteria may, in some cases, be less restrictive. The SEP approach is to identify safety issues and provide a balanced, integrated approach to assessing capacity.

This assessment of LACBWR focuses on the integrity of the containment building. The original intent of the evaluation was to concentrate on the overall behavior of the containment building to withstand the combined seismic and LOCA pressure and thermal loads, and to identify any areas where additional effort is required. Therefore, numerous details such as hatches and penetrations are not included. The containment shell is assumed to be adequately reinforced around these openings so that the effects of these discontinuities on the overall containment shell response are assumed to be small. No jet impingement or pipe whip forces are being considered during this phase of the SEP. Thus, analytical techniques and models capable of describing the overall behavior of the structure to the prescribed load condition are considered adequate without the need to concentrate on local effects and details.

In the evaluation described in this report, no effects of LOCA were included as was originally planned. This was at the direction of the NRC. Since the original scope included only an evaluation of the integrity of the containment vessel, no in-structure response spectra were generated, and no seismic capacities of piping or equipment were investigated. Although the assessment of the containment vessel integrity does not normally require a detailed consideration of the reactor building internal structures, some results are included in this report since they are a direct result of the overall structure seismic model analysis.

1.2 EVALUATION CRITERIA

In general, the current review is not based on demonstrating compliance with specific design codes or other current acceptance criteria. This has also been the approach used to date in conducting the seismic evaluation of the SEP plants (Reference 3). While capacity reduction factors (ϕ factors) and similar approaches are necessary in the design codes, the evaluation conducted for LACBWR is based on unfactored loads. However, some original loads used in the LACBWR design as obtained from the FSAR are also included for comparison, although the calculations used to develop the design loads were not reviewed nor were the design stress analyses available.

The load combination investigated for the SEP includes the normal operating loads together with the seismic loads resulting from the Safe Shutdown Earthquake (SSE). Other factored load combinations such as would be required for current licensing analyses were not considered. The SSE loads were developed for a 0.11g peak ground acceleration earthquake. The site specific earthquake characteristics including the peak ground acceleration level and corresponding free-field ground response spectra were developed for the LACBWR site by LLNL (Reference 4).

2. STRUCTURE DESCRIPTION

The La Crosse reactor containment building is a welded steel cylinder with reinforced concrete internal structures. The cylinder has a hemispherical upper dome which encloses an integral 42,000 gallon water storage tank. The lower steel head is ellipsoidal and is supported by a reinforced concrete, pile supported, foundation which is approximately three feet thick. Additional concrete is placed above the lower head to form the basement floor and support the concrete internal structures. The overall height of the containment shell is approximately 144 feet and the inside diameter is 60 feet. Grade elevation is 639'-0". The cylinder is embedded 26'-6" below grade to the extremity of the lower ellipsoidal head. Major penetrations are located at near-grade and include the air lock with the fuel transfer equipment, the freight door, and the emergency air lock. The centerline of the reactor core is located at elevation 660'- 2-1/2" and the main operating floor is at elevation 701'-0". A nine inch thick outer shield wall is located inside the steel containment shell and extends up to and supports the main crane girder. The outer shield wall is integral with the concrete internal structures, but is separated from the steel shell by one-half inch of premolded joint filler. The general arrangement of the LACBWR reactor building is shown in Figure 2-1.

2.1 STEEL CONTAINMENT VESSEL

The steel containment vessel for LACBWR was fabricated and erected by the Chicago Bridge & Iron Company. The vessel was designed using 52 psig and -0.5 psig design pressures to Sections II, VIII, and IX of the ASME Boiler and Pressure Vessel Code with Nuclear Code Cases 1270N, 1271N, and 1272N. Design temperatures were 280^{o}F maximum and -20^{o}F minimum. All plate parts subject to internal pressure were fabricated from A201B to A300 steel with Charpy keyhole test of 15 ft-lb at -50^{o}F. All butt welds were 100% radiographed.

The inside radius of the cylinder and the hemisperical top head is 30'-0". The overall height is 144' - 1-3/32" inside plate dimension. The cylinder and lower heads are fabricated from 1.16 inch thick plate. The top head is fabricated from 0.60 inch thick plate for the lower 45^{o} segment and 0.705 inch thick plate for the remainder of the head. The bottom head is supported by the reinforced concrete foundation slab and additional concrete up to approximately six foot thick is placed above the bottom head to form the basement and sump floors. No shear ties or other means of positive anchorage exist between the steel shell and concrete.

2.2 PILE FOUNDATION

The La Crosse Boiling Water Reactor building is supported on a total of 230 steel encased concrete piles. The minimum specified bearing capacity of the cast in-place piles is 50 tons per pile (Reference 5). Union Metal Company mono-tube, cold rolled, seven gage piles were used. The bottom section had a tip diameter of eight inches and was tapered 0.14 inches per foot over a 30 foot length to the 12 inch butt diameter. The final length of the pile was attained using a constant 12 inch diameter extension with the same gage as the bottom section.

The piles were driven from an average elevation of approximately 609'. The appropriate resistance to develop the 50 ton capacity was generally encountered between elevations 577' to 581'. No jetting or pre-boring was necessary. The piles are all vertical and no batter angle used for any of the piles. The approximate elevation of bedrock is 507' or an additional 73 feet below the average bottom of the piles.

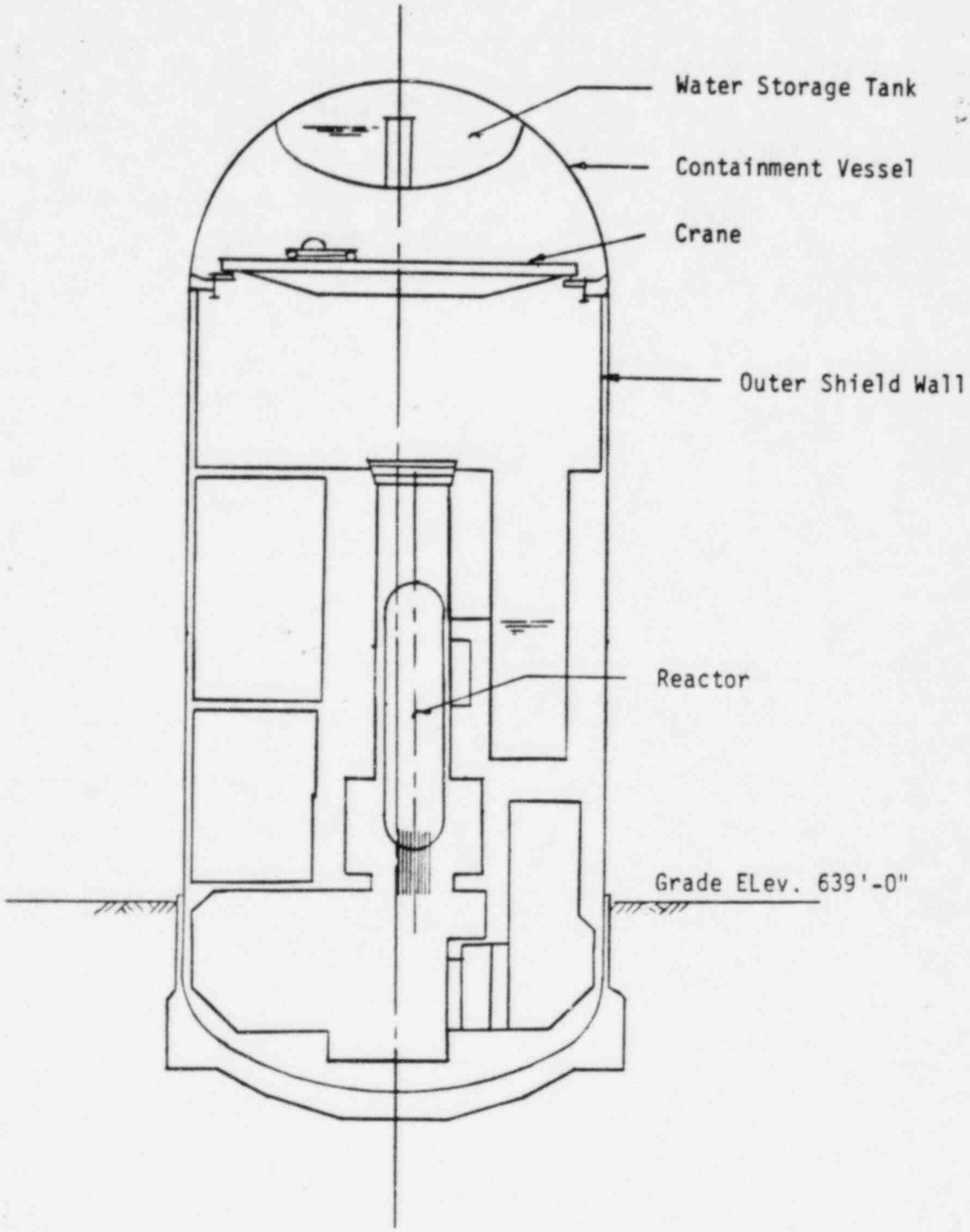

FIGURE 2-1: LACROSSE BOILING WATER REACTOR BUILDING

3. ORIGINAL ANALYSIS

The design and construction of the La Crosse Boiling Water Reactor were completed before current seismic licensing criteria for nuclear power plants were firmly established. However, a seismic analysis of the important LACBWR structures and some equipment was conducted in 1974 by Gulf United Services (Reference 1). These calculations were not reviewed as part of the SEP. However, some results for the reactor building are included in this report for comparison with those generated in the current investigation.

3.1 GEOTECHNICAL INVESTIGATION

As part of the initial seismic analysis, a geotechnical investigation was conducted by Dames & Moore (Reference 1). This investigation included an evaluation of the site seismicity, geology, and liquefaction potential.

3.1.1 Seismicity Evaluation

Included in the geotechnical investigation are an evaluation of the historical seismicity of the area and a recommendation of the Safe Shutdown Earthquake (SSE) characteristics for the site. A peak horizontal ground acceleration level of 0.12g for the SSE was developed for the site. The horizontal ground response spectra recommended for use in the original analysis are shown on Figure 3-1 from Reference 1.

3.1.2 Geology Evaluation

The LACBWR structures including the reactor building are situated on 15 to 20 feet of hydraulically placed fill. The hydraulic fill overlies approximately 100 to 130 feet of glacial outwash and fluvial deposits. The bedrock below the site consists of nearly flatlying sandstones and shales.

Boring logs from prior investigations as well as those drilled as part of the Dames & Moore investigation were used to develop the profile of the soil characteristics. The soil beneath the reactor building consists of fine-to-medium sands with shear wave velocities in the 820 to 917 ft/sec range. Although characteristics of the soil are described for various layers, with the possible exception of the hydraulic fill, the soils exhibit a very uniform gradation down to bedrock without significant discontinuities in the soil values. Table 3-1 from Reference 1 shows the soil overburden configuration, and Table 3-2 lists the engineering properties including the shear strain effects for the 0.12g SSE earthquake.

3.2 STRUCTURE SEISMIC ANALYSIS

The initial seismic analysis of the LACBWR structures included the development of a two-dimensional model of the reactor containment building.

3.2.1 Reactor Building Structural Model

The initial reactor model was a lumped-mass shear beam model with a total of 36 masses. The model included a representation of the outer steel shell, the concrete internal structure and biological shielding, the reactor vessel, the water storage tank, and the pile foundation (Figure 3-2). The structure foundation system was analyzed using the SIM Code, an acronym standing for Structure-In-Medium which utilized the free-free beam modes as input.

3.2.2 Pile Foundation Model

The original stiffness and damping of the pile foundation were developed using a single equivalent beam for the piles. The details of the calculations used to develop the equivalent beam were not available for review. Apparently, the stiffness was determined from the summation of the individual pile stiffnesses with no reduction for pile group effects. The mass of the soil within the pile group was included but no stiffness was attributed to this soil. Elastic half-space frequency independent springs and dashpots were computed assuming a rigid disk with the same dimensions of the foundation slab. However, these springs and dashpots were apparently attached at a location corresponding to the bottom of the piles. Also, an embedment stiffness was included which appears to have been developed for a single equivalent cylinder representing the pile group. These two assumptions would imply that plane sections through pile group would remain plane under lateral load conditions. In addition, a dashpot was added to account for the pile damping which was developed from Reference 6 for cylinders buried in foam.

3.2.3 Structure Response

Using the soil-structure interaction model described above, shears and moments throughout the structure model were computed using a time history analysis. The peak values for the SSE are plotted in Figures 3-3 and 3-4.

A stress evaluation for the LACBWR reactor building based on the seismic loads described above was also conducted. At several locations in the lower elevations, the maximum seismic moment was found to exceed the yield moment capacity. All seismic moments were found to be less than the ultimate moment capacity of the structure, and no seismic shear loads were found to be in excess of the ultimate shear capacities throughout the structure.

An evaluation of the water sloshing in the top head storage tank was also conducted in the original analysis. A separate analysis was conducted using an equivalent circular cylindrical tank and the seismic input from the reactor building structural model. Low stresses in the tank were computed.

TABLE 3-1

SOIL OVERBURDEN CONFIGURATION

Soil Layer	Thickness (ft)	Depth to Bottom (ft)	Dry Weight (pcf)	Description
1	18	18	105	Hydraulic fill, medium sand
2	12	30	101	Fine to medium sand
3	70	100	107	Fine to medium sand, some fine gravel
4	15	115	124	Fine to medium sand, some fine to medium gravel
5	20	135	115	Fine to medium sand, some fine to medium gravel

TABLE 3-2

STRAIN COMPATIBLE SOIL PROPERTIES FOR
0.12 G SSE MOTION AT SURFACE

Zone Number	Soil Layer	Shear Modulus (PSF x 10^6)	Damping Ratio %	Shear Speed (FPS)	RMS Shear Strain %
1	1	0.997	11.5	553	0.318×10^{-2}
2	2	2.16	5.0	830	0.356×10^{-2}
3	3	1.94	8.1	820	0.601×10^{-2}
4	3	2.23	8.8	820	0.715×10^{-2}
5	3	2.51	9.3	869	0.800×10^{-2}
6	4	2.65	3.0	829	0.880×10^{-2}
7	5	3.01	3.5	917	0.892×10^{-2}

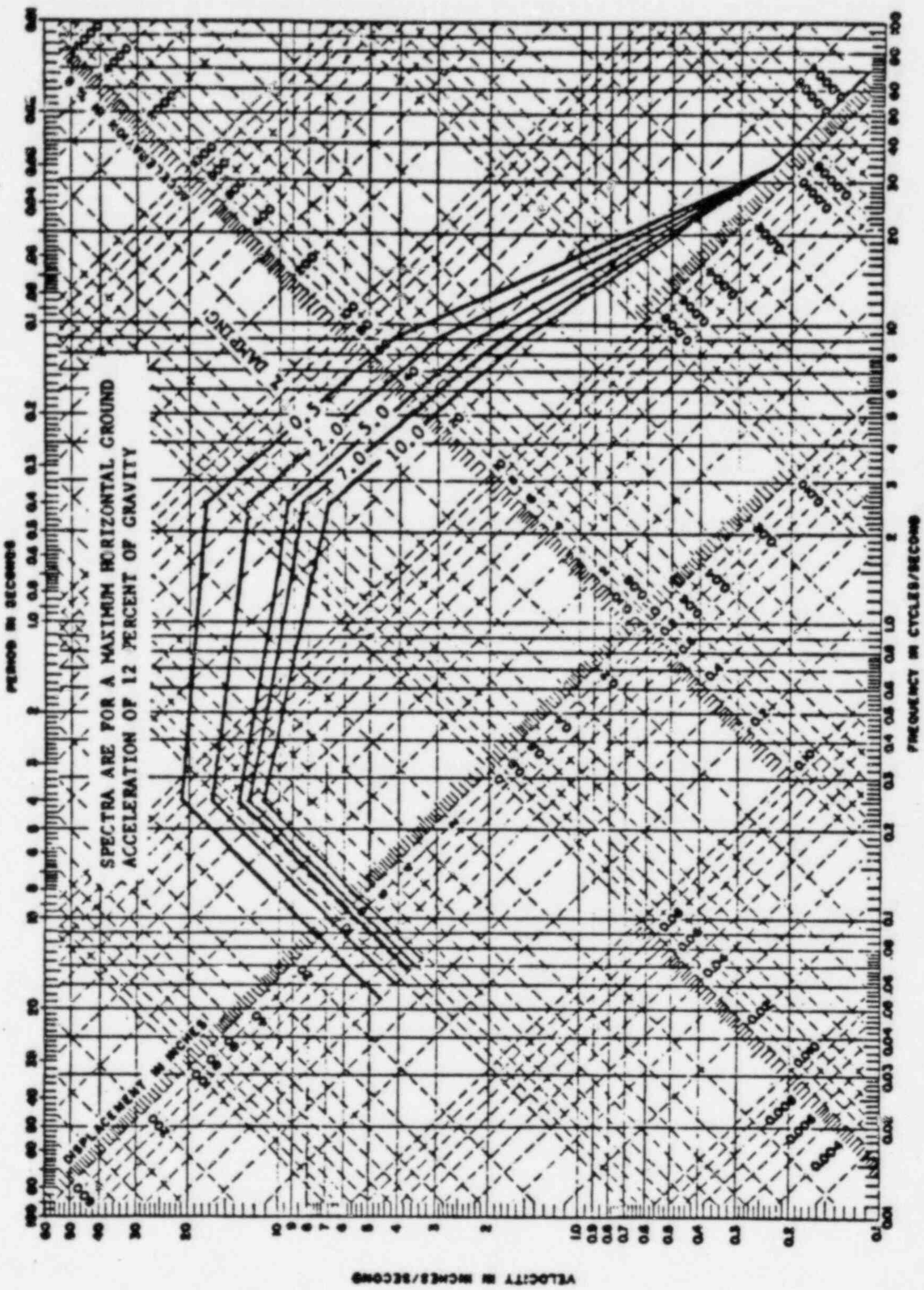

FIGURE 3-1: HORIZONTAL RESPONSE SPECTRA - ORIGINAL ANALYSIS

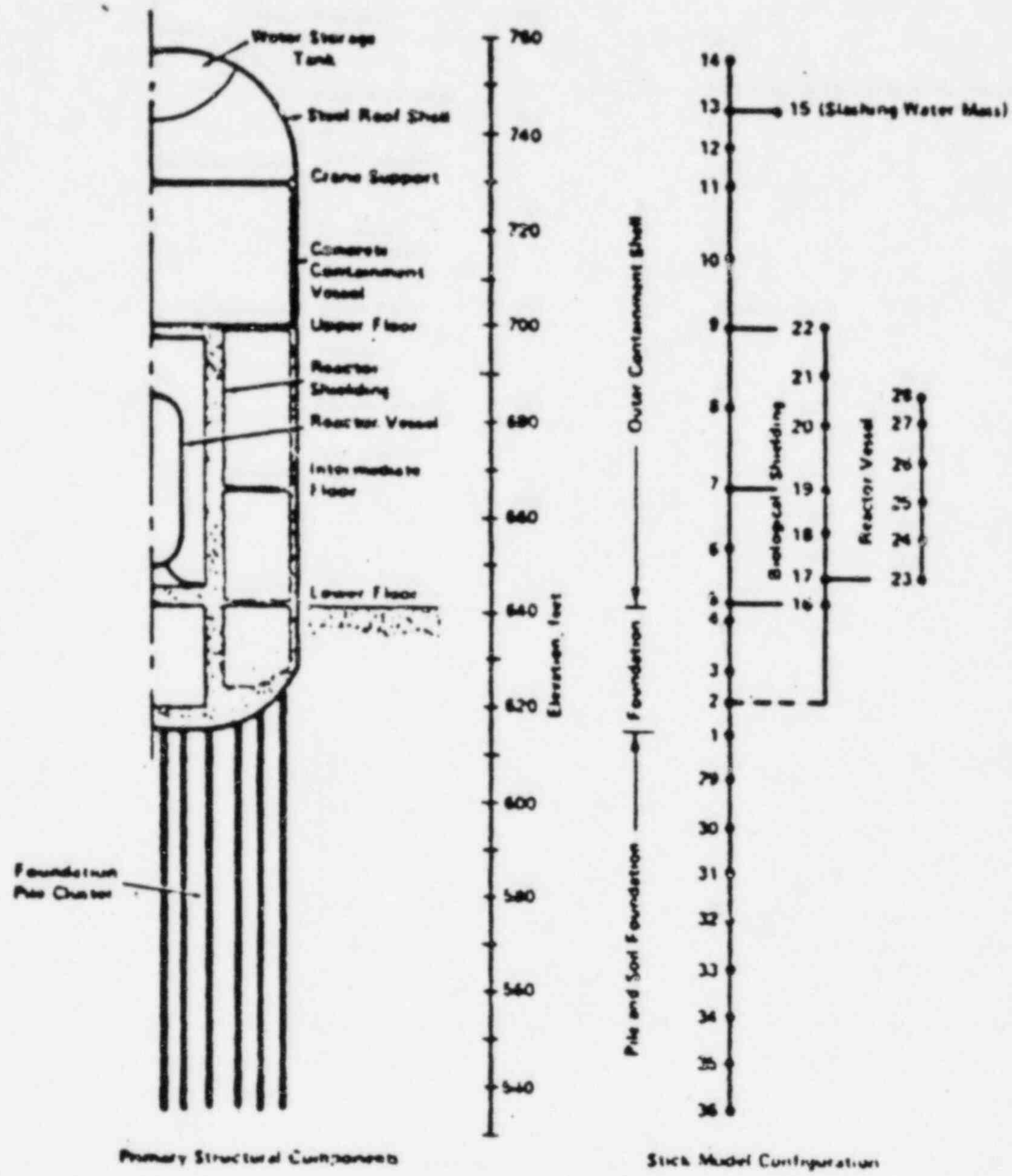

FIGURE 3-2: ORIGINAL SEISMIC ANALYSIS MODEL

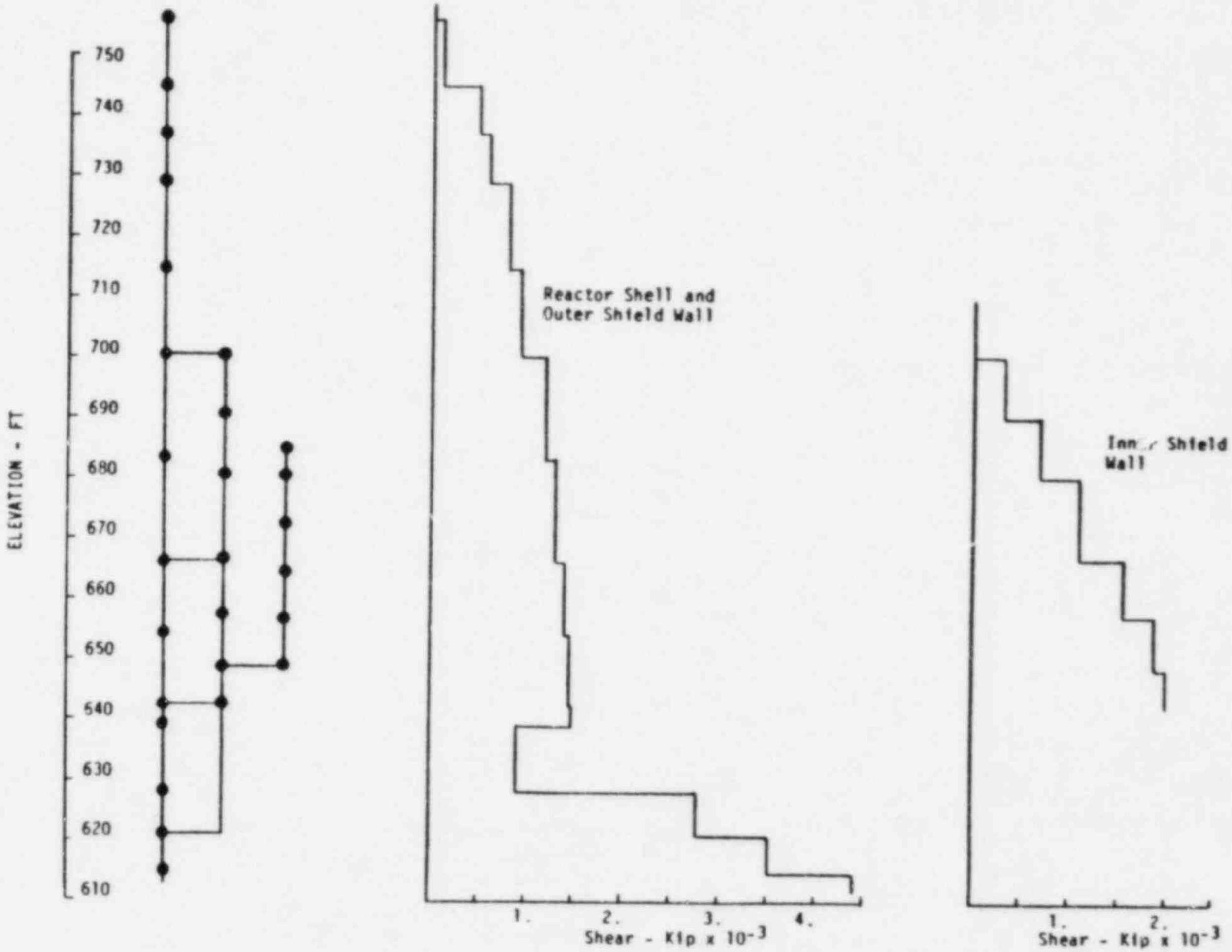

FIGURE 3-3. ORIGINAL ANALYSIS PEAK SHEAR DISTRIBUTION

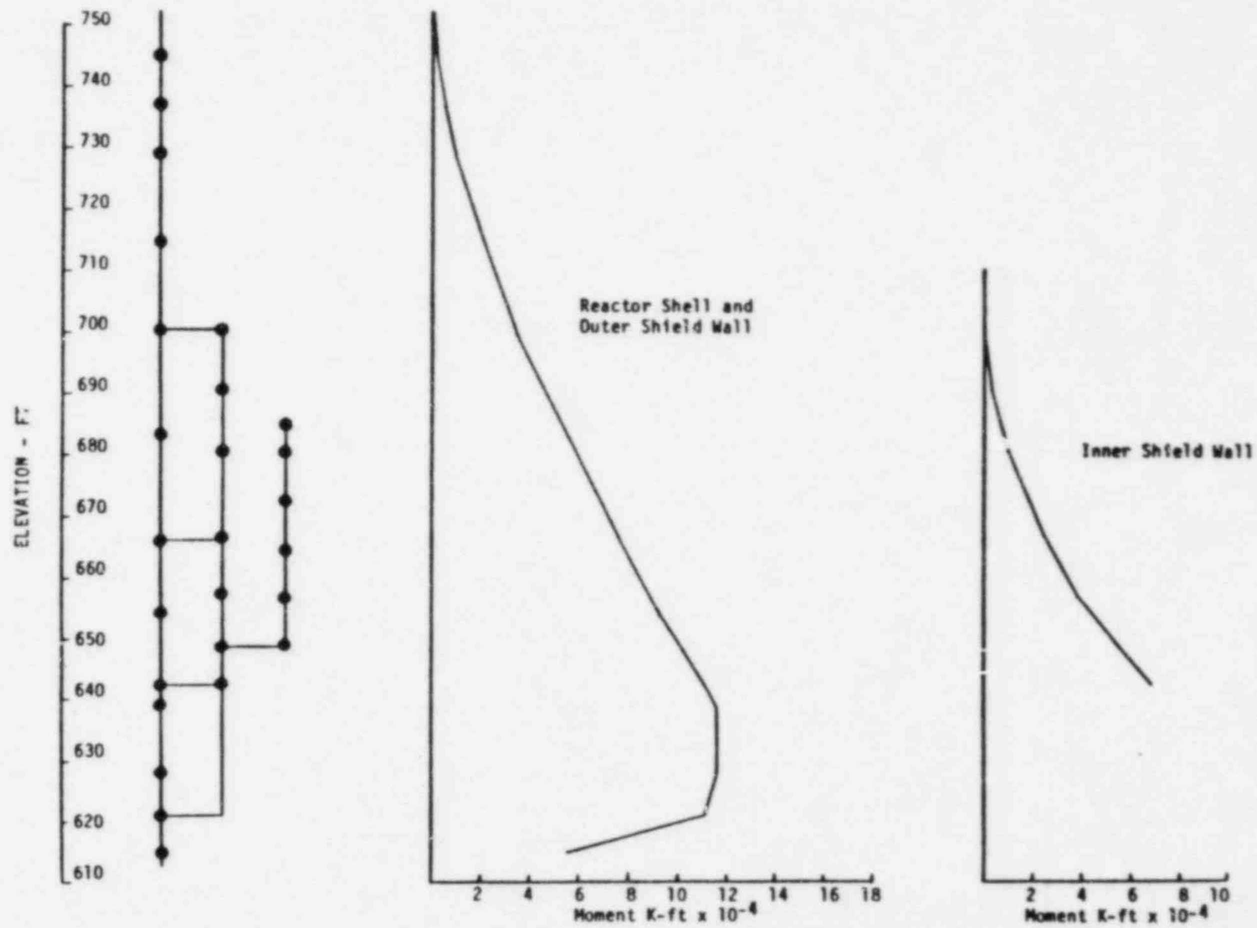

FIGURE 3-4. ORIGINAL ANALYSIS PEAK MOMENT DISTRIBUTION

4. SEISMIC MODEL

The evaluation of the capacity of the La Crosse Boiling Water Reactor for the SEP was conducted using a new soil-structure interaction model of the reactor building. The original scope of work was to evaluate the containment vessel for combined seismic and LOCA loads. The concrete internal structure (inner and outer shielding) and reactor vessel were included in the model since they could influence the containment vessel response. However, no other items of equipment were included as discrete elements, and no in-structure response spectra or time-histories were developed since no investigation of the piping or equipment seismic capacity was included in this effort. The SEP evaluation of the structure to include LOCA loads was subsequently discontinued.

4.1 SEISMIC INPUT

The seismic analysis of the LACBWR reactor building conducted for the SEP was based on the site specific spectra developed by LLNL (Reference 4). Figure 4-1 shows the 5% damped site specific spectrum. For vertical input, two-thirds of the horizontal spectra were used. The peak horizontal ground acceleration for the site specific spectra is approximately 0.11g. This may be compared to the 0.12g peak ground acceleration used in the original seismic analysis. A comparison of the 5% damped spectra for the LACBWR site developed by LLNL (Reference 4) and by Dames & Moore (Reference 1) is shown in Figure 4-2. Throughout the frequency range of interest, the original analysis spectrum is seen to considerably exceed the site specific spectrum used for the SEP evaluation. Since the analysis for this investigation was based on response spectrum analysis, no artificial earthquake time-histories were generated nor were any actual earthquake records used as input.

4.2 SOIL PROPERTIES

The soil properties used in the current SEP evaluation were based on values developed by Dames & Moore for the original seismic investigation (Reference 1). Shear modulus and hysteretic soil damping for the overburden are shown in Table 3-2. The values include the effects of the soil strains expected for the 0.12g SSE. In the original seismic investigation, only one value of each soil property was used. However, in accordance with Reference 7, a range of soil properties was used for the current evaluation in order to account for both the soil uncertainties, including some variation with depth, as well as the pile-soil interaction.

The site specific earthquake for the LACBWR site (Reference 4) has a peak ground acceleration of 0.11g. This is nearly the same peak ground acceleration as was used in the original seismic analysis. Although the free-field ground response spectra exhibit somewhat different shapes in the amplified region of the spectra, it is expected that the soil strains for both earthquakes will be comparable. Consequently, no modifications for soil-strain effects were made for the SEP analysis beyond those originally developed by Dames & Moore.

As shown in Table 3-2, some variation of the soil properties with depth is indicated. A weighted average of all the soil zones identified by Dames & Moore results in a best estimate shear modulus of approximately 2.37×10^6 psf. An average of the more important zones where the piles are founded results in a shear modulus of approximately 2.11×10^6 psf. For the SEP investigation reported here, a best estimate for the soil-shear modulus of 2.23×10^6 psf and shear wave velocity of 820 fps was judged to adequately represent the overburden with the exception of the hydraulic fill layer. The 2.23×10^6 psf value corresponds to Zone 4 identified in Reference 1. Figure 4-3 shows the original Dames & Moore soil properties for the hydraulic fill layer and Figure 4-4 shows the properties for Zone 4 which is judged to adequately represent the remainder of the overburden for the best estimate case. A weighted average of the soil hysteretic damping expected in the overburden results in a value of approximately 7% of critical.

In order to account for the uncertainty in the soil properties, a range of soil properties was used in determining the soil foundation stiffness. Reference 7 recommends a value for the lower bound soil shear modulus of 50% of the best estimate high strain shear modulus, and 90% of the best estimate low strain modulus for the upper bound soil modulus. For the current evaluation, this recommendation was used for the lower bound case which results in a 1.12×10^6 psf shear modulus. Following the recommendations of Reference 7 for the upper bound case results in an increase of less than 12% above the best estimate case, however. Therefore, in order to account for a greater possible uncertainty in the soil properties, a soil shear modulus equal to 3.35×10^6 psf, or a 50% increase above the best estimate, was used for this investigation. Hysteretic soil damping equal to 7% was assumed for both the upper and lower bound conditions.

4.3 PILE FOUNDATION

Effects of pile foundation on the response of La Crosse containment were taken into account by adding soil springs at the base of the structure. An approximate method developed by Novak (References 8 and 9) was used to calculate stiffness and damping properties of a single pile. A group effect factor was then applied to the total stiffness of all piles to account for pile group interaction (Reference 10). The approach to the problem is described in the following paragraphs.

In general, stiffness and damping of foundations on elastic soil are frequency dependent. However, for the current analysis, a set of frequency independent stiffness and damping terms are calculated. It is shown (Reference 8) that the effect of frequency is not very strong in the frequencies typical of piles, and therefore the above assumption is considered justified. Soil-shear modulus is assumed to vary parabolically with depth, and the value of shear modulus at the pile tip (G_t) is varied $\pm$ 50 percent. The important parameters affecting the stiffness and damping of a single friction pile are $\sqrt{G_t/E}$, ρ/ρ_p, ℓ/r, and V_b/V_t, where G_t = soil-shear modulus at the tip, E = Young's modulus of pile, ρ/ρ_p = ratio of soil density to pile density, ℓ/r = ratio of pile length to pile radius, and V_b/V_t = ratio of shear wave velocity below tip to shear wave velocity at tip. Of the above parameters, the slenderness ratio (ℓ/r) and G_t/E ratio have a more profound effect on pile stiffness and damping. Table 4-1 shows stiffness and damping for a single pile for three values of soil shear modulus, i.e., upper bound, best estimate, and lower bound. Rocking and torsional stiffness and damping effects are considered negligible for a single pile in comparison to the overall group pile foundation rocking and torsion values. In general, damping is seen to increase for the softer soil conditions. Stiffness values are seen to decrease with lower soil moduli but not in direct proportion indicating the effects of the relative stiffness of the pile itself which remains constant. Once individual pile stiffness and damping are determined, the overall foundation stiffness and damping can be determined as follows:

$$K_w = \frac{n}{c_w} k_w \quad \text{vertical translation}$$

$$K_u = \frac{n}{c_u} k_u \quad \text{horizontal translation}$$

$$K_\psi = \sum_{i=1}^{n} k_w x_i^2 \quad \text{rocking}$$

$$K_\theta = \sum_{i=1}^{n} k_u r_i^2 \quad \text{Torsion}$$

In the above equations, the left hand side represents an overall foundation stiffness, n is the total number of piles, c_w and c_u are group interaction factors (Reference 10), x_i is the distance from the rotational axis in rocking to pile i, and r_i is the distance from center of rotation in torsion to pile i. For the present analysis, it was assumed that all piles are fixed in the pile cap, although attachment details were not available for review. Also, the interaction term between rocking and horizontal translation was neglected in computing the foundation impedance terms. It may be noted that the damping terms, which are represented as complex stiffness terms in this approach, are treated exactly the same as stiffness terms. The effects of reactor vessel and foundation slab embedment on stiffness and damping impedance terms were computed separately (Reference 11) and added to their corresponding terms.

