February 12, 1982

Docket No. 50-255
LS05-82- 02-061

SEE Repts. #8202190306

Mr. David J. VandeWalle
Nuclear Licensing Administrator
Consumers Power Company
1945 W. Parnall Road
Jackson, Michigan 49201

Dear Mr. VandeWalle:

SUBJECT: SYSTEMATIC EVALUATION PROGRAM TOPIC III-7.B, "DESIGN CODES, DESIGN CRITERIA AND LOAD COMBINATIONS" - PALISADES

Enclosed is our final evaluation of SEP Topic III-7.B for Palisades. This evaluation supersedes the draft evaluation sent to you on November 16, 1981. You did not provide any comments regarding the content of the draft evaluation.

This evaluation identifies areas of the codes used in the design of your facility where changes have occurred to decrease safety margins. It also identifies loads applicable to some or all of the structures at Palisades which have increased in magnitude. 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. This assessment may be revised in the future if your facility design is changed, or if NRC criteria relating to this subject are modified before the integrated safety assessment is completed.

Add: Gary Staley
Ted Michaels

SE04
5/1

DSu USE EX (04)

Sincerely,

Thomas V. Wambach, Project Manager
Operating Reactors Branch No. 5
Division of Licensing

Enclosure:
As stated

cc w/enclosure:
See next page

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DATE	2/9/82	2/8/82	2/9/82	2/11/82	2/11/82	2/11/82	

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SYSTEMATIC EVALUATION PROGRAM
TOPIC III-7.B

PALISADES

TOPIC: III-7.B, Design Codes, Design Criteria and Loading Combinations

I. INTRODUCTION

SEP plants were generally designed and constructed during the time span from the late 1950's to late 1960's. They were designed according to criteria and codes which differ from those accepted by the NRC for new plants.

The purpose of this topic is to assess the safety margins existing in Category I structures as a result of changes in design codes and criteria.

II. REVIEW GUIDELINES

The current licensing criteria which governs the safety issue in this topic is 10 CFR 50, Appendix A, GDC 1, 2, and 4 as interpreted by Standard Review Plan 3.8.

III. RELATED SAFETY TOPICS

The following SEP topics are related to III-7.B:

1. II-3.B, Flooding Potential and Protection Requirements
2. III-2, Wind and Tornado Loadings
3. III-3.A, Effects of High Water Level on Structures
4. III-4.A, Tornado Missiles
5. III-5.A, Effects of Pipe Breaks Inside Containment
6. III-5.B, Effects of Pipe Breaks Outside Containment
7. III-6, Seismic Design Considerations
8. VI-2.D, Mass and Energy Release for Postulated Pipe Break Inside Containment

IV. EVALUATION

The evaluation is based on a Technical Evaluation Report (TER) prepared by the Franklin Research Center (FRC) in conjunction with the NRC staff through contract. The report is entitled "Design Codes, Design Criteria and Loading Combinations" and is attached to this Safety Evaluation Report as Enclosure (1).

We have compared structural design codes employed in the design of Category I structures at Palisades to present codes. This was done through generic code versus code comparison without investigating specifically how the original code was applied to the Palisades design; however, after reviewing drawings of structures at Palisades, we concluded that certain portions of the codes were not applicable because the types of structures to which the codes are referring were non-existent. We have compared the loads and loading combinations employed in the design of Palisades as described in the FSAR to those required today.

A result of these comparisons is that a number of code changes could potentially impact significantly margins of safety (denoted by scale A and Ax in Enclosure 1). This can be attributed to several factors such as:

1. New codes have imposed stricter limitations than old,

2. New codes have included sections governing design of certain types of structures which were not included in the older codes,

3. Design loads required today were not included in the plant design, and

4. Certain load combinations judged to be significant were not included in plant design.

In Enclosure (1), some items have been judged to potentially impact margins of safety regarding the containment as a result of comparing ACI 318-63 to ASME BPV Section 3, Division 2. These items are discussed in Section 11 of the report. One item, cc-3421.5 of the BPV Code, Section III, Division 2, 1980, is not significant based upon the additional information contained in Enclosure (2).

The code changes of concern from Enclosure (1) are: (See next page)

Structural Elements to be Examined	Code Change Affecting These Elements: New Code	Old Code
Beams	AISC 1980	AISC 1963
a. Composite Beams		
1. Shear connectors in composite beams	1.11.4	1.11.4
2. Composite beams or girders with formed steel deck	1.11.5	--*
b. Hybrid Girders		
Stress in flange	1.10.6	1.10.6
Compression Elements	AISC 1980	AISC 1963
With width-to-thickness ratio higher than specified in 1.9.1.2	1.9.1.2 and Appendix C	1.9.1
Tension Members	AISC 1980	AISC 1963
When load is transmitted by bolts or rivets	1.14.2.2	--
Connections	AISC 1980	AISC 1963
a. Beam ends with top flange coped, if subject to shear	1.5.1.2.2	--
b. Connections carrying moment or restrained member connection	1.15.5.2 1.15.5.3 1.15.5.4	--

*Double dash (--) indicates that no provisions were provided in the older code.

Structural Elements to be Examined	Code Change Affecting These Elements New Code	Old Code
Members Designed to Operate in an Inelastic Regime	AISC 1980	AISC 1963
Spacing of lateral bracing	2.9	2.8
Short Brackets and Corbels having a shear span-to-depth ratio of unity or less	ACI 349-76 11.13	ACI 318-63 --
Shear Walls used as a primary load-carrying member	ACI 349-76 11.16	ACI 318-63 --
Precast Concrete Structural Elements, where shear is not a member of diagonal tension	ACI 349-76 11.15	ACI 318-63 --
Concrete Regions Subject to High Temperatures	ACI 349-76	ACI 318-63
Time-dependent and position-dependent temperature variations	Appendix A	--
Columns with Spliced Reinforcement	ACI 349-76	ACI 318-63
subject to stress reversals; f_y in compression to 1/2 f_y in tension	7.10.3	805
Steel Embedments used to transmit load to concrete	ACI 349-76 Appendix B	ACI 318-63 --
Containment and Other Elements, transmitting In-plane shear	B&PV Code Section III, Div. 2, 1980 CC-3421.5	ACI 318-63 --
Region of shell carrying concentrated forces normal to the shell surface (see case study 13 for details)	B&PV Code, Section III, Div. 2, 1980 CC-3421.6	ACI 318-63 1707

Structural Elements to be Examined	Code Change Affecting These Elements New Code	Old Code
Region of shell under torsion	B&PV Code Section III, Div. 2, 1980 CC-3421.7	ACI 318-63 921
Elements Subject to Biaxial Tension	B&PV Code, Section III, Div. 2, 1980 CC-3532.1.2	ACI 318-63 —
Brackets and Corbels	B&PV Code, Section III, Div. 2, 1980 CC-3421.8	ACI 318-63 —

Section 10 of Enclosure (1) address load and load combination changes which occured as a result of criteria changes and identifies specific plant structures for which various loads and load combinations may be significant. Based upon a lack of detailed information on the stress results for loads and load combinations used during design of structures at Palisades, these loads and load combinations may be potentially significant.

Enclosure (2) provides details of a reanalysis of the containment for combined seismic and LOCA loadings which was performed by our contractor, Lawrence Livermore Laboratory. It is concluded that the containment will perform its intended function if subjected to combined seismic and LOCA loads.

V. CONCLUSIONS

We conclude that after comparing design codes, criteria, loads and load combinations, a number of changes have occurred which could potentially impact margins of safety. These changes are identified above. These differences between plant design and current licensing criteria should be resolved as follows:

1. Review Seismic Category 1 Structures at Palisades to determine if any of the structural elements for which a concern exists are a part of the facility design of Palisades. For those that are, assess the impact of the code changes on margins of safety on a plant specific basis, and

2. Examine on a sampling basis the margins of safety of Seismic Category 1 structures for loads and load combinations not covered by another SEP topic and denoted by Ax in Enclosure (1). (The load tables should be reviewed to assure their technical accuracy concerning applicability of the loads for each of the structures and their significance.)

Regarding the ability of the Palisades containment to resist the seismic and LOCA loads described in Enclosure (2), we conclude that the containment will perform its intended function if subjected to combined seismic and LOCA loads.

SYSTEMATIC EVALUATION PROGRAM
TOPIC III-7.B

PALISADES

TOPIC: III-7.B, Design Codes, Design Criteria and Loading Combinations

I. INTRODUCTION

SEP plants were generally designed and constructed during the time span from the late 1950's to late 1960's. They were designed according to criteria and codes which differ from those accepted by the NRC for new plants.

The purpose of this topic is to assess the safety margins existing in Category I structures as a result of changes in design codes and criteria.

II. REVIEW GUIDELINES

The current licensing criteria which governs the safety issue in this topic is 10 CFR 50, Appendix A, GDC 1, 2, and 4 as interpreted by Standard Review Plan 3.8.

III. RELATED SAFETY TOPICS

The following SEP topics are related to III-7.B:

1. II-3.B, Flooding Potential and Protection Requirements
2. III-2, Wind and Tornado Loadings
3. III-3.A, Effects of High Water Level on Structures
4. III-4.A, Tornado Missiles
5. III-5.A, Effects of Pipe Breaks Inside Containment
6. III-5.B, Effects of Pipe Breaks Outside Containment
7. III-6, Seismic Design Considerations
8. VI-2.D, Mass and Energy Release for Postulated Pipe Break Inside Containment

IV. EVALUATION

The evaluation is based on a Technical Evaluation Report (TER) prepared by the Franklin Research Center (FRC) in conjunction with the NRC staff through contract. The report is entitled "Design Codes, Design Criteria and Loading Combinations" and is attached to this Safety Evaluation Report as Enclosure (1).

We have compared structural design codes employed in the design of Category I structures at Palisades to present codes. This was done through generic code versus code comparison without investigating specifically how the original code was applied to the Palisades design; however, after reviewing drawings of structures at Palisades, we concluded that certain portions of the codes were not applicable because the types of structures to which the codes are referring were non-existent. We have compared the loads and loading combinations employed in the design of Palisades as described in the FSAR to those required today.

A result of these comparisons is that a number of code changes could potentially impact significantly margins of safety (denoted by scale A and Ax in Enclosure 1). This can be attributed to several factors such as:

1. New codes have imposed stricter limitations than old,

2. New codes have included sections governing design of certain types of structures which were not included in the older codes,

3. Design loads required today were not included in the plant design, and

4. Certain load combinations judged to be significant were not included in plant design.

In Enclosure (1), some items have been judged to potentially impact margins of safety regarding the containment as a result of comparing ACI 318-63 to ASME BPV Section 3, Division 2. These items are discussed in Section 11 of the report. One item, cc-3421.5 of the BPV Code, Section III, Division 2, 1980, is not significant based upon the additional information contained in Enclosure (2).

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Brackets and Corbels	B&PV Code, Section III, Div. 2, 1980 CC-3421.8	ACI 318-63 —

Section 10 of Enclosure (1) address load and load combination changes which occured as a result of criteria changes and identifies specific plant structures for which various loads and load combinations may be significant. Based upon a lack of detailed information on the stress results for loads and load combinations used during design of structures at Palisades, these loads and load combinations may be potentially significant.

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2. Examine on a sampling basis the margins of safety of Seismic Category 1 structures for loads and load combinations not covered by another SEP topic and denoted by Ax in Enclosure (1). (The load tables should be reviewed to assure their technical accuracy concerning applicability of the loads for each of the structures and their significance.)

Regarding the ability of the Palisades containment to resist the seismic and LOCA loads described in Enclosure (2), we conclude that the containment will perform its intended function if subjected to combined seismic and LOCA loads.