In order to account for possible separation of the containment vessel from the soil due to lack of tensile capacity of the cohesionless soil, only one-half the theoretical embedment effect was used in this analysis. Overall foundation stiffness and damping for the upper and lower bound soil shear moduli, as well as the best estimate shear modulus, are listed in Table 4-2.

Shown in Table 4-3 is a comparison of the solution for a footing on an elastic half-space with the present model using best estimate of soil-shear modulus. The last column of Table 4-3 lists values of stiffness and damping for the pile group without any embedment effects. Comparison of the last two columns of Table 4-3 reveals that taking into account the embedment effect increases damping for all modes of vibration, but stiffnesses are increased only slightly over the pile group without foundation embedment. It has been noted in the literature that the use of piles decreases geometric damping, particularly in the horizontal translation mode (Reference 13). Therefore, the foundation embedment in this case contributes relatively heavily to the overall foundation geometric damping. As mentioned previously, in addition to the geometric damping values listed in Tables 4-2 and 4-3, soil hysteretic damping equal to 7 percent of critical was also added to all three soil cases for the seismic response analysis.

4.4 STRUCTURE MODEL

In the present study, the reactor building and its inner structures were modeled as several three-dimensional beam sticks, with the masses lumped at the nodes. The structure is essentially symmetric about the N-S axis. This axis is denoted by X in the following discussion. Stiffnesses and masses for the steel containment, outer shield structure, and the inner shield structure were computed separately. The mathematical model for the reactor vessel used in the original anaysis was employed in the current analysis. It was included in case its seismic response could influence the overall structure response. However, no details of the reactor vessel were available to independently determine its dynamic characteristics and the original calculations were not checked as part of the SEP analysis. The inner and outer shield walls are closely coupled due to the stiffness of the connecting diaphragms and shear walls. Therefore, the masses of both inner and outer shield walls at a given elevation, except for the reactor vessel, were lumped together. The water tank was modeled as several elements including a sloshing mode connected to steel containment. Soil flexibility was accounted for by adding the pile foundation springs, described in the previous section, at the base of the model.

Nodal connectivity in the model is shown in Figure 4-5. The beam stick between nodes 18 and 48 represents the reactor vessel, and the water tank is in between nodes 70 and 74. Nodal coordinates for the model shown in Figure 4-5 are listed in Table 4-4. Locations of the nodal masses in the N-S plane and their respective elevations are depicted in Figure 4-6, and their values are listed in Table 4-5. It may be noted that the reactor building masses, including the reactor vessel, are eccentric with respect to the centerline of the building in most cases. For instance, the reactor vessel is eccentric 30 inches from the centerline of the building. The locations of the centers of resistance for the beam stick models of the reactor building are shown in Figure 4-7. Comparing Figures 4-6 and 4-7 shows that for most elevations, the center of mass and center of resistance of the building do not coincide. For the present analysis, the sticks representing the outer shield wall and the inner shield wall are connnected with stiff elements at every elevation point representing the stiff shear walls and concrete diaphragms.

This model assumes that although there is 1/2" premolded joint expansion filler between the outer shield wall and the steel containment vessel, they would displace horizontally together. Taking into consideration elastic properties of the expansion filler material and the dimensions of the structure, it is most unlikely that the two stick models can respond with any significant relative horizontal displacement between the concrete and steel. No vertical shear connection between the steel containment vessel and the outer shield wall was assumed, however. Table 4-6 lists all the beam section properties, and Table 4-7 lists all the beam elements, their connectivity, material property (steel vs concrete), and section properties as described in Table 4-6.

For the present analysis, material damping of 3 percent of critical for steel and 4 percent of critical for concrete were assumed. These values for material damping are consistant with the SEP guidelines (Reference 3) for damping under working stress conditions, i.e., less than about one-half yield point. Stiffness proportional composite modal damping in each mode, including the soil damping, were computed for all three soil cases. For the actual response spectra analysis, modal damping including soil hysteretic damping was limited to a maximum of 20 percent of critical. Modal damping ratios for the three soil cases are listed in Table 4-8.

TABLE 4-1

SINGLE PILE STIFFNESS AND DAMPING FOR THE THREE SOIL CASES

Mode of Vibration	Impedance	Upper Bound Soil Modulus	Best Estimate Soil Modulus	Lower Bound Soil Modulus
Vert. Translation	Stiffness (k/in)	3970	3100	2035
	Damping (%)	3.2	6.4	22
Horiz. Translation	Stiffness (k/in)	616	486	260
	Damping (%)	3.0	6.1	19

TABLE 4-2

FOUNDATION STIFFNESS AND DAMPING INCLUDING THE EFFECT OF EMBEDMENT

Mode of Vibration	Impedance	Upper Bound Soil Modulus	Best Estimate Soil Modulus	Lower Bound Soil Modulus
Vertical Translation	Stiffness (k/in)	104,400	73,400	48,100
	Damping (%)	14	14	17
Horiz. Translation	Stiffness (k/in)	62,300	48,200	24,000
	Damping (%)	24	24	31
Rocking	Stiffness (in-k/rad)	4.3×10^9	3.0×10^9	1.9×10^9
	Damping (%)	2	2.3	3.5
Torsion	Stiffness (in-k/rad)	4.5×10^9	3.5×10^9	1.7×10^9
	Damping (%)	90	91	119

TABLE 4-3

COMPARISON OF HALF-SPACE SOLUTIONS WITH CALCULATED STIFFNESS AND DAMPING USING BEST ESTIMATE OF SOIL SHEAR MODULUS

Vibration Mode	Impedance	Half-Space w/o Piles or Embedment	Piles W/Embedment	Piles w/o Embedment
Vertical Translation	Stiffness (k/in)	31,000	73,400	69,900
	Damping (%)	41	14	2
Horiz. Translation	Stiffness (k/in)	26,300	48,200	43,000
	Damping (%)	23	24	3.8
Rocking	Stiffness (in-k/rad)	2.7×10^9	3.0×10^9	2.6×10^9
	Damping (%)	1.0	2.3	0.6
Torsion	Stiffness (in-k/rad)	3.7×10^9	3.5×10^9	3.4×10^9
	Damping (%)	38	91	14

TABLE 4-4

NODAL POINT COORDINATES (IN)

Node	X Coord.	Y Coord.	Z Coord.
1	0.000	0.000	0.000
2	0.000	0.000	108.000
3	84.000	0.000	108.000
4	0.000	0.000	186.000
5	11.000	0.000	186.000
6	84.000	0.000	186.000
7	95.000	0.000	186.000
8	0.000	0.000	277.200
9	84.000	0.000	277.200
10	95.000	0.000	277.200
11	100.000	0.000	277.200
12	150.000	0.000	277.200
13	0.000	0.000	369.000
14	33.000	0.000	369.000
15	72.000	0.000	369.000
16	100.000	0.000	369.000
17	150.000	0.000	369.000
18	30.000	0.000	369.000
19	30.000	0.000	429.000
20	30.000	0.000	450.500
21	0.000	0.000	466.300
22	33.000	0.000	466.300
23	81.000	0.000	466.300
24	150.000	0.000	466.300
25	30.000	0.000	490.600
26	30.000	0.000	534.500
27	0.000	0.000	563.400
28	33.000	0.000	563.400
29	81.000	0.000	563.400
30	94.000	0.000	563.400
31	124.000	0.000	563.400
32	150.000	0.000	563.400
33	30.000	0.000	578.500
34	30.000	0.000	629.400
35	30.000	0.000	680.000
36	0.000	0.000	660.000
37	56.000	0.000	660.000
38	81.000	0.000	660.000
39	124.000	0.000	660.000
40	30.000	0.000	730.900

TABLE 4-4 (Continued)

NODAL POINT COORDINATES (IN)

Node	X Coord.	Y Coord.	Z Coord.
41	0.000	0.000	742.600
42	52.000	0.000	742.600
43	81.000	0.000	742.600
44	110.000	0.000	742.600
45	124.000	0.000	742.600
46	30.000	0.000	781.800
47	30.000	0.000	825.000
48	30.000	0.000	861.100
49	0.000	0.000	825.000
50	52.000	0.000	825.000
51	110.000	0.000	825.000
52	0.000	0.000	906.000
53	52.000	0.000	906.000
54	69.000	0.000	906.000
55	110.000	0.000	906.000
56	0.000	0.000	987.000
57	52.000	0.000	987.000
58	69.000	0.000	987.000
59	110.000	0.000	978.000
60	0.000	0.000	1068.000
61	27.000	0.000	1068.000
62	52.000	0.000	1068.000
63	110.000	0.000	1068.000
64	0.000	0.000	1215.000
65	-29.000	0.000	1215.000
66	0.000	0.000	1374.000
67	0.000	0.000	1458.000
68	0.000	0.000	1542.000
69	0.000	0.000	1626.000
70	0.000	0.000	1680.000
71	0.000	0.000	1590.000
72	0.000	0.000	1626.000
73	0.000	0.000	1735.200
74	0.000	0.000	1542.000

TABLE 4-5

NODAL MASSES

Node Number	Mass ($kip\text{-}sec^2/in$)	Mass-Moment X ($in\text{-}kip\text{-}sec^2$)	Mass-Moment Y ($in\text{-}kip\text{-}sec^2$)	Mass-Moment Z ($in\text{-}kip\text{-}sec^2$)
1	7.41	2.3×10^5	2.3×10^5	4.6×10^5
2	3.53	1.3×10^5	1.3×10^5	2.6×10^5
5	3.14	1.2×10^5	1.2×10^5	2.4×10^5
11	2.35	6.5×10^4	5.8×10^4	1.2×10^5
15	5.46	1.5×10^5	1.3×10^5	2.8×10^5
19	0.145	0	0	0
20	0.147	0	0	0
23	4.05	8.9×10^4	7.7×10^4	1.7×10^5
25	0.153	0	0	0
26	0.164	0	0	0
30	3.29	8.1×10^4	6.7×10^4	1.5×10^5
33	0.174	0	0	0
34	0.174	0	0	0
35	0.174	0	0	0
37	5.36	1.5×10^5	1.4×10^5	2.9×10^5
40	0.163	0	0	0
44	2.12	6.1×10^4	4.9×10^4	1.1×10^5
46	0.140	0	0	0
47	0.109	0	0	0
51	2.04	6.1×10^4	4.9×10^4	1.1×10^5
54	1.67	5.0×10^4	4.5×10^4	9.5×10^4
58	1.67	5.0×10^4	4.5×10^4	9.5×10^4
61	4.92	1.3×10^5	1.3×10^5	2.6×10^5
65	1.20	7.4×10^4	6.7×10^4	1.4×10^5
66	1.24	6.7×10^4	6.7×10^4	1.3×10^5
67	0.07	0	0	0

TABLE 4-5 (Continued)

NODAL MASSES

Node Number	Mass ($kip-sec^2/in$)	Mass-Moment X ($in-kip-sec^2$)	Mass-Moment Y ($in-kip-sec^2$)	Mass-Moment Z ($in-kip-sec^2$)
68	0.07	0	0	0
69	0.07	0	0	0
70	0.28	0	0	0
71	0.26	0	0	0
72	0.44	0	0	0
73	0.02	0	0	0
74	0.03	0	0	0

TABLE 4-6

BEAM GEOMETRIC PROPERTIES

Section Number	Axial Area A(1)	Shear Area A(2)	Shear Area A(3)	Torsion J(1)	Inertia I(2)	Inertia I(3)
1	.1000E+07	.1000E+07	.1000E+07	.2000E+13	.1000E+13	.1000E+13
2	.2090E+03	.1110E+03	.1110E+03	.5600E+06	.2800E+06	.2800E+06
3	.2600E+03	.1380E+03	.1380E+03	.1140E+06	.5700E+05	.5700E+05
4	.6280E+03	.3330E+03	.3330E+03	.1600E+07	.8000E+06	.8000E+06
5	.3960E+03	.2100E+03	.2100E+03	.4000E+06	.2000E+06	.2000E+06
6	.2500E+04	.1323E+04	.1323E+04	.3000E+09	.1500E+09	.1500E+09
7	.2620E+04	.1390E+04	.1390E+04	.3400E+09	.1700E+09	.1700E+09
8	.1338E+04	.7090E+03	.7090E+03	.1700E+09	.8400E+08	.8400E+08
9	.1259E+04	.6670E+03	.6670E+03	.1400E+09	.7000E+08	.7000E+08
10	.1086E+04	.5750E+03	.5750E+03	.9000E+08	.4500E+08	.4500E+08
11	.8360E+03	.4430E+03	.4430E+03	.3000E+08	.1500E+08	.1500E+08
12	.4150E+03	.2200E+03	.2200E+03	.3400E+07	.1700E+07	.1700E+07
13	.6080E+03	.3220E+03	.3220E+03	.1600E+08	.7900E+07	.7900E+07
14	.7700E+03	.4080E+03	.4080E+03	.3200E+08	.1600E+08	.1600E+08
15	.8210E+03	.4350E+03	.4350E+03	.3900E+08	.1950E+08	.1950E+08
16	.2200E+06	.1150E+06	.1150E+06	.2800E+11	.1400E+11	.1400E+11
17	.3370E+05	.1070E+05	.2400E+05	.3100E+10	.1320E+10	.1800E+10
18	.4880E+05	.1070E+05	.3910E+05	.3520E+10	.1320E+10	.2200E+10
19	.3480E+05	.1070E+05	.2510E+05	.3320E+10	.1320E+10	.2000E+10
20	.3240E+05	.1070E+05	.2270E+05	.3320E+10	.1320E+10	.2000E+10
21	.2040E+05	.1070E+05	.1070E+05	.2640E+10	.1320E+10	.2000E+10
22	.1510E+05	.1200E+05	.9000E+04	.2000E+09	.1320E+10	.1320E+10
23	.1730E+05	.1380E+05	.1040E+05	.2300E+09	.7500E+08	.1300E+09
24	.5960E+05	.4770E+05	.3580E+05	.1200E+10	.7500E+08	.1600E+09
25	.5430E+05	.4340E+05	.3260E+05	.8600E+09	.5700E+09	.6200E+09
26	.4590E+05	.3670E+05	.2750E+05	.5500E+09	.2340E+09	.6260E+09
27	.1580E+02	0.	0.	.1000E-01	.1750E+09	.3760E.09
					.1000E-01	.1000E-01

TABLE 4-7

BEAM ELEMENT DATA

(Material: 1 = Concrete, 2 = Steel)

Beam Number	Node -I	Node -J	Material Number	Section Number
1	1	2	2	1
2	2	3	2	1
3	4	5	2	1
4	5	6	2	1
5	6	7	2	1
6	8	9	2	1
7	9	10	2	1
8	10	11	2	1
9	11	12	2	1
10	13	14	2	1
11	14	15	2	1
12	15	16	2	1
13	16	17	2	1
14	21	22	2	1
15	22	23	2	1
16	23	24	2	1
17	27	28	2	1
18	28	29	2	1
19	29	30	2	1
20	30	31	2	1
21	31	32	2	1
22	36	37	2	1
23	37	38	2	1
24	38	39	2	1
25	41	42	2	1
26	42	43	2	1
27	43	44	2	1
28	44	45	2	1
29	49	50	2	1
30	50	51	2	1
31	52	53	2	1
32	53	54	2	1
33	54	55	2	1
34	56	57	2	1
35	57	58	2	1
36	58	59	2	1
37	60	61	2	1
38	61	62	2	1
39	62	63	2	1
40	18	13	2	1

TABLE 4-7 (Continued)

BEAM ELEMENT DATA

(Material: 1 = Concrete, 2 = Steel)

Beam Number	Node -I	Node -J	Material Number	Section Number
41	47	49	2	27
42	64	65	2	1
43	18	19	2	1
44	19	20	2	2
45	20	25	2	3
46	25	26	2	4
47	26	33	2	4
48	33	34	2	4
49	34	35	2	4
50	35	40	2	4
51	40	46	2	4
52	46	47	2	4
53	47	48	2	5
54	2	4	2	6
55	4	8	2	6
56	8	13	2	6
57	13	21	2	6
58	21	27	2	6
59	27	36	2	6
60	36	41	2	6
61	41	49	2	6
62	49	52	2	6
63	52	56	2	6
64	56	60	2	6
65	60	64	2	6
66	64	66	2	7
67	66	67	2	8
68	67	68	2	9
69	68	69	2	10
70	69	70	2	11
71	70	73	2	12
72	74	71	2	13
73	71	72	2	14
74	72	70	2	15
75	2	4	1	16
76	7	10	1	17
77	12	17	1	18
78	17	24	1	18
79	24	32	1	18
80	31	39	1	19

TABLE 4-7 (Continued)

BEAM ELEMENT DATA

(Material: 1 = Concrete, 2 = Steel)

Beam Number	Node -I	Node -J	Material Number	Section Number
81	39	45	1	19
82	44	51	1	20
83	51	55	1	20
84	55	59	1	20
85	59	63	1	20
86	60	64	1	21
87	64	66	1	21
88	3	6	1	22
89	6	9	1	22
90	11	16	1	23
91	14	22	1	24
92	22	28	1	24
93	29	38	1	25
94	38	43	1	25
95	42	50	1	26
96	50	53	1	26
97	53	57	1	26
98	57	62	1	26

TABLE 4-8

MODAL DAMPINGS FOR RESPONSE SPECTRA ANALYSIS

Mode	Modal Dampings		
	Upper Bound G	Best Estimate G	Lower Bound G
1	0.11	0.11	0.13
2	0.11	0.11	0.13
3	0.03	0.03	0.20
4	0.20	0.20	0.03
5	0.20	0.20	0.20
6	0.20	0.20	0.20
7	0.20	0.20	0.20
8	0.06	0.05	0.04
9	0.06	0.05	0.04
10	0.03	0.03	0.03
11	0.10	0.09	0.07
12	0.04	0.04	0.04
13	0.04	0.04	0.04
14	0.03	0.03	0.03
15	0.04	0.04	0.04
16	0.03	0.04	0.04
17	0.03	0.03	0.03
18	0.04	0.04	0.04
19	0.03	0.03	0.03
20	0.03	0.03	0.03

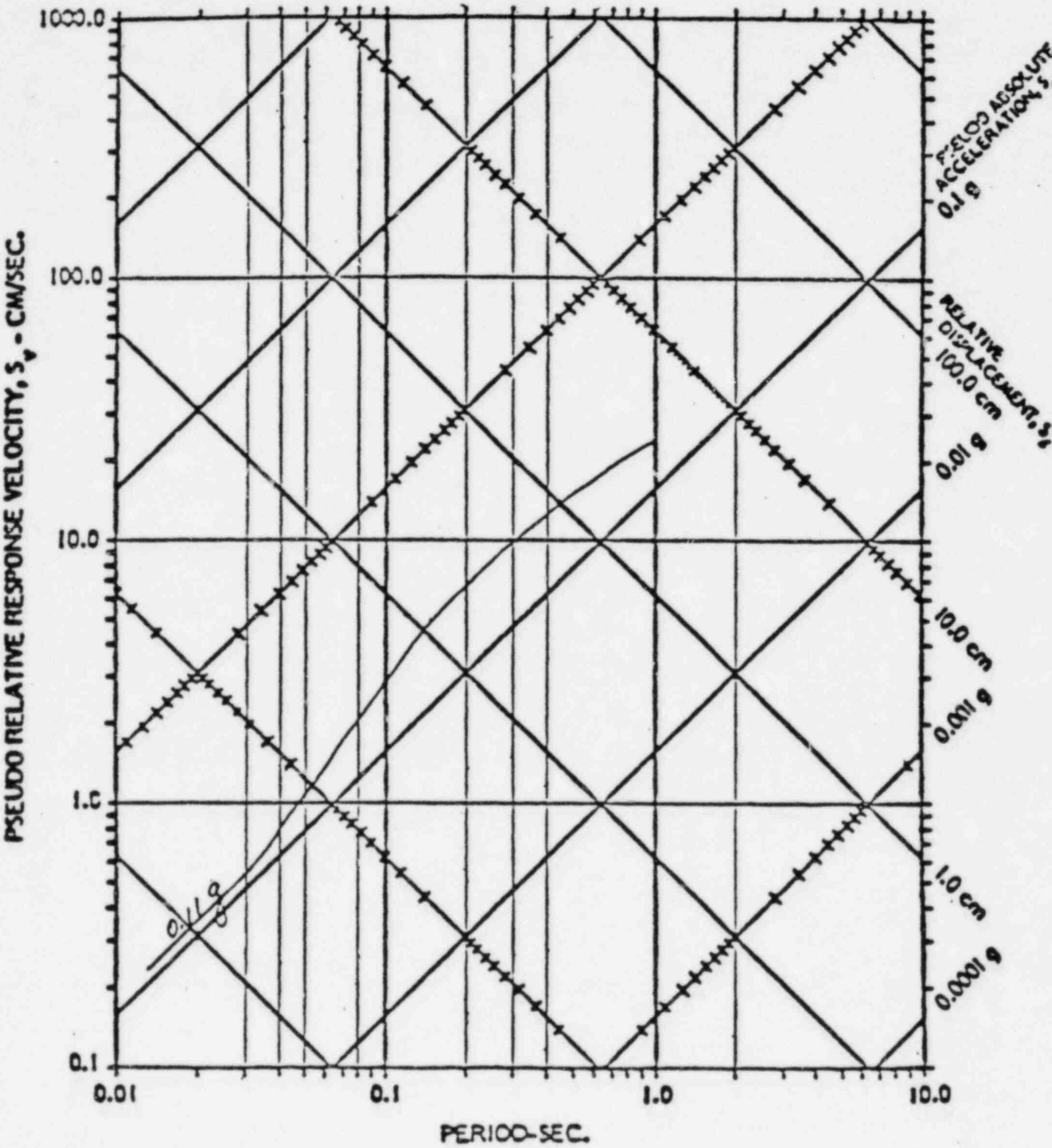

FIGURE 4-1: LACBWR SITE SPECIFIC GROUND RESPONSE SPECTRUM (5% DAMPING)

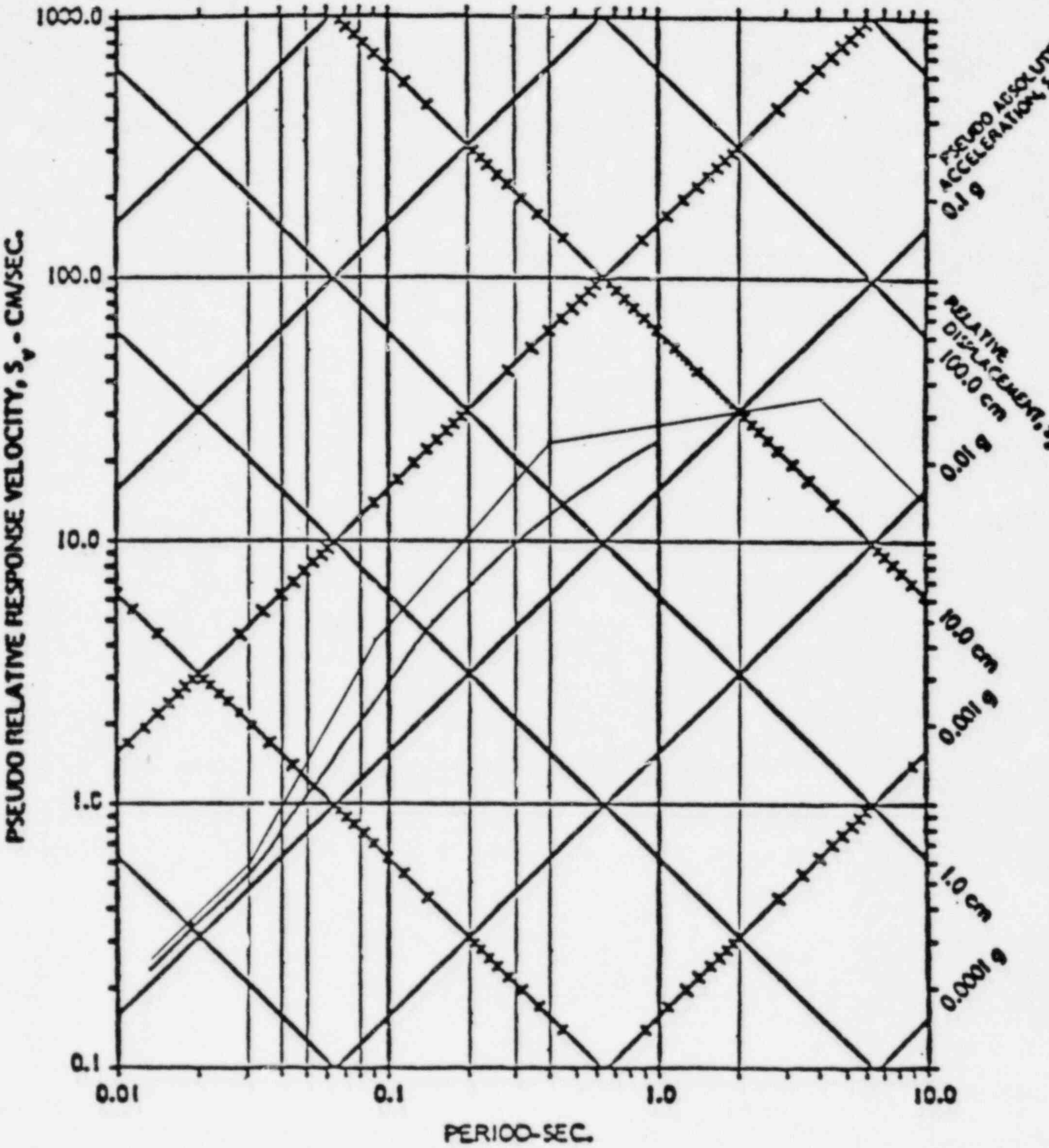

FIGURE 4-2: COMPARISON OF LLNL SITE SPECIFIC GROUND RESPONSE SPECTRUM WITH ORIGINAL ANALYSIS SPECTRUM (5% DAMPING)

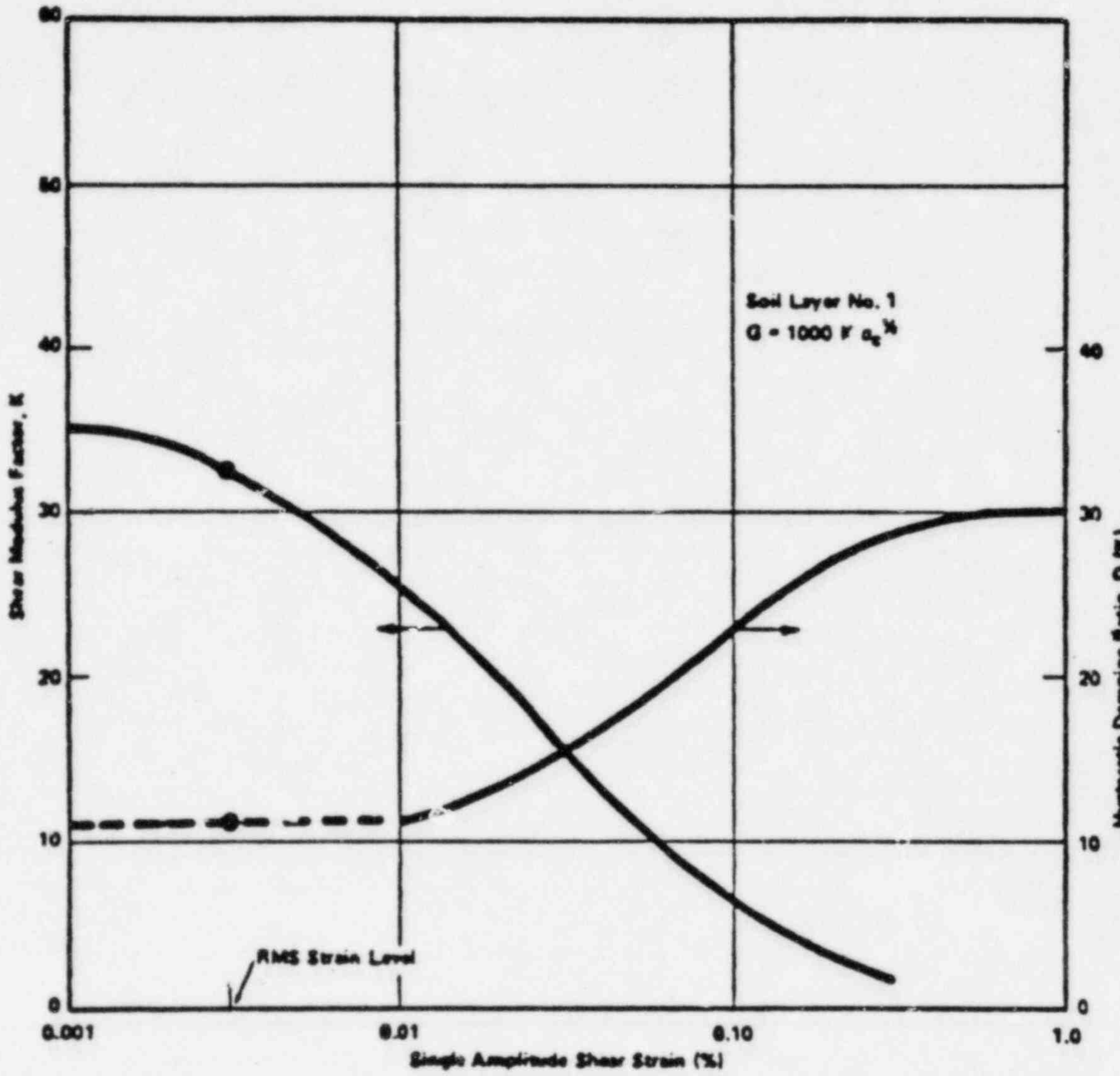

FIGURE 4-3: SOIL PROPERTIES FOR HYDRAULIC FILL

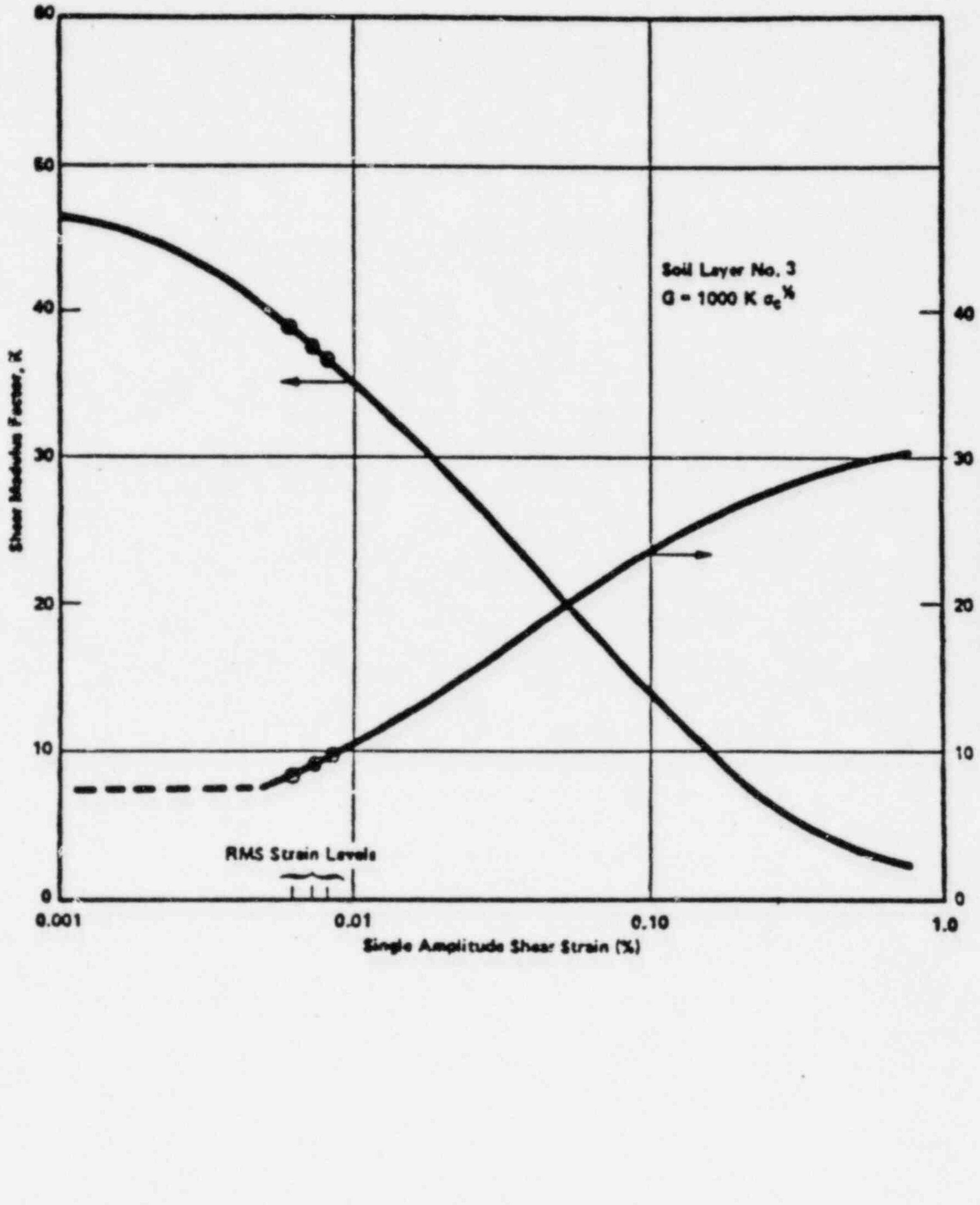

FIGURE 4-4: SOIL PROPERTIES FOR PILE FOUNDATION REGION

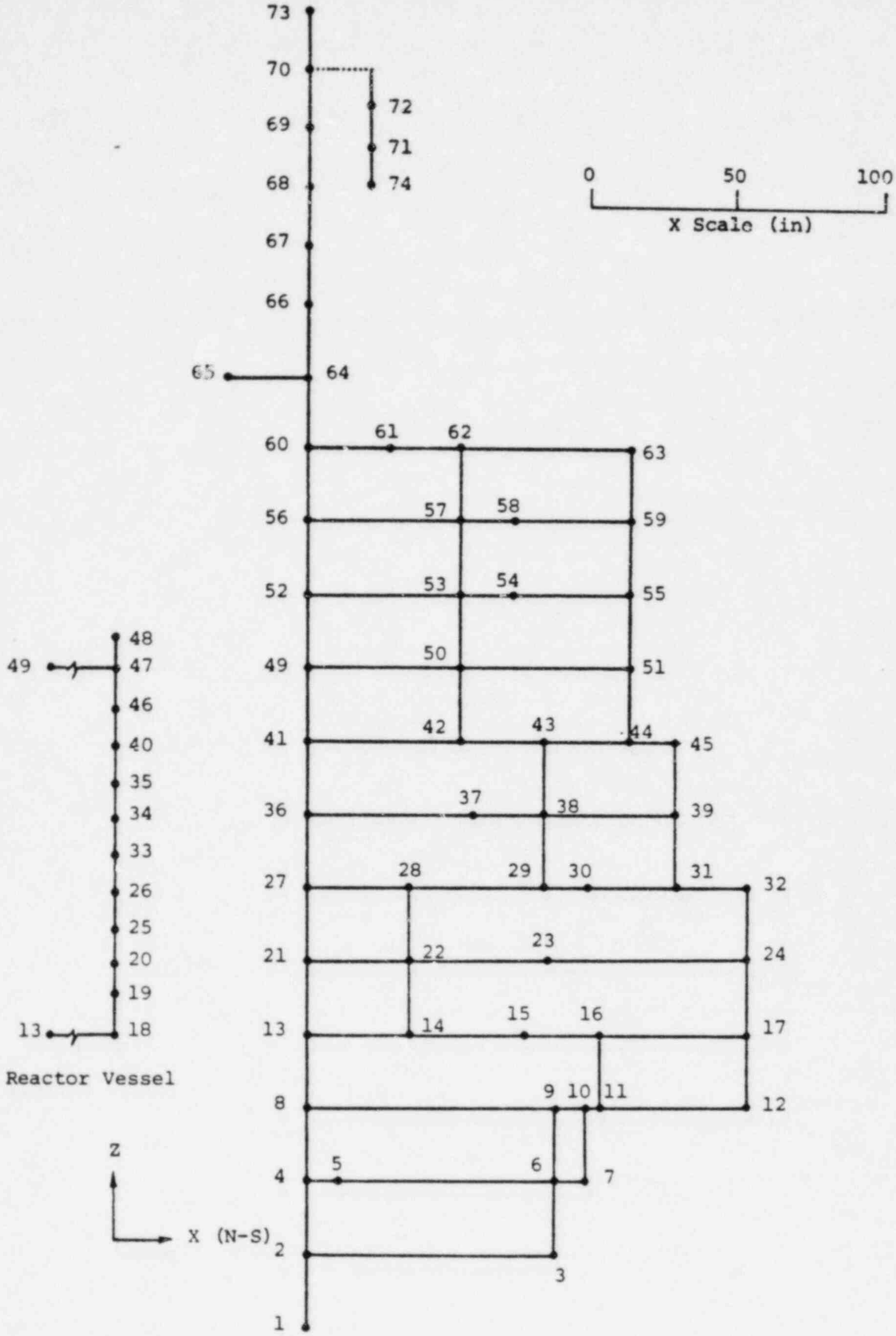

FIGURE 4-5. NODAL CONNECTIVITY OF STRUCTURES MODEL

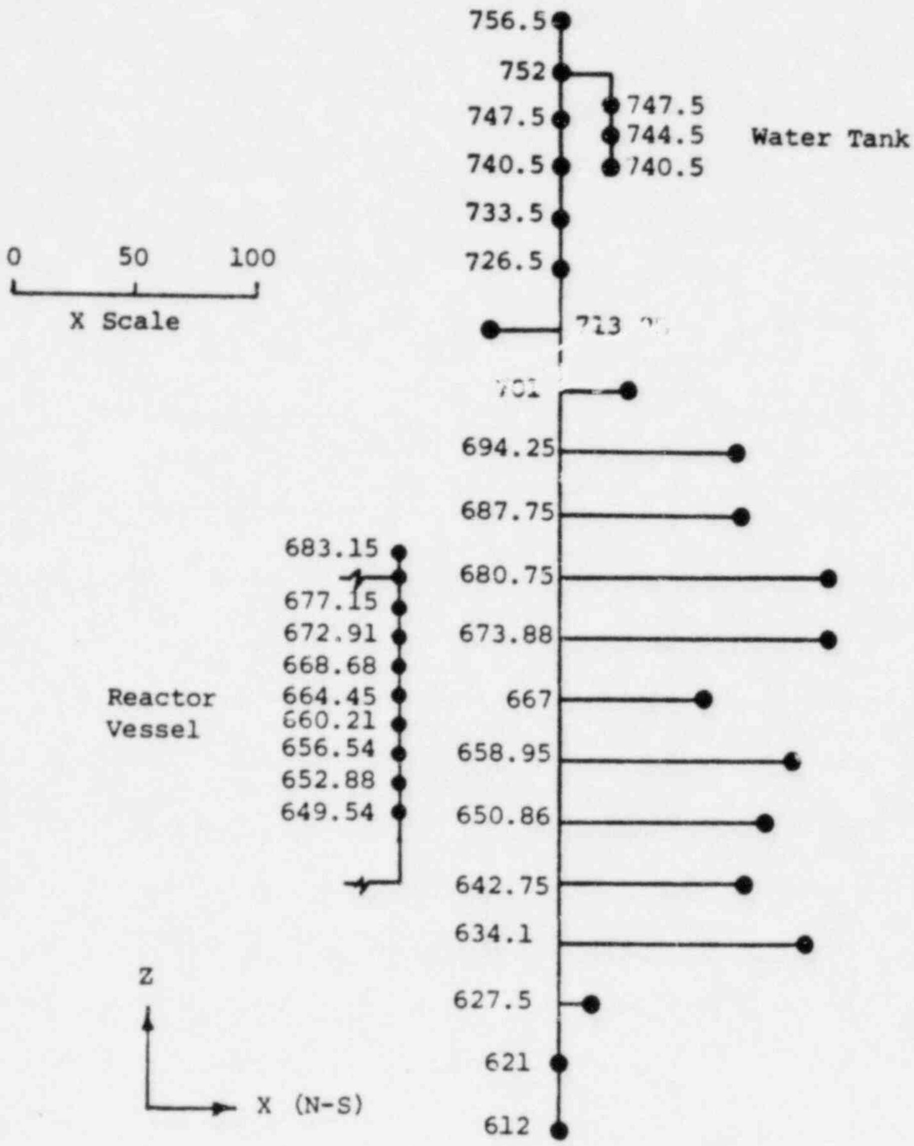

FIGURE 4-6. MASS LOCATIONS AND THEIR RESPECTIVE ELEVATIONS (FT)

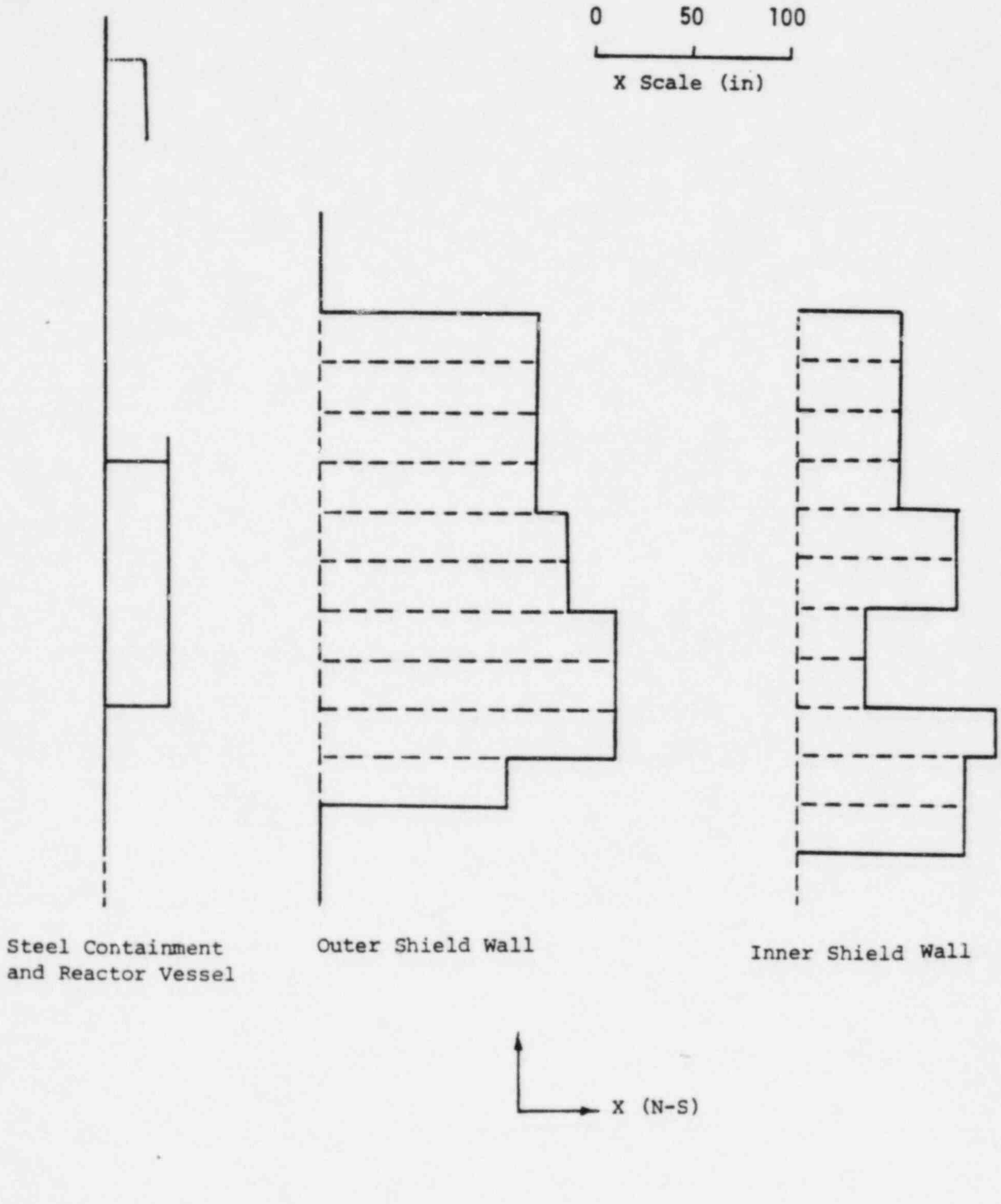

FIGURE 4-7. BEAM STICK MODELS OF LA CROSSE REACTOR BUILDING

5. SEISMIC RESPONSE

5.1 STRUCTURE RESPONSE

Natural frequencies and mode shapes of the La Crosse structural model described in Chapter 4 were found using a modified version of the computer program SAP IV (Reference 12). Tables 5-1, 5-2, and 5-3 list the first 20 structural frequencies and describe the dominant modes of vibration for each soil case. A response spectrum approach was used to calculate the response of the structure to the site specific spectra for various modal frequencies and damping values. Vertical response was found by scaling the site specific horizontal spectra by 2/3. A modified square-root-of-the-sum-of-the-squares (SRSS) approach was used to combine the modal response. In this approach, all modes having closely spaced frequencies were assembled into groups, and the response of each group was found by taking the sum of the absolute responses. The total response is then obtained by taking the square-root-of-the-sum-of-the-squares of the response for each group and the remaining modes.

Figures 5-1 through 5-24 show shear diagrams for the outer shield structure, steel containment, the inner shield structure, and the total shear diagram for all three soil cases. Moment diagrams for these cases are shown in Figures 5-25 through 5-42. A comparison of peak seismic shear and moment with the La Crosse FSAR seismic analysis performed by Gulf United is presented in Tables 5-4 and 5-5. In the Gulf United model, the outer shield concrete and the steel containment were modeled together with one stick below elevation 726'-6". Steel reinforcement details in concrete were not available for the SEP review. As a result, values of yield moment, ultimate moment and ultimate shear capacities were assumed to be those calculated in the original La Crosse seismic analysis. Based on this assumption, Tables 5-4 and 5-5 show that the peak seismic shears and moments in the foundation, the inner shield wall and the outer shield wall are all below their corresponding ultimate shears and moments. In fact, the present analysis indicates that all computed moments are below yield moments.

In order to find the peak moment and axial load acting on an individual pile, maximum base shear, base moment, and base torsional moment from the response spectra analysis were applied on the foundation. A rocking moment on the foundation induces axial forces on the piles which are combined with the dead weight loads. Total dead load on the piles is equal to weight of the containment minus weight of the displaced soil. Dead load was assumed to be uniformly distributed among the piles. Peak axial loads on any individual pile were found to be 9.9 kips in tension and 105.9 kips in compression. Torsion and shear on the foundation induce shear loads in the piles. Assuming that the piles are fixed in the cap, moment in a pile may be computed from the stiffness coupling terms between horizontal translation and rocking. Peak moment on a pile was calculated to be 241 kip-in. Since details of the reinforcement in the piles were not available, no determination of the axial and moment capacities of piles could be made in the present study.

The minimum design capacity of the piles was specified as 50 tons per pile or 100 kips. It is expected that the maximum additional 5.9 kips above the minimum design load can be accepted by the pile with no distress since piles of this type are expected to have a substantial strength margin beyond the axial design value. The details of the pile caps are unknown. Some tension capacity probably exists. If this is exceeded, some relative motion between the pile caps and the foundation slab may occur for the piles in locations of maximum tension. This would introduce a small amount of nonlinearity into the seismic response, as well as a slight shift in the pile foundation neutral axis, and a slight increase in the maximum compressive loads in the piles on the opposite side of the foundation. None of these effects are expected to result in significant variation in the seismic response of the structure or in significant structural damage for the 0.11g earthquake. It is recommended that the ability of the pile to withstand the bending moment be verified if the details of the pile reinforcing can be determined.

5.2 VESSEL STRESS ANALYSIS

Stresses in the steel containment vessel were calculated from shear and moment diagrams presented in Section 5.1. Peak longitudinal membrane stress due to seismic loads was calculated to be 1.27 ksi which occurs near the base of the vessel. Including the longitudinal compressive stress due to dead weight, total stress was found to be 1.67 ksi. This is less than the code allowable buckling stress of 6.3 ksi, which was computed by evaluating the classical linear design buckling stress of a cylinder under axial loads (Reference 14), and then multiplying the computed stress by 1/3 according to the ASME code (Reference 15). Also, peak shear stress in the steel containment was computed to be 0.55 ksi. Since other loads such as pressure and temperature loads which also contribute to shear stresses in the containment were not considered here, peak seismic shear stress alone was not compared with the code allowable shear stress. However, substantial margins of safety exist for the containment vessel subjected to the 0.11g site specific earthquake.

TABLE 5-1

MODAL FREQUENCIES FOR UPPER BOUND ESTIMATE OF SOIL SHEAR MODULUS

Mode Number	Frequency (Hz)	Vibration Mode
1	1.87	Rocking, Y
2	1.88	Rocking, X
3	3.89	Reactor vessel
4	5.83	Torsion
5	6.84	Vertical Translation
6	8.44	Horizontal Translation, Y
7	8.46	Horizontal Translation, X
8	17.0	Structure plus soil
9	17.4	Structure plus soil
10	20.2	Structure plus soil
11	24.6	Structure plus soil
12	26.0	Structure plus soil
13	26.8	Structure plus soil
14	29.8	Structure plus soil
15	34.1	Structure plus soil
16	37.0	Structure plus soil
17	37.7	Structure plus soil
18	39.3	Structure plus soil
19	40.7	Structure plus soil
20	44.7	Structure plus soil

TABLE 5-2

MODAL FREQUENCIES FOR BEST ESTIMATE OF SOIL SHEAR MODULUS

Mode Number	Frequency (Hz)	Vibration Mode
1	1.59	Rocking, Y
2	1.60	Rocking, X
3	3.89	Reactor Vessel
4	5.20	Torsion
5	5.80	Vertical Translation
6	7.58	Horizontal Translation, Y
7	7.58	Horizontal Translation, X
8	16.7	Structure plus soil
9	17.1	Structure plus soil
10	20.2	Structure plus soil
11	24.4	Structure plus soil
12	25.8	Structure plus soil
13	26.6	Structure plus soil
14	29.8	Structure plus soil
15	34.0	Structure plus soil
16	36.8	Structure plus soil
17	37.6	Structure plus soil
18	39.2	Structure plus soil
19	40.6	Structure plus soil
20	44.6	Structure plus soil

TABLE 5-3

MODAL FREQUENCIES FOR LOWER BOUND ESTIMATE OF SOIL SHEAR MODULUS

Mode Number	Frequency (Hz)	Vibration Mode
1	1.27	Rocking, Y
2	1.27	Rocking, X
3	3.70	Torsion
4	3.88	Reactor vessel
5	4.72	Vertical Translation
6	5.58	Horizontal Translation, Y
7	5.59	Horizontal Translation, X
8	16.2	Structure plus soil
9	16.7	Structure plus soil
10	20.2	Structure plus soil
11	24.0	Structure plus soil
12	25.5	Structure plus soil
13	26.4	Structure plus soil
14	29.7	Structure plus soil
15	33.8	Structure plus soil
16	36.6	Structure plus soil
17	37.6	Structure plus soil
18	39.0	Structure plus soil
19	40.5	Structure plus soil
20	44.6	Structure plus soil

TABLE 5-4

COMPARISON OF PEAK SEISMIC SHEARS WITH FSAR SEISMIC ANALYSIS

Location	Gulf United		SEP	
	V_{max} (kips)	V_{max}/V_u	V_{max}(kips)	V_{max}/V_u
Foundation	3530	0.092	1946	0.051
Outer Shield Wall	1480	0.366	1063	0.263
Inner Shield Wall	2060	0.176	1067	0.091

TABLE 5-5

COMPARISON OF PEAK SEISMIC MOMENTS WITH FSAR SEISMIC ANALYSIS

Location	GULF UNITED			SEP		
	M_{max}(kip-in)	M_{max}/M_y	M_{max}/M_u	M_{max} (kip-in)	M_{max}/M_y	M_{max}/u
Foundation	6.7×10^5	0.135		1.6×10^6	0.322	
Outer Shield Moment	1.4×10^6	1.008	0.694	4.1×10^5	0.295	0.203
Inner Shield Moment	8.2×10^5	0.558		1.8×10^5	0.122	