ENCLOSURE 1

(DRAFT)

TECHNICAL EVALUATION REPORT

DESIGN CODES, DESIGN CRITERIA, AND LOADING COMBINATIONS

CONSUMERS POWER COMPANY
PALISADES NUCLEAR POWER STATION

NRC DOCKET NO. 50-255
NRC TAC NO. 41502
NRC CONTRACT NO. NRC-03-79-118

FRC PROJECT C5257
FRC ASSIGNMENT 11
FRC TASK 324, SEP Topic III-7.B

Prepared by

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Lead NRC Engineer: D. Persinko

January 20, 1982 (Rev. 3)

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(DRAFT)

TECHNICAL EVALUATION REPORT

DESIGN CODES, DESIGN CRITERIA, AND LOADING COMBINATIONS

CONSUMERS POWER COMPANY
PALISADES NUCLEAR POWER STATION

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Lead NRC Engineer: D. Persinko

January 20, 1982 (Rev. 3)

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CONTENTS

CONTENTS (Cont.)

CONTENTS (Cont.)

FOREWORD

This Technical Evaluation Report was prepared by Franklin Research Center under a contract with the U.S. Nuclear Regulatory Commission (Office of Nuclear Reactor Regulation, Division of Operating Reactors) for technical assistance in support of NRC operating reactor licensing actions. The technical evaluation was conducted in accordance with criteria established by the NRC.

PALISADES SER ADDENDA - SEP TOPIC III-7.B

To be inserted before Section 10.2 in FRC report:

Current criteria require consideration during plant design of thirteen load combinations for most structures, as shown in the load combination tables. These specific requirements were not in effect at the time when SEP plants were designed. Consequently, other sets of load-combinations were used. In comparing actual and current criteria, an attempt was made to match each of the load combinations actually considered to its nearest counterpart under present requirements. For example, consider a plant where the SSE was addressed in combination with other loads, but not in combination with the effects of a LOCA (load combination 13). The load combination tables would reflect this by showing that load case 9 was addressed, but that load case 13 was not. If six load cases were considered, only six (nearest counterpart) load cases are indicated in the table---not partial fulfillment of all 13.

The scale rankings assigned to loads and load combinations in tables are intended as an appraisal of plant status, with respect to demonstration of compliance with current design criteria, based on information available to the NRC prior to the inception of the SEP review. A number of structurally related SEP topics review some loads and load combinations in detail based upon current calculational methods. In order that a consistant basis for the tables be maintained, they are based upon load combination considered in the original design of the facility, or in the case of facility modifications, they are based upon the combinations used in the design of the modification. Loads which were not included in the original design or have increased in magnitude and have not been specifically addressed in another SEP topic should be addressed by the licensee.

1. INTRODUCTION

For the Seismic Category I buildings and structures at the Palisades Nuclear Power Station, this report provides a comparison of (a) the structural design codes and loading criteria used in the design with (b) the corresponding codes and criteria used for current licensing of new plants.

The objective of the code comparison review is to identify deviations in design criteria from current criteria, and to assess the effect of these deviations on margins of safety, as they were originally perceived and as they would be perceived today.

The work was conducted as part of the Nuclear Regulatory Commission's (NRC) Systematic Evaluation Program (SEP) and provides technical assistance for Topic III-7.B, "Design Codes, Design Criteria, and Load Combinations." The report was prepared at the Franklin Research Center under NRC Contract No. NRC-03-79-118.

2. BACKGROUND

With the development of nuclear power, provisions addressing facilities for nuclear applications were progressively introduced into the codes and standards to which plant building and structures are designed. Because of this evolutionary development, older nuclear power plants conform to a number of different versions of these codes, some of which have since undergone considerable revision.

There has likewise been a corresponding development of other licensing criteria, resulting in similar non-uniformity in many of the requirements to which plants have been licensed. With this in mind, the NRC undertook an extensive program to evaluate the safety of 11 older plants (and eventually all plants) to a common set of criteria. The program, entitled the Systematic Evaluation Program (SEP), employs current licensing criteria (as defined by NRC's Standard Review Plan) as the common basis for these evaluations.

To make the necessary determinations, the NRC is investigating, under the SEP, 137 topics spanning a broad spectrum of safety-related issues. The work reported herein constitutes the results of part* of the investigation of one of these topics, Topic III-7.B, "Design Codes, Design Criteria, and Load Combinations."

This topic is charged with the comparison of structural design criteria in effect in the late 1950's to the late 1960's (when the SEP plants were constructed) with those in effect today. Other SEP topics also address other aspects of the integrity of plant structures. All these structurally oriented tasks, taken together, will be used to assess the structural adequacy of the SEP plants with regard to current requirements. The determinations with respect to structural safety will then be integrated into an overall SEP evaluation encompassing the entire spectrum of safety-related topics.

*The report addresses only the Palisades plant.

3. REVIEW OBJECTIVES

The broad objective of the NRC's Systematic Evaluation Program (SEP) is to reassess the safety of 11 older nuclear power plants in accordance with the intent of the requirements governing the licensing of current plants, and to provide assurance, possibly requiring backfitting, that operation of these plants conforms to the general level of safety required of modern plants.

Task III-7.B of the SEP effort seeks to compare actual and current structural design criteria for the major civil engineering structures at each SEP plant site, i.e., those important to shutdown, containment, or both, and therefore designated Seismic Category I structures. The broad safety objective of SEP Task III-7.B is (when integrated with several other interfacing SEP topics) to assess the capability of all Seismic Category I structures to withstand all design conditions stipulated by the NRC, at least to a degree sufficient to assure that the nuclear power plant can be safely shut down under all circumstances.

The objective of FRC's present effort under Task III-7.B is to provide, through code comparisons, a rational basis for making the required technical assessments, and a tool which will assist in the structural review.

Finally, the objective of the present report is to present the results of FRC's Task III-7.B work as they relate to the Palisades Nuclear Power Station.

4. SCOPE

FRC was asked to review the provisions of the structural codes and standards used for design of SEP plant Seismic Category I civil engineering structures* and compare them with the corresponding provisions governing current licensing practice. The review includes the containment and all Category I structures within and exterior to it. Explicit among the criteria to be reviewed are loads and loading combinations postulated for these structures.

To carry out the review, FRC was assigned the following tasks:

1. Identify current design requirements, based on a review of NRC Regulations; 10CFR50.55a, "Codes and Standard"; and the NRC Standard Review Plan (SRP).

2. Review the structural design codes, design criteria, design and analysis procedures, and load combinations (including combinations involving seismic loads) used in the design of all Category I structures as defined in the Final Safety Analysis Report (FSAR) for each SEP plant.

3. Based upon the plant-specific design codes and standards identified in Task 2 and current licensing codes and standards from Task 1, identify plant-specific deviations from current licensing criteria for design codes and criteria.

4. Assess the significance of the identified deviations, performing (where necessary) comparative analyses to quantify significant deviations. Such analyses may be made on typical elements (beams, columns, frames, and the like) and should be explored over a range of parameters representative of plant structures.

5. Prepare a Technical Evaluation Report for each SEP plant including:

    a. comparisons of plant design codes and criteria to those currently accepted for licensing

    b. assessment of the significance of the deviations

    c. results of any comparative stress analyses performed in order to make an assessment of the significance of the code changes upon safety margins

*In general, these are the structures normally examined in licensing reviews under Section 3.8 of the SRP (but note the list at the end of this section of structures specifically excluded from FRC's scope).

d. overall evaluation of the acceptability of structural codes used at each SEP plant.

A number of SEP topics examine aspects of the integrity of the structures composing SEP facilities. Several of these interface with the Task III-7.B effort as shown below:

Topic	Designation
III-1	Classification of Structures, Components, Equipment, and Systems (Seismic and Quality)
III-2	Wind and Tornado Loading
III-3	Hydrodynamic Loads
III-4	Missile Generation and Protection
III-5	Evaluation of Pipe Breaks
III-6	Supports
III-7.A	Inservice Inspection of Structures
III-7.C	Delamination of Prestressed Concrete Structures
III-7.D	Structural Integrity Tests

Because they are covered either elsewhere within the SEP review or within other NRC programs, the following matters are explicitly excluded from the FRC scope:

Mark I torus shell, supports, vents, local region of drywell at vent penetrations	Reviewed in Generic Task A-7.
Reactor pressure vessel supports, steam generator supports, pump supports	Reviewed in Generic Task A-2, A-12.
Equipment supports in SRP 3.8.3	Reviewed generally in Topic III-6, Generic Task A-12.

Other component supports (steel and concrete)	Specific supports have been analyzed in detail in Topic III-6. (Component supports may be included later if items of concern applicable to component supports are found as a result of reviewing the structural codes.)
Testing of containment	Reviewed in Topic III-7.D.
Inservice inspection; quality control/assurance	Should be considered in FRC review only to the extent that it affects design criteria, design allowables. Aspects of inservice inspection are being reviewed in Topics III-7.A and III-3.C
Determination of structures that should be classified Seismic Category I	Not in FRC scope.
Shield walls and subcompartments inside containment	Reviewed in Generic Task A-2.
Masonry walls	Reviewed generically in IE Bulletin.
Seismic analysis	Being reviewed by Lawrence Livermore Laboratory.

5. MARGINS OF SAFETY

There are several bases upon which margins of safety* may be defined and discussed.

The most often used is the margin of safety based on yield strength. This is a particularly useful concept when discussing the behavior of steels, and became ingrained into the engineering vocabulary at the time when steel was the principal metal of engineering structures. In this usage, the margin of safety reflects the reserve capacity of a structure to withstand extra loading without experiencing an incipient permanent change of shape anywhere throughout the structure. Simultaneously, it reflects the reserve load carrying capacity existing before the structure is brought to the limit for which an engineer could be certain the computations (based on elastic behavior of the metal) applied.

This is the conventional use of the term and the meaning which engineers take as intended, unless the term is further qualified to show something else is meant. Thus, if a structure is stated to have a margin of safety of 1.0 under a given set of loads, then it will be generally understood that every load on the structure may be simultaneously doubled without encountering (anywhere) inelastic stresses or deflections. On the other hand, if (under load) a structure has no margin of safety, any increment to any load will cause the structure to experience, in a least one (and possibly more than one) location, some permanent distortion (however small) of its original shape.

However, because the yield strengths of common structural steels are generally well below their ultimate strengths, the engineer knows that in most (but not in all) cases, the structure possesses substantial reserve capacity--beyond his computed margin--to carry additional load.

There are other useful ways, however, to speak of safety margins and these (not the conventional one) are particularly relevant to the aims of the SEP program.

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*Factors of safety (FS) are related to margins of safety (MS) through the relation MS = FS - 1.

One may speak of margins of safety with respect to code allowable limits. This margin reflects the reserve capacity of a structure to withstand extra loading while still conforming to all criteria governing its design.

One may also speak (if it is made clear in advance that this is the intended meaning) of margins of safety against actual failure. Both steel and concrete structures exhibit much higher "margins of safety" on this second basis than is shown by computation of margins of safety based on code allowables.

These latter concepts of "margin of safety" are very significant to the SEP review. Indeed the basic review concept, at least as it relates to structural integrity, cannot be easily defined in any quantitative manner without considering both. The SEP review concept is predicated on the assumption that it is unrealistic to expect that plants which were built to, and were in compliance with, older codes will still conform to current criteria in all respects. The SEP review seeks to assess whether or not plants meet the "intent" of current licensing criteria as defined by the Standard Review Plan (SRP). The objective is not to require that older plants be brought into conformance with all SRP requirements to the letter, but rather to assess whether or not their design is sufficient to provide the general level of safety that current licensing requirements assure.

With respect to aspects of the SEP program that involve the integrity of structures, the SEP review concept can be rephrased in a somewhat more quantitative fashion in terms of these two "margins of safety." Thus, it is not expected or demanded that all structures show positive margins of safety based upon code allowables in meeting all current SRP requirements; but it is demanded that margins of safety based upon ultimate strength are not only positive, but ample. In fact, the critical judgments to be made (for SEP plants) are:

1. to what extent may current code margins be infringed upon.
2. what minimum margin of safety based on ultimate strength must be assured.