FIGURE 5-1: SHEAR IN THE OUTER SHIELD WALL IN X DIRECTION USING BEST ESTIMATE OF SOIL SHEAR MODULUS

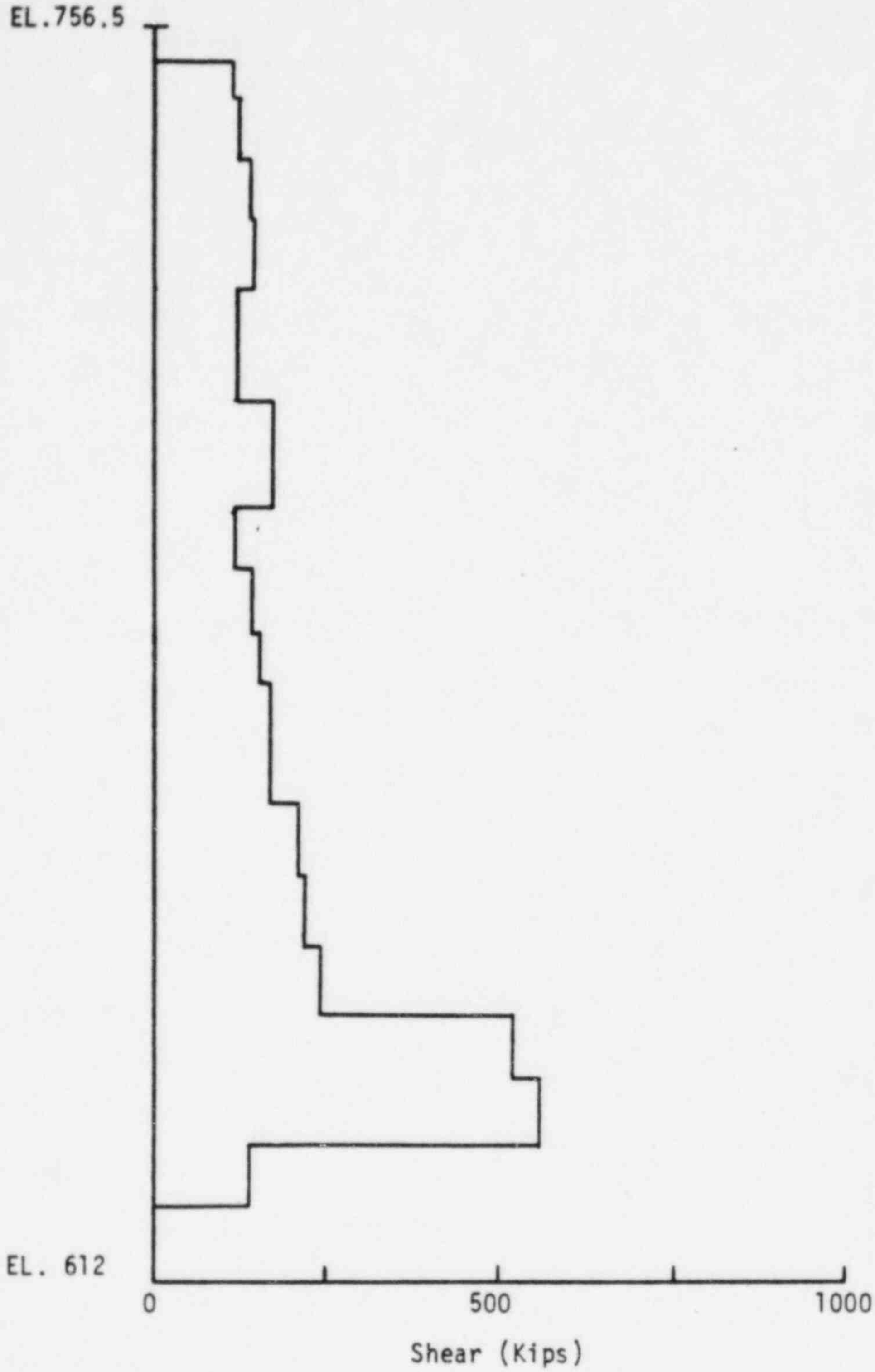

FIGURE 5-2: SHEAR IN STEEL CONTAINMENT IN X DIRECTION
USING BEST ESTIMATE OF SOIL SHEAR MODULUS

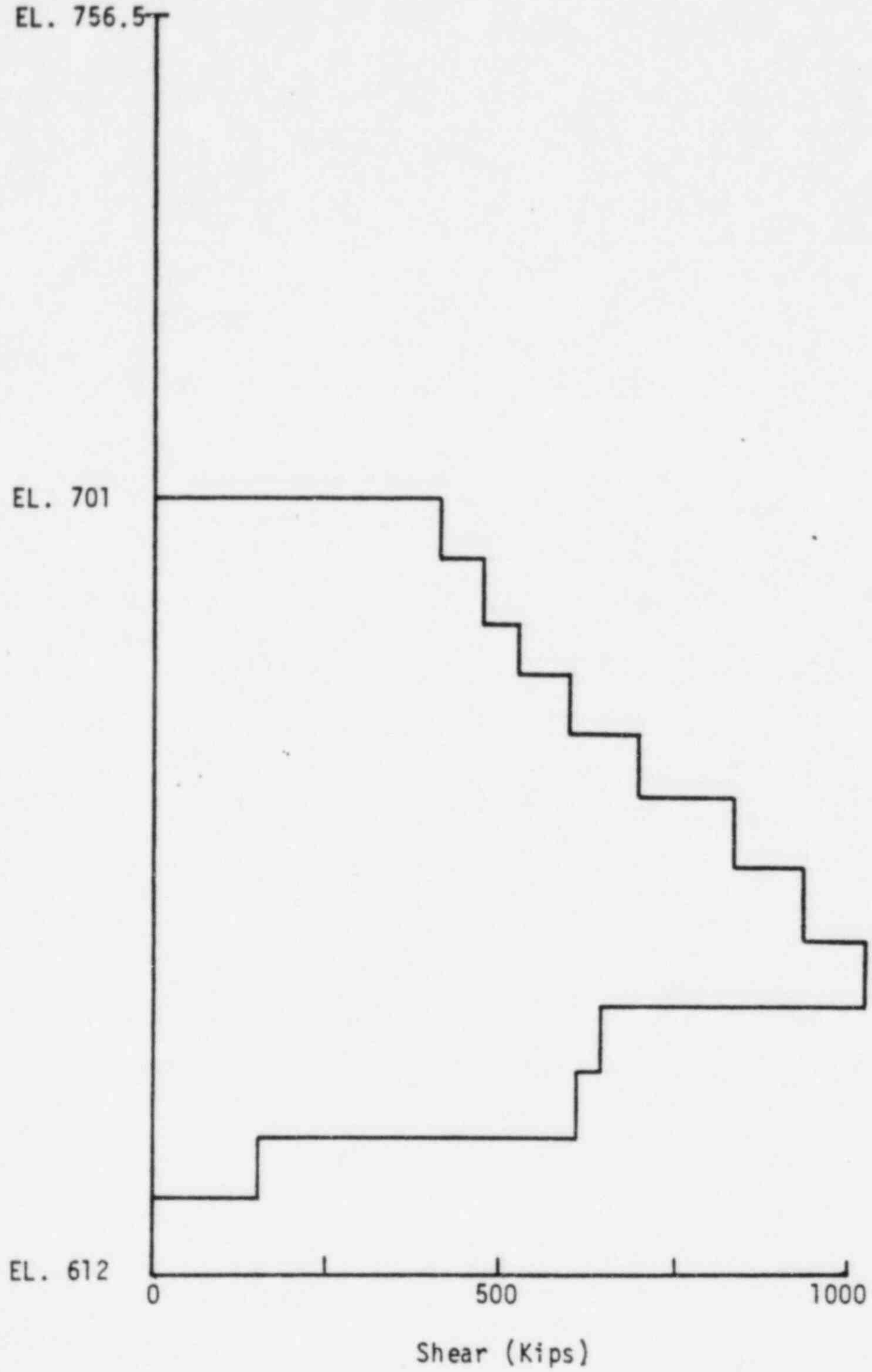

FIGURE 5-3: SHEAR IN THE INNER SHIELD STRUCTURE IN X DIRECTION USING BEST ESTIMATE OF SOIL SHEAR MODULUS

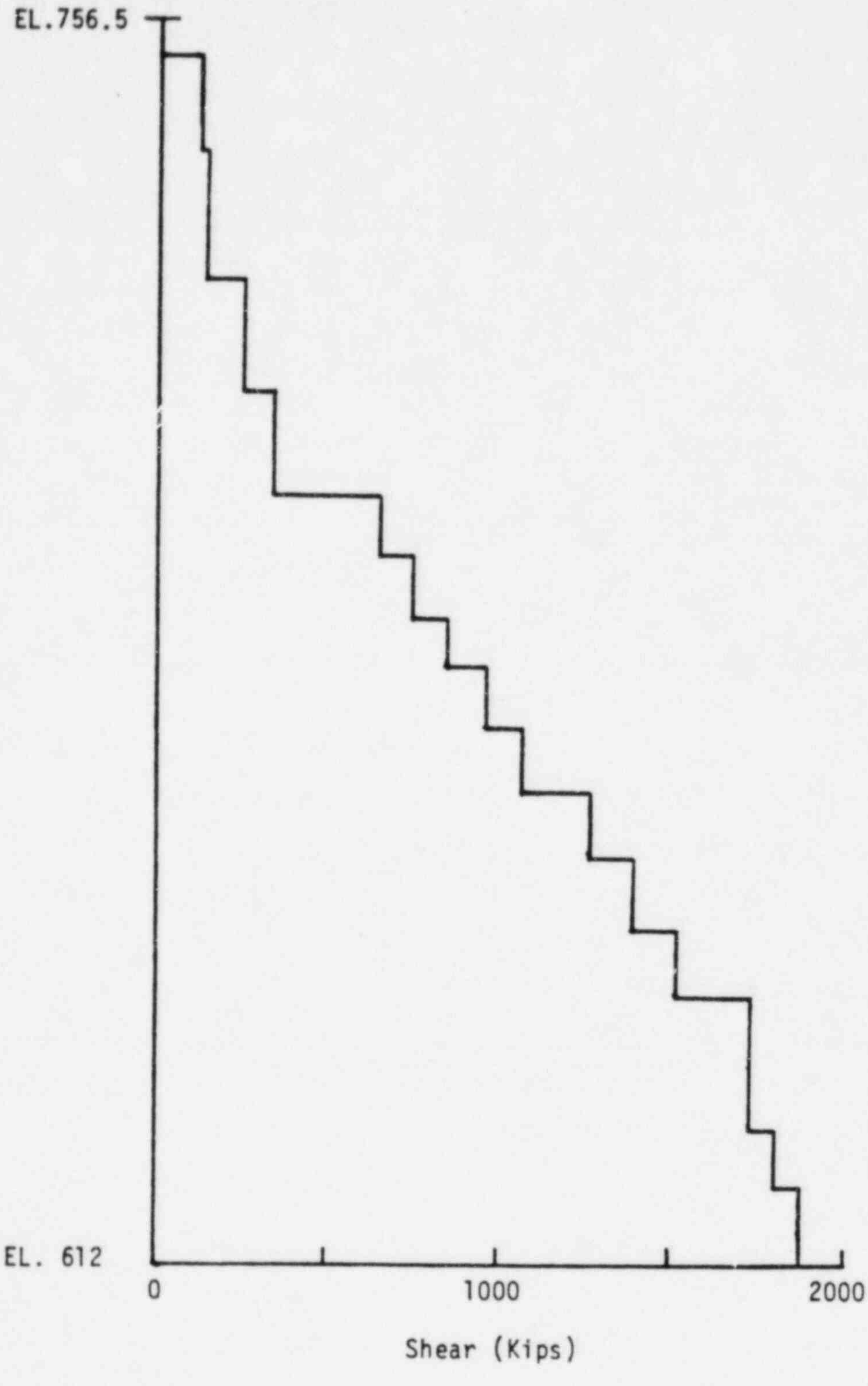

FIGURE 5-4: TOTAL SHEAR IN X DIRECTION USING BEST ESTIMATE OF SOIL SHEAR MODULUS

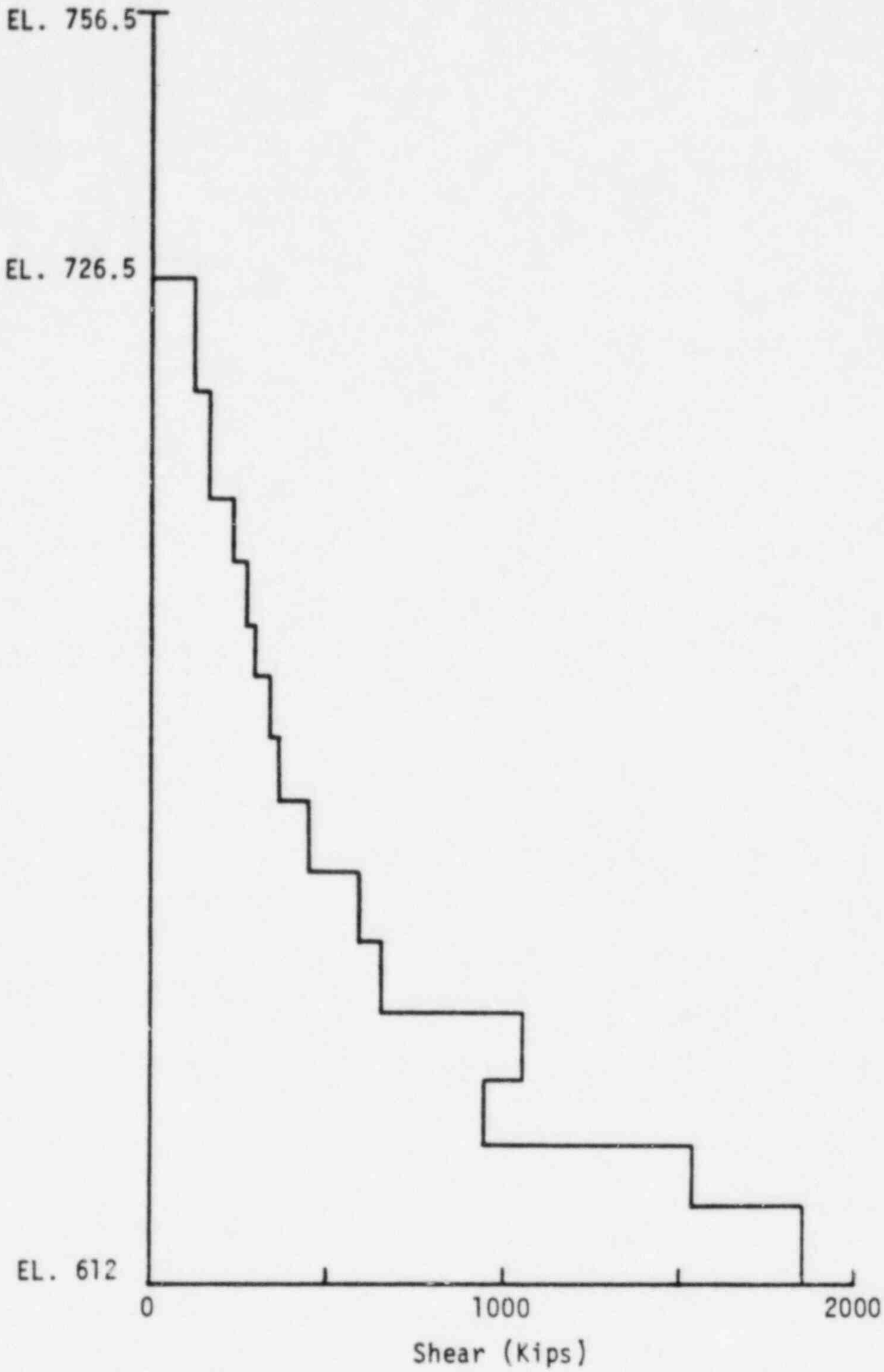

FIGURE 5-5: SHEAR IN THE OUTER SHIELD WALL IN Y DIRECTION USING BEST ESTIMATE OF SOIL SHEAR MODULUS

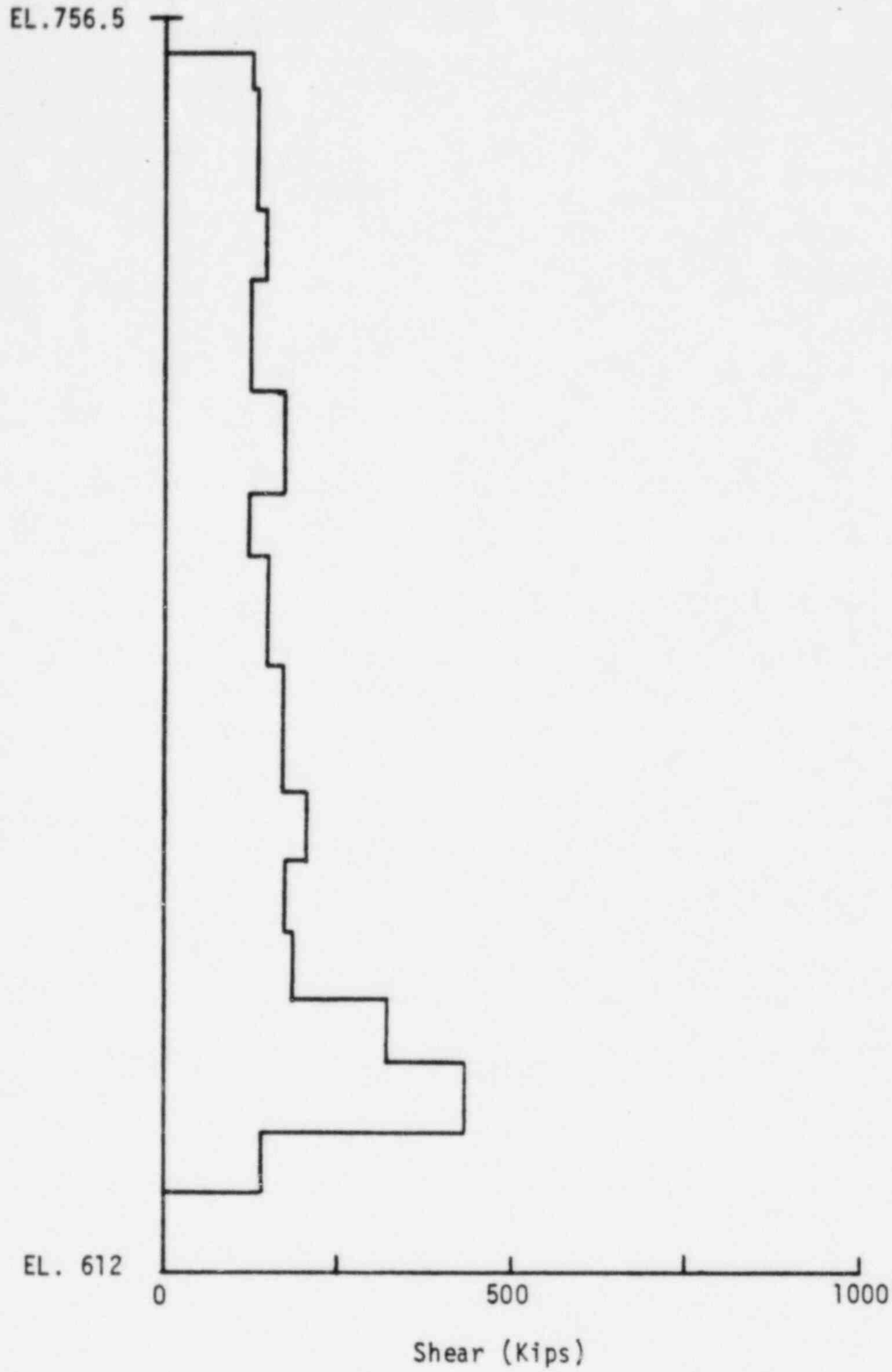

FIGURE 5-6: SHEAR IN STEEL CONTAINMENT IN Y DIRECTION USING BEST ESTIMATE OF SOIL SHEAR MODULUS

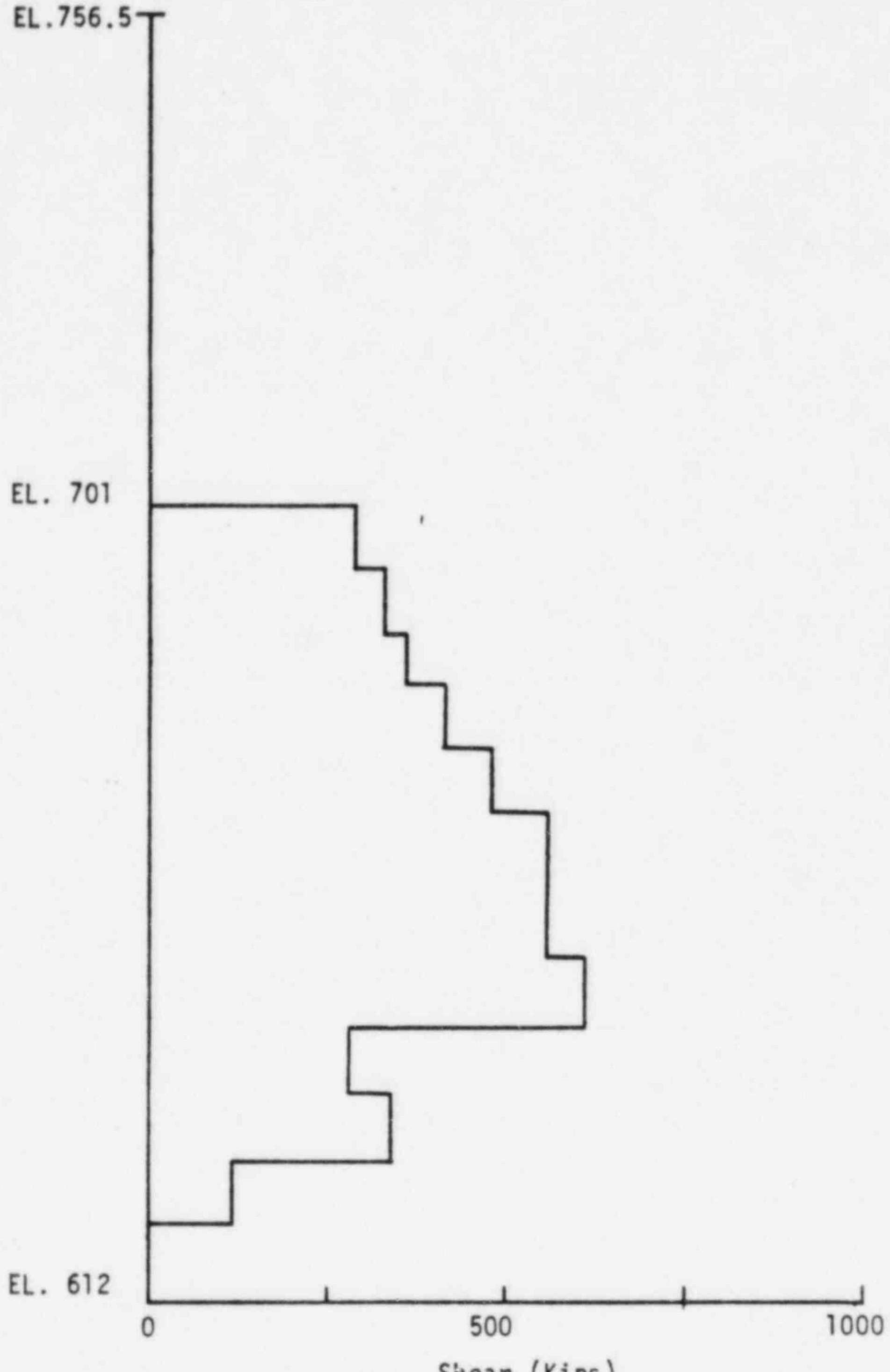

FIGURE 5-7: SHEAR IN THE INNER SHIELD STRUCTURE IN Y DIRECTION USING BEST ESTIMATE OF SOIL SHEAR MODULUS

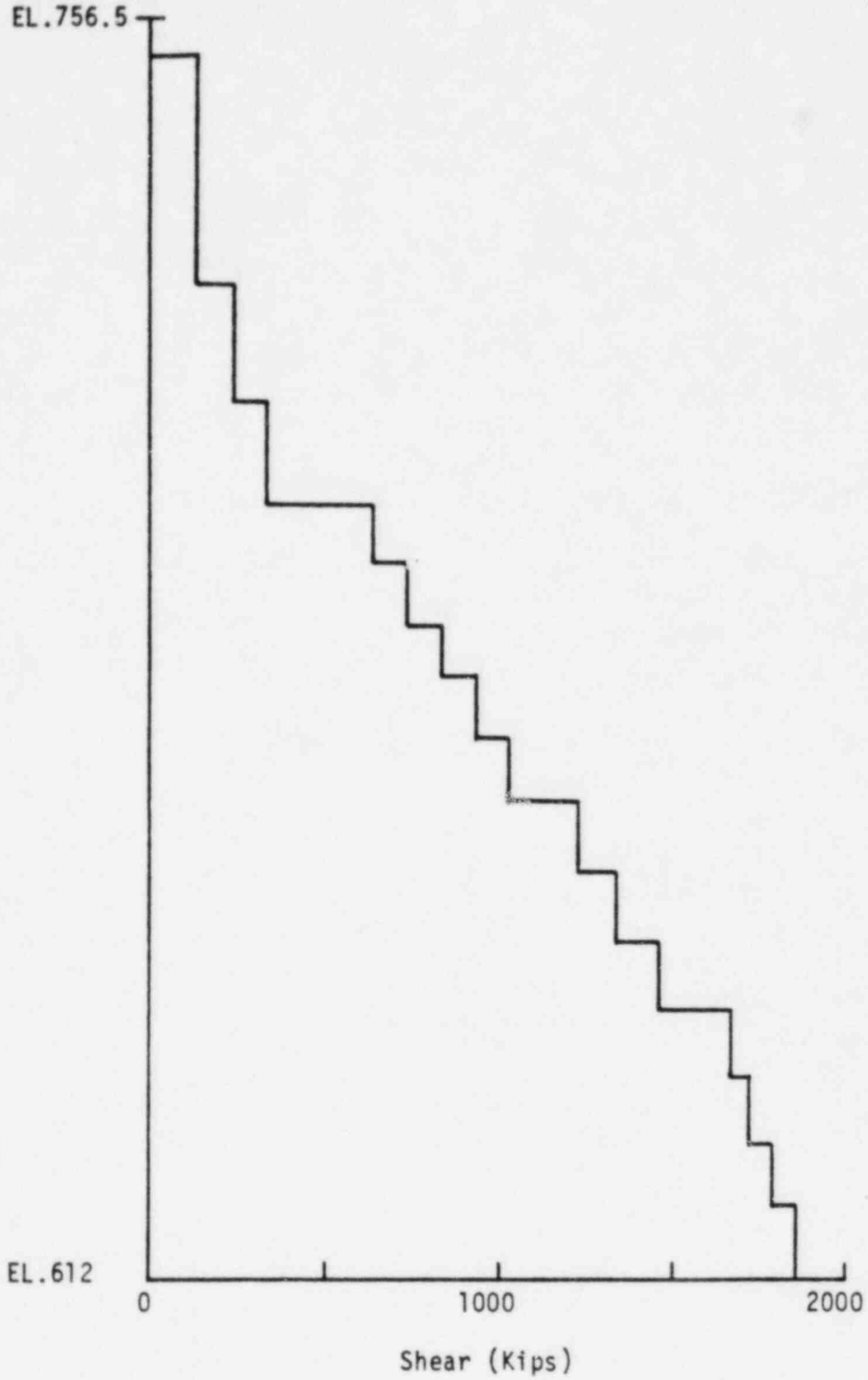

FIGURE 5-8: TOTAL SHEAR IN Y DIRECTION USING BEST ESTIMATE OF SOIL SHEAR MODULUS

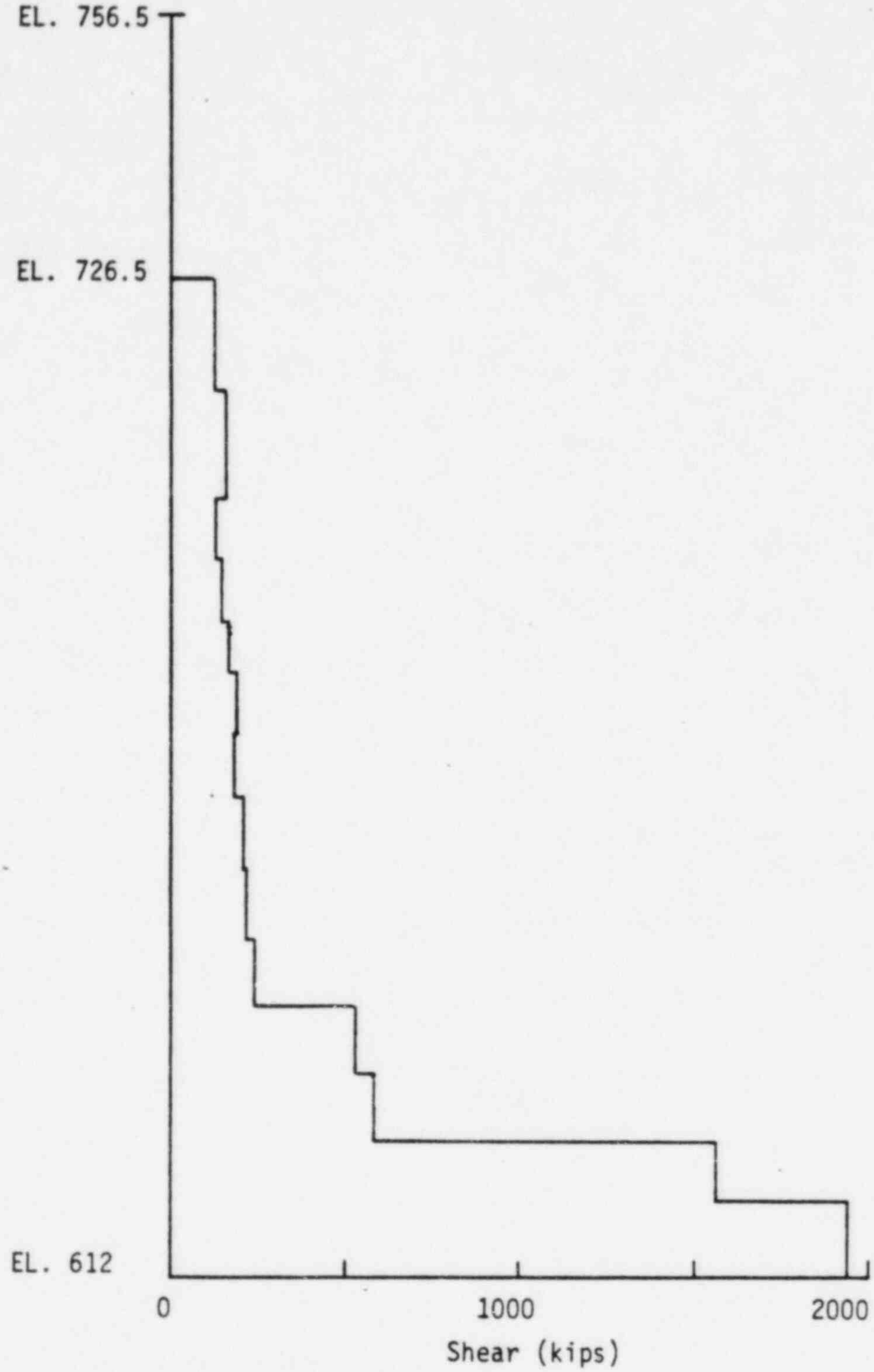

FIGURE 5-9. SHEAR IN OUTER SHIELD WALL IN X DIRECTION USING UPPER BOUND ESTIMATE OF SOIL SHEAR MODULUS

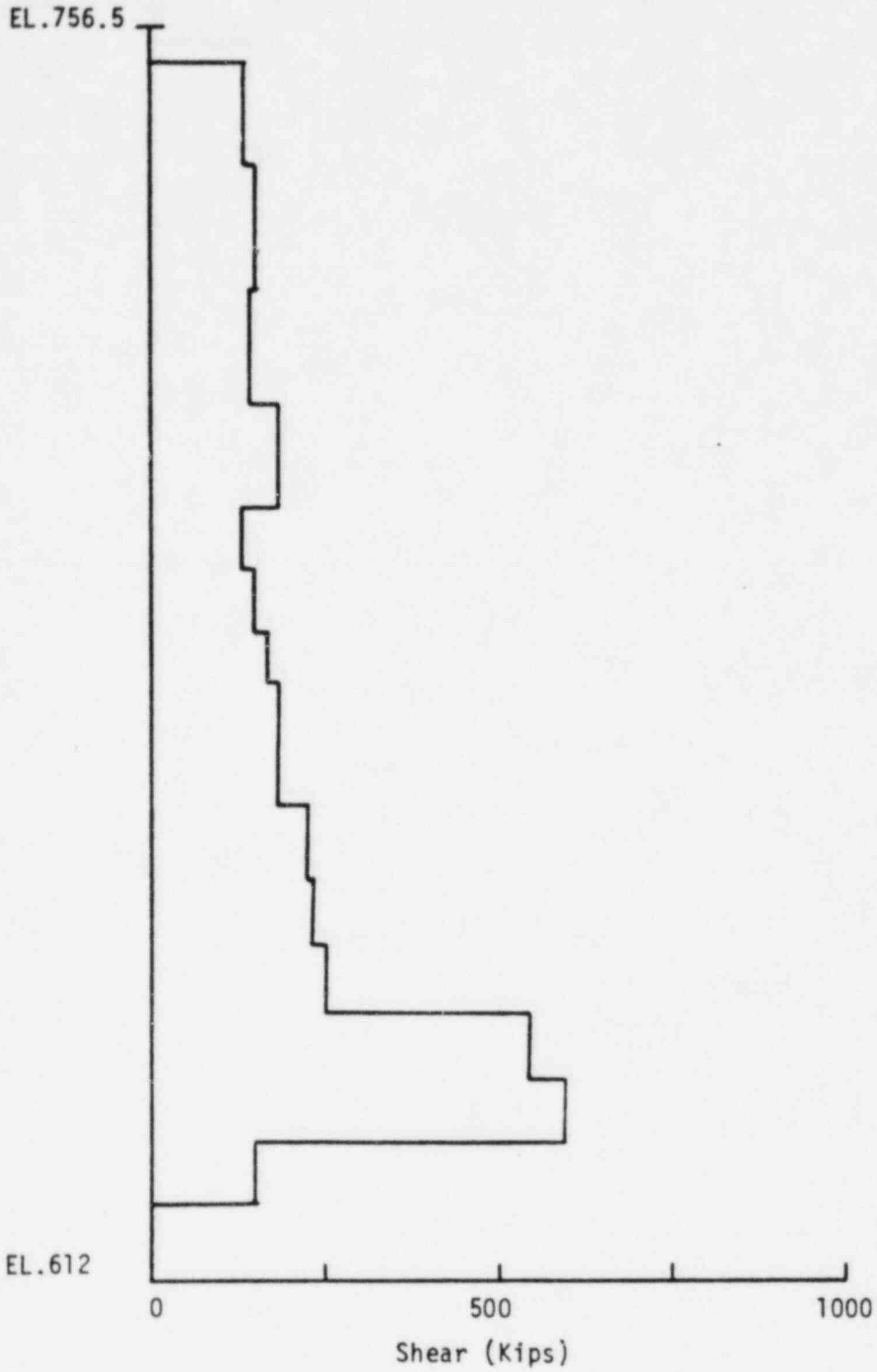

FIGURE 5-10: SHEAR IN STEEL CONTAINMENT IN X DIRECTION
USING UPPER BOUND ESTIMATE OF SOIL SHEAR MODULUS

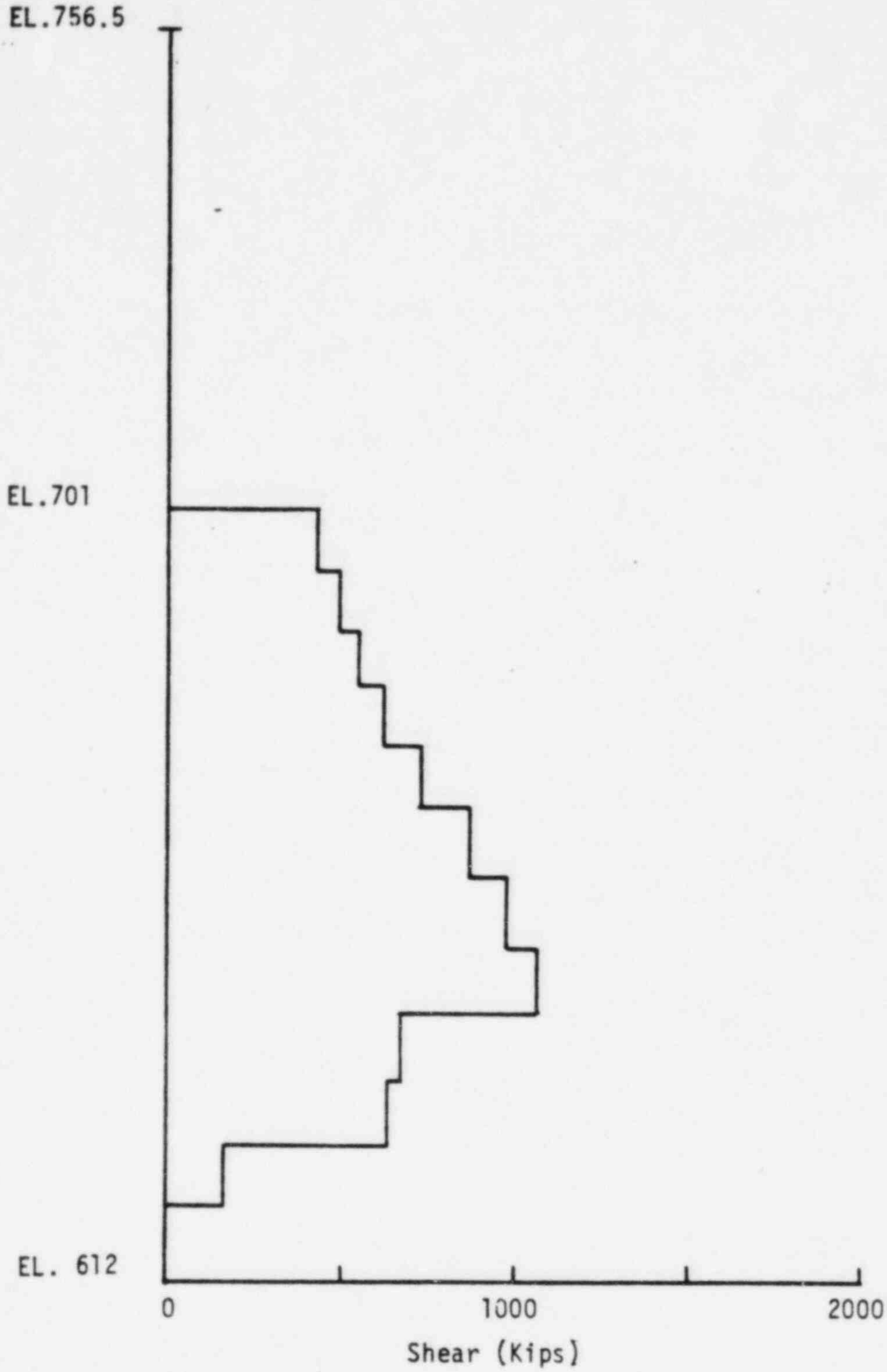

FIGURE 5-11. SHEAR IN THE INNER SHIELD STRUCTURE IN X DIRECTION USING UPPER BOUND ESTIMATE OF SOIL SHEAR MODULUS

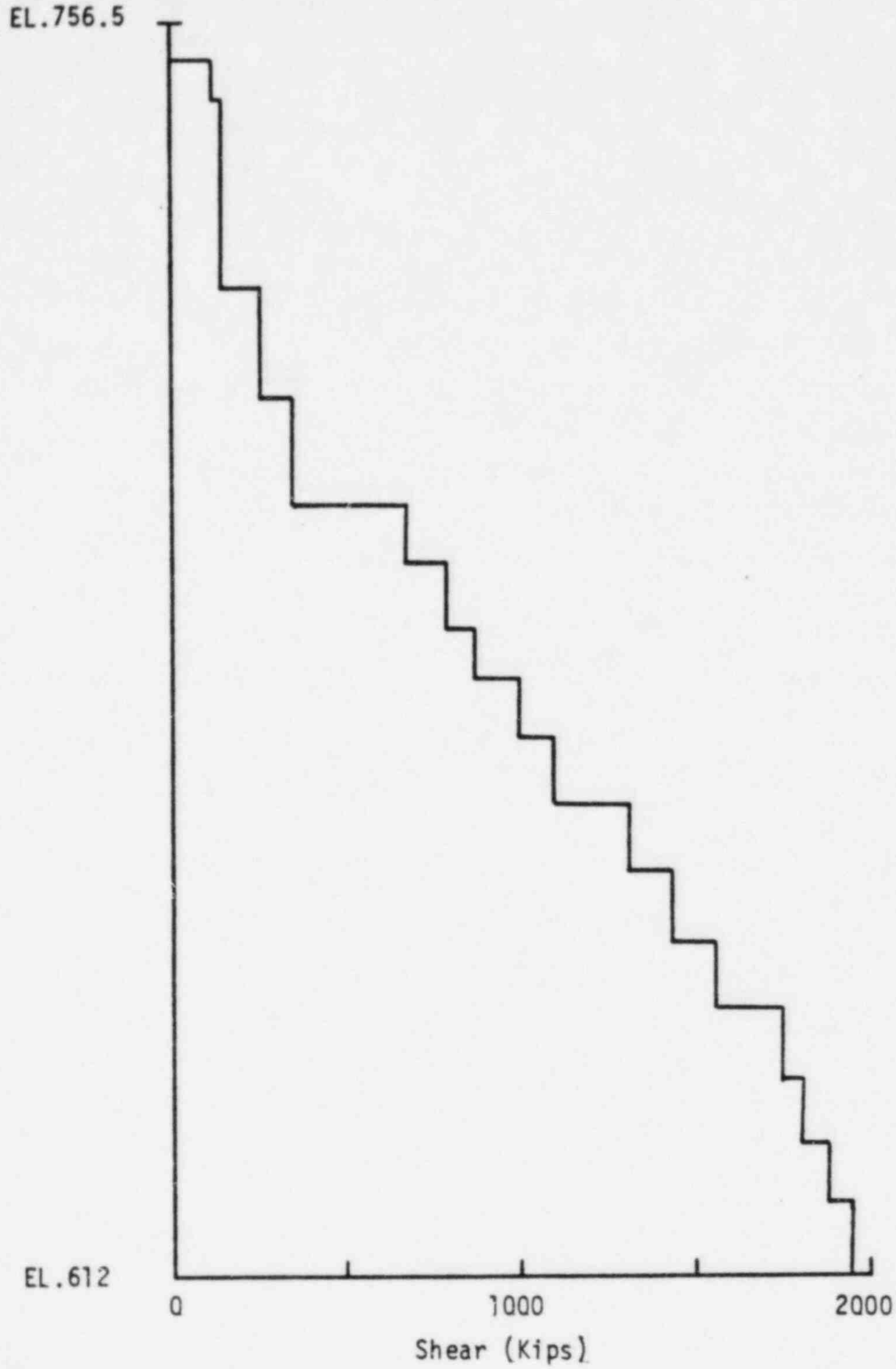

FIGURE 5-12: TOTAL SHEAR IN X DIRECTION USING UPPER BOUND ESTIMATE OF SOIL SHEAR MODULUS

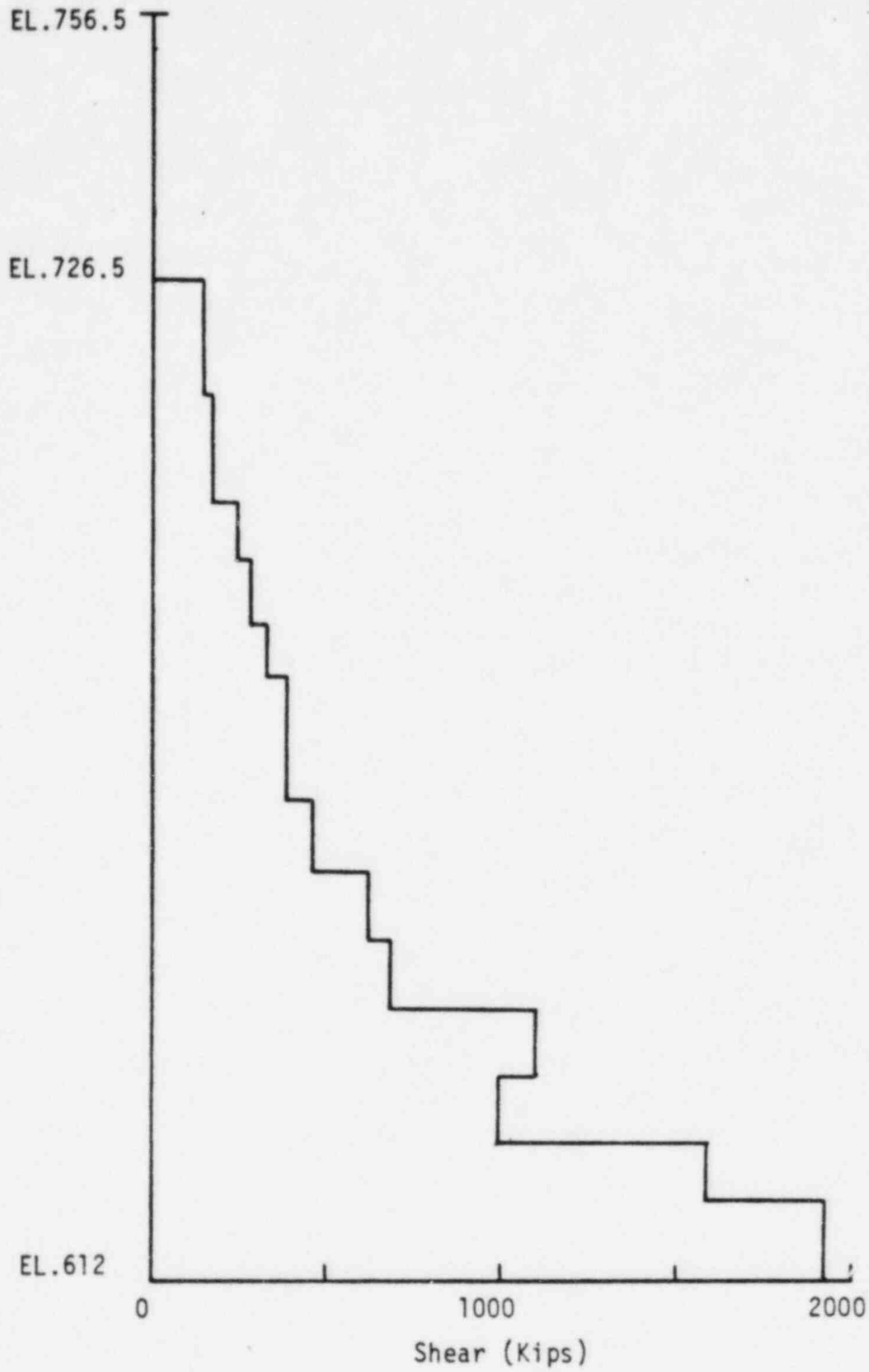

FIGURE 5-13: SHEAR IN THE OUTER SHIELD WALL IN Y DIRECTION USING UPPER BOUND ESTIMATE OF SOIL SHEAR MODULUS

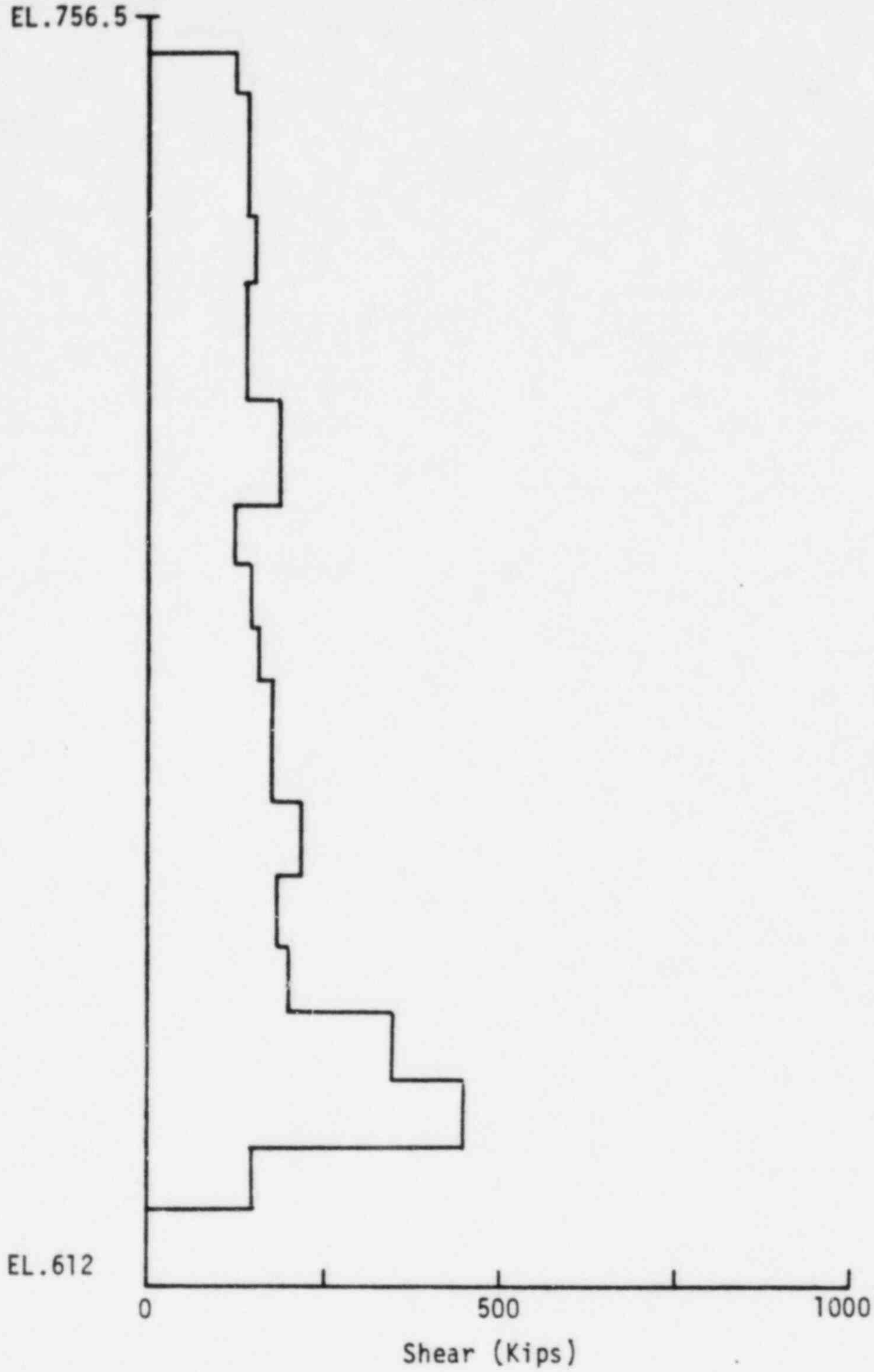

FIGURE 5-14: SHEAR IN STEEL CONTAINMENT IN Y DIRECTION
USING UPPER BOUND ESTIMATE OF SOIL SHEAR MODULUS

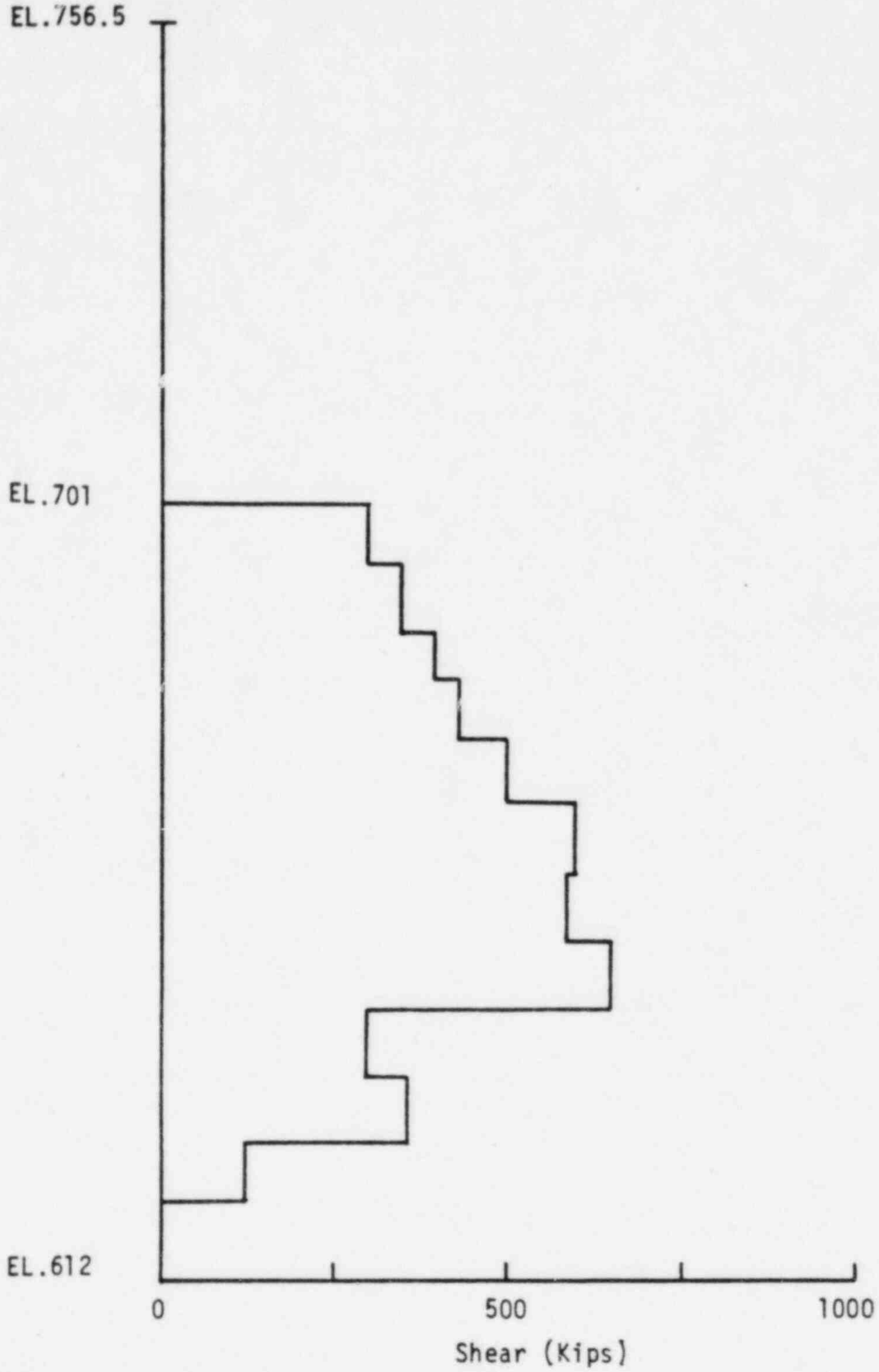

FIGURE 5-15: SHEAR IN THE INNER SHIELD STRUCTURE IN Y DIRECTION USING UPPER BOUND ESTIMATE OF SOIL SHEAR MODULUS

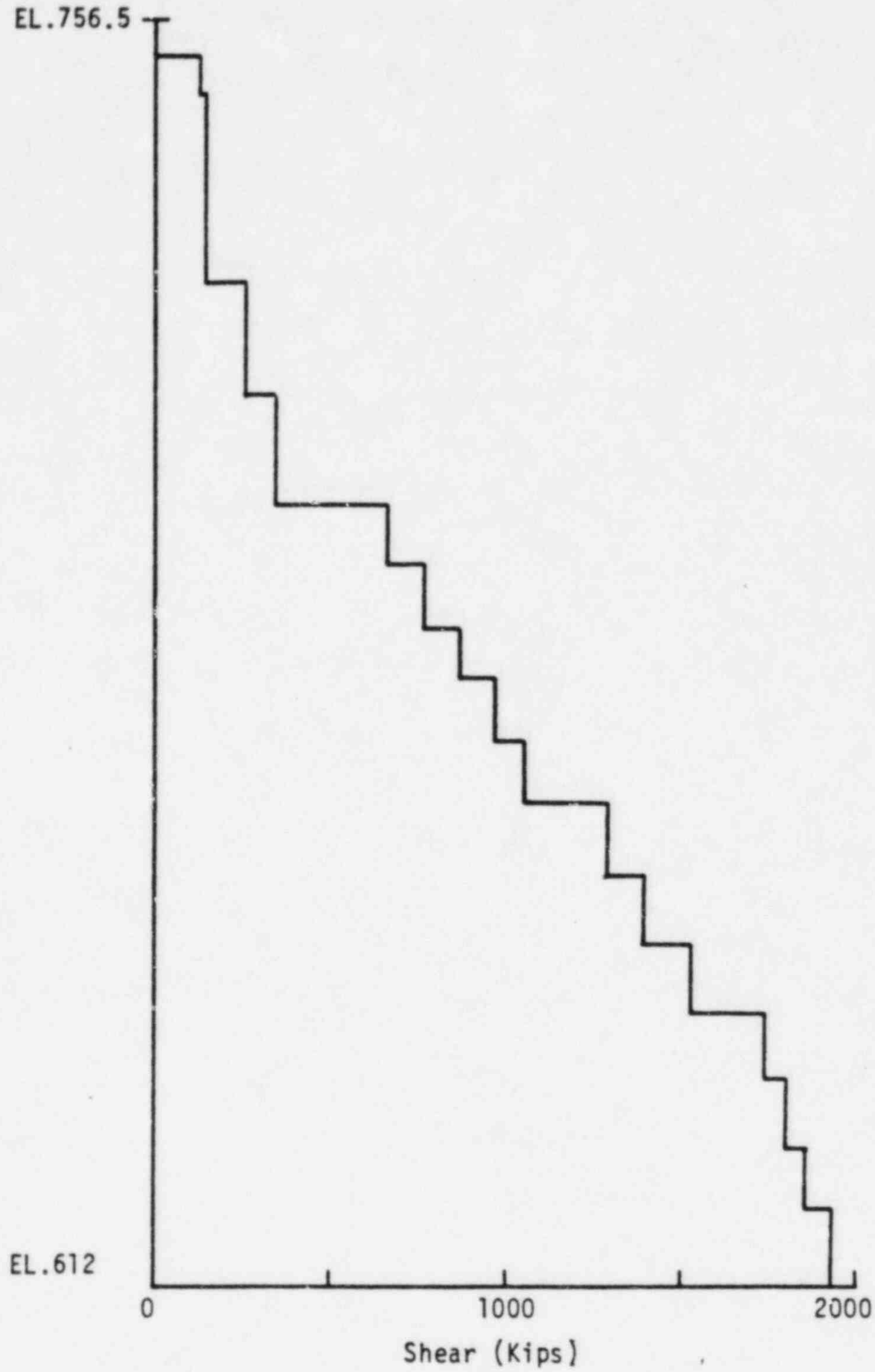

FIGURE 5-16: TOTAL SHEAR IN Y DIRECTION USING UPPER BOUND ESTIMATE OF SOIL SHEAR MODULUS

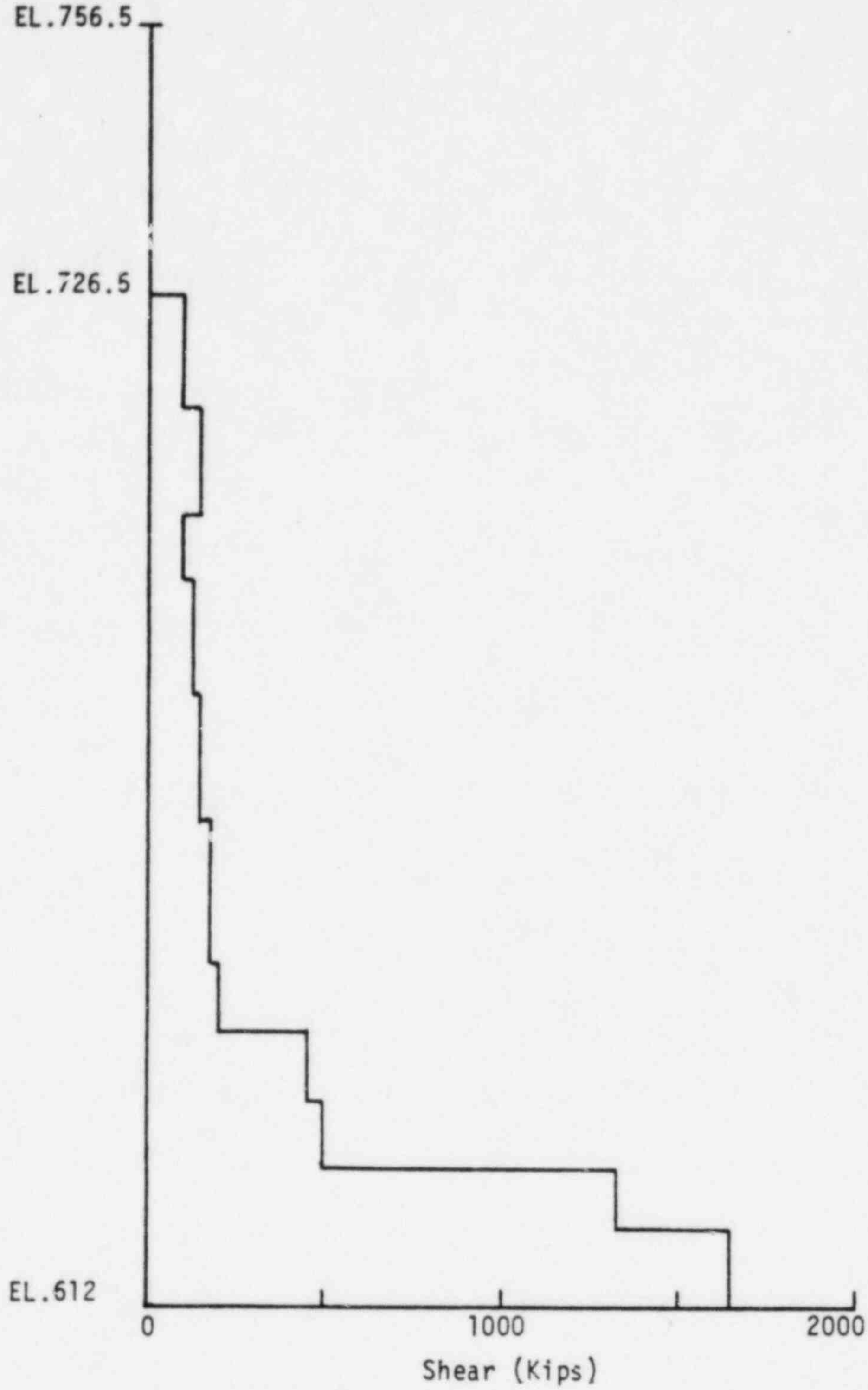

FIGURE 5-17: SHEAR IN THE OUTER SHIELD WALL IN X DIRECTION USING LOWER BOUND ESTIMATE OF SOIL SHEAR MODULUS

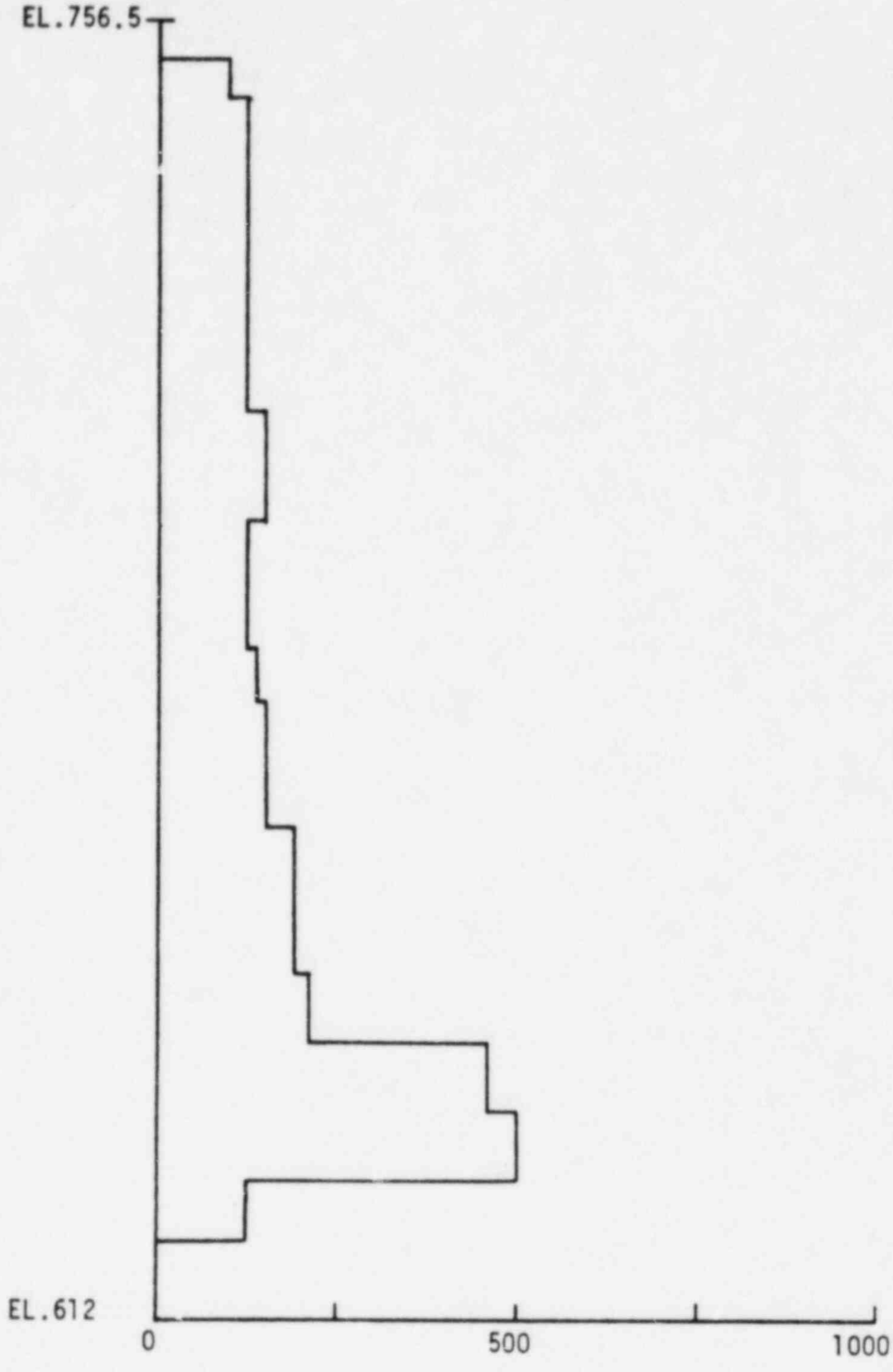

FIGURE 5-18: SHEAR IN STEEL CONTAINMENT IN X DIRECTION USING LOWER BOUND ESTIMATE OF SOIL SHEAR MODULUS

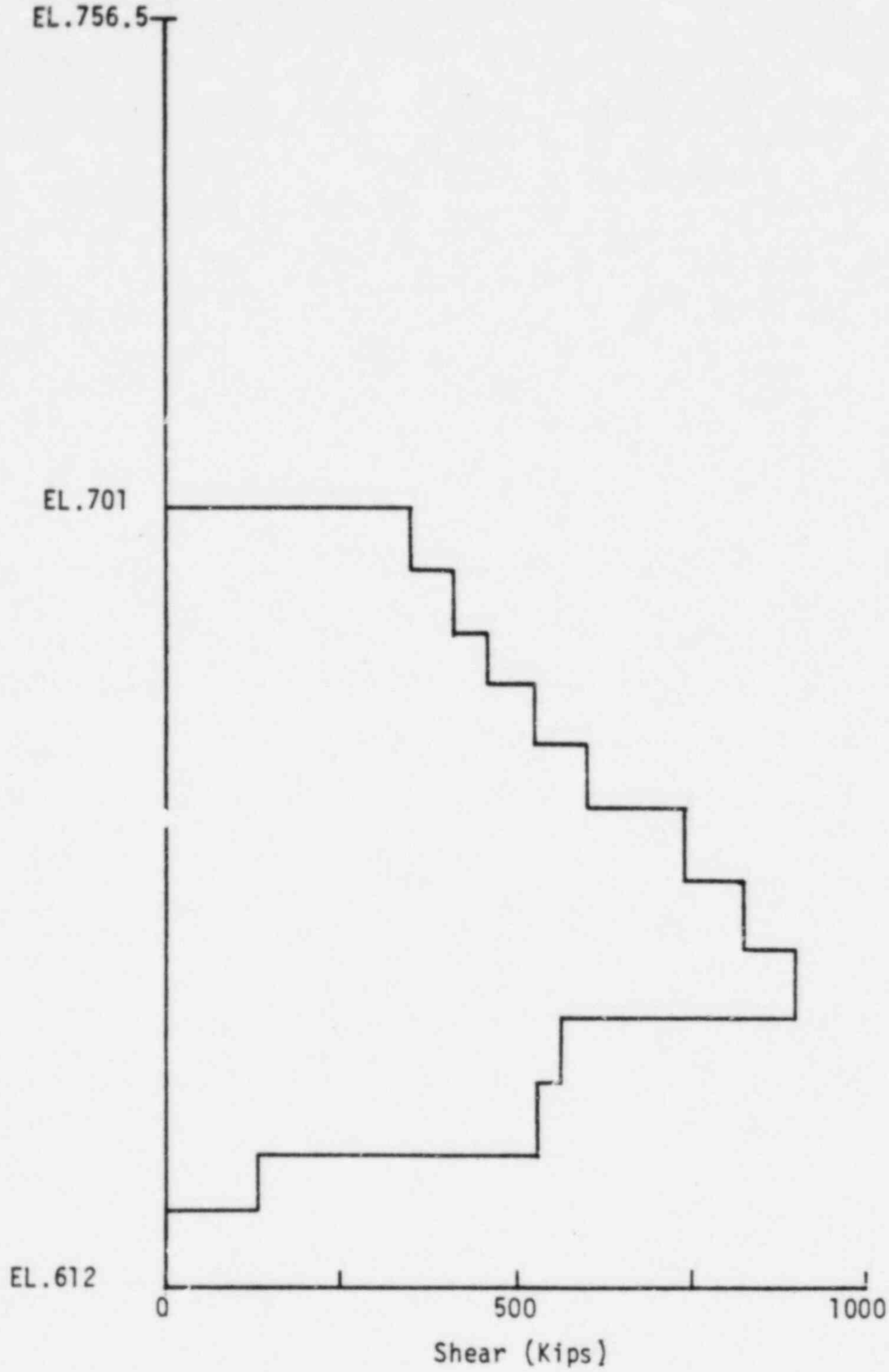

FIGURE 5-19: SHEAR IN THE INNER SHIELD STRUCTURE IN X DIRECTION USING LOWER BOUND ESTIMATE OF SOIL SHEAR MODULUS

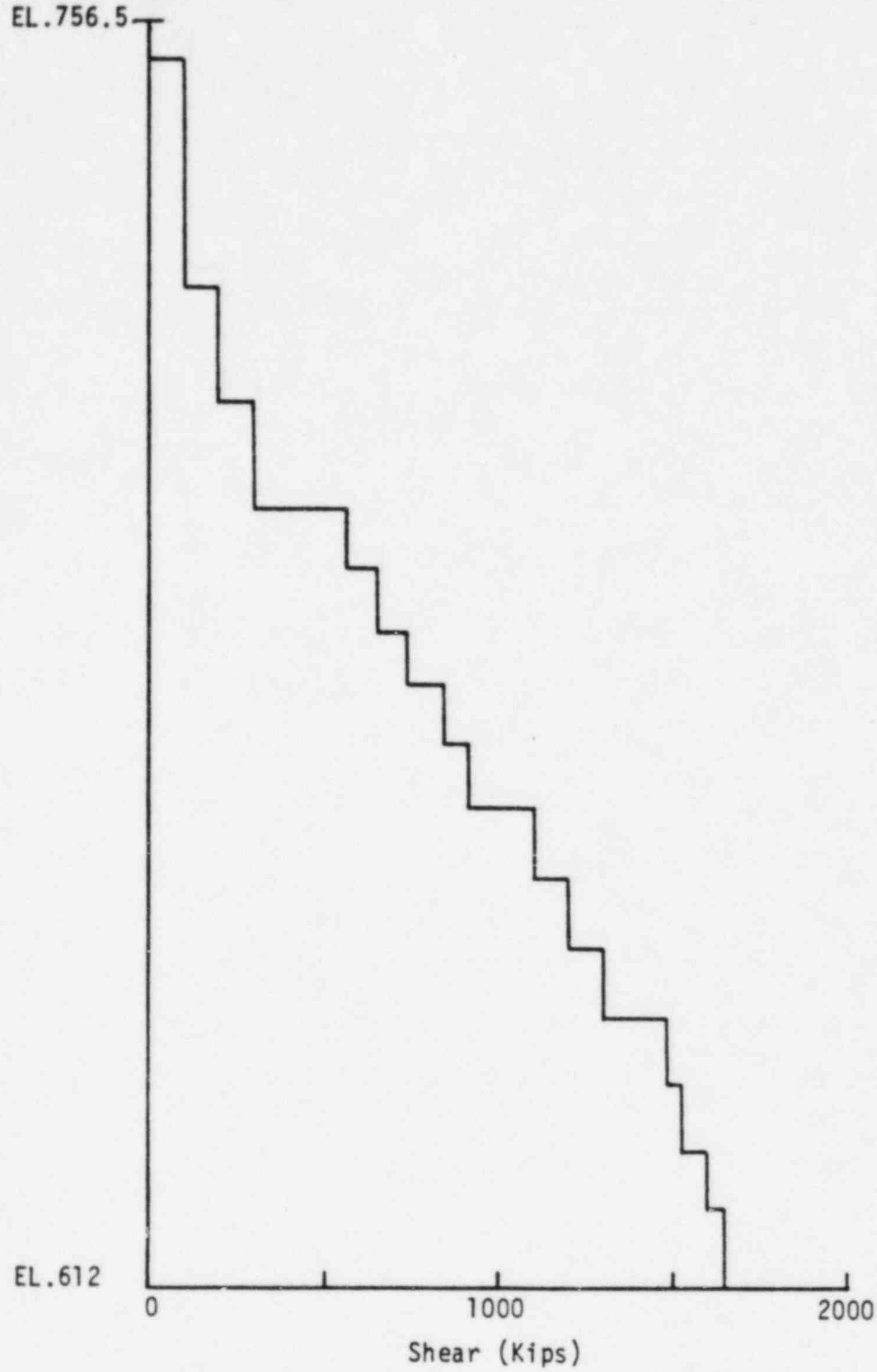

FIGURE 5-20: TOTAL SHEAR IN X DIRECTION USING LOWER BOUND ESTIMATE OF SOIL SHEAR MODULUS

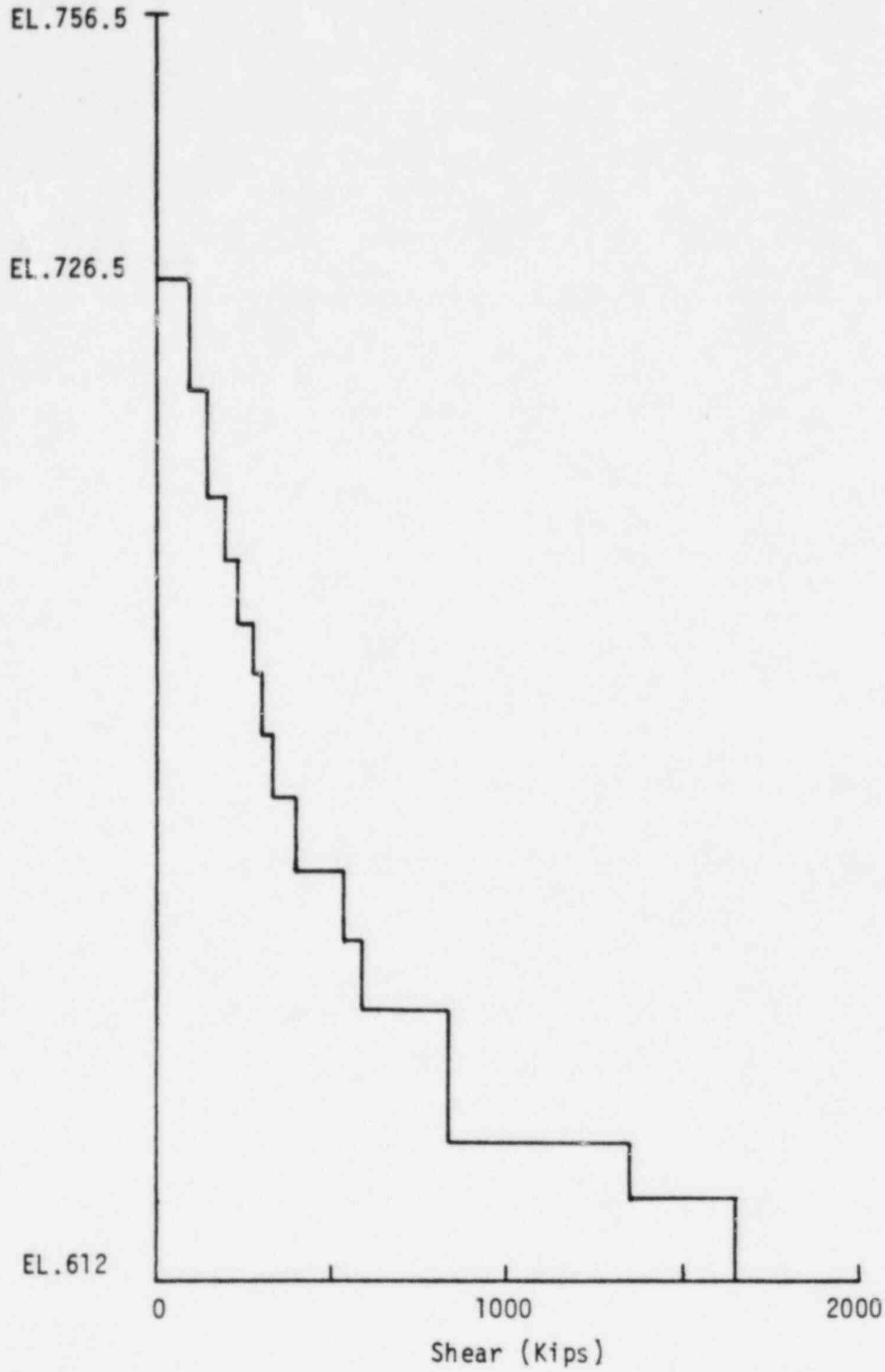

FIGURE 5-21: SHEAR IN THE OUTER SHIELD WALL IN Y DIRECTION USING LOWER BOUND ESTIMATE OF SOIL SHEAR MODULUS

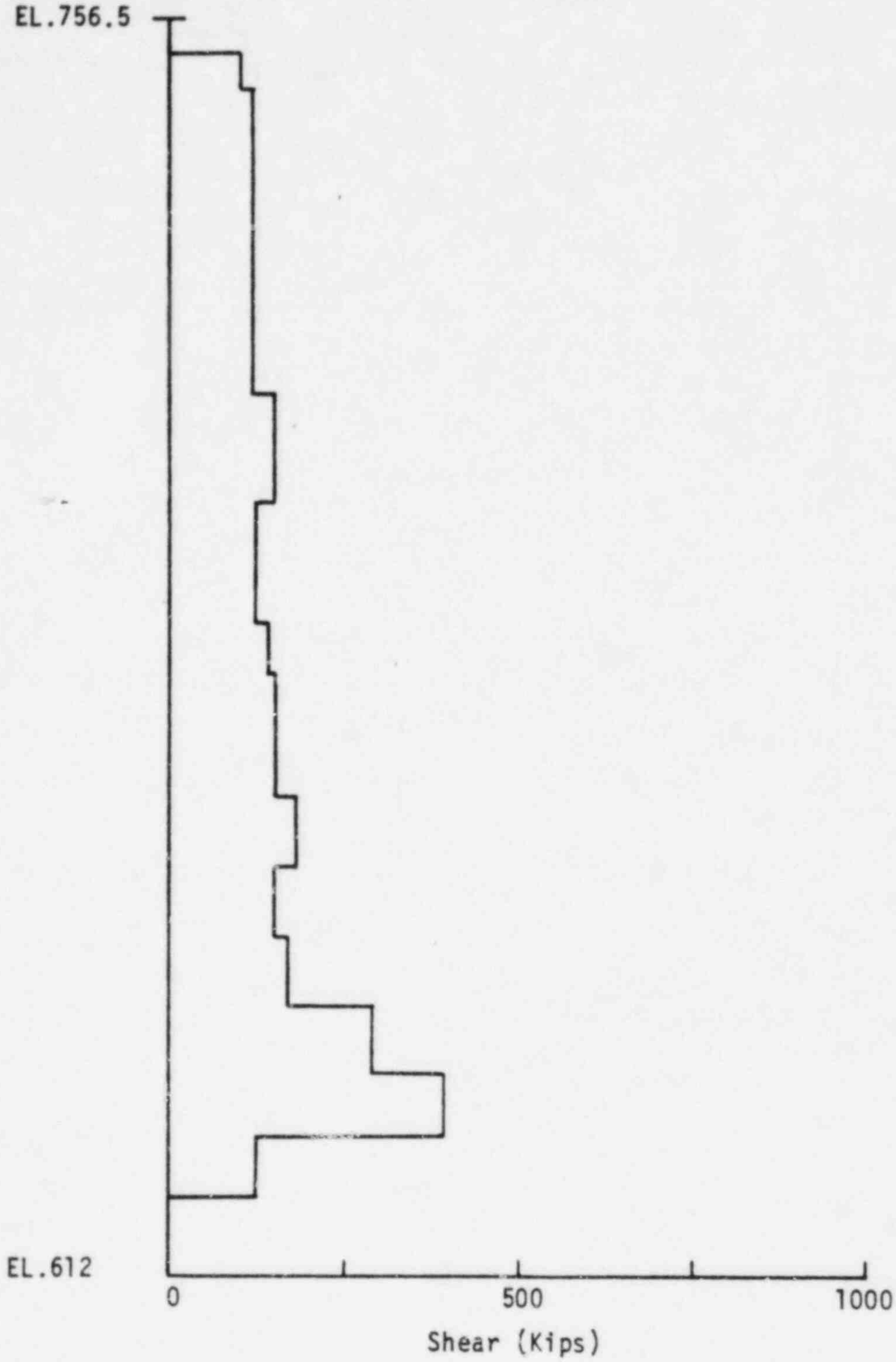

FIGURE 5-22: SHEAR IN STEEL CONTAINMENT IN Y DIRECTION USING LOWER BOUND ESTIMATE OF SOIL SHEAR MODULUS

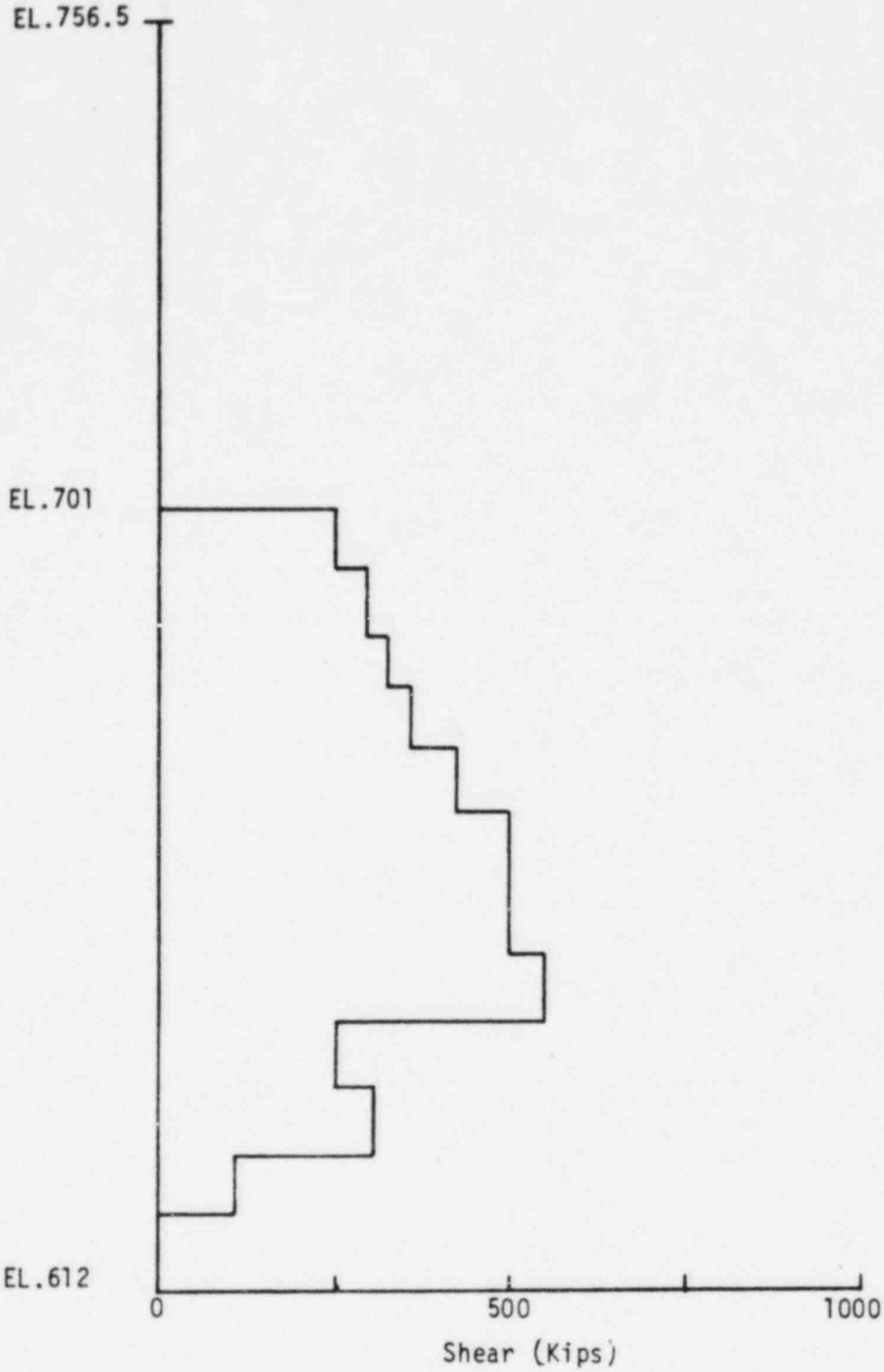

FIGURE 5-23: SHEAR IN THE INNER SHIELD STRUCTURE IN Y DIRECTION USING BOUND ESTIMATE OF SOIL SHEAR MODULUS

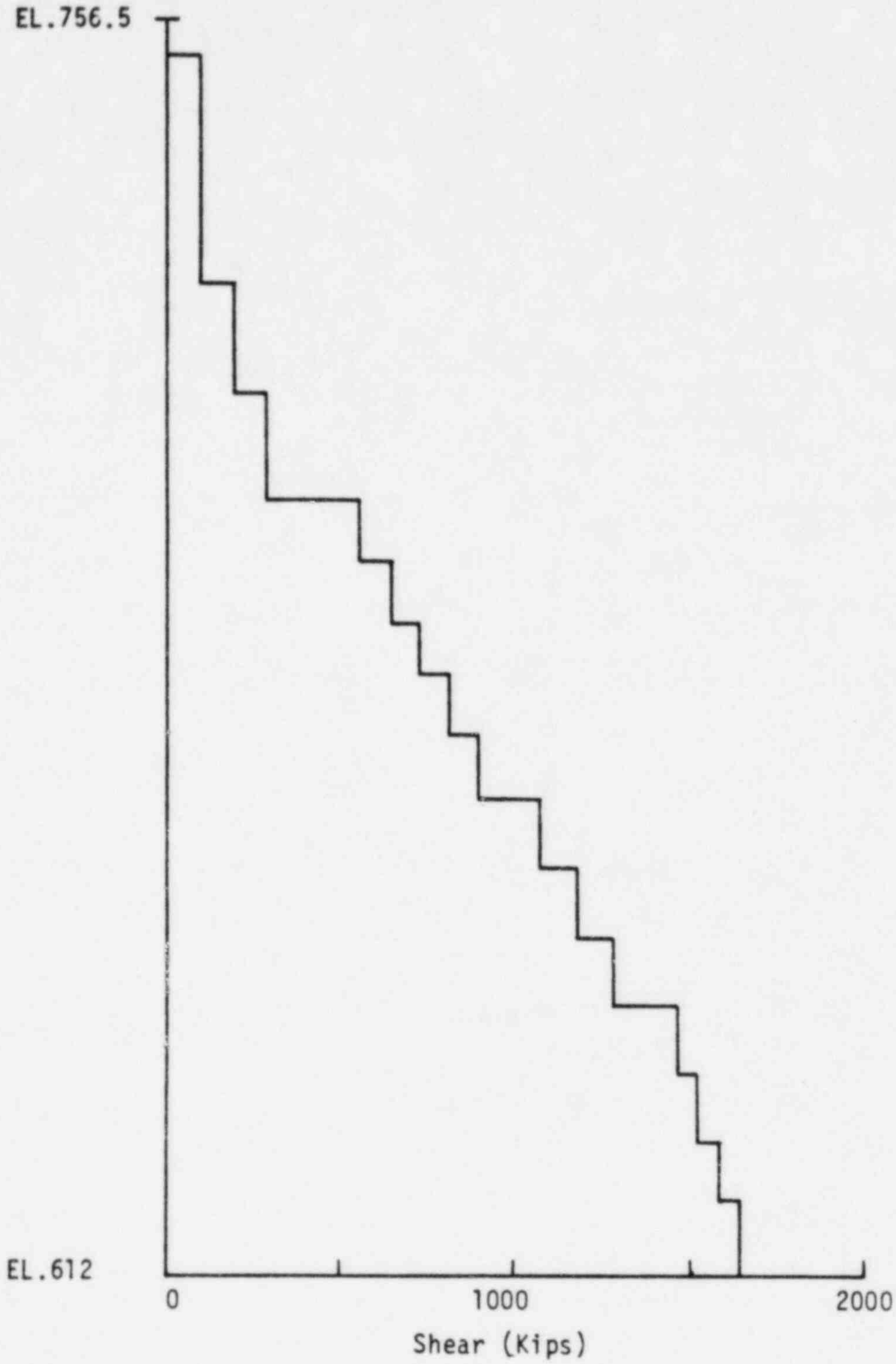

FIGURE 5-24: TOTAL SHEAR IN Y DIRECTION USING LOWER BOUND ESTIMATE OF SOIL SHEAR MODULUS

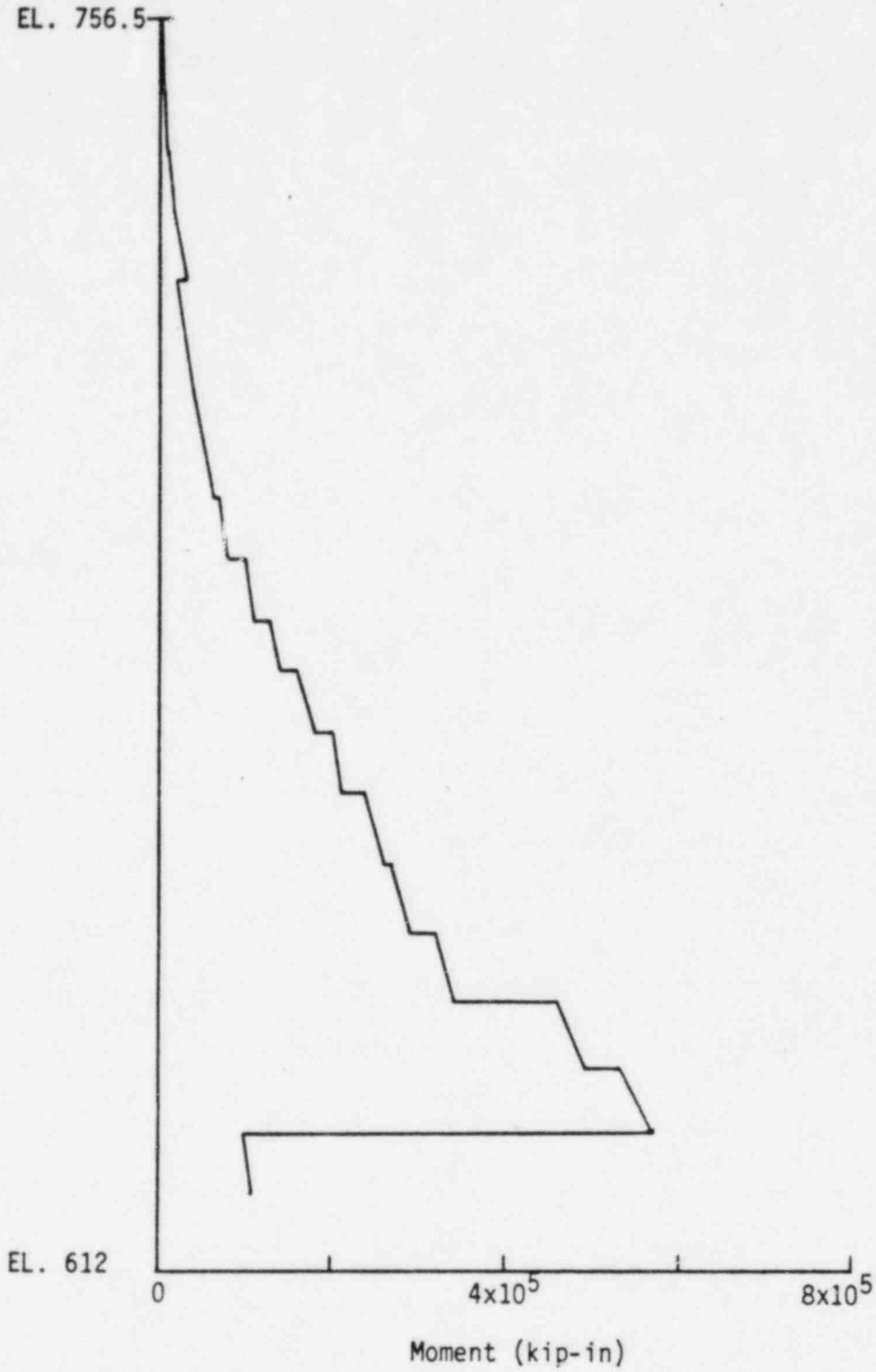

FIGURE 5-25. MOMENT IN STEEL CONTAINMENT IN X DIRECTION USING BEST ESTIMATE OF SOIL SHEAR MODULUS

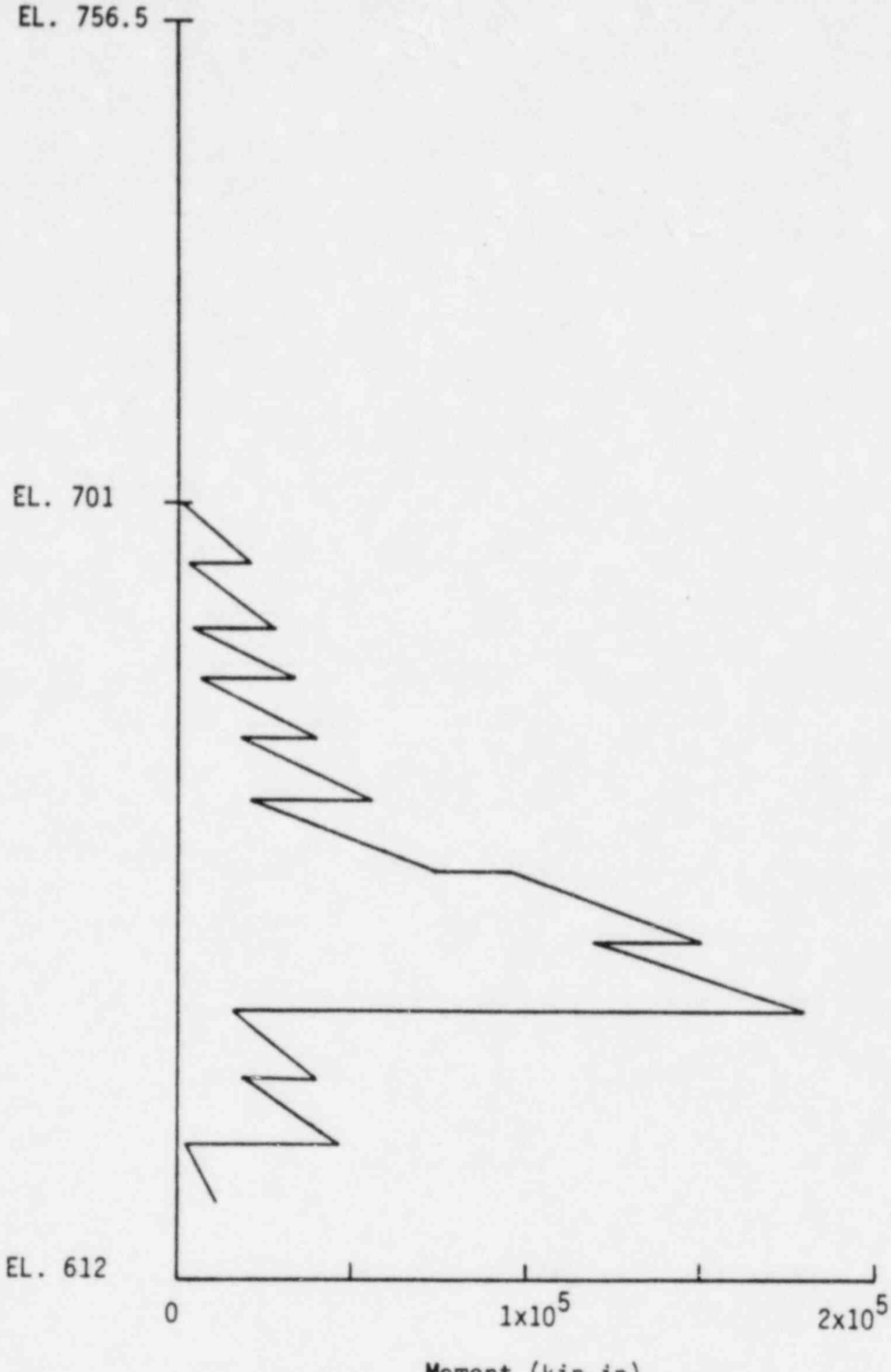

FIGURE 5-26. MOMENT IN INNER SHIELD WALL IN Y DIRECTION USING BEST ESTIMATE OF SOIL SHEAR MODULUS

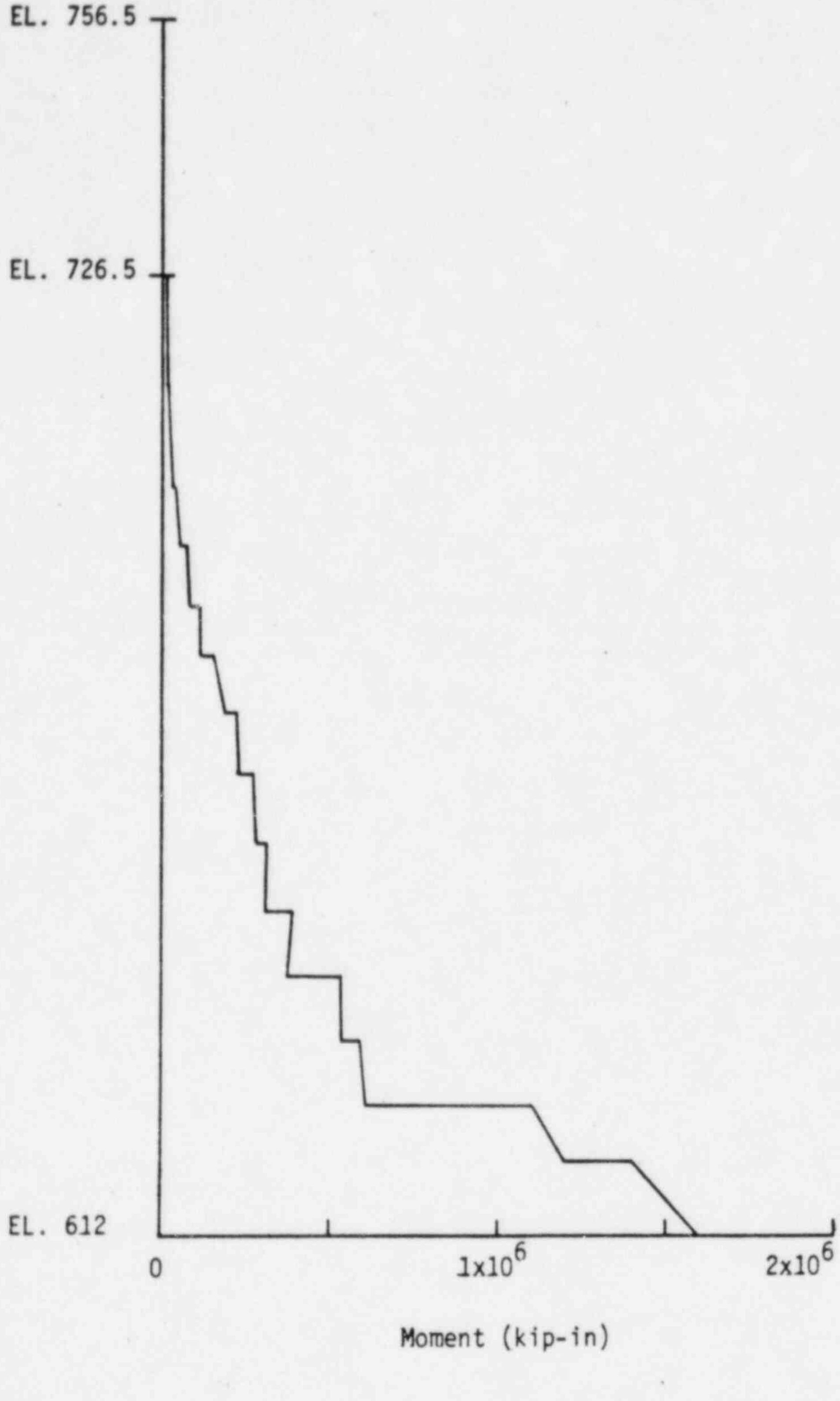

FIGURE 5-27. MOMENT IN OUTER SHIELD WALL IN X DIRECTION USING BEST ESTIMATE OF SOIL SHEAR MODULUS

FIGURE 5-28. MOMENT IN STEEL CONTAINMENT IN Y DIRECTION USING BEST ESTIMATE OF SOIL SHEAR MODULUS

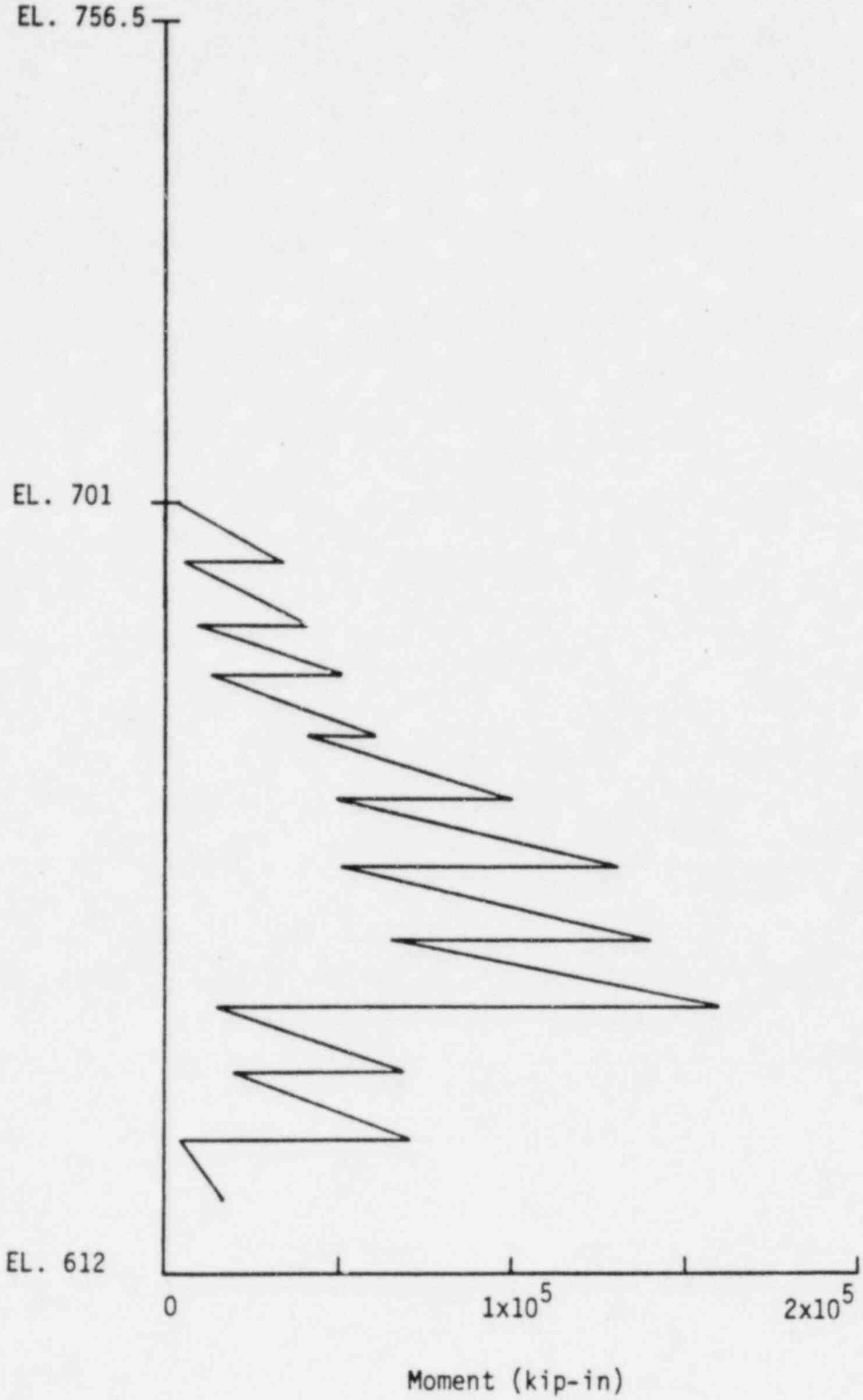

FIGURE 5-29. MOMENT IN INNER SHIELD WALL IN Y DIRECTION USING BEST ESTIMATE OF SOIL SHEAR MODULUS

FIGURE 5-30. MOMENT IN OUTER SHIELD WALL IN Y DIRECTION USING BEST ESTIMATE OF SOIL SHEAR MODULUS

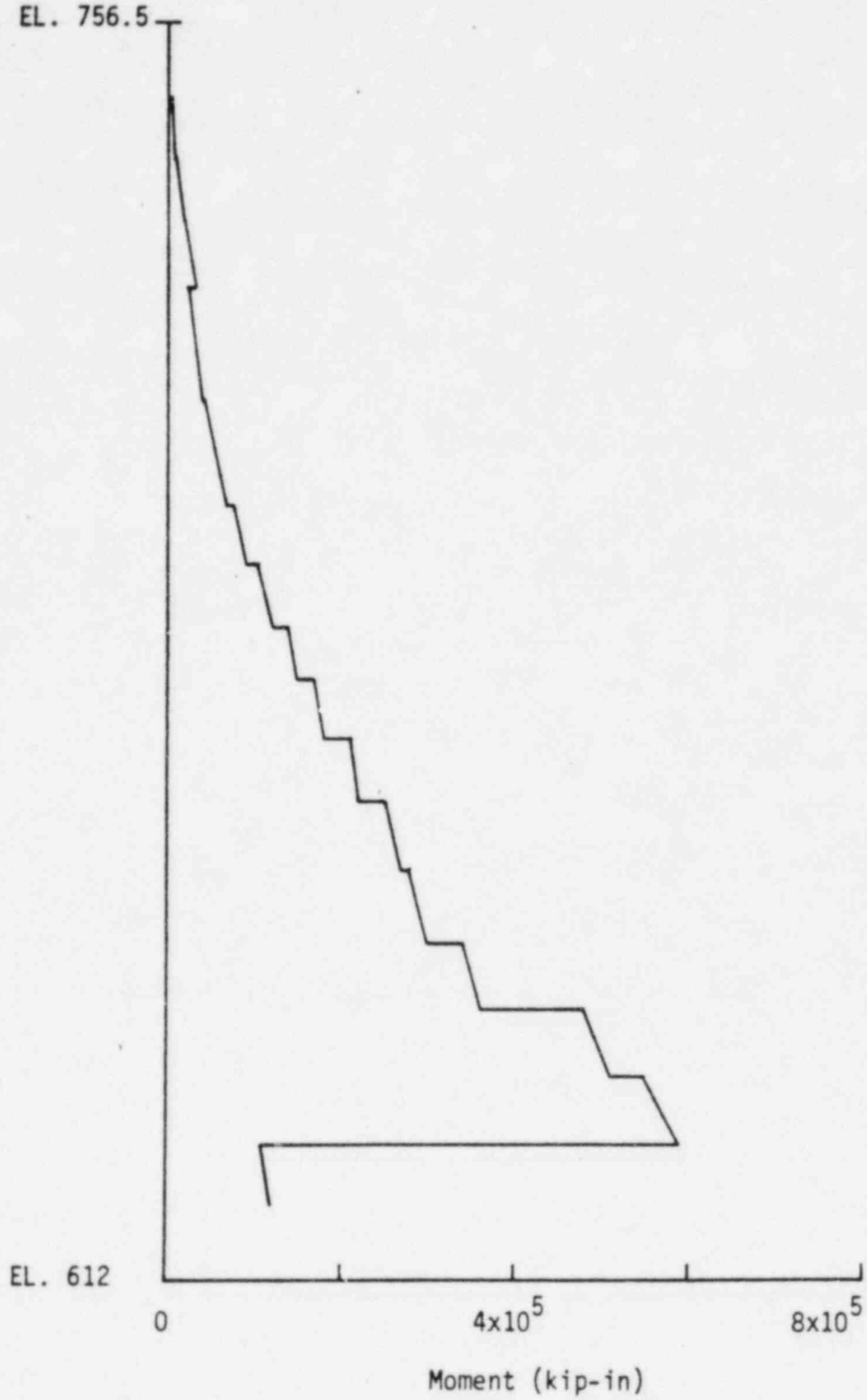

FIGURE 5-31. MOMENT IN STEEL CONTAINMENT IN X DIRECTION USING UPPER BOUND ESTIMATE OF SOIL SHEAR MODULUS

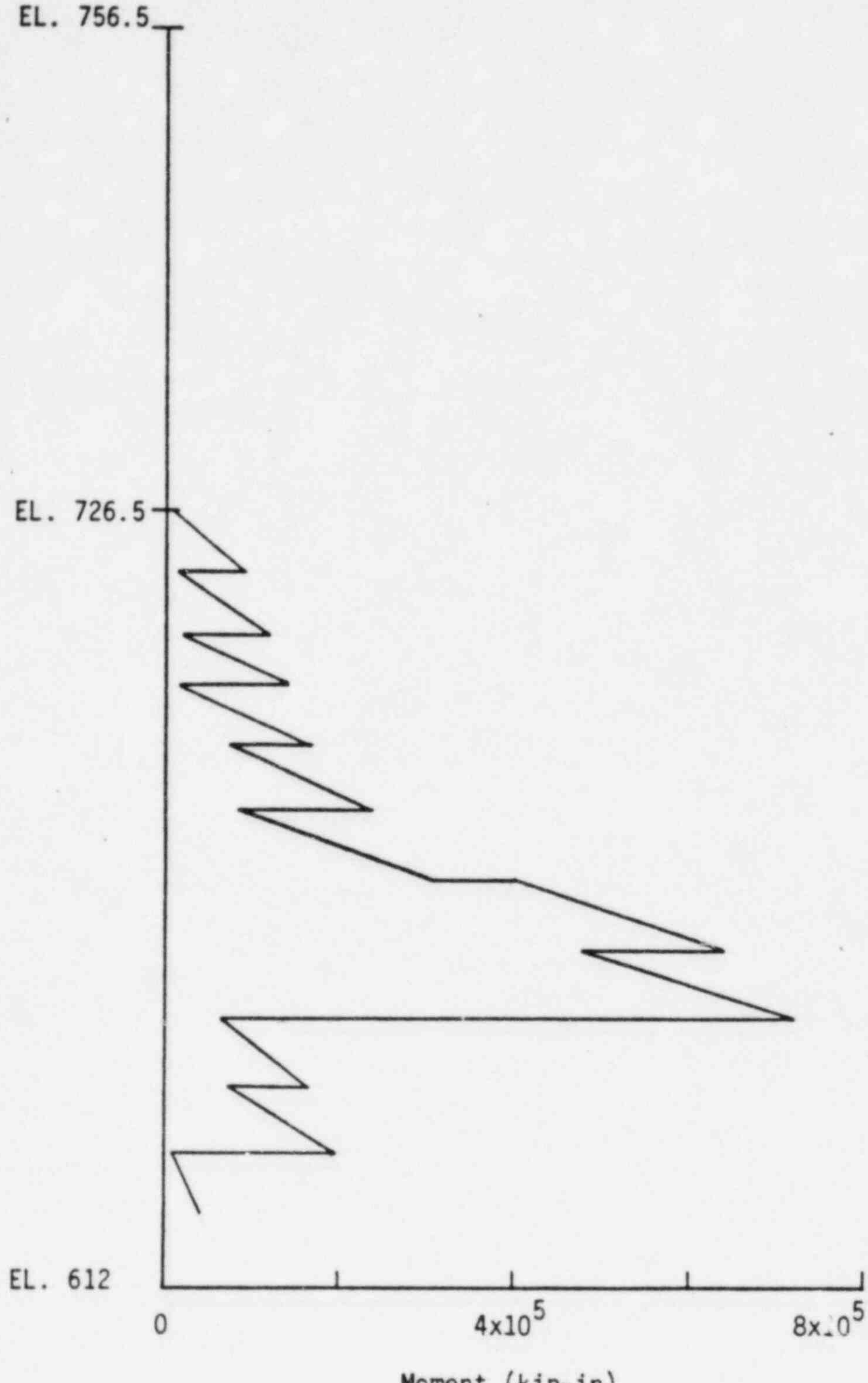

FIGURE 5-32. MOMENT IN INNER SHIELD WALL IN X DIRECTION USING UPPER BOUND ESTIMATE OF SOIL SHEAR MODULUS

FIGURE 5-33. MOMENT IN OUTER SHIELD WALL IN X DIRECTION USING UPPER BOUND ESTIMATE OF SOIL SHEAR MODULUS

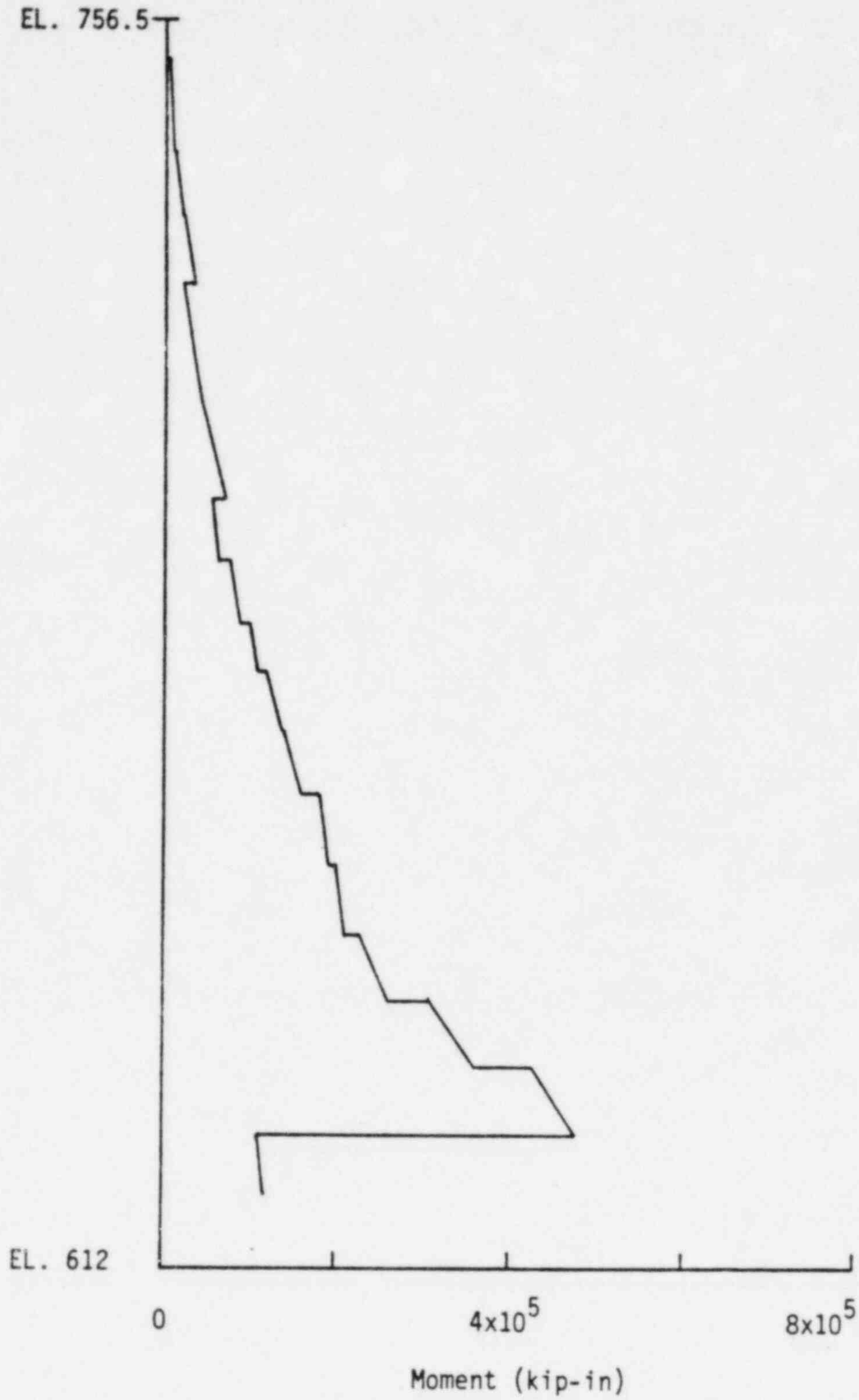

FIGURE 5-34. MOMENT IN STEEL CONTAINMENT IN Y DIRECTION USING UPPER BOUND ESTIMATE OF SOIL SHEAR MODULUS

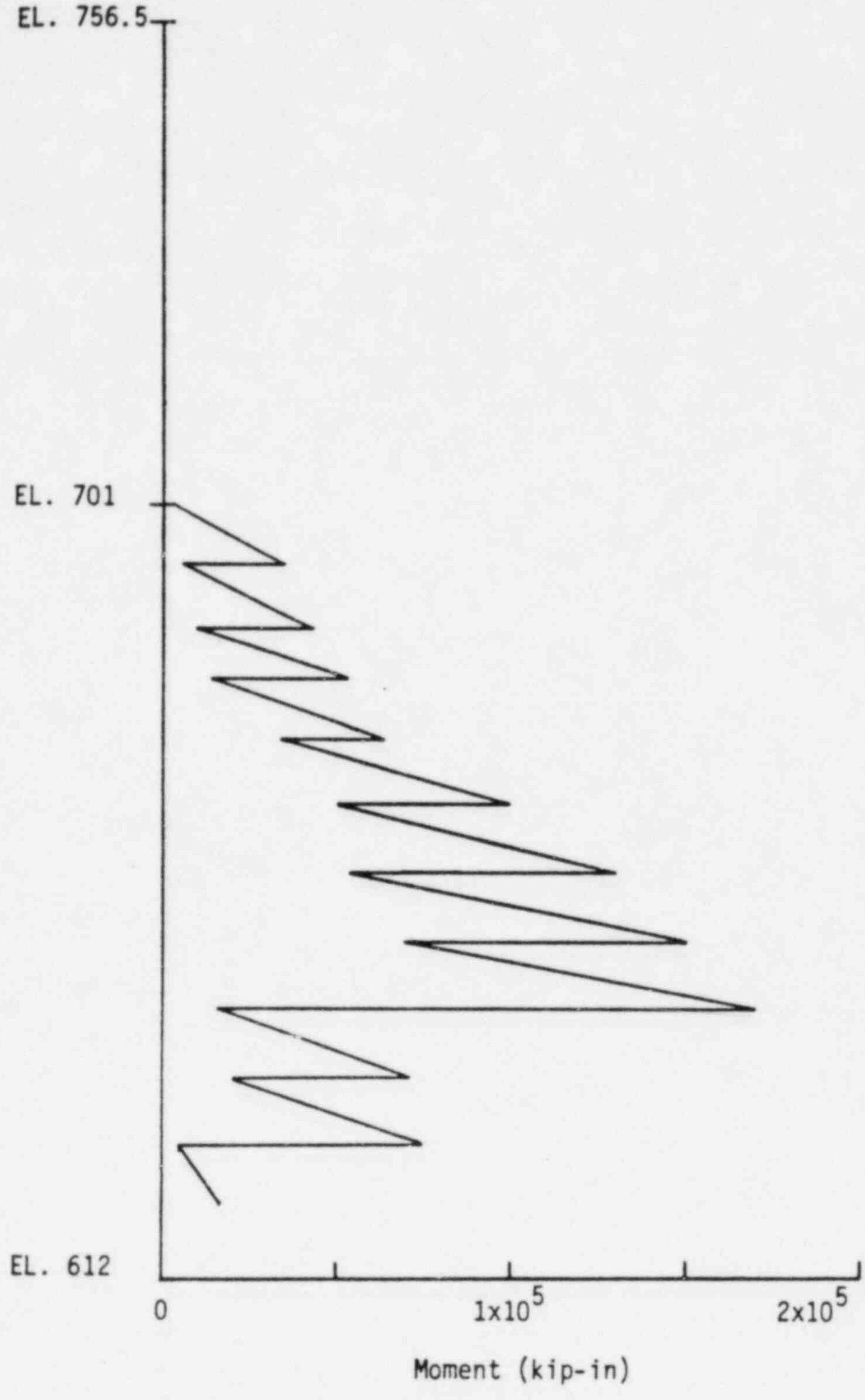

FIGURE 5-35. MOMENT IN INNER SHIELD WALL IN Y DIRECTION USING UPPER BOUND ESTIMATE OF SOIL SHEAR MODULUS