The choice of method for Topic III-7.B review can be discussed in terms of these two key considerations.

6. CHOICE OF REVIEW APPROACH

The approach taken in the review process depends, to a large degree, upon which of the two previously stated key questions one chooses to emphasize and address first.

One could give primary consideration to the second. If this approach is chosen, one first sets up a minimum margin of safety (based on failure) that will be acceptable for SEP plants. This margin is to be computed in accordance with current criteria. Then, one investigates structures designed in accordance with earlier code provisions, and to different loading combinations, to see if they meet the chosen SEP margin when challenged by current loading combinations and evaluated to current criteria. This approach gives the appearance of being efficient. The review proceeds from the general (the chosen minimum margin of safety) to the particular (the ability of a previously designed structure to meet the chosen margin). Moreover, issues are immediately resolved on a "go; no-go" basis. However, the initial step is not easy; neither are the necessary evaluations. One is dealing with highly loaded structures in regions where materials behave inelastically. Rule-making in such areas is sure to be difficult, and likely to be highly controversial.

The alternative approach is taken in this review. It proceeds from the particular to the general, and places initial emphasis upon seeking to answer (for SEP plants) questions as to what, how many, and of what magnitude are the infringements on current criteria. No new rulemaking is involved (at least at the outset). All initial assessments are based on existing criteria.

Current and older codes are compared paragraph-by-paragraph to see the effects that code changes may have on the load carrying ability of individual elements (beams, columns, frames, and the like). It should be noted that this process, although involving judgments, is basically fact-finding -- not decisionmaking.

This kind of review is painstaking, and there is no assurance in advance that it in itself will be decisive. It may turn out, after examination of the facts, that designs predicated upon the older criteria infringe upon current design allowables in many cases and to extensive depths. If so, such information will certainly be of value to the final safety assessment, but many open questions will remain.

On the other hand, it may turn out that infringements upon current criteria are infrequent and not of great magnitude. If this is the case, many issues will have been resolved, and questions of structural integrity sharply focused upon a few remaining key issues.

7. METHOD

A brief description of the approach used to carry out SEP Topic III-7.B follows. For discussion of the work, it is convenient to divide it into six areas:

1. information retrieval and assembly
2. appraisal of information content
3. code comparison reviews
4. code change impact assessment
5. plant-specific review of the relevancy of code change impacts
6. summarizing plant status vis-a-vis design criteria changes.

7.1 INFORMATION RETRIEVAL

The initial step (and to a lesser extent an ongoing task of the review) was to collect and organize necessary information. At the beginning of FRC's work assignment, NRC forwarded files relevant to the work. These submittals included pertinent sections of plant FSARs, Standard Review Plan (SRP) 3.8, response to questions on Topic III-7.B previously requested of licensees by the NRC, and other relevant data and reports.

FRC organized these submittals into Topic III-7.B files on a plant-by-plant basis. The files also house additional information, subsequently received, and other documents developed for the plant review.

A number of channels were used to gather additional information. These included information requests to NRC; letter requests for additional information sent to licensees; plant site visits; and retrieval of representative structural drawings, design calculations, and design specifications.

In addition, a separate file was set up to maintain past and present structural codes, NRC Regulatory Guides, Staff Position Papers, and other relevant documents (including, where available, reports from SEP tasks interfacing with the III-7.B effort).

7.2 APPRAISAL OF INFORMATION CONTENT

Most of the information sources were originally written for purposes other than those of the Task III-7.B review. Consequently, much of the information sought was embedded piecemeal in the documents furnished. These sources were searched for the relevant information that they did contain. Generally it was found that information gaps remained (i.e., some needed items were not referenced at all or, when they were found, often were not specific enough for Task III-7.B purposes). The information found was assembled and the gaps were filled through the information retrieval efforts mentioned earlier.

7.3 CODE COMPARISON REVIEWS

The codes and standards used to represent current licensing practice were selected as described in Appendix I of this report. Briefly summarized, the criteria selection corresponds to NUREG-800, of NRC's SRP, the operative document providing guidance to NRC reviewers on licensing matters (see Reference 1).

Next, the Seismic Category I structures at the Palisades Nuclear Power Station were identified (see Section 8). For these, on a structure-by-structure basis, the codes and standards which were used for actual design were likewise identified (see Section 9). Each code was then paired with its counterpart that would govern design were the structure to be licensed today.

Workbooks were prepared for each code pair. The workbook format consisted of paragraph-by-corresponding paragraph photocopies of the older and the current versions laid out side-by-side on 11 by 17-inch pages. A central column between the codes was left open to provide space for reviewer comments.

The code versions were initially screened to discover areas where the text either remained identical in both versions or had been reedited without changing technical content. Code paragraphs which were found to be essentially the same in both versions were so marked in the comments column.

The review then focused on the remaining portions of the codes where textual disparities existed. Pertinent comments regarding such changes were entered. Typical comments address either the reason the change had been introduced, or the intent of the change, or its impact upon safety margins, or a combination of such considerations.

As can be readily appreciated, many different circumstances arise in such evaluations--some simple, some complex. A few examples are cited and briefly discussed below.

Provisions were found where code changes liberalized requirements, i.e., less stringent criteria are in force today than were formerly required. Such changes are introduced from time to time as new information becomes available regarding the provision in question. Not infrequently code committees are called upon to protect against failure modes where the effects are well known; but too little is yet clear concerning the actual failure mechanism and the relative importance of the contributing factors. The committee often cannot defer action until a full investigation has been completed, but must act on behalf of safety. Issues such as these are usually resolved with prudence and caution--sometimes by the adoption of a rule (based upon experience and judgment) known to be conservative enough to assure safety. Subsequent investigation may produce evidence showing the adopted rule to be over-cautious, and provide grounds for its relaxation.

On the other hand, some changes which on first view may appear to reflect a relaxation of code requirements do not in fact actually do so. Structural codes tend to be documents with interactive provisions. Sometimes apparent liberalization of a code paragraph may really reflect a general tightening of criteria, because the change is associated with stiffening of requirements elsewhere.

To cite a simple example, a newly introduced code provision may be found making it unnecessary to check thin flanged, box section beams of relatively small depth-to-width ratio for buckling. This might appear to be a relaxation of requirements. However, elsewhere the code has also introduced a requirement that the designer must space end supports closely enough to preclude buckling. Thus, code requirements have been tightened, not relaxed.

In the code comparison review, wherever it was found that code requirements had truly been relaxed, this was noted in the reviewer's comments. Because liberalization of code criteria clearly cannot give rise to safety issues concerning structures built to more stringent requirements, such matters were not considered further.

On the other hand, whenever it was clear that a code change introduced more stringent criteria, the potential impact of the change on margins of safety shown for the structure was assessed. When it was felt that the change (although more restrictive) would not significantly affect safety margins, this judgment was entered in the commentary. When it was clear that the code change had the potential to significantly affect the perceived margin of safety, this was noted in the comments and the paragraph was flagged for further consideration.

Sometimes the effects of a code change are not easily seen. Indeed, depending upon a number of factors,* the change may reflect a tightening of requirements for some structures and a liberalization for others. When doubtful or ambiguous situations were encountered, the effect of the code change was explored analytically using simple models.

A variety of analytical techniques were used, depending on the situation at hand. One general approach was to select a basic structural element (a beam, a column, a frame, a slab, or the like) and analytically test it, under both the older and the current criteria. For example, selecting a typical structural element and a simple loading, the element was designed to the older code requirements. The load carrying capacity of this structure was then reexamined, this time using current code criteria. Finally, the load carrying capacity of the element, as shown by the older criteria and determined by the

*Geometry, material properties, magnitude or type of loading, type of supports—to name a few.

current criteria, was compared. Examples of investigations performed to assess code change impacts are found in Appendix B.

In making these studies, an attempt was made to use structural elements, model dimensions, and load magnitudes that were representative of actual structures. For studies that were parametized, an attempt was made to span the parametric range encountered in nuclear structures.

Although one must be cautious about claiming that results from simplified models may be totally applicable to the more complex situations occurring in real structures, it was felt that such examples provided reasonable guidance for making rational judgments concerning the impact of changed code provisions on perceived margins of safety.

7.4 ASSESSMENT OF THE POTENTIAL IMPACT OF CODE CHANGES

As the scope of the Task III-7.B assignment makes clear, a limited objective is sought (for the present) with respect to assessment of the effects of code changes on Seismic Category I structures.

The scope of review is not set at the level of appraisal of individual, as-built structures on plant sites. Correspondingly, the review does not attempt to make quantitative assessments as to the structural adequacy under current NRC criteria of specific structures at particular SEP plants.

To the contrary, the scope of the review is confined to the comparison of former structural codes and criteria with counterpart current requirements. Correspondingly, the assessment of the impact of changes in codes and criteria is confined to what can be deduced solely from the provisions of the codes and criteria.

Although the review is therefore carried out with minimal reference to actual structures in the field, the assessments of code change impacts that can be made at the code comparison level hold considerable significance for actual structures.

In this respect, two important points should be noted:

1. The review brings sharply into focus the changes in code provisions that may give rise to concern with respect to structural margins of safety as perceived from the standpoint of the requirements that NRC now imposes upon plants currently being licensed.

   The review simultaneously culls away a number of code changes that do not give rise to such concerns, but which (because they are there) would otherwise have to be addressed, on a structure-by-structure basis.

2. The effects of code changes that can be determined from the level of code review are confined to potential or possible impacts on actual structures.

   Review, conducted at the code comparison level, cannot determine whether or not potentially adverse impacts are actually realized in a given structure. The review may only <u>warn</u> that this <u>may</u> be the case.

   For example, current criteria may require demonstration of integrity of a structure under a loading combination that includes an additional load not specified in the corresponding loading combination to which the structure was designed. If the non-considered load is large (i.e., in the order of or larger than other major loads that were included), then it is quite possible that some members in the structure would appear overloaded as viewed by current criteria. Thus a potential concern exists.

   However, no determination as to actual overstress in any member can be made by code review alone. Actual margins of safety in the controlling member (and several others*) must certainly be examined before even a tentative judgment of this kind may be attempted.

In order to carry out the code review objective of identifying criteria changes that had the potential to give rise to concern about possible impairment of perceived margins of safety, the following scheme classifying code change impacts was adopted.

7.4.1 <u>Classification of Code Changes</u>

Where code changes involve technical content (as opposed to those which are editorial, organizational, administrative, and the like), the changes are classified according to the following scheme.

*The addition of a new load can change the location of the point of highest stress.

Each such code change is classified according to its potential to alter perceived margins of safety* in structural elements to which it applies. Four categories are established:

Scale A Change - The new criteria have the potential to substantially impair margins of safety as perceived under the former criteria.

Scale A_x Change - The impact of the code change on margins of safety is not immediately apparent. Scale A_x code changes require analytical studies of model structures to assess the potential magnitude of their effect upon margins of safety.

Scale B Change - The new criteria operate to impair margins of safety but not enough to cause engineering concern about the adequacy of any structural element.

Scale C Change - The new criteria will give rise to larger margins of safety than were exhibited under the former criteria.

7.4.1.1 General and Conditional Classifications of Code Change Impacts

Scale ratings of code changes are found in two different forms in this report. For example, some may be designated as "Scale A," and others as "Scale C." Others may have dual designation, such as "Scale A if --- [a condition statement] or Scale C if --- [a second condition statement]."

In assigning scale classifications, an <u>efficient</u> design to original criteria is assumed. That is, it is postulated that (a) the provision in question controls design and (b) the structural member to which the code provision applies was proportioned to be at (or close to) the allowable limit. The impact scale rating is assigned accordingly.