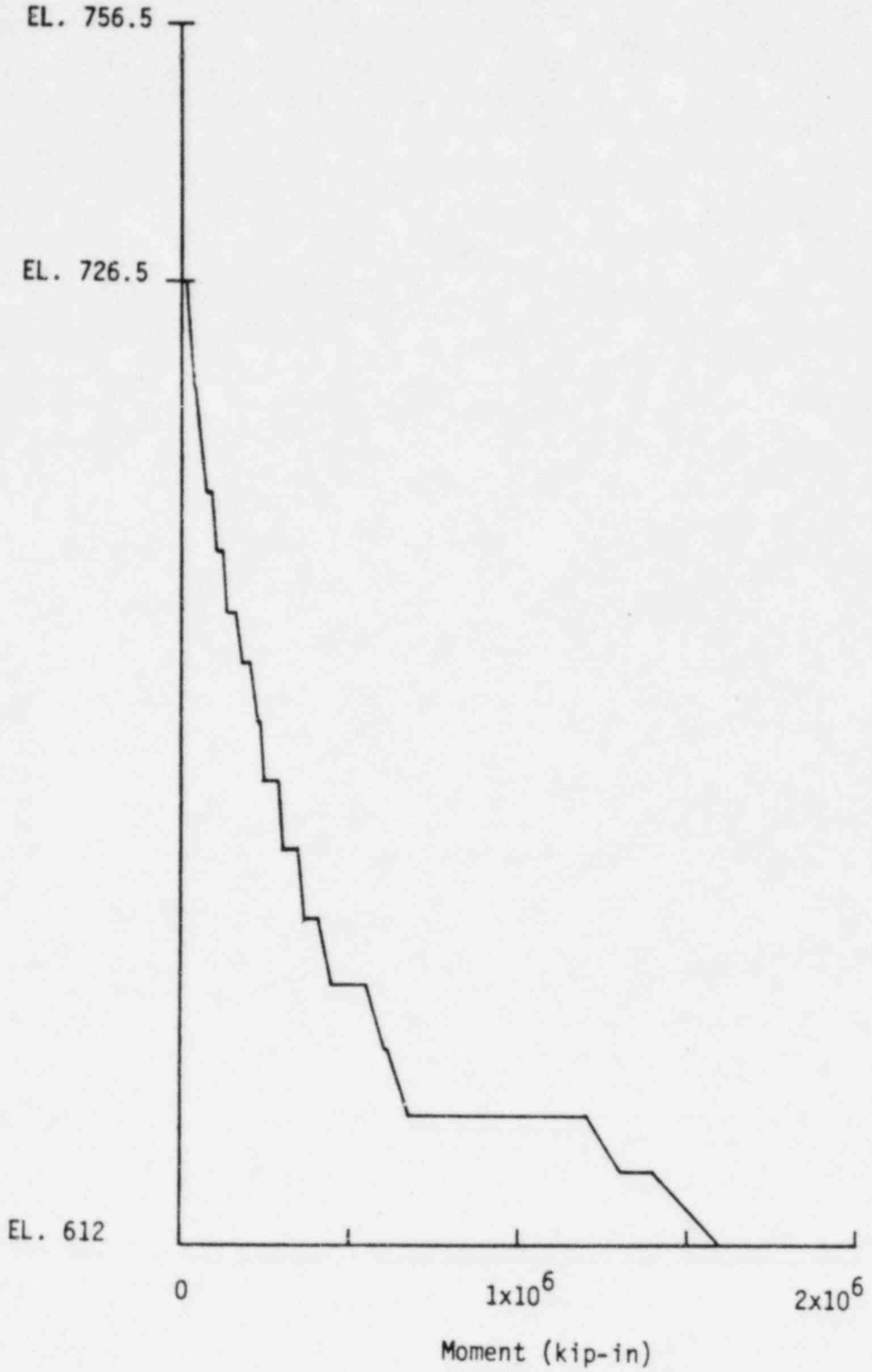

FIGURE 5-36. MOMENT IN OUTER SHIELD WALL IN Y DIRECTION USING UPPER BOUND ESTIMATE OF SOIL SHEAR MODULUS

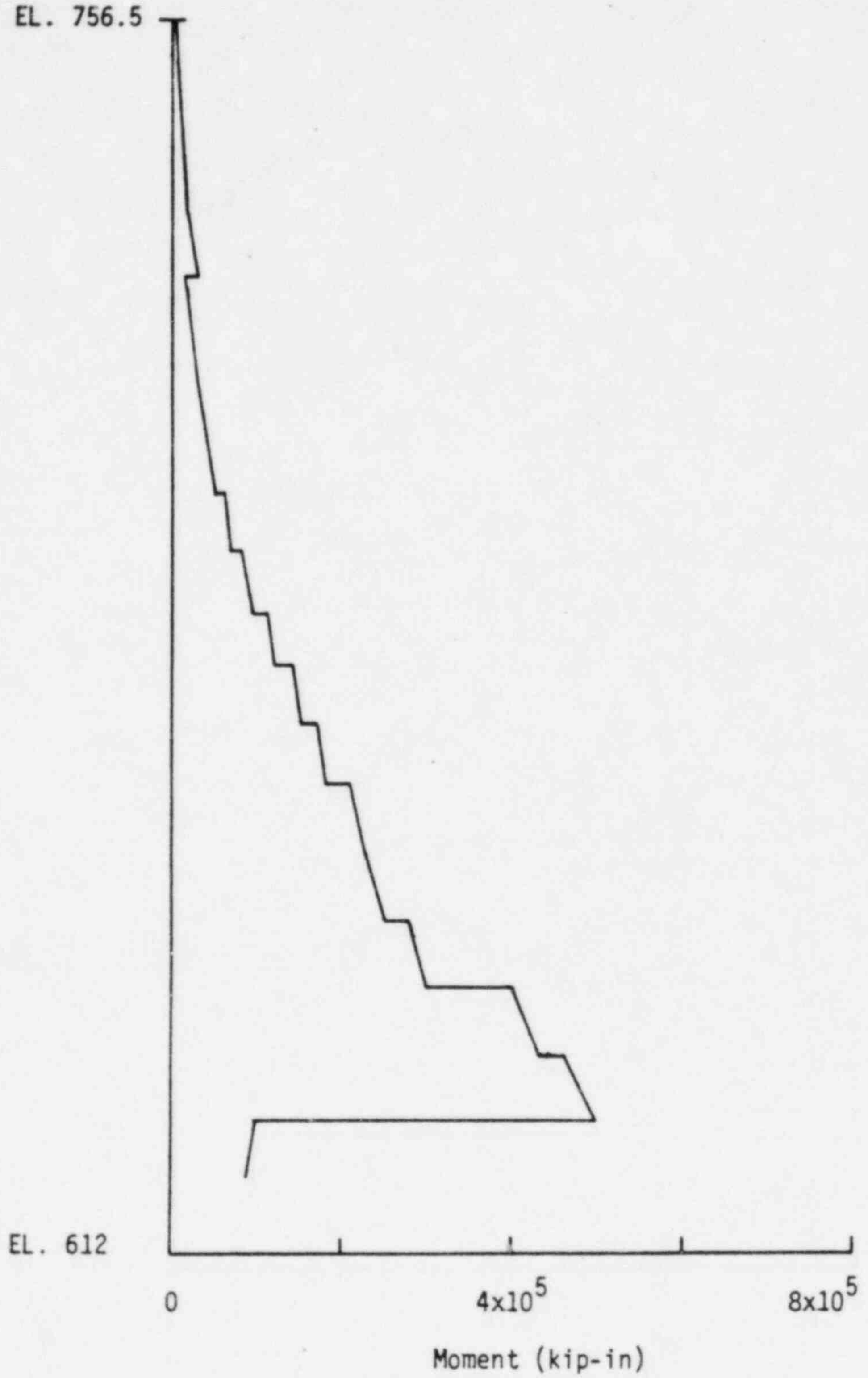

FIGURE 5-37. MOMENT IN STEEL CONTAINMENT IN X DIRECTION USING LOWER BOUND ESTIMATE OF SOIL SHEAR MODULUS

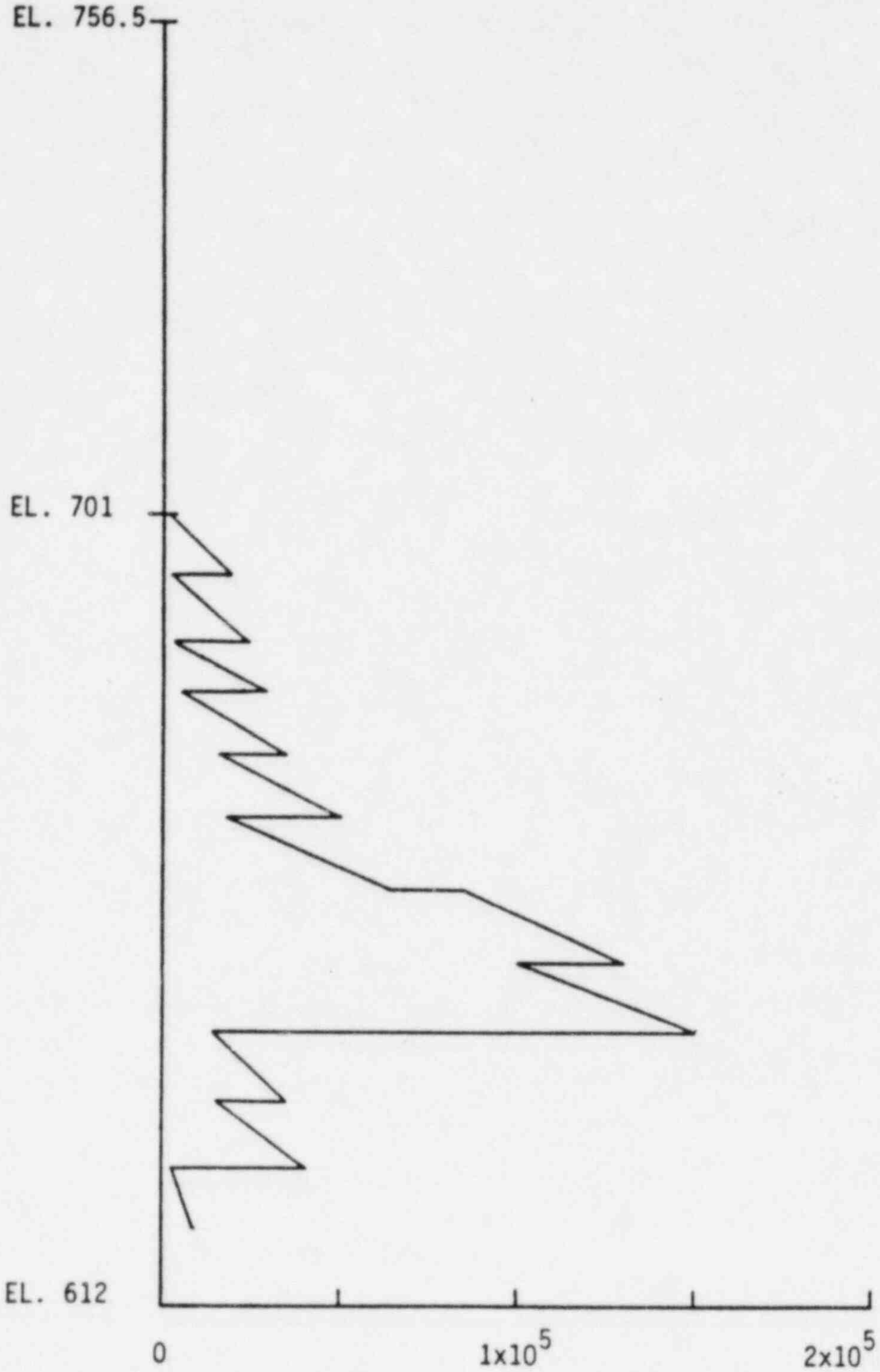

FIGURE 5-38. MOMENT IN INNER SHIELD WALL IN X DIRECTION USING LOWER BOUND ESTIMATE OF SOIL SHEAR MODULUS

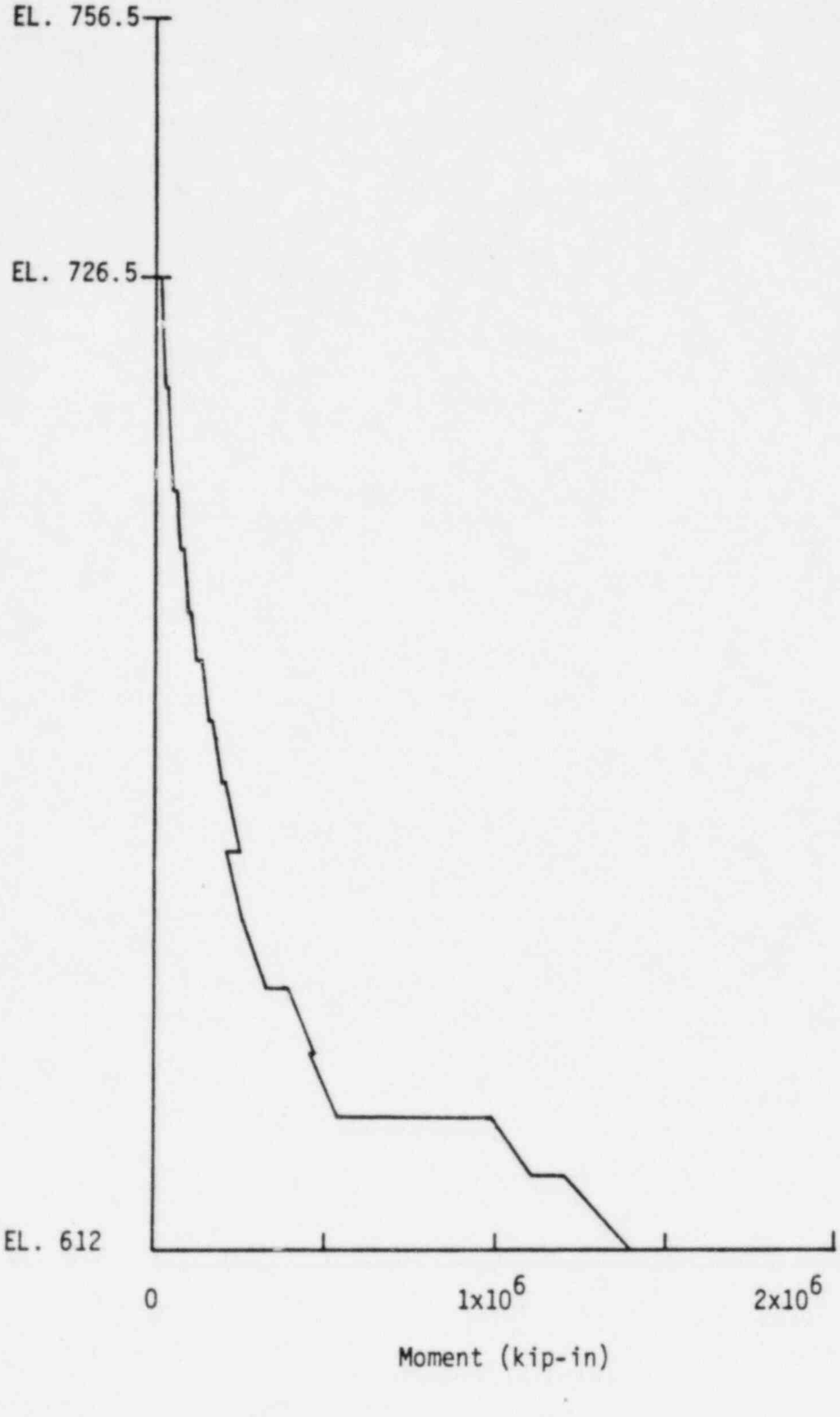

FIGURE 5-39. MOMENT IN OUTER SHIELD WALL IN X DIRECTION USING LOWER BOUND ESTIMATE OF SOIL SHEAR MODULUS

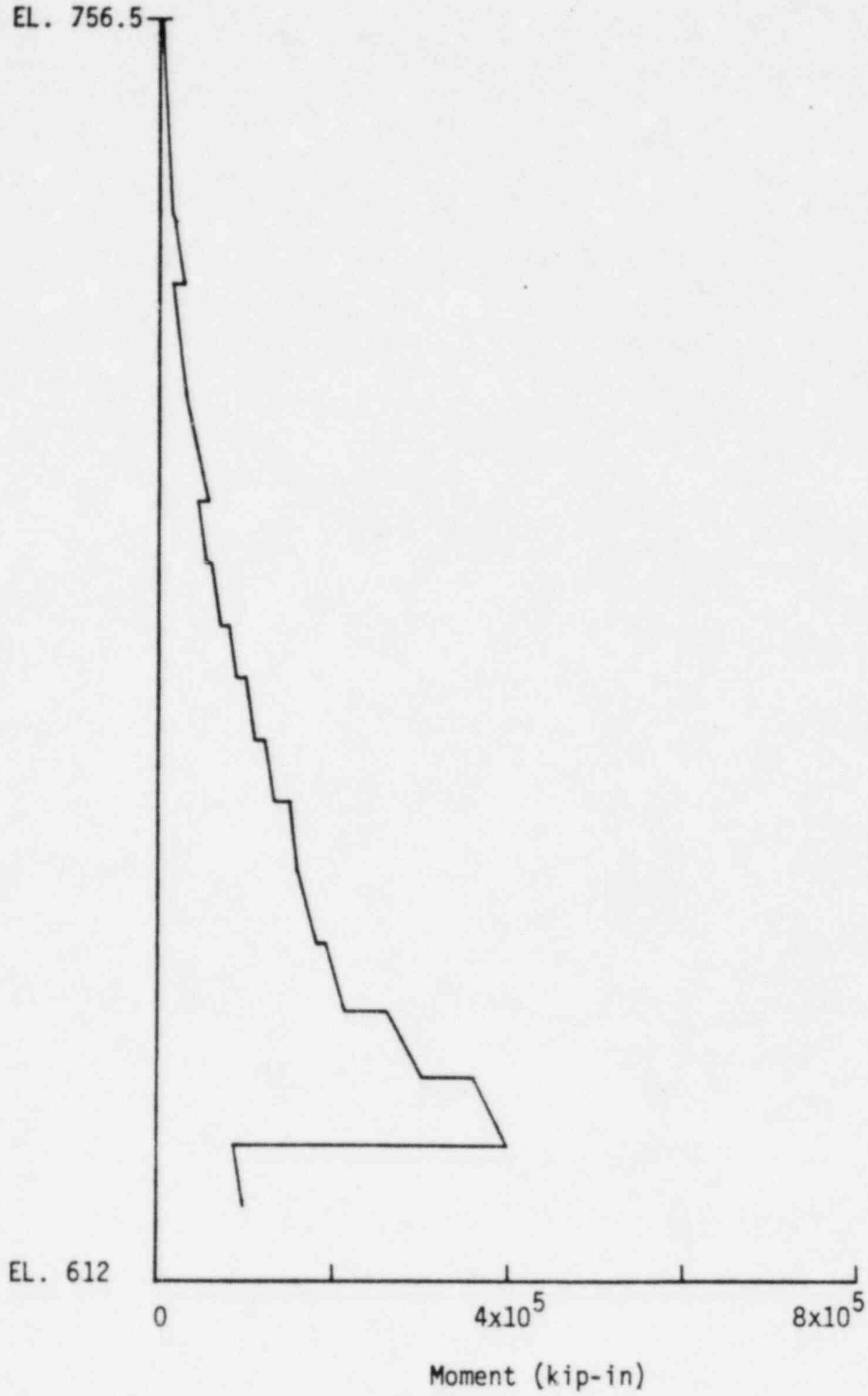

FIGURE 5-40. MOMENT IN STEEL CONTAINMENT IN Y DIRECTION USING LOWER BOUND ESTIMATE OF SOIL SHEAR MODULUS

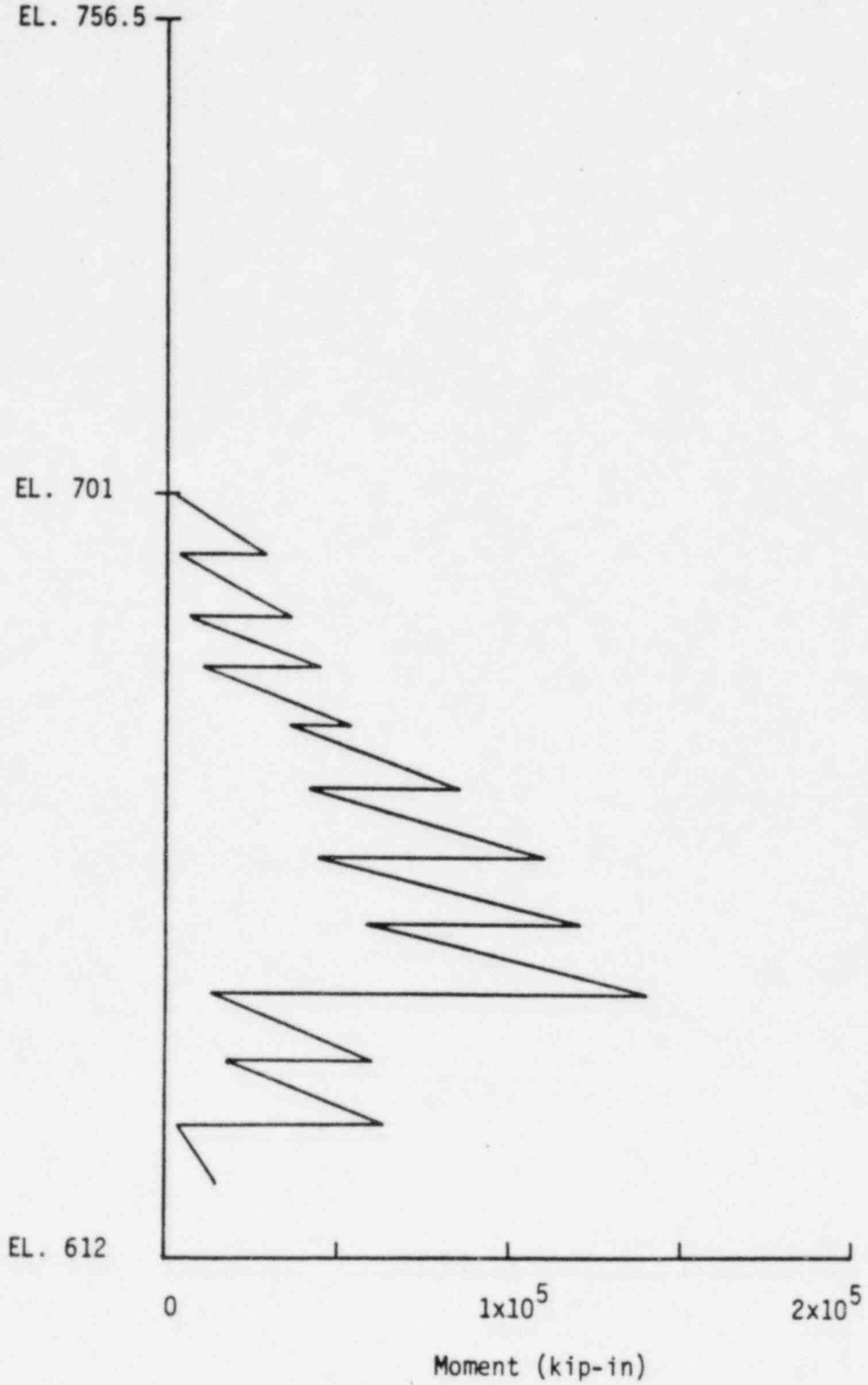

FIGURE 5-41. MOMENT IN INNER SHIELD WALL IN Y DIRECTION USING LOWER BOUND ESTIMATE OF SOIL SHEAR MODULUS

FIGURE 5-42. MOMENT IN OUTER SHIELD WALL IN Y DIRECTION USING LOWER BOUND ESTIMATE OF SOIL SHEAR MODULUS

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Attachment 4

EGG-EA-6053

September 1982

TECHNICAL EVALUATION OF LA CROSSE BOILING WATER REACTOR POWER STATION SEISMIC DESIGN

Engineering Analysis Division
Applied Mechanics Branch

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RGRahl/

RCGuenzler/ R. C. Guenzler

EGG-EA-6053

SEptember 1982

TECHNICAL EVALUATION OF LA CROSSE BOILING WATER REACTOR POWER STATION SEISMIC DESIGN

T. L. Bridges

Prepared for the
U.S. NUCLEAR REGULATORY COMMISSION
Under DOE Contract No. DE-AC07-76ID01570
FIN No. A-6426

EG&G Idaho, Inc

FORM EG&G 398
(Rev 11-81)

INTERIM REPORT

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Washington, D.C.
Under DOE Contract No. DE-AC07-76ID01570
NRC FIN No. A-6426

INTERIM REPORT

ABSTRACT

A Systematic Evaluation Program (SEP) was initiated by the Nuclear Regulatory Commission (NRC) to bring eleven older operating nuclear power plants to a level of safety consistent with current standards of acceptability. Dairyland Power Cooperative's consultant analyzed the La Crosse Boiling Water Reactor (LACBWR) Nuclear Power Station's safety related piping, mechanical and electrical equipment, and component supports. NRC personnel and their consultants from EG&G Idaho, Inc., formed a review team that evaluated the licensee's analyses. The analyses presented to the review team by Dairyland Power Cooperative's consultant were generally acceptable for the areas of SEP which they addressed, although several suggestions, comments, and questions must be resolved. The major deficiency with the material submitted by the licensee is that it does not address all the areas of concern for the SEP program. The results of the analyses performed indicate that modifications are required to bring this plant to an acceptable level of safety.

SUMMARY

A Systematic Evaluation Program (SEP) was initiated by the Nuclear Regulatory Commission (NRC) with the goal of bringing eleven older nuclear power plants to a level of safety consistent with current seismic design standards of acceptability. The La Crosse Boiling Water Reactor (LACBWR) Nuclear Power Station is one of these plants. The NRC and their consultants from EG&G Idaho, Inc., formed a review team for evaluating seismic reevaluation analyses presented by Dairyland Power Cooperative's consultant, Nuclear Energy Services, Inc. (NES). These analyses were performed for safety related equipment required to function during a Safe Shutdown Earthquake (SSE).

In accordance with the SEP audit review plan for LACBWR (Appendix A), a SEP audit meeting was conducted at NES's offices in Danbury, Connecticut, on August 10-11, 1982. The evaluation of information presented at this meeting is the subject of this report. The SEP review team developed an acceptance criteria for guidance in evaluating the SEP seismic reevaluation analyses. The criteria guideline contained in Appendix B coupled with current seismic analysis procedures provided the basis for this technical evaluation of the LACBWR's seismic reevaluation analyses of the safety related equipment.

This report is divided into individual sections covering the piping, electrical equipment, mechanical equipment, and component supports. These sections contain procedures utilized by NES for the seismic reevaluation analyses performed. Each section also contains the review team's evaluation of the analyses presented.

The analyses and procedures presented by NES to the review team were generally acceptable. However, some open items still remain and must be addressed for this review to be complete. The major deficiency with the material submitted by the licensee is that it does not address all the areas of concern for the SEP Program. The results indicate that modifications are required to bring this plant to an acceptable level of safety.

CONTENTS

TECHNICAL EVALUATION REPORT
LA CROSSE BOILING WATER REACTOR (LACBWR)
NUCLEAR POWER STATION--SEISMIC DESIGN

INTRODUCTION

In October of 1977, the Office of the Nuclear Reactor Regulation (NRR), an office of the Nuclear Regulatory Commission (NRC), initiated a Systematic Evaluation Program (SEP) by selecting eleven older operating nuclear power plants with the goal of bringing these plants to a level of safety consistent with current standards of acceptability. These plants were divided into two groups based on their original seismic design. The La Crosse Boiling Water Reactor (LACBWR) Nuclear Power Station, operated by the Dairyland Power Cooperative, is included with the Group II plants. A reanalysis was performed to demonstrate that the structural integrity of the safety related piping systems, mechanical equipment, electrical equipment, and component supports would not be impaired when subjected to a Safe Shutdown Earthquake (SSE) combined with other normal design loadings.

The LACBWR Nuclear Power Station is a boiling water reactor designed to produce 50 MW of gross electric power. This plant became operational in November 1969. The plant architect engineer was Sargent and Lundy Engineers. The plant reactor and generator were supplied by Allis-Chalmers. The plant was not originally designed for seismic loading.

A decision was made by the NRC to review the reevaluation analyses performed by the licensee and their consultants rather than performing their own analyses on the plant. A review team consisting of NRC staff personnel and NRC consultants from EG&G Idaho, Inc., evaluated the piping, mechanical, and electrical equipment analyses. The licensee and their consultants were required to present their seismic reevaluation criteria, typical analyses, and results to the review team.

The audit review consisted of a working level meeting between the review team and Dairyland Power Cooperative's consultant, Nuclear Energy Services, Inc. (NES). This meeting proved to be an efficient method of exchanging

information among the review team and NES with a minimum of formal written communication. The review team obtained a general idea of methods utilized by NES through these meetings. Sample analyses and calculations were presented and reviewed in detail for some systems. Questions, comments, and open items were formulated and submitted to the licensee at the conclusion of this meeting. Prior to the audit meeting, the review team developed an audit plan (Appendix A) and presented it to the licensee. This plan was developed to aid the utility and their consultants in presenting information the review team considered important.

The review team developed an acceptance criteria (Appendix B) for guidance in evaluating the seismic reevaluation analyses. The licensee was requested to justify major deviations which appear less conservative than those in the review team acceptance criteria.

The scope of review for the seismic reevaluation program included the systems, structures, and components (including emergency power supply and distribution, instrumentation, and actuation systems) with the following functions:

1. The reactor coolant pressure boundary as well as the core and vessel internals. This also includes those portions of the steam and feedwater system extending from and including the secondary side of the steam generator up to and including the outermost containment isolation valve and connected piping of 2-1/2 inch or larger nominal pipe size, up to and including the first valve that is either normally closed or is capable of automatic closure during all modes of normal reactor operation.

2. Systems or portions of systems that are required for safe shutdown as identified in the SE safe shutdown review (SEP Topic VII-3). The system boundary includes those portions of the system required to perform the safety function and connected piping up to and including the first valve that is either normally closed or capable of automatic closure when the safety function is required.

3. Systems or portions of systems that are required to mitigate design basis events, i.e., accidents and transients (SEP Topics XV-1 to XV-24). The functions to be provided include emergency core cooling, post-accident containment heat removal, post-accident containment atmosphere cleanup, as well as support systems, such as cooling water, needed for proper functioning of these systems.

4. Systems and structures required for fuel storage (SEP Topic IX-1). Integrity of the spent fuel pool structure including the racks is needed. Failure of the liner plate due to the safe shutdown earthquake must not result in significant radiological releases, or in loss of ability to keep the fuel covered. Failure of cooling water systems or other systems connected to the pool should not permit draining of the fuel pool. Means to supply make-up to the pool as needed must be provided.