If the code change is Scale A, and it applies (in a particular structure) to a member which is not highly stressed, then this may afford excellent grounds for asserting that this particular member is adequate; but it does not

*That is, if (all other considerations remaining the same) safety margins as computed by the older code rules were to be recomputed for an as-built structure in accordance with current code provisions, would there be a difference due only to the code change under consideration?

thereby downgrade the ranking to, say, a Scale B change for that member. The scale ranking is not a function of member stress* nor a ranking of member adequacy. The scale system ranks code change impact, not individual members.

However, a number of code provisions are framed so that the allowable limit is made a function of member proportion. When this kind of a code provision is changed, the change may affect members of certain proportions one way and members of other proportions differently.

For example, assume a change in column design requirements is introduced in the code and this is framed in terms of radius of gyration. The new rule acts to tighten design requirements for slender columns, but liberalizes former requirements for columns that are not slender. This change may be rated Scale A for slender columns, and simultaneously, Scale C for non-slender ones. Although some columns now appear to be Scale A columns while others appear to be Scale C columns, the distinction between them resides in the code, and is not a reflection of member adequacy. Clearly, it is still code changes that are ranked; but, in this case, the code change does not happen to affect all columns in a unilateral way.

7.4.1.2 Code Impacts on Structural Margins

This classification of code changes identifies both (a) changes that have the potential to significantly impair perceived margins of safety (Scale A changes) and (b) changes that have the potential to enhance perceived margins of safety (Scale C changes).

Emphasis is subsequently placed on Scale A changes, not on Scale C changes. The purpose of the code comparison review is to narrow down and bring into sharper focus the areas where structures shown adequate under former criteria may not fully comply with current criteria. Once such criteria changes have been identified, actual structures may be checked to see if the potential concern is applicable to the structure. Depending upon a number of structure-specific circumstances, this may or may not be so.

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*There are exceptions, but these are code-related, not adequacy-related.

The same thing is true of Scale C changes, i.e., those that may enhance perceived structural margins. Specific structures must be examined to see if the potential benefit is actually applicable to the structure. If it is applicable, credit may be taken for it. However, this step can only be taken at the structural level, not at the code level.

A simple example may help clarify this point. Assume a steel beam exists in a structure designed by AISC 1963 rules for the then-specified loading combination. Current criteria require inclusion of an additional load in the loading combination (Scale A change), but the current structural code permits a higher allowable load if the beam design conforms to certain stipulated proportions (Scale C change). Several circumstances are possible for beams in actual structures, as shown below.

New Load	Higher Stress Limit	Results
Maximum stress in beam under original loading conditions was low with ample margin for additional load	Applicability immaterial	Beam adequate under current criteria
Maximum stress in beam under original loading condition was near former allowable limit	Beam qualifies for higher stress limit	Beam may be adequate under current criteria
Maximum stress in beam under original loading condition was near former allowable limit	Beam does not qualify for increased stress limit	Beam unlikely to be adequate under current criteria

It is clear from this example that the function of the code review is to point out code changes that might impair perceived margins of safety, and that assessment of the applicability of the results of the review is best accomplished at the structure-specific level.

7.5 PLANT-SPECIFIC CODE CHANGES

There is substantial overlap among the SEP plants in the codes and standards used for structural design. For example, several plants followed the provisions of ACI-318, 1963 edition, in designing major concrete structures.

Thus, the initial work (comparing older and current criteria) is not plant-specific. However, when the reviewed codes are packaged in sets containing only those code comparisons relevant to design of Seismic Category I structures in a particular SEP plant, the results begin to take on plant-specific character.

The code changes potentially applicable to particular structures at a particular SEP plant have then been identified. However, this list is almost surely overly long because the list has been prepared without reference to actual plant structures. For example, the code change list might include an item relating to recently introduced provisions for the design of slender columns, and none actually exist in any structures in that particular plant.

In-depth examination of design drawings, audit of structural analyses, and review of plant specifications were beyond the scope of the III-7.B task. Accordingly, FRC did not attempt such activities. However, occasional reference to such documents was necessary to the review work. Consequently, FRC was able to cull from the list some items that were obviously inappropriate to the plant structures. Wherever this was done, the reason for removal was documented, but no attempt was made to remove every such item.

Code changes that, for structures in general, may be significant but did not appear applicable to any of the Category I structures at Palisades were relegated to Appendix A. The Scale A or Scale A_x changes that remained are listed on a code-by-code basis in Section 11.

8. PALISADES SEISMIC CATEGORY I STRUCTURES

SEP Topic III-1 has for its objectives the classification of components, structures, and systems with respect to both quality group and seismic designation. The task force charged with this responsibility has presented its findings in Reference 5, and the following structures have been determined to be Seismic Category I:

A. Containment

Includes:
- Cylindrical wall, dome, and slab
- Liner (no credit for structural strength under mechanical loads)
- Equipment hatch
- Personnel locks

B. Internal Structures

- Reactor cavity
- Steam generator compartments (reviewed in Generic Task A-2)
- Biological shield (reviewed in Generic Task A-2)

C. External Structures

1. Auxiliary building (entire building except for administrative and access control areas)

   Includes:
   - Control room
   - Diesel generator compartments
   - Switchgear room
   - (The above three items are in a common enclosure with three floor levels)
   - Spent fuel pool
   - New fuel storage area
   - Radwaste area
   - Pump rooms (for ECCS and feedwater)

2. Turbine building
   (only the basement area which houses auxiliary feedwater pumps is Seismic Category I)

3. Intake/discharge structures including pump house for service water pumps.

9. STRUCTURAL DESIGN CRITERIA

The structural codes governing design of the major Seismic Category I structures for the Palisades Nuclear Power Generating Station are detailed in the following table.

Structure	Design Criteria	Current Criteria
A. Containment		
1. Concrete (including shell, dome, and slab)	ACI 318-63	ASME B&PV Code, Section III, Division 2, 1980 (subtitled ACI 359-80)
	ACI 301-63 (specifications for concrete)	ACI 301-72 (Rev. 1975)
2. Liner	ASME B&PV Section III, 1965 (Provisions of Article 4*) ASME B&PV Section VIII (undated), (Fabrication Practices for Welded Vessels Only) ASME B&PV Section IX (undated), (welding procedure and welders qualifications only)	ASME B&PV Code, Section III, Division 2, 1980 (Subtitled ACI 359-80)
3. Personnel locks and equipment hatches	ACI 318-63 for Concrete ASME B&PV Section III, 1965, for steel	ASME B&PV Code, Section III, Division 2, 1980 (subtitled ACI 359-80)
B. Internal Structures	ACI 318-63 AISC 1963	ACI 349-80

*The two significant applications of this article are:
1. determination of thermal stresses in the liner
2. analysis of pipe penetration attached to the liner.

Structure	Design Criteria	Current Criteria
C. External Structures		
1. Auxiliary building Control room Fuel pool Diesel generator room Radwaste facility	AISC 1963 ACI 318-63	AISC 1980 ACI 349-76
2. Service water, intake, pump house, and discharge structures	AISC 1963 ACI 318-63	AISC 1980 ACI 349-76
3. Turbine building auxiliary feedwater pump enclosure	AISC 1963 ACI 318-63	AISC 1980 ACI 349-76

REFERENCES:

Identification of the Original Design Codes:

1. Palisades FSAR Section 5 and Appendix B (Identifies codes for Items A and B)
2. Seismic Review of Palisades Nuclear Power Plant Unit I, Phase I Report - Subject: Review and documentation of existing seismic analysis and design (identifies codes for Items A through C above)

10. LOADS AND LOAD COMBINATION CRITERIA

10.1 DESCRIPTION OF TABLES OF LOADS AND LOAD COMBINATIONS

The requirements governing loads and load combinations to be considered in the design of civil engineering structures for nuclear service have been revised since the older nuclear power plants were constructed and licensed. Such changes constitute a major aspect of the general pattern of evolving design requirements; consequently, they are singled out for special consideration in the present section of this report.

The NRC Regulatory Guides and Standard Review Plans provide guidance regarding what loads and load combinations must be considered. In some cases, the required loads and load combinations are also specified within the governing structural design code; other structural codes have no such provisions and take loads and load combinations as given a priori. In this report, loads and load combinations are treated within the present section whether or not the structural design codes also include them.

Later sections of this report address, paragraph by paragraph, changes in text between design codes current at the time the plant was constructed and those governing design today; however, to avoid repetition, code changes related to loads and load combinations will not be evaluated again although they may appear as provisions of the structural design codes.

To provide a compact and systematic comparison of previous and present requirements, the facts are marshalled in tabular form. Two sets of tables are used:

1. load tables
2. load combination tables.

Both sets of tables are constructed in accordance with current requirements for Seismic Category I structures, i.e., the load tables list all loads that must be considered in today's design of these structures, and the load combination tables list all combinations of these loadings for which current licensing procedures require demonstration of structural integrity.

In general, the loads and load combinations to be considered are determined by the structure under discussion. The design loads for the structure housing the emergency power diesel generator, for example, are quite different than those for the design of the containment vessel. Consequently, structures must be considered individually. Each structure usually requires a load table and load combination table appropriate to its specific design requirements.

The design requirements for the various civil engineering structures within a nuclear power plant are echoed in applicable sections of NRC's Standard Review Plan (SRP) 3.8. The tables in the present report correspond to, and summarize, these requirements for each structure. A note at the bottom of each table provides the reference to the applicable section of the Standard Review Plan; Section 10.2 of this report lists, for reference, the load symbols used in the charts together with their definitions.

The loads actually used for design are considered, structure by structure, and the load tables are filled in according to the following scheme:

1. The list of potentially applicable loads (according to current requirements) is examined to eliminate loads which either do not occur on, or are not significant for, the structure under consideration.

2. The loads included in the actual design basis are then checked against the reduced list to see if all applicable loads (according to current requirements) were actually considered during design.

3. Each load that was considered during design is next screened to see if it appears to correspond to current requirements. Questions such as the following are addressed: Were all the individual loads encompassed by the load category definition represented in the applied loading? Do all loads appear to match present requirements (1) in magnitude? (2) in method of application?

4. An annotation is made as to whether deviations from present requirements exist, either because of load omissions or because the loads do not correspond in magnitude or in other particulars.

5. If a deviation is found, a judgment (in the form of a scale ranking) is made as to the potential impact of the deviation on perceived margins of safety.

6. Relevant notes or comments are recorded.

Of particular importance to the Topic III-7.B review are comments indicating that the effects of certain loadings (tornado and seismic loads, in particular) are being examined under other SEP topics. In all such cases, the findings of these special SEP topics (where review in depth of the indicated loading conditions will be undertaken) will be definitive for the overall SEP effort. Consequently, no licensee investigation of such issues is required under Topic III-7.B nor is such effort within the scope of Topic III-7.B (see Section 4). Licensee participation in the resolution of such issues may, however, be requested under the scope of other SEP topics devoted to such issues.

After the load tables have been filled out, the load combination tables are compiled. Like the load tables, the load combination tables are drawn up to current requirements and the load combinations actually used in the design basis are matched against these requirements.

For ease of comparison, the load combinations actually used are superimposed on the load combinations currently required. This is accomplished in two steps:

1. Currently specified load combinations include loads sufficient for the most general cases. In particular applications, some of these are either inappropriate or insignificant. Therefore, the first step is to strike all loads that are not applicable to the structure under consideration from all load combinations in which they appear.

2. Next, loads actually combined are indicated by encircling (in the appropriate load combinations) each load contributing to the summation considered for design.

Thus, the comparison between what was actually done and what is required today is readily apparent. If the load combinations used are in complete accord with current requirements, each load symbol on the sheet appears as either struck or encircled. Load combinations not considered and loads omitted from the load combinations stand out as unencircled items.

A scale ranking is next assigned to the load combinations; however (unlike the corresponding ranking of loads), a scale ranking is not necessarily assigned to each one. When the load combinations used for design correspond closely to current requirements, scale ratings may be assigned to all combinations. However, when the number of load combinations considered in design was substantially fewer than current criteria prescribe, it did not appear to serve any engineering purpose to rank the structure for each currently required load combination. Instead, a limited number of loading cases (usually two) were ranked.

The following considerations guided the selection of these cases:

1. For purposes of the SEP review, it was not believed necessary to require an extensive reanalysis of structures under all load combinations currently specified.

2. SEP plants have been in full power operation for a number of years. During this time, they have experienced a wide spectrum of operating and upset conditions. There is no evidence that major Seismic Category I structures lack integrity under these operating conditions.

3. The most severe load combinations occur under emergency and accident conditions. These are also the conditions associated with the greatest consequences to public health and safety.

4. If demonstration of structural adequacy under the most severe load combinations currently specified for emergency and accident conditions is provided, a reasonable inference can be drawn that the structure is also adequate to sustain the less severe loadings associated with less severe consequences.

10.2 LOAD DEFINITIONS

D Dead loads or their related internal moments and forces (such as permanent equipment loads).

E or E_o Loads generated by the operating basis earthquake.

E' or E_{ss} Loads generated by the safe shutdown earthquake.

F Loads resulting from the application of pre-stress.

H Hydrostatic loads under operating conditions.

H_a Hydrostatic loads generated under accident conditions, such as post-accident internal flooding. (F_L is sometimes used by others* to designate post-LOCA internal flooding.)

*See, for example, SRP 3.8.2.

L	Live loads or their related internal moments and forces (such as movable equipment loads).
P_o or P_v	Loads resulting from pressure due to normal operating conditions.
P_a	Pressure load generated by accident conditions (such as those generated by the postulated pipe break accident).
P_s	All pressure loads which are caused by the actuation of safety relief valve discharge including pool swell and subsequent hydrodynamic loads.
R_o	Pipe reactions during startup, normal operating, or shutdown conditions, based on the critical transient or steady-state condition.
R_r or R_a	Pipe reactions under accident conditions (such as those generated by thermal transients associated with an accident).
R_s	All pipe reaction loads which are generated by the discharge of safety relief valves.
T_a	Thermal loads under accident conditions (such as those generated by a postulated pipe break accident).
T_o	Thermal effects and loads during startup, normal operating, or shutdown conditions, based on the most critical transient or steady-state condition.
T_s	All thermal loads which are generated by the discharge of safety relief valves.
W	Loads generated by the design wind specified for the plant.
W' or W_t	Loads generated by the design tornado specified for the plant. Tornado loads include loads due to tornado wind pressure, tornado-created differential pressure, and tornado-generated missiles.
Y_r	Equivalent static load on the structure generated by the reaction on the broken pipe during the design basis accident.
Y_j	Equivalent static load on the structure generated by the impingement of the fluid jet from the broken pipe during the design basis accident.
Y_m	Missile impact equivalent static load on the structure generated by or during the design basis accident, such as pipe whipping.

The load combination charts correspond to loading cases and load definitions as specified in the appropriate SRP. Each chart is associated with a specific SRP as identified in the notes accompanying the chart. Guidance with respect to the specific loads which must be considered in forming each load combination is provided by the referenced SRP. All SRPs are prepared to a standard format; consequently, subsection 3 of each plan always contains the appropriate load definitions and load combination guidance.

10.3 DESIGN LOAD TABLES

"COMPARISON OF DESIGN BASIS LOADS"

STRUCTURE: CONTAINMENT STRUCTURE (CONCRETE)

COMPARISON OF DESIGN BASIS LOADS

PLANT: PALISADES

	Current Design Basis Loads	Is Load Applicable To This Structure?	Is Load Included In Plant Design Basis?	Does Load Magnitude Correspond To Present Criteria?	Does Deviation Exist In Load Basis?	Code Impact Scale Ranking	Comments
Gravity	D	YES	YES	YES	NO	—	
	L	YES	YES	YES	NO	Bx	3.)
Pressure	F	YES	YES	YES	NO	—	
	H	YES	YES	—	—	Bx	2.)
	P_o	NO 7.)	—	—	—	—	
	P_a	YES	YES	YES	NO	—	4.) SEP TOPIC VI-2.D, III-7B
	P_s	NO	—	—	—	—	
Thermal	T_o	YES	YES	YES	NO	—	
	T_a	YES	YES	NO	YES	—	4.) SEP TOPIC VI-2.D, III-7B
	T_s	NO	—	—	—	—	
Pipe & Mech.	R_o	YES	YES	YES	NO	—	
	R_a	YES	YES	YES	NO	—	
	R_s	NO	—	—	—	—	
Environmental	E'	YES	YES	NO	YES	Ax	1.) SEP TOPIC III-6
	E	YES	YES	NO	YES	—	
	W'	YES	YES	YES	NO	Ax	1.), 5.) SEP TOPIC III-2
	W	YES	YES	—	—	—	
Impulse	Y_r	YES	YES	NO	YES	Ax	SEP TOPIC III-5.A
	Y_j	YES	YES	NO	YES	Ax	SEP TOPIC III-5.A
	Y_m	YES	YES	—	—	Ax	6.) SEP TOPIC III-5.A

Comments

1.) This load is being reviewed as a separate SEP Topic

2.) TREATMENT OF SOIL PRESSURE NOT DESCRIBED IN FSAR ALTHOUGH PROVISION TO ACCOUNT FOR IT UNDER LOAD CATEGORY 'D' IS STATED.

3) ROOF LOADS HAVE INCREASED PER SEP TOPIC II-2.A; II-3B (parapet roofs)

4.) LOAD MAGNITUDE AND IMPACT BASED ON RESULTS OF SEP TOPIC VI-2.D

5.) FSAR STATES THAT "SEISMIC LOADING CONTROL DESIGN AND THAT 360 MPH TORNADO CONSIDERED," BUT OFFER NO SPECIFIC SUBTANTIATION.

6.) FSAR STATES PIPE WHIP RESTRAINT OR CONCRETE WALL OR SLAB BARRIERS PROVIDED.

7.) FSAR TECH. SPEC. STATES "THE REACTOR SHALL NOT BE CRITICAL IF CONTAINMENT INTERNAL PRESSURE EXCEEDS 2 PSIG

COMPARISON OF DESIGN BASIS LOADS

STRUCTURE: AUXILIARY BLDG 3-STORY ENCLOSURE FOR: CONTROL ROOM DIESEL GEN. & SWITCHGEAR

PLANT: PALISADES

	Current Design Basis Loads	Is Load Applicable To This Structure?	Is Load Included In Plant Design Basis?	Does Load Magnitude Correspond To Present Criteria?	Does Deviation Exist In Load Basis?	Code Impact Scale Ranking	Comments
Gravity	D	YES	YES	YES	NO	—	
	L	YES	YES	YES	NO	Ax	5.)
Pressure	F	NO	—	—	—	—	
	H	YES	YES	—	—	Bx	3.), 4.)
	P_a	—	—	—	—	Ax	1.) SEP TOPIC III-5B
Thermal	T_o	YES	YES	YES	NO	—	
	T_a	—	—	—	—	Ax	1.) SEP TOPIC III-5B
Pipe & Mech.	R_o	YES	YES	YES	NO	—	
	R_a	—	—	—	—	Ax	
Environmental	E'	YES	YES	NO	YES	Ax	1.) SEP TOPIC III-6
	E	YES	YES	—	—	—	
	W'	YES	YES	YES	NO	—	1.), 2.) SEP TOPIC III-2
	W	YES	YES	—	—	—	
Impulse	Y_r	—	—	—	—	Ax 6.)	SEP TOPIC III-5.B
	Y_j	YES	YES	—	—	Ax 6.)	SEP TOPIC III-5.B
	Y_m	YES	—	—	—	Ax 6.)	SEP TOPIC III-5.B

Comments

1.) This load is being reviewed as a separate SEP Topic

2.) TORNADO MISSILE INCLUDED IN W'

3.) SOIL PRESSURE TREATMENT NOT DESCRIBED IN FSAR (EXCEPT THAT PALISADES DESIGN BASIS PROVIDES FOR ITS CONSIDERATION UNDER LOAD DESIGNATION 'D')

4.) H IN PALISADES' LOADING COMBINATIONS IS RESERVED FOR PIPE REACTIONS DURING NORMAL OPERATION. FRC USES R_o FOR THIS.

5.) Roof LOADS HAVE INCREASED PER SEP TOPIC II-2 A; II-3B (parapet roofs)

6.) RESULTS FROM SEP TOPIC III-5.B WILL DETERMINE IF DEVIATION EXISTS AND IF DEVIATION IS FOUND ACCESS ITS SIGNIFICANCE

COMPARISON OF DESIGN BASIS LOADS

STRUCTURE: AUXILIARY BLDG. SPENT FUEL POOL (CONCRETE)

PLANT: PALISADES

	Current Design Basis Loads	Is Load Applicable To This Structure?	Is Load Included In Plant Design Basis?	Does Load Magnitude Correspond To Present Criteria?	Does Deviation Exist In Load Basis?	Code Impact Scale Ranking	Comments
Gravity	D	YES	YES	YES	NO	—	
	L	YES	YES	YES	NO	—	
Pressure	F	NO	—	—	—	—	
	H	YES	YES	YES	NO	—	
	P_a	NO	—	—	—	—	1.) SEP TOPIC III-5B
Thermal	T_o	NEGL.	—	—	—	—	
	T_a	YES	YES	YES	NO	—	6.) SEP TOPIC III-5B
Pipe & Mech.	R_o	NO	—	—	—	—	
	R_a	NO	—	—	—	—	
Environmental	E'	YES	YES	NO	YES	Ax	1.) SEP TOPIC III-6
	Z	YES	YES	NO	YES	—	
	W'	YES 3.)	YES 4.)	YES	— 5.)	Ax	1.), 2.), 3.) SEP TOPIC III-2
	W	NO	—	—	—	—	
Impulse	Y_r	—	—	—	—	Ax	PIPE BREAK EXTERNAL TO CONTAINMENT IS EVALUATED IN SEP TOPIC III-5.B
	Y_j	—	—	—	—	Ax	
	Y_m	—	—	—	—	Ax	

Comments

1.) This load is being reviewed as a separate SEP Topic

2.) TORNADO MISSILE INCLUDED IN W'

3.) APPLICABLE ONLY SINCE ROOF OVER SPENT FUEL POOL IS NOT TORNADO RESISTANT.

4.) FSAR STATES "POOL WALLS ARE RESISTANT TO TORNADOES AND THE ASSOCIATED CREDIBLE MISSILES"

5.) SEP TOPIC III-2 WILL DETERMINE WHETHER OR NOT POOL EXPOSURE TO POSSIBLE TORNADO EFFECTS IS AN ALLOWABLE SPENT FUEL POOL LOAD.

6.) FSAR STATES THAT SPENT FUEL POOL IS DESIGNED TO WITHSTAND TEMP. STRESSES CAUSED BY POOL WATER TEMP. RISING TO 150° F UNDER ABNORMAL CONDITION.