5. Structures that house the above equipment.

For the LACBWR Nuclear Power Station, the review team required the following systems, associated structures, and components to be addressed:

a) Reactor Coolant System (RCS)
b) Portions of Main Steam System
c) Portions of Main Feedwater System
d) Portions of other systems directly connected to RCS up to and including isolation valves
e) Control Rod Drives
f) Shutdown Condenser
g) Portions of Demineralized Water Transfer or High Pressure Service Water System
h) Portions of High Pressure Core Spray System
i) Portions of Alternate Core Spray System
j) Manual Depressurization System
k) Spent Fuel Pool and Makeup.

As discussed previously, a "system" also includes the power supply, instrumentation and actuation systems.

The report was divided into individual sections covering piping, electrical equipment, mechanical equipment, and component supports. Each section explains in detail NES's procedures, acceptance criteria, and typical analyses. Each section also contains the review team's evaluation of the analyses performed by NES. The review team's conclusions were based upon the presentations and documents provided by NES.

PIPING SYSTEMS

Licensee Evaluations

The licensee's consultant (NES) performed the seismic reevaluation analyses for the safety related piping systems of the LACBWR Nuclear Power Station. The piping systems analyzed were:

1. Recirculation Piping System

2. Main Steam Piping System

3. Feedwater Piping System

4. Shutdown Condenser Vent Piping System

5. High Pressure Core Spray (HPCS) Discharge Line Piping System

6. HPCS Suction Line Piping System.

NES is currently performing analyses for the Manual Depressurization Piping System and the Emergency Service Water Supply Piping System.

In general, the SEP seismic reevaluation analyses of the safety related piping systems were performed using conventional piping analysis modeling procedures. These analyses were performed using the computer code PIPESD. The analyses considered all of the static load conditions (deadweight, pressure, thermal expansion, thermal anchor movements, and seismic anchor movements) and dynamic seismic inertia loadings using the requirements of the ASME Code for either Class 1 or Class 2 piping. Most of the analyses (see Table 1) contained evaluations of the Operating Basis Earthquake (OBE) and the Safe Shutdown Earthquake (SSE) in combination with normal operating design static loadings. The seismic portions of these analyses were performed by applying each of the two horizontal spectra separately with the vertical spectrum (x+y and y+z) rather than simultaneous three directional spectra input. First, horizontal global x-direction spectrum with simultaneous vertical global y-direction spectrum was input. Then, horizontal global z-direction spectrum with simultaneous vertical global y-direction spectrum was input. Worst case results from the two load cases were used in evaluating the components at each model node point location. The horizontal spectra used to perform these analyses were for the appropriate elevation obtained from a previous analysis (Reference 1). In general, the vertical spectra used were not analytically determined for the appropriate elevations as were the horizontal spectra. These spectra were determined based on the horizontal spectra, or no vertical amplification was assumed and vertical ground spectra were used. The spectra used corresonded to 1% damping for the OBE spectra and 2% damping for the SSE spectra. Generally, the input spectra used was not peak broadened. Table 1 summarizes the seismic loading used to perform the seismic reevaluation analyses for LACBWR's safety related piping systems.

Table 2 provides a summary of the ASME Code editions used to perform the reevaluation analyses, whether the piping system was analyzed using the rules for Class 1 or Class 2 piping, and the allowable stress for evaluating SSE loading in combination with normal design pressure and deadweight loading.

The results of all the seismic reevaluation analyses were within the allowable stress limits for all load combinations evaluated. Initial stress calculations, using the conservative stress intensification factors of the ASME Code for the socket weld coupling/reactor nozzle region, resulted in an excessive peak stress value and therefore a very low number of permissible HPCS operating cycles (100 cycles). NES then reanalyzed this socket weld coupling using a detailed finite element model and the ANSYS computer code. The transient thermal loading produced by the HPCS initiation were determined using the same basic model developed for the stress analysis and using the LION computer program. The allowable number of full stres cycles was determined to be 2900 based on this detailed analysis.