STRUCTURE	AUXILIARY BLDG SPENT FUEL POOL (ROOF - STEEL)

COMPARISON OF DESIGN BASIS LOADS

PLANT: PALISADES

	Current Design Basis Loads	Is Load Applicable To This Structure?	Is Load Included In Plant Design Basis?	Does Load Magnitude Correspond To Present Criteria?	Does Deviation Exist In Load Basis?	Code Impact Scale Ranking	Comments
Gravity	D	YES	YES	YES	NO	—	
	L	YES	YES	YES	NO	Ax	3.)
Pressure	F	NO	—	—	—	—	
	H	NO	—	—	—	—	
	P_a	NO	—	—	—	—	1.) SEP TOPIC III-5B
Thermal	T_o	YES	NEGLIGIBLE	—	—	—	
	T_a	NO	—	—	—	—	1.) SEP TOPIC III-5B
Pipe & Mech.	R_o	NO	—	—	—	—	
	R_a	NO	—	—	—	—	
Environmental	E'	YES	YES	NO	YES	Ax	1.) SEP TOPIC III-6
	E	YES	YES	—	—	—	
	W'	YES	NO	—	YES	Ax	1.), 2.) SEP TOPIC III-2
	W	YES	YES	—	—	—	
Impulse	Y_r	NO	—	—	—	—	
	Y_j	NO	—	—	—	—	
	Y_m	NO	—	—	—	—	

Comments

1.) This load is being reviewed as a separate SEP Topic

2.) ROOF OVER SPENT FUEL POOL IS NOT DESIGNED AS TORNADO RESISTANT

3.) ROOF LOADS HAVE INCREASED, SEP TOPIC II-2.A; II-3B (parapet roofs)

COMPARISON OF DESIGN BASIS LOADS

PLANT: PALISADES

STRUCTURE: ~~BALANCE~~ AUXILIARY BLDG; NEW FUEL AREA; PUMP ROOMS; PENETRATION ROOMS; RADWASTE TREATMENT AREA

	Current Design Basis Loads	Is Load Applicable To This Structure?	Is Load Included In Plant Design Basis?	Does Load Magnitude Correspond To Present Criteria?	Does Deviation Exist In Load Basis?	Code Impact Scale Ranking	Comments
Gravity	D	YES	YES	YES	NO	—	
	L	YES	YES	YES	NO	A_x	4.)
Pressure	F	NO	—	—	—	—	
	H	YES	YES	—	—	—	
	P_a	—	—	—	—	A_x	1.) SEP TOPIC III-5B
Thermal	T_o	YES	YES	NEGLIGIBLE	—	—	
	T_a	—	—	—	—	A_x	1.) SEP TOPIC III-5B
Pipe & Mech.	R_o	YES	YES	YES	NO	—	
	R_a	—	—	—	—	A_x	
Environmental	E'	YES	YES	NO	YES	A_x	1.) SEP TOPIC III-6
	E	YES	YES	—	—	—	
	W'	YES	YES	YES	NO	—	1.), 2.) SEP TOPIC III-2
	W	YES	YES	—	—	—	
Impulse	Y_r	YES	YES	—	—	A_x	1.) 3.) SEP TOPIC III-5.B
	Y_j	YES	YES	—	—	A_x	1.), 3.)
	Y_m	YES	—	—	—	A_x	1.), 3.)

Comments

1.) This load is being reviewed as a separate SEP Topic

2.) TORNADO MISSILE INCLUDED IN W' (NEW FUEL AREA ROOF IS NOT TORNADO RESISTANT)

3.) NO DESCRIPTION, OF HOW LOAD IS TREATED, IS FOUND IN FSAR. PROVISION FOR IT EXISTS IN PALISADES DESIGN BASIS.

4.) ROOF LOADS HAVE INCREASED - SEP TOPIC II-2.A; II-3B (parapet roofs)

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COMPARISON OF DESIGN BASIS LOADS

STRUCTURE: INTAKE STRUCTURE (INCL. ENCLOSURE FOR SERVICE WATER PUMPS) DISCHARGE STRUCTURE

PLANT: PALISADES

	Current Design Basis Loads	Is Load Applicable To This Structure?	Is Load Included In Plant Design Basis?	Does Load Magnitude Correspond To Present Criteria?	Does Deviation Exist In Load Basis?	Code Impact Scale Ranking	Comments
Gravity	D	YES	YES	YES	NO	—	
	L	YES	YES	YES	NO	Ax	2.)
Pressure	F	NO	—	—	—	—	
	H	YES	YES	YES	NO	—	
	P_a	NO	—	—	—	—	1) SEP TOPIC III-5B
Thermal	T_o	NEGLIGIBLE	—	—	—	—	
	T_a	NO	—	—	—	—	1) SEP Topic III-5B
Pipe & Mech.	R_o	YES	NEGLIGIBLE	—	—	—	
	R_a	NO	—	—	—	—	
Environmental	E'	YES	YES	NO	YES	Ax	1.) SEP TOPIC III-6
	E	YES	YES	—	—	—	
	W'	YES	YES	YES	NO	—	1.) SEP TOPIC III-2
	W	YES	YES	—	—	—	
Impulse	Y_r	NO	—	—	—	—	SEP TOPIC III-5B
	Y_j	NO	—	—	—	—	
	Y_m	NO	—	—	—	—	

Comments

1.) This load is being reviewed as a separate SEP Topic

2.) ROOF LOADS HAVE INCREASED PER SEP TOPIC II-2.A ; II-3B (parapet roofs)

COMPARISON OF DESIGN BASIS LOADS

PLANT: PALISADES

STRUCTURE: TURBINE BLDG. AUX. FEED WATER PUMP ENCLOSURE (ONLY)

	Current Design Basis Loads	Is Load Applicable To This Structure?	Is Load Included In Plant Design Basis?	Does Load Magnitude Correspond To Present Criteria?	Does Deviation Exist In Load Basis?	Code Impact Scale Ranking	Comments
Gravity	D	YES	YES	YES	NO	—	
	L	YES	YES	YES	NO	—	4.)
Pressure	F	NO	—	—	—	—	
	H	YES	YES			Bx	2.), 3.)
	P_a	—	—	—	—	Ax	1.) SEP Topic III-5B
Thermal	T_o	YES	YES	NEGLIGIBLE	—	—	
	T_a	—	—	—	—	Ax	1.) SEP Topic III-5B
Pipe & Mech.	R_o	YES	YES	YES	NO	—	
	R_a	—	—	—	—	Ax	
Environmental	E'	YES	YES	NO	YES	Ax	1.) SEP Topic III-6
	E	YES	YES	—	—	—	
	W'	YES	YES	YES	NO	—	1.) SEP Topic III-2
	W	YES	YES	—	—	—	
Impulse	Y_r	YES	YES	—	—	Ax	1.) SEP Topic III-5 B
	Y_j	YES	YES	—	—	Ax	1.)
	Y_m	YES	YES	—	—	Ax	1.)

Comments

1.) This load is being reviewed as a separate SEP Topic

2.) SOIL PRESSURE TREATMENT NOT DESCRIBED IN FSAR (EXCEPT THAT PALISADES DESIGN BASIS PROVIDES FOR ITS CONSIDERATION UNDER LOAD DESIGNATION 'D'

3.) H IN PALISADES' LOAD COMBINATIONS DESIGNATES PIPE-REACTION UNDER OPERATING CONDITIONS. FRC USES R_o FOR THIS.

4.) ROOF LOADS HAVE INCREASED, SEP TOPIC II-2.A; II-3B (parapet roofs)

10.4 LOAD COMBINATION TABLES

"COMPARISON OF LOADING COMBINATION CRITERIA"

STRUCTURE
CONCRETE CONTAINMENT

COMPARISON OF LOADING COMBINATION CRITERIA

PLANT: PALISADES

Category	Combined Loading Cases	Gravity Dead, Live	Prestress Load	Pressure	Thermal	Severe Environment	Natural Phenomena	Mechanical	Scale Ranking
Normal	1	D + L	F	$\cancel{P_v}$	T_o			R_o	
Severe Environmental	2	D + L	F	$\cancel{P_v}$	T_o	E_o		R_o	
	3	D + L	F	$\cancel{P_v}$	T_o		W	R_o	
Severe Environmental (Factored)	4	$\boxed{D}$ + 1.$\boxed{L}$	$\boxed{F}$	$\cancel{P_v}$	$\boxed{T_o}$	1.$\boxed{E_o}$ $\boxed{1.25}$		$\boxed{1.25R_o}$	
	5	D + 1.3L	F	$\cancel{P_v}$	T_o		1.5W	R_o	
Extreme Environmental	6	$\boxed{D + L}$	$\boxed{F}$	$\cancel{P_v}$	$\boxed{T_o}$	$\boxed{E_{ss}}$		$\boxed{R_o}$	
	7	$\boxed{D + L}$	$\boxed{F}$	$\cancel{P_v}$	$\boxed{T_o}$		W_t	$\boxed{1.25R_o}$	4.
Abnormal	8	$\boxed{D + L}$	$\boxed{F}$	$\boxed{1.5\ P_a}$	$\boxed{T_a}$			R_a	A_x
	9	D + L	F	P_a	T_a			1.25 R_a	
Abnormal/ Severe Environmental	10	$\boxed{D + L}$	$\boxed{F}$	$\boxed{1.25\ P_a}$	$\boxed{T_a}$	$\boxed{1.25\ E_o}$		$\boxed{1.25R_a}$	
	11	D + L	F	1.25 P_a	T_a		1.25W	R_a	
	12	D + L	F	H_a	T_o	E_o			
	13	D + L	F	H_a	T_o		W		
Abnormal/ Extreme Environmental	14	$\boxed{D + L}$	$\boxed{F}$	$\boxed{P_a}$	$\boxed{T_a}$	$\boxed{E_{ss}}$		$\boxed{R_o}$ $R_a + R_r$	A_x (2)

Ref.: 1. SRP Section 3.8.1 Concrete Containment
2. ASME Section III, Div. 2 Article CC-3000

Notes

1. Encircled loads are those considered in the design. When load factors different from those currently required were used, the factor used is also encircled.
2. The FSAR states that; forces or pressure on structure due to rupture of one pipe, is considered. However no specific details are found.
3. For purposes of the SEP Review, demonstration that structural integrity is maintained for load case 14, 8 (per current criteria) may be considered as providing reasonable assurance that this structure meets the intent of current design criteria.
4. FSAR STATES SEISMIC LOADING CONTROLS OVER TORNADO

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STRUCTURE
CONTAINMENT LINER

COMPARISON OF LOADING COMBINATION CRITERIA

PLANT: PALISADES

Category	Combined Loading Cases	Gravity Dead, Live	Prestress Load	Pressure	Thermal	Severe Environment	Natural Phenomena	Mechanical	Scale Ranking
Normal	1	D + L	F	$\cancel{P_v}$	T_o			R_o	
Severe Environmental	2	D + L	F	$\cancel{P_v}$	T_o	E_o		R_o	
	3	D + L	F	$\cancel{P_v}$	T_o		W	R_o	
Severe Environmental (Factored)	4	$\boxed{D + L}$	$\boxed{F}$	$\cancel{P_v}$	$\boxed{T_o}$	$\boxed{E_o \; 1.25}$		$\boxed{1.5 R_o}$	
	5	D + L	F	$\cancel{P_v}$	T_o		W	R_o	
Extreme Environmental	6	$\boxed{D + L}$	$\boxed{F}$	$\cancel{P_v}$	$\boxed{T_o}$	$\boxed{E_{ss}}$		$\boxed{R_o}$	
	7	$\boxed{D + L}$	$\boxed{F}$	$\cancel{P_v}$	$\boxed{T_o}$		W_t	$\boxed{1.5 R_o}$	4,
Abnormal	8	$\boxed{D + L}$	$\boxed{F}$	$\boxed{1.5 P_a}$	$\boxed{T_a}$			R_a	A_x
	9	D + L	F	P_a	T_a			R_a	
Abnormal/ Severe Environmental	10	$\boxed{D + L}$	$\boxed{F}$	$\boxed{1.25 P_a}$	$\boxed{T_a}$	$\boxed{E_o \; 1.25}$		$\boxed{1.25 R_a}$	
	11	D + L	F	P_a	T_a		W	R_a	
	12	D + L	F	H_a	T_o	E_o			
	13	D + L	F	H_a	T_o		W		
Abnormal/ Extreme Environmental	14	$\boxed{D + L}$	$\boxed{F}$	$\boxed{P_a}$	$\boxed{T_a}$	$\boxed{E_{ss}}$		$\boxed{R_o}$ $R_a + R_r$	2. A_x

Ref.: 1. SRP Section 3.8.1 Concrete Containment
2. ASME Section III, Div. 2 Article CC-3000

Notes

1. Encircled loads are those considered in the design. When load factors different from those currently required were used, the factor used is also encircled.

2. The FSAR states that; forces or pressure on structure due to rupture of one pipe, is considered. However no specific details are found.

3. For purposes of the SEP Review, demonstration that structural integrity is maintained for load case 14, 8 (per current criteria) may be considered as providing reasonable assurance that this structure meets the intent of current design criteria.

4. FSAR STATES SEISMIC LOADS CONTROLS OVER TORNADO

5. PARAGRAPH CC-3720 OF ASME SECTION III. DIV. 2 STATES THAT FOR THE LINER THE LOAD FACTORS FOR ALL CASES MAY BE TAKEN AS 1.0, BUT THE LOAD FACTORS SHOWN ABOVE WERE CONSIDERED IN THE ANALYSIS.

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COMPARISON OF LOADING COMBINATION CRITERIA

CONCRETE STRUCTURES

PLANT: PALISADES

STRUCTURE: AUXILIARY BUILDING 3-STORY ENCLOSURE FOR CONTROL ROOM, DIESEL GEN & SWITCHGEAR

Combined Loading Cases	Gravity Dead, Live	Thermal	Pressure	Mechanical	Natural Phenomena	Impulsive Loading	Scale Ranking
1	1.4D + 1.7L						
2	1.4D + 1.7L				1.9E		
3	1.4D + 1.7L				1.7W		
4	.75 (1.4D + 1.7L)	.75 x 1.7 T_o		.75 x 1.7 R_o			
5	.75 (1.4D + 1.7 L) $\boxed{1.25(D+L)}$	.75 x 1.7 T_o		.75 x 1.7 $\boxed{R_o}$ $\boxed{1.25}$	.75 x 1.9 $\boxed{E}$ $\boxed{1.25}$		
6	.75 (1.4D + 1.7L) $\boxed{1.25(D+L)}$	.75 x 1.7 T_o		.75 x 1.7 $\boxed{R_o}$ $\boxed{1.25}$	.75 x 1.7 $\boxed{W}$ $\boxed{1.25}$		
7	1.2D				1.9E		
8	1.2D				1.7W		
9	$\boxed{D + L}$	T_o		$\boxed{R_o}$	$\boxed{E'}$		
10	$\boxed{D}$ + L	T_o		R_o	$\boxed{W_t}$		A_x **
11	D + L	T_a	1.5 P_a	R_a			
12	$\boxed{1.25 (D + L)}$	T_a	1.25 P_a	R_a	$\boxed{1.25E}$	Y_r + $\boxed{Y_j}$ + Y_m	
13	$\boxed{D + L}$	T_a	P_a	R_a	$\boxed{E'}$	Y_r + $\boxed{Y_j}$ + Y_m	A_x *

Ref; SRP (1981) Sect. 3.8.4 Other Category I structures (concrete)

Notes – Ultimate strength method required by ACI-349 (1977)

- Method used in design { working stress / ultimate strength ✓
- Loads deemed inapplicable or negligible struck from loading combinations.
- Encircled loads are those actually considered in the design. When load factors different from those currently required were used, the factor used is also encircled.

* The FSAR states that; forces or pressure on structure due to rupture of one pipe, is considered. However no specific details are found.

** Wind velocity used is 360 mph as referenced in the FSAR, 360 mph is required by the Reg. Guide 1.76; FSAR states no significant live loads other than crane loads

For purposes of the SEP Review, demonstration that structural integrity is maintained for load cases 10 and 13 (per current criteria) may be considered as providing reasonable assurance that this structure meets the intent of current design criteria.

COMPARISON OF LOADING COMBINATION CRITERIA

CONCRETE STRUCTURES

PLANT: PALISADES

STRUCTURE:
AUXILIARY BUILDING
SPENT FUEL POOL
(CONCRETE)

Combined Loading Cases	Gravity Dead, Live	Thermal	Pressure	Mechanical	Natural Phenomena	Impulsive Loading	Scale Ranking
1	1.4D + 1.7L						
2	$\boxed{1.25(D+L)}$ ~~1.4D + 1.7L~~				1.9 $\boxed{E \times 1.25}$		
3	1.4D + 1.7L				1.7W		
4	.75 (1.4D + 1.7L	~~.75 x 1.7 T_o~~		~~.75 x 1.7 R_o~~			
5	.75 (1.4D + 1.7 L) $\boxed{1.25(D+L)}$	~~.75 x 1.7 T_o~~		.~~75 x 1.7 R_o~~	.75 x 1.9 $\boxed{E\ 1.25}$		
6	.75 (1.4D + 1.7L) $\boxed{1.25(D+L)}$	.~~75 x 1.7 T_o~~		.~~75 x 1.7 R_o~~	.75 x 1.7 $\boxed{W\ 1.25}$		
7	1.2D				1.9E		
8	1.2D				1.7W		
9	$\boxed{D + L}$	~~T_o~~		~~R_o~~	$\boxed{E'}$		
10	D + L	~~T_a~~		~~R_a~~	W_t (1)		A_x *
11	D + L	T_a	~~1.5 P_a~~	~~R_a~~			
12	$\boxed{1.25\ (D + L)}$	T_a	~~1.25 P_a~~	~~R_a~~	$\boxed{1.25E}$	$Y_r + Y_j + Y_m$	
13	$\boxed{D + L}$	T_a	~~P_a~~	~~R_a~~	$\boxed{E'}$	$Y_r + Y_j + Y_m$	A_x

Ref; SRP (1981) Sect. 3.8.4 Other Category I structures (concrete)

Notes - • Ultimate strength method required by ACI-349 (1977)

- Method used in design { working stress / ultimate strength ✓
- Loads deemed inapplicable or negligible struck from loading combinations.
- Encircled loads are those actually considered in the design. When load factors different from those currently required were used, the factor used is also encircled.

* Wind velocity used is 360 mph as referenced in the FSAR, 360 mph is required by the Reg. Guide 1.76.

For purposes of the SEP Review, demonstration that structural integrity is maintained for load cases 10 and 13 (per current criteria) may be considered as providing reasonable assurance that this structure meets the intent of current design criteria.

1. ONLY MISSILE LOAD APPLICABLE

COMPARISON OF LOADING COMBINATION CRITERIA

STEEL STRUCTURES (Plastic Analysis)

PLANT: PALISADES

STRUCTURE: AUXILIARY BLDG. SPENT FUEL POOL (STEEL ROOF)

Combined Loading Cases	Gravity Dead, Live	Thermal	Pressure	Mechanical	Natural Phenomena	Impulsive Loading	Scale
1	1.7 (D + L)						
2	1.7 (D + L)				1.7E		
3	1.7 (D + L)				1.7W		
4	1.3 (D + L)	~~1.3 T_o~~		~~1.3 R_o~~			
5	1.3 $\boxed{(D + L)}$ 1.25	~~1.3 T_o~~		~~1.3 R_o~~	1.3 $\boxed{E}$ 1.25		
6	1.3 $\boxed{(D + L)}$ 1.25	~~1.3 T_o~~		~~1.3 R_o~~	1.3 $\boxed{W}$ 1.25		
7	$\boxed{D + L}$	~~T_o~~		~~R_o~~	$\boxed{E'}$		
8	D + L	~~T_o~~		~~R_o~~	W_t		A(2.)
9	D + L	~~T_a~~	~~1.5 P_a~~	~~R_a~~			
10	$\boxed{D + L}$ 1.25	~~T_a~~	~~1.25 P_a~~	~~R_a~~	$\boxed{1.25E}$	~~Y_j~~ + ~~Y_r~~ + ~~Y_m~~	
11	$\boxed{D + L}$	~~T_a~~	~~1.0 P_a~~	~~R_a~~	$\boxed{E'}$	~~Y_j~~ + ~~Y_r~~ + ~~Y_m~~	

Ref; SRP (1981) SECT. 3.8.4 Other Category I structures (steel)

Loads deemed inapplicable or negligible struck from loading combinations.

Notes

1. Encircled loads are those actually considered in the design. When load factors are different from those currently required were used, the factor used is also encircled.

2. 360 mph is required by the Reg. Guide 1.76

For purposes of the SEP Review, demonstration that structural integrity is maintained for load cases 8 and 11 (per current criteria) may be considered as providing reasonable assurance that this structure meets the intent of current design criteria.

COMPARISON OF LOADING COMBINATION CRITERIA

CONCRETE STRUCTURES

PLANT: PALISADES

STRUCTURE: ~~AUXILIARY~~ BALANCE AUXILIARY BLDG.; NEW FUEL AREA PUMP ROOMS; PENETRATION ROOMS RADWASTE TREATMENT AREA

Combined Loading Cases	Gravity Dead, Live	Thermal	Pressure	Mechanical	Natural Phenomena	Impulsive Loading	Scale Ranking
1	1.4D + 1.7L						
2	1.4D + 1.7L				1.9E		
3	1.4D + 1.7L				1.7W		
4	.75 (1.4D + 1.7L	~~.75 x 1.7 T_o~~		.75 x 1.7 R_o			
5	.75 (1.4D + 1.7 L) 1.25(D+L)	~~.75 x 1.7 T_o~~		.75 x 1.7 R_o 1.25	.75 x 1.9E 1.25		
6	.75 (1.4D + 1.7L) 1.25(D+L)	~~.75 x 1.7 T_o~~		.75 x 1.7 R_o 1.25	.75 x 1.7W 1.25		
7	1.2D				1.9E		
8	1.2D				1.7W		
9	D + L	T_o		R_o	E'		
10	D + L	T_o		R_o	W_t		Ax**
11	D + L	T_a	$1.5P_a$	R_a			
12	(D + L) 1.25	T_a	$1.25P_a$	R_a	1.25E	$Y_r + Y_j + Y_m$	
13	D + L	T_a	P_a	R_a	E'	$Y_r + Y_j + Y_m$	Ax*

Ref; SRP (1981) Sect. 3.8.4 Other Category I structures (concrete)

Notes - Ultimate strength method required by ACI-349 (1977)

Method used in design { working stress / ultimate strength ✓

Loads deemed inapplicable or negligible struck from loading combinations.

Encircled loads are those actually considered in the design. When load factors different from those currently required were used, the factor used is also encircled.

*The FSAR states that; forces or pressure on structure due to rupture of one pipe, is considered. However no specific details are found.

**Wind velocity used is 360 mph as referenced in the FSAR, 360 mph is required by the Reg. Guide 1.76; FSAR states no significant live loads other than crane loads.

For purposes of the SEP Review, demonstration that structural integrity is maintained for load cases 10 and 13 (per current criteria) may be considered as providing reasonable assurance that this structure meets the intent of current design criteria.