TABLE 1. SEISMIC LOADING SUMMARY

Piping System	NES Report No.	Seismic Global Loading Combinations	Basis for Vertical Spectrum	Spectra Peak Broadening Utilized
Recirculation	81A0089	OBE x+y and z+y	From Reference 1	None
Main Steam	81A0088	OBE x+y and z+y SSE x+y and z+y	SSE--Assumed no vertical amplification, used 2/3 of horizontal SSE ground spectrum OBE--used 1/2 of SSE vertical	None
Feedwater	81A0087	OBE x+y and z+y SSE x+y and z+y	Same as above	None
HPCS Discharge	81A0091 Rev. 1	OBE x+y and z+y SSE x+y and z+y	Same as above	None
HPCS Suction	81A0090	OBE x+y and z+y SSE x+y and z+y	Same as above	None
Shutdown Condenser Vent	81A0051	OBE x+y+z SSE x+y+z	The vertical response spectra for SSE and OBE were taken as 2/3 of the respective horizontal spectra	None
Manual Depressurization	Draft Preliminary	SSE x+y+z	The SSE vertical spectrum was taken as 2/3 the horizontal SSE spectrum	$\pm$ 10%

TABLE 2. ALLOWABLE STRESS SUMMARY

Piping System	Edition of ASME Code Used	Analysis Piping Class	Allowable Stress Used to Evaluate SSE Loading in Combination with Pressure, Deadweight and Sustained Mechanical Loads
Recirculation	1974	2	OBE $1.2\ S_h$
			SSE $1.8\ S_h$*
Main Steam	1974	2	$1.8\ S_h$
Feedwater	1974	2	$1.8\ S_h$
HPCS Discharge	1974	1	$3.0\ S_m$
HPCS Suction	1974	1	$3.0\ S_m$
Shutdown Condenser Vent	1980	2	$1.2\ S_h$
Manual Depressurization	1980	2	$1.2\ S_h$

* Evaluation was not performed.

Review Team Evaluations

In general, the seismic reevaluation analyses of the safety related piping systems were adequately performed. The modeling techniques utilized by NES provide a complete and practical representation of the piping systems. The mass point spacing used was examined and considered acceptable. The response spectra damping values utilized by NES were in accordance with Regulatory Guide 1.61. The load combinations evaluated were more than adequate for most of the analyses in that both OBE and SSE loadings were evaluated. The systems evaluated using the rules for Class 1 piping were also evaluated for fatigue. The allowable stresses used by NES to reevaluate the safety related piping meet or exceed SEP allowable stress requirements specified in Appendix B.

The seismic portions of NES's piping reevaluation analyses were performed by using two simultaneous global directions of acceleration input at a time for most of the reevaluation analyses. The method of combining the responses due to these two global directional inputs was not specified. It is believed PIPESD provides two options for combining these responses, either absolute sum or square root of the sum of squares (SRSS). This is not a major issue; however, considering that only two directions of simultaneous input were used, absolute sum is more acceptable. In addition, the reevaluation analyses of the safety related piping systems were accomplished by combining modal responses by the SRSS method. This method is not consistent with current Regulatory Guide 1.92 requirements if closely spaced modes exist. EG&G Idaho, Inc. performed an independent analysis of LACBWR's HPCS discharge piping system (Reference 2) using three directional simultaneous spectra input per current Regulatory Guide 1.92 requirements. The piping stresses were somewhat higher than those determined by NES and still well within allowable stresses. However, the piping support loads were determined to be significantly greater (sometimes 75%) for the EG&G analysis. Should LACBWR's piping reevaluation analyses be redone for other reasons, it is recommended that simultaneous three directional spectra input be used and modal responses be combined per the current requirements of Regulatory Guide 1.92.

The piping reevaluation analyses performed by NES are based on piping support modifications which have not been installed at the present time. Support modifications have been installed for the HPCS discharge and suction systems; however, for these two piping systems, several discrepancies between piping support conditions as designed and or installed from those modeled in the piping system analyses are present. These discrepancies appear to be significant and require either additional justification or these analyses should be redone using actual support conditions (location and stiffness).

For the recirculation piping system, the SSE loading was not evaluated based on the assumption that the SSE loading is only 20% greater than the OBE loading and the stress allowables are 50% greater for the SSE loading. The SSE and OBE input spectra for this location was reviewed and it appears the assumption that the SSE loading is only 20% greater than the OBE loading is not valid. Above four CPS, it appears that the SSE spectra is approximately twice the OBE spectra. Additional justification for not evaluating SSE loading is required or this evaluation should be performed.

For most of the piping reevaluation analyses performed, the vertical spectra utilized was not analytically determined as was done for the horizontal spectra. The vertical spectra was either assumed to be 2/3 of the horizontal spectra or ground vertical spectra was used assuming no vertical structural amplification. Adequate justification for either of these assumptions was not provided. It is recommended that additional justification be provided to support these assumptions. In addition to this vertical spectra concern, there is another concern with all of the spectra used in that spectra peak broadening was not utilized for most of the piping reevaluation analyses performed. Justification for not complying with this current seismic analysis requirement should also be provided.

The reevaluation analysis of the manual depressurization system was performed using a horizontal spectra which did not account for the horizontal flexibility of the condenser platform. Justification for this simplified assumption should be provided or the horizontal flexibility of the condenser platform should be considered.

Regulatory Guide 1.61 requires using lower damping values than those in Table 1 of Regulatory Guide 1.61 if stress levels are low, otherwise dynamic response will be underestimated. NES did not address this requirement. It has little effect from the standpoint of evaluating piping stresses; however, other aspects of the analyses can be significantly affected such as determining support loads or determining component accelerations. This requirement should not always be overlooked. For example the shutdown condenser vent piping stresses were quite low. The pipe support loads for this system will be underestimated if this reduction in damping is not addressed.

ELECTRICAL EQUIPMENT

Seismic reevaluation analyses have not been performed for LACBWR's safety related electrical equipment nor has the licensee submitted a plan for performing such evaluations. The SEP requirements for the safety related electrical equipment are that structural adequacy of this equipment and its support structures be demonstrated for the SSE loading in combination with normal design operating loads. Demonstration of structural adequacy of the equipment's anchorage is also required. Since the licensee has not addressed the seismic reevaluation of the safety related electrical equipment, this area remains a major open item for the LACBWR Nuclear Plant.

MECHANICAL EQUIPMENT

Licensee Evaluation

Seismic reevaluation analyses of the safety related mechanical eqipment were performed by the licensee's consultant, NES. These analyses were performed for the shutdown condenser and the shutdown condenser platform. The reevaluation analysis of the shutdown condenser was performed using the ASME Code Section VIII, Division 2. Primary shell stresses and local stresses at saddles and nozzles were evaluated for the SSE loading in combination with normal design loading; dead weight, pressure, and mechanical loads. Sloshing was not included in the evaluation. Evaluation of the condenser internals was not performed. Evaluation of the condenser shell was performed using conventional hand calculation methods. The seismic portion of this analysis was performed using the static equivalent method. Local nozzle stresses were calculated using the computer code WERCO, which was based on Welding Research Council Bulletin No. 107. Saddle local stresses were calculated using a three dimensional model of the saddle region of the condenser. The computer code ANSYS was used to perform this analysis. The stress results of the condenser analysis were within the allowables specified by the ASME Code Section VIII, Division 2.

The seismic reevaluation analysis of the shutdown condenser platform was performed based on the requirements for acceptability of Standard Review Plan 3.8.3. This analysis was performed for the SSE loading in combination with dead loads and live loads. The computer code STARDYNE was used to perform this analysis. The seismic portion of this analysis was performed using the static equivalent method. The natural frequency of the platform was determined in three orthogonal global directions considering the stiffness of the platform and the mass of the platform and condenser. The horizontal acceleration values for the SSE loading were then obtained from the acceleration spectra for the platform elevation. The vertical acceleration was assumed to be 2/3 the horizontal acceleration from the corresponding vertical frequency. Results of a preliminary analysis of the condenser platform indicated a need for several platform modifications.

The results of NES's seismic reevaluation analysis of the condenser platform correspond to the modified structure. The results of this analysis are within allowable stresses for the structure members. An evaluation of the member connections was not performed.

Review Team Evaluations

The seismic reevaluation analysis of the shutdown condenser was not totally acceptable. The evaluation of the local nozzle stresses was adequately performed. The evaluation of shell primary stresses was adequately performed, with the exception that sloshing was not included in the evaluation. The boundary conditions used to model the condenser saddle support region appear to be unrealistic and nonconservative. Additional evaluation of the saddle supports region of the condenser is requested. The seismic reevaluation analysis of the condenser did not include an evaluation of the condenser internals. A demonstration of the structural adequacy of the condenser internals must also be provided.

In general, the seismic reevaluation analysis of the shutdown condenser platform was adequately performed from the standpoint of a structural member evaluation. The modeling techniques utilized by NES provide a complete and practical representation of the platform structure. The load combinations utilized in this analysis are consistent with SEP requirements, one major deficiency contained in the analysis of the condenser platform is that structural adequacy of member connections was not demonstrated. Additional justification must be provided for not evaluating the platform structural connections or evaluation of these connections should be provided.

COMPONENT SUPPORTS

The licensee's consultant, NES, performed the seismic reevaluation analyses for component supports on the HPCS piping systems. Evaluation of the HPCS piping supports was performed using AISC allowable stresses times 1.6. This is not consistent with the SEP criteria (Appendix B) for compression loads. The SEP buckling allowable stress is limited to 2/3 critical buckling stress, where the critical buckling stress includes residual stresses, imperfections, and appropriate boundary conditions. The load combinations utilized in the HPCS piping supports analyses were not specified. The SEP component support criteria requires an evaluation of the following two service Level D (faulted) load combinations:

1. $(W + P_D) + (SSE) + (SAM)$

2. $(W + P_D + T) + (SSE) + (SAM)$

where

W	=	loads due to weight effects
P_D	=	pressure plus design mechanical loads
T	=	thermal expansion loads (includes thermal anchor movement effects)
SSE	=	inertial loads due to safe shutdown earthquake
SAM	=	loads due to SSE anchor movement effects

The load combinations used for the HPCS piping supports reevaluation analyses should be provided by the licensee.

Since seismic reevaluation of the component supports for the bulk of the LACBWR plant's safety related equipment has not been performed, this area remains a major open item for the LACBWR Nuclear Plant.

CONCLUSIONS

Based upon the SEP seismic reevaluation analyses reviewed, the structural adequacy of the LACBWR Nuclear Plant to withstand the effects of postulated SSE loading in combination with normal design loading has not been demonstrated. Seismic reevaluation analyses were not performed for all of the LACBWR's safety related electrical and mechanical equipment. The licensee should perform reevaluation analyses for the following additional safety related equipment or provide justification for not evaluating them.

1. Piping from the inside main stream isolation valve (MSIV) to the outside MSIV, including the valve's drive motor.

2. Control Rod Drive Mechanisms

3. Electrical Equipment in Control Room

4. Deisel Fuel Oil Tank

5. Reactor Vessel and Reactor Vessel Supports.

After reviewing the seismic reevaluation analyses performed for the LACBWR Nuclear Plant's safety related equipment, for those analyses provided, the following major items remain open:

General

Transient mechanical design loading (other than seismic) was not addressed in any of the LACBWR Plant's seismic reevaluation analyses. The licensee should provide justification for evaluating sustained mechanical loads instead of all design mechanical design loads.

Piping

1. For the HPCS suction and discharge piping systems, the designed and installed locations of piping supports are significantly different than those used for the seismic reevaluation analyses. Justification for not redoing these analyses must be provided or these analyses must be redone using the actual support locations and stiffnesses.

2. For the recirculation piping system, the SSE loading was not evaluated based on the assumption that the SSE loading is only 20% greater than the OBE loading and the stress allowables are 50% greater for the SSE loading. The SSE and OBE input spectrum for this location were reviewed and it appears the assumption that the SSE loading is only 20% greater than the OBE loading is not valid. Above four CPS, it appears that the SSE spectrum is approximately twice the OBE spectrum. Additi , justification for not evaluating SSE loading is required or this evaluation should be performed.

3. For most of the piping reevaluation analyses performed, the vertical spectrum utilized was not analytically determined as was done for the horizontal spectrum. The vertical spectrum was either assumed to be 2/3 of the horizontal spectrum or ground vertical spectrum was used assuming no vertical structural amplification. Adequate justification for either of these assumptions was not provided. It is recommended that additional justification be provided to support these assumptions.

4. The spectra used to perform most of the seismic reevaluation analyses was not peak broadened as requried by current seismic criteria. Justification for not complying with this current seismic analyses requirement should be provided.

Mechanical Equipment

1. The seismic reevaluation analysis of the shutdown condenser did not consider sloshing loading. Justification for not including this loading must be provided or this loading should be included in the reevaluation analysis.

2. The boundary conditions used to model the condenser saddle support region appear to be unrealistic and nonconservative. Additional evaluation of the saddle supports region utilizing more realistic boundary conditions is requested.

3. The analysis of the condenser did not include an evaluation of the consender internals. A demonstration of the structural adequacy of the condenser internals must also be provided.

4. The analysis of the shutdown condenser platform did not include an evaluation of the structural member connections. Additional justification must be provided for not evaluating the platform structural connections or evaluation of these connections should be provided.

Component Supports

1. Seismic reevaluation analyses were performed for component supports on the HPCS piping systems only. Since seismic reevaluation of the component supports for the bulk of the LACBWR Plant's safety related equipment has not been performed, this area remains a major open item.

2. The load combination's used to perform the seismic reevaluation analyses of the HPCS piping systems' component supports were not provided. The licensee should provide these to NRC so they can be reviewed for compliance with SEP requirements.

3. Evaluation of the HPCS piping supports was performed using the AISC allowable stresses times 1.6 for Service Level D (faulted) conditions. This is not consistent with the SEP criteria (Appendix B) for compression loads. The SEP buckling allowable stress is limited to 2/3 critical buckling stress, where the critical buckling stress accounts for residual stresses, imperfections, and appropriate boundary conditions. Reevaluation of the compression members using the SEP allowable buckling stresses must be performed.

REFERENCES

1. Seismic Evaluation of the La Crosse Boiling Water Reactor, Gulf United Services Report No. SS-1162, January 11, 1974.

2. D. K. Morton, Summary of the La Crosse Boiling Water Reactor Piping Audit Calculations and Review Performed for the Systematic Evaluation Program, EG&G Idaho, Inc., Report No. EGG-EA-5525, January 1980.

APPENDIX A

LA CROSSE AUDIT PLAN FOR SEISMIC QUALIFICATION
OF PIPING, MECHANICAL, AND ELECTRICAL EQUIPMENT

LA CROSSE AUDIT PLAN FOR SEP SEISMIC QUALIFICATION OF PIPING, MECHANICAL, AND ELECTRICAL EQUIPMENT

I. Background

In October, 1977, the office of Nuclear Reactor Regulation (NRR) initiated Phase I of the Systematic Evaluation Program (SEP) to determine the margin of safety relative to current standards for eleven selected operating nuclear power plants and to define the nature and extent of retrofitting required to bring these plants to acceptable levels of safety if they are not already at these levels. Phase I of SEP involved Group I plants, where Phase II involves Group II plants, consisting of San Onofre 1, La Crosse, Big Rock Point, Yankee Rowe, and Haddam Neck. The review for seismic requalification of SEP Group II plants will be performed by two teams. One team consisting of NRC staff personnel and NRC consultants from Lawrence Livermore National Laboratory (LLNL) will evaluate the Group II plants' structures. A second team consisting of NRC staff personnel and NRC consultants from EG&G Idaho, Inc., will evaluate the Group II plants' piping, mechanical, and electrical equipment important to safety. This audit plan provides a description of how the SEP seismic requalification of La Crosse piping, mechanical, and electrical equipment important to safety will be reviewed.

II. Scope

The scope of review for the SEP seismic re-evaluation program will include the systems and components (including emergency power supply and distribution, instrumentation, and actuation systems) with the following functions:

1. The reactor coolant pressure boundary as well as the core and vessel internals. This should also include those portions of the steam and feedwater system extending from and including the secondary side of the steam generator up to and including the outermost containment isolation valve and connected piping for

1. NUREG/CR-0098, "Development of Criteria for Seismic Review of Selected Nuclear Power Plants," N. M. Newmark and W. J. Hall, May 1978.

2. Standard Review Plan, Sections 3.2, 3.7, 3.8, 3.9, 3.10.

3. Regulatory Guides, 1.29, 1.48, 1.60, 1.61, 1.89, 1.92, 1.100, 1.124, 1.130.

4. ANSI/IEEE Standard 344-1975.

5. ASME Boiler and Pressure Vessel Code Section III, 1980 Edition or subsequent.

6. AISC, "Manual of Steel Construction," Eighth Edition.

The intent of Phase II of SEP is to demonstrate that the structural integrity of the systems and components being re-evaluated will not be impaired when subjected to a postulated Safe Shutdown Earthquake (SSE) in combination with other normal design loadings. As a minimum, component primary stresses must be evaluated using current criteria provided in the above standards for Level D (faulted) service limits.

IV. <u>Review Procedures</u>

A. <u>General</u>

The review team (NRC and NRC consultants) will perform the review effort parallel with the licensee's seismic re-evaluation efforts. A minimum of three working level meetings among the review team, licensee, and licensee's consultants are anticipated. This method of review has been selected in order to expedite the review. The working level meetings will permit an exchange of information which will minimize formal written communication, thus expediting the program. One of the meetings will be conducted at the plant so the review team can perform a field inspection of the equipment being re-evaluated.

1. Detailed presentation of seismic re-evaluation program plan by licensee or licensee's consultants.[a]

2. Discussion and resolution of concerns which the review team has with the program plan.[a]

3. Presentation of licensee's progress towards completion of seismic re-evaluation program by licensee.

4. Presentation of anticipated schedule for completing program by licensee.

5. Summary presentation of seismic re-evaluation analyses results (include identification of systems and components which require retrofitting) by licensee.

6. Detailed review of completed seismic re-evaluation analyses for selected systems and equipment (include detailed review of required retrofits).

7. Exit briefing identifying acceptable areas of review and areas of concern requiring additional information to resolve by review team.

For the meeting at the plant, the following agenda is anticipated:

1. Presentation of licensee's progress towards completion of seismic re-evaluation program by licensee.

2. Presentation of anticipated schedule for completing program by licensee.

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a. Required at initial meeting only.

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VI. Review Schedule

The anticipated schedule for completing Phase II of SEP for La Crosse nuclear power plant is as follows:

1.	First working meeting	Not scheduled
2.	Plant visit	Not Scheduled
3.	Final working meeting	Not Scheduled
4.	Complete TER	09-30-82

c. If static equivalent method is used:

(1) Is justification provided for performing a static equivalent analysis?

(2) How was required level of input determined?

3. Has the piping system been properly modeled?

a. Have valves been properly modeled including any eccentricity?

b. Has adequate mass point spacing been utilized?

c. Are adjacent element length ratios reasonable?

d. Have all significant branch piping systems been included?

e. Have all supports been specified with correct imposed loads (if any), direction and stiffness?

f. Have supports with significant nonlinear characteristics been properly handled?

g. Have correct pipe sizes, geometry, thicknesses, and uniform weights been specified?

h. Have correct design and operating pressure and temperature data been specified?

4. Has the piping system been evaluated against proper criteria?

a. Has a proper minimum thickness check been performed?

b. Have excessive deflections been considered?

A-6

(3) Is spectra properly broadened?

(4) Do system frequencies straddle any peaks?

(5) How were directional components of input applied (combined)?

c. If time history method is used:

(1) Is sufficient system response achieved?

(2) Is an adequate time step utilized?

(3) Is proper damping utilized?

(4) How were directional components of input applied (combined)?

d. If testing was used for requalification:

(1) What type of test was performed?

(2) What justification is provided for the type of test used?

(3) How were system natural frequencies determined?

(4) How was the required response spectra (RRS) determined?

(5) How does the test response spectra (TRS) compare to the RRS?

(6) What g level was used in the test?

III. Analysis Audit Format (Electrical Equipment)

1. Is the equipment rigid or flexible?

   a. How were the natural frequencies determined?

   b. If flexible, is its response single-directional or multi-directional?

   c. If flexible, is its response at one predominant frequency or at several frequencies?

2. What type of analysis was performed?

   a. Static g level

      (1) How was required level of input determined?

   b. If response spectra method is used:

      (1) Is correct spectra and damping utilized?

      (2) Is sufficient system response achieved?

      (3) Is spectra properly broadened?

      (4) Do system frequencies straddle any peaks?

      (5) How were directional components of input applied (combined)?

   c. If time history method is used:

      (1) Is sufficient system response achieved?

4. Has the system been properly modeled?

   a. Has adequate mass point spacing and distribution been used?

   b. Have all supports and boundary conditions, including anchor bolts, been properly modeled?

   c. Have significant nonlinear effects been properly handled?

5. Has the system been evaluated against proper criteria?

   a. Have the proper load combinations been analyzed?

   b. Have proper stress intensities been evaluated?

   c. Have deflections been considered?

   d. Have proper allowable stress limits been selected?

   e. How were computer output responses combined (directional and modal)?

APPENDIX B

REEVALUATION GUIDELINE FOR SEP GROUP II PLANTS
(EXCLUDING STRUCTURES)

B-1

REEVALUATION GUIDELINE
FOR
SEP GROUP II PLANTS
(EXCLUDING STRUCTURES)

INTRODUCTION

In support of NRC's Systematic Evaluation Program (SEP) for Group II Plants, the following Reevaluation Criteria have been established. These criteria include recommended load combinations with allowable stresses and/or loads for piping systems, component supports, concrete attachments, and equipment. These criteria are based on linear elastic analyses having been performed. The acceptance criteria are generally based on the ASME Code. For situations not covered by these criteria, (i.e. items constructed of cast iron) compatible criteria shall be developed by the licensee and will be reviewed on a case-by-case basis. The licensee is requested to justify major deviations in criteria which appear less conservative than those specified herein. 1

DEFINITIONS

Code = ASME Boiler and Pressure Vessel Code, Section III, "Nuclear Power Plant Components," 1980 Edition, Winter 1980 Addenda.

σ_m = General membrane stress. This stress is equal to the average stress across the solid section under consideration, excludes discontinuities and concentrations, and is produced only by mechanical loads.

σ_b = Bending stress. This stress is equal to the linear varying portion of the stress across the solid section under consideration, excludes discontinuities and concentrations, and is produced only by mechanical loads. 1

P_D = Design or maximum operating pressure loads and design mechanical loads.

SSI = Inertial loads due to Safe Shutdown Earthquake (SSE) and design mechanical loads where applicable.

T = Loads due to thermal expansion of attached pipe (constraint of free end displacement).

W = Loads due to weight effects.

AM = Loads due to SSE anchor movement effects.

S_{bk} = Critical buckling stress.

S_m = Allowable stress intensity at temperature listed in ASME Code.

S_y = Yield strength at temperature listed in ASME Code.

S_u = Ultimate tensile strength at temperature listed in ASME Code.

σ_ℓ = Local membrane stress. This stress is the same as σ_m except that it includes the effect of discontinuities. 1

S = ASME Code Class 2 allowable stress value. The allowable stress shall correspond to the metal temperature at the section under consideration.

P_m = General Primary Membrane Stress Intensity. This stress intensity is derived from the average value across the thickness of a section of the general primary stresses produced by design internal pressure and other specified Design Mechanical Loads, but excluding all secondary and peak stresses. Averaging is to be applied to the stress components prior to determination of the stress intensity values. 1

P_{ℓ} = Local Membrane Stress Intensity. This stress intensity is the same as P_m except that it includes the effects of discontinuities.

P_b = Primary Bending Stress Intensity. This stress intensity is derived from the linear varying portion of stresses across 1
the solid section under consideration produced design pressure and other specified design mechanical loads. Secondary and peak stresses are not included.

SPECIAL LIMITATIONS

1. Critical buckling loads (stresses) must be determined taking into account combined loading (i.e., axial, bending, and shear), initial imperfections, residual stresses, inelastic deformation, and boundary conditions. Both gross and local buckling must be evaluated. Critical buckling loads (stresses) shall be determined using accepted methods such as those contained in NASA Plates and Shells Manual or ASME Code Case N-284. 1

2. Where stresses exceed material yield strength, it shall be demonstrated that brittle failures and detrimental cyclic effects are precluded, and that dynamic analysis assumptions are not nonconservatively affected. Where significant cyclic effects are identified, it shall be demonstrated that the structure or component is capable of withstanding ten full peak deformation cycles.

3. Where results of analysis indicate that the allowable stresses of the original construction code are exceeded in any of the load combinations specified herein, it shall be demonstrated that the in-situ item was designed and fabricated using rules compatible with those required for the appropriate ASME Code Class (Subsection NX2000,

4000, 5000, and 6000). In cases where compatibility with the appropriate ASME Code Subsections was not substantially achieved, appropriate reductions in these limits shall be established, justified, and applied.

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ACCEPTANCE CRITERIA FOR PIPING

Using Code[a] Class 2 analytical procedures [Equation (9), NC-3653.1], the following stresses are not to be exceeded for the specified piping: 1

Class 1: $P_m + P_b = |W + P_D| + |SSE| \leq 1.8\ S$ 1

Class 2: $P_m + P_b = |W + P_D| + |SSE| \leq 2.4\ S$

The effects of thermal expansion must meet the requirements of Equation (10) or (11) of NC-3653, including moment effects of anchor displacements due to SSE if anchored displacement effects are omitted from Equation (9) of NC-3653. Class 1 analytical procedures (NB-3600) can also be utilized if appropriate allowable stresses specified in NB-3650 are used. 1

Branch lines shall be analyzed including the inertial and displacement input due to the response of the piping to which it is attached at the attachment point.

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a. The references to ASME Code equation and paragraph numbers on this page correspond to the 1980 edition of the code, 1981 winter addenda. This was done in order to avoid confusion introduced by the initial 1980 edition of the code which renumbered the equations differently from past and present editions of the code. Equation numbers presented on this page reflect common nomenclature utilized in the nuclear industry. 1

ACCEPTANCE CRITERIA FOR CLASS 1 COMPONENT SUPPORTS 1

Imposed Load Combinations	Acceptance Criteria[a] Linear	Acceptance Criteria[a] Plate and Shell[b]
The higher of: $\left\|W + P_D\right\|$ or $\left\|W + P_D + T\right\|$	Code Subsection NF Design, Level A, and Level B Limits	$P_m \le 1.0\ S_m$ $P_\ell + P_b \le 1.5\ S_m$ 1
The higher of: $\left\|W + P_D\right\| + \left\|SSE\right\| + \left\|AM\right\|$ or $\left\|W + P_D + T\right\| + \left\|SSE\right\| + \left\|AM\right\|$	Code Subsection NF Level D Limits	$P_m \le 1.5\ Sm$ or $1.2\ Sy$[c] not to exceed 0.7 Su $P_\ell + P_b$ 2.25 Sm or 1.85 Sy [c] not to exceed 1.05 Su 1

In addition to the above criteria, the allowable buckling stress shall be limited to 2/3 S_{bk}, where S_{bk} is determined in accordance with Special Limitation 1.

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a. These load combinations shall be used in lieu of those specified in ASME Code Subsection NF. In addition, for brittle types of material not specified in the Code, appropriate stress intensification factors for notches and stress discontinuities shall be applied in the analysis.

b. The 1.5 Sm value from NB 3221 on which these are based (Code Appendix F 1323.1) shall be limited by Code Section NB 3221.3. 1

c. Use larger of.

ACCEPTANCE CRITERIA FOR CLASS 2 COMPONENT SUPPORTS

Imposed Load Combinations	Acceptance Criteria[a] Linear	Plate and Shell
The higher of: $\lvert W + P_D \rvert$ or $\lvert W + P_D + T \rvert$	Code Subsection NF Design, Level A, and Level B Limits	$\sigma_\ell \leq 1.0\ S$ $\sigma_\ell + \sigma_b \leq 1.5\ S$
The higher of: $\lvert W + P_D \rvert + \lvert SSE \rvert + \lvert AM \rvert$ or $\lvert W + P_D + T \rvert + \lvert SSE \rvert + \lvert AM \rvert$	Code Subsection NF Level D Limits	$\sigma_\ell \leq 1.5\ S$ or $0.4\ S_u$ (b) $\sigma_\ell + \sigma_b \leq 2.25\ S$ or $0.6\ S_u$ (b)

In addition to the above criteria, the allowable buckling stress shall be limited to 2/3 S_{bk}, where S_{bk} is determined in accordance with Special Limitation 1.

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a. These load combinations shall be used in lieu of those specified in ASME Code Subsection NF. In addition, for brittle types of material not specified in the Code, appropriate stress intensification factors for notches and stress discontinuities shall be applied in the analysis.

b. Use lesser of.

ACCEPTANCE CRITERIA FOR CONCRETE ATTACHMENTS

I. Concrete Expansion Anchor Bolts[a]

Load Combinations: Same as for component supports.

Acceptance Criteria:[b]

Wedge type: 1/4 ultimate as specified by manufacturer.

Shell type: 1/5 ultimate as specified by manufacturer.

II. Grouted Bolts: Replace[a],[b],[c]

I'' oncrete Embedded Anchors[a]

Load Combinations: Same as for component supports.

Acceptance Criteria[b]: 0.7 S_u

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a. Base plate flexibility effects must be considered.

b. Both pullout and shear loads must be considered in combined loading situations.

c. Unless stresses in the bolts and structure to which they are attached are shown to be sufficiently low to preclude concrete/grout/steel interface bond failures. Load combinations are the same as those for component supports.

ACCEPTANCE CRITERIA FOR CLASS 1 MECHANICAL EQUIPMENT

Component	Loading Combination[b]	Criteria[d] [g]
Pressure vessels and heat-exchangers	$\lvert W + P_D \rvert + \lvert SSE \rvert + \lvert \text{Nozzle Loads} \rvert$	$P_m \leq 2.4\ S_m$ or $0.7\ S_u$ [e] $(P_m \text{ or } P_\ell) + P_b \leq 3.6\ S_m$ or $1.05\ S_u$ [e]
Active pumps and other mechanical components[a],[c],[d]	$\lvert W + P_D \rvert + \lvert SSE \rvert + \lvert \text{Nozzle Loads} \rvert$	$P_m \leq 1.2\ S_m$ or S_y [f] $(P_m \text{ or } P_\ell) + P_b \leq 1.8\ S_m$ or $1.5\ S_y$ [f]
Inactive pumps and other mechanical components[c]	$\lvert W + P_D \rvert + \lvert SSE \rvert + \lvert \text{Nozzle Loads} \rvert$	$P_m \leq 2.4\ S_m$ or $0.7\ S_u$ [e] $(P_m \text{ or } P_\ell) + P_b \leq 3.6\ S_m$ or $1.05\ S_u$ [e]
Active valves[a],[c],[d]	$\lvert W + P_D \rvert + \lvert SSE \rvert + \lvert \text{Nozzle Loads} \rvert$	$P_m \leq 1.2\ S_m$ or S_y [f] $(P_m \text{ or } P_\ell) + P_b \leq 1.8\ S_m$ or $1.5\ S_y$ [f]
Inactive valves[c]	$\lvert W + P_D \rvert + \lvert SSE \rvert + \lvert \text{Nozzle Loads} \rvert$	$P_m \leq 2.4\ S_m$ or $0.7\ S_u$ [e] $(P_m \text{ or } P_\ell) + P_b \leq 3.6\ S_m$ or $1.05\ S_u$ [e]
Bolt stress shall be limited to:		Tension = Sy or $0.7\ S_u$[e] Shear = 0.6 Sy or $0.42\ S_u$[e]

a. Active pumps, valves, and other mechanical components (e.g., CRDs) are defined as those that must perform a mechanical motion to accomplish a system safety function.

b. Nozzle loads shall include all piping loads (including seismic and thermal anchor movement effects) transmitted to the component during the SSE.

c. Scope and evaluation of pumps and valves are to be in accordance with NB 3411, NB 3412, and NB 3546 of the Code, including seismic and thermal anchor movement effects.

d. For active mechanical equipment contained in safe shut down systems, it shall be demonstrated that deformation induced by the loading on these pumps, valves and other mechanical components (e.g., CRDs) do not introduce detrimental effects which would preclude function of this equipment following a postulated SSE event. For valve operators integrally attached to valve bodies, binding can be considered precluded if stresses in the valve body and operator housing and supports are shown to be less than yield. In these evaluations, all loads (including seismic and thermal anchor movement effects) shall be included. 1

e. Use lesser of two values.

f. Use greater of two values.

g. The 1.5 Sm value from NB 3221 on which these are based (Code Appendix F 1323.1) shall be limited by Code Section NB 3221.3.

ACCEPTANCE CRITERIA FOR CLASS 2 MECHANICAL EQUIPMENT

Component	Loading Combination[b]	Criteria[d]
Pressure vessels	$\lvert W + P_D \rvert + \lvert SSE \rvert + \lvert \text{Nozzle Loads} \rvert$	$\sigma_m \leq 2.0\ S$
and heat-exchangers		$(\sigma_m \text{ or } \sigma_\ell) + \sigma_b \leq 2.4\ S$
Active pumps and other mechanical components[a],[c],[d]	$\lvert W + P_D \rvert + \lvert SSE \rvert + \lvert \text{Nozzle Loads} \rvert$	$\sigma_m \leq 1.5\ S$
		$(\sigma_m \text{ or } \sigma_\ell) + \sigma_b \leq 1.8\ S$
Inactive pumps and other mechanical components[c]	$\lvert W + P_D \rvert + \lvert SSE \rvert + \lvert \text{Nozzle Loads} \rvert$	$\sigma_m \leq 2.0\ S$
		$(\sigma_m \text{ or } \sigma_\ell) + \sigma_b \leq 2.4\ S$
Active valves[a],[c],[d]	$\lvert W + P_D \rvert + \lvert SSE \rvert + \lvert \text{Nozzle Loads} \rvert$	$\sigma_m \leq 1.5\ S$
		$(\sigma_m \text{ or } \sigma_\ell) + \sigma_b \leq 1.8\ S$
Inactive valves[c]	$\lvert W + P_D \rvert + \lvert SSE \rvert + \lvert \text{Nozzle Loads} \rvert$	$\sigma_m \leq 2.0\ S$
		$(\sigma_m \text{ or } \sigma_\ell) + P_b \leq 2.4\ S$
Bolt stresses shall be limited to:		Tension = Sy or 0.7 S_u[e]
		Shear = 0.6 Sy or 0.42 S_u[e]

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a. Active pumps, valves, and other mechanical components (e.g., CRDs) are defined as those that must perform a mechanical motion to accomplish a system safety function.

b. Nozzle loads shall include all piping loads (including seismic and thermal anchor movement effects) transmitted to the component during the SSE.

c. Scope and evaluation of pumps and valves are to be in accordance with NC 3411, NC 3412, and NC 3521 of the Code, including seismic and thermal anchor movement effects.

d. For active mechanical equipment contained in safe shut down systems, it shall be demonstrated that deformation induced by the loading on these pumps, valves and other mechanical components (e.g., CRDs) do not introduce detrimental effects which would preclude function of this equipment following a postulated SSE event. For valve operators integrally attached to valve bodies, binding can be considered precluded if stresses in the valve body and operator housing and supports are shown to be less than yield. In these evaluations, all loads (including seismic and thermal anchor movement effects) shall be included. 1

e. Use lesser of two values.

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ACCEPTANCE CRITERIA FOR TANKS

Load Combinations: $|W + P_D| + |SSE|$
+ |Dynamic Fluid Pressure Loads|[a]

Acceptance Criteria: Smaller of S_y or 0.7 S_u. In addition, the allowable buckling stress shall be limited to 2/3 S_{bk}, where S_{bk} is determined in accordance with Special Limitation 1.

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a. Dynamic fluid pressure shall be considered in accordance with accepted and appropriate procedures; e.g., USAEC TID-7024. Horizontal and vertical loads shall be determined by appropriately combining the loads due to vertical and horizontal earthquake excitation considering that the loads are due to pressure pulses within the fluid. These loads shall also be applied, in combination with other loads, in tank support evaluations.