COMPARISON OF LOADING COMBINATION CRITERIA

CONCRETE STRUCTURES

PLANT: PALISADES

STRUCTURE: INTAKE STRUCTURE (INCL. ENCLOSURE FOR SERVICE-WATER PUMPS) DISCHARGE STRUCTURE

Combined Loading Cases	Gravity Dead, Live	Thermal	Pressure	Mechanical	Natural Phenomena	Impulsive Loading	Scale Ranking
1	1.4D + 1.7L						
2	1.4D + 1.7L				1.9E		
3	1.4D + 1.7L				1.7W		
4	.75 (1.4D + 1.7L)	.75 x 1.7 T_o		.75 x 1.7 R_o			
5	.75 (1.4D + 1.7 L) $\boxed{1.25 (D + L)}$	$\cancel{.75 \times 1.7 T_o}$		$\cancel{.75 \times 1.7 R_o}$	.75 x 1.$\boxed{E}$ 1.25		
6	.75 (1.4D + 1.7L) $\boxed{1.25 (D + L)}$	$\cancel{.75 \times 1.7 T_o}$		$\cancel{.75 \times 1.7 R_o}$	.75 x 1.$\boxed{W}$ 1.25		
7	1.2D				1.9E		
8	1.2D				1.7W		
9	$\boxed{D + L}$	$\cancel{T_o}$		$\cancel{R_o}$	$\boxed{E'}$		
10	$\boxed{D}$ + L	$\cancel{T_o}$		$\cancel{R_o}$	$\boxed{W_t}$		
11	D + L	$\cancel{T_a}$	$\cancel{1.5 P_a}$	$\cancel{R_a}$			
12	$\boxed{1.25 (D + L)}$	$\cancel{T_a}$	$\cancel{1.25 P_a}$	$\cancel{R_a}$	$\boxed{1.25E}$	$\cancel{Y_r} + \cancel{Y_j} + \cancel{Y_m}$	
13	$\boxed{D + L}$	$\cancel{T_a}$	$\cancel{P_a}$	$\cancel{R_a}$	$\boxed{E'}$	$\cancel{Y_r} + \cancel{Y_j} + \cancel{Y_m}$	

Ref; SRP (1981) Sect. 3.8.4 Other Category I structures (concrete)

Notes - Ultimate strength method required by ACI-349 (1977)

Method used in design { working stress / ultimate strength ✓

Loads deemed inapplicable or negligible struck from loading combinations.

Encircled loads are those actually considered in the design. When load factors different from those currently required were used, the factor used is also encircled.

* Wind velocity used is 360 mph as referenced in the FSAR, 360 mph is required by the Reg. Guide 1.76.; FSAR states no significant live loads other than crane loads

For purposes of the SEP Review, demonstration that structural integrity is maintained for load case 10, 13 (per current criteria) may be considered as providing reasonable assurance that this structure meets the intent of current design criteria.

COMPARISON OF LOADING COMBINATION CRITERIA

CONCRETE STRUCTURES

PLANT: PALISADES

STRUCTURE: TURBINE BUILDING AUX. FEED WATER PUMP ENCLOSURE (ONLY)

Combined Loading Cases	Gravity Dead, Live	Thermal	Pressure	Mechanical	Natural Phenomena	Impulsive Loading	Scale Ranking
1	1.4D + 1.7L						
2	1.4D + 1.7L				1.9E		
3	1.4D + 1.7L				1.7W		
4	.75 (1.4D + 1.7L)	~~.75 x 1.7 T_o~~		.75 x 1.7 R_o			
5	.75 (1.4D + 1.7 L) 1.25(D + L)	~~.75 x 1.7 T_o~~		.75 x 1.7 R_o 1.25	.75 x 1.9E 1.25		
6	.75 (1.4D + 1.7L) 1.25(D+L)	~~.75 x 1.7 T_o~~		.75 x 1.7 R_o 1.25	.75 x 1.W 1.25		
7	1.2D				1.9E		
8	1.2D				1.7W		
9	D + L	~~T_o~~		R_o	E'		
10	D + L	~~T_o~~		R_o	W_t		A_x**
11	D + L	T_a	$1.5P_a$	R_a			
12	1.25 (D + L)	T_a	$1.25P_a$	R_a	1.25E	$Y_r + Y_j + Y_m$	
13	D + L	T_a	P_a	R_a	E'	$Y_r + Y_j + Y_m$	A_x*

Ref; SRP (1981) Sect. 3.8.4 Other Category I structures (concrete)

Notes - Ultimate strength method required by ACI-349 (1977)

- Method used in design { working stress / ultimate strength ✓
- Loads deemed inapplicable or negligible struck from loading combinations.
- Encircled loads are those actually considered in the design. When load factors different from those currently required were used, the factor used is also encircled.

* The FSAR states that; forces or pressure on structure due to rupture of one pipe, is considered. However no specific details are found.

** Wind velocity used is 360 mph as referenced in the FSAR, 360 mph is required by the Reg. Guide 1.76.; FSAR states no significant live loads other than crane loads

For purposes of the SEP Review, demonstration that structural integrity is maintained for load cases 10 and 13 (per current criteria) may be considered as providing reasonable assurance that this structure meets the intent of current design criteria.

11. REVIEW FINDINGS

The most important findings of the review are summarized in this section in tabular form.

The major structural codes used for design of Seismic Category I buildings and structures for the Palisades Nuclear Power Station were:

1. AISC, "Specification for Design, Fabrication, and Erection of Structural Steel for Buildings," 1963
2. ACI 318-63, "Building Code Requirements for Reinforced Concrete," 1963
3. ACI 301-63, "Suggested Specifications for Structural Concrete for Buildings," 1963.

Each of these design codes has been compared with the corresponding structural code governing current licensing criteria. Tables follow, in the order listed above, summarizing important results of these comparisons for each code.

These tables provide:

1. identification by paragraph number (both of the orginal code and of its current counterpart) of code provisions where Scale A or Scale A_x deviations exist.
2. identification of structural elements to which each such provision may apply.

Some listed provisions may apply only to elements that do not exist in the Palisades structures. When FRC could determine that this was the case, such provisions were struck from the list. Any provisions that appeared to be inapplicable for other reasons also were eliminated. Items so removed are listed in Appendix A to this report.

Access to further information concerning code provision changes is provided by additional appendixes. Each pair of codes (the design and the current ones) has a tabular summary within the report (Appendix B) which lists all code changes by scale ranking.

In addition, a separately bound appendix exists for each code pair. This provides:

1. full texts of each revised provision in both the former and current versions

2. comments or conclusions, or both, relevant to the code change

3. the scale ranking of the change.

11.1 MAJOR FINDINGS OF AISC-1963 VS. AISC-1980 CODE COMPARISON

MAJOR FINDINGS OF AISC 1963 VS. AISC 1980 CODE COMPARISON

(Summary of Code Changes with the Potential to Significantly Degrade Perceived Margin of Safety)

Scale A

Referenced Subsection AISC 1980	AISC 1963	Structural Elements Potentially Affected	Comments
1.5.1.2.2	--	Beam end connection where the top flange is coped and subject to shear, or failure by shear along a plane through fasteners or by a combination of shear along a plane through fasteners plus tension along a perpendicular plane	See case study 1 for details.
1.9.1.2 and Appendix C	1.9.1	Slender compression unstiffened elements subject to axial compression or compression due to bending when actual width-to-thickness ratio exceeds the values specified in subsection 1.9.1.2	New provisions added in the 1980 Code, Appendix C See case study 10 for details.
1.10.6	1.10.6	Hybrid girder - reduction in flange stress	New requirement added in the 1980 Code. Hybrid girders were not covered in the 1963 Code. See case study 9 for details.

MAJOR FINDINGS OF AISC 1963 VS. AISC 1980 CODE COMPARISON

(Summary of Code Changes with the Potential to Significantly Degrade Perceived Margin of Safety)

Scale A (Cont.)

Referenced Subsection AISC 1980	AISC 1963	Structural Elements Potentially Affected	Comments
1.11.4	1.11.4	Shear connectors in composite beams	New requirements added in the 1980 Code regarding the distribution of shear connectors (eqn. 1.11-7). The diameter and spacing of the shear connectors are also subject to new controls.
1.11.5	--	Composite beams or girders with formed steel deck	New requirement added in the 1980 Code
1.14.2.2	--	Axially loaded tension members where the load is transmitted by bolts or rivets through some but not all of the cross-sectional elements of the members	New requirement added in the 1980 Code
1.15.5.2 1.15.5.3 1.15.5.4	--	Restrained members when flange or moment connection plates for end connections of beams and girders are welded to the flange of I or H shaped columns	New requirement added in the 1980 Code
2.9	2.8	Lateral bracing of members to resist lateral and torsional displacement	Scale A $0.0 < M/M_p < 1.0$ C $0.0 > M/M_p > -1.0$ See case study 7 for details.

11.2 MAJOR FINDINGS OF ACI 318-63 VS. ACI 349-76 CODE COMPARISON

MAJOR FINDINGS OF ACI 318-63 VS. ACI 349-76 CODE COMPARISON

(Summary of Code Changes with the Potential to Significantly Degrade Perceived Margin of Safety)

Scale A

Referenced Subsection ACI 349-76	Referenced Subsection ACI 318-63	Structural Elements Potentially Affected	Comments
7.10.3	805	Columns designed for stress reversals with variation of stress from f_y in compression to 1/2 f_y in tension	Splices of the main reinforcement in such columns must be reasonably limited to provide for adequate ductility under all loading conditions.
11.13		Short brackets and corbels which are primary load-carrying members	As this provision is new, any existing corbels or brackets may not meet these criteria and failure of such elements could be non-ductile type failure. Structural integrity may be seriously endangered if the design fails to fulfill these requirements.
11.15	--	Applies to any elements loaded in shear where it is inappropriate to consider shear as a measure of diagonal tension and the loading could induce direct shear type cracks.	Structural integrity may be seriously endangered if the design fails to fulfill these requirements.

MAJOR FINDINGS OF ACI 318-63 VS. ACI 349-76 CODE COMPARISON

(Summary of Code Changes with the Potential to Significantly Degrade Perceived Margin of Safety)

Scale A (Cont.)

Referenced Subsection ACI 349-76	ACI 318-63	Structural Elements Potentially Affected	Comments
11.16	--	All structural walls - those which are primary load carrying, e.g., shear walls and those which serve to provide protection from impacts of missile-type objects.	Guidelines for these kinds of wall loads were not provided by older codes; therefore, structural integrity may be seriously endangered if the design fails to fulfill these requirements.
Appendix A	--	All elements subject to time-dependent and position-dependent temperature variations and restrained so that thermal strains will result in thermal stresses.	For structures subject to effects of pipe break, especially jet impingement, thermal stresses may be significant (Scale A). For structures not subject to effects of pipe break accident, thermal stresses are unlikely to be significant (Scale B).

MAJOR FINDINGS OF ACI 318-63 VS. ACI 349-76 CODE COMPARISON

(Summary of Code Changes with the Potential to Significantly Degrade Perceived Margin of Safety)

Scale A (Cont.)

Referenced Subsection ACI 349-76	ACI 318-63	Structural Elements Potentially Affected	Comments
Appendix B	--	All steel embedments used to transmit loads from attachments into the reinforced concrete structure.	New appendix; therefore, considerable review of older designs is warranted. Since stress analysis associated with these conditions is highly dependent on definition of failure planes and allowable stress for these special conditions, past practice varied with designers' opinions. Stresses may vary significantly from those thought to exist under previous design procedures.

11.3 MAJOR FINDINGS OF ACI 301-63 VS. ACI 301-72 (REVISED 1975) COMPARISON

No Scale A or A_x changes were found in the ACI 301 Code Comparison.

11.4 MAJOR FINDINGS OF ACI 318-63 VS. ASME B&PV CODE, SECTION III, DIVISION 2, 1980 CODE COMPARISON

MAJOR FINDINGS OF ACI 318-63 VS. ASME B&PV CODE, SECTION III, DIVISION 2, 1980 CODE COMPARISON

(Summary of Code Changes with the Potential to Significantly Degrade Perceived Margin of Safety)

Scale A

Referenced Subsection Sec. III 1980	Referenced Subsection ACI 318-63	Structural Elements Potentially Affected	Comments
CC-3421.5	---	Containment and other elements transmitting in-plane shear	New concept. There is no comparable section in ACI 318-63, i.e., no specific section addressing in-plane shear. The general concept used here (that the concrete, under certain conditions, can resist some shear, and the remainder must be carried by reinforcement) is the same as in ACI 318-63. Concepts of in-plane shear and shear friction were not addressed in the old codes and therefore a check of old designs could show some significant decrease in overall prediction of structural integrity.
CC-3421.6	1707	Regions subject to peripheral shear in the region of concentrated forces normal to the shell surface	These equations reduce to $V_c = 4\sqrt{f'_c}$ when membrane stresses are zero, which compares to ACI 318-63 [Sections 1707 (c) and (d)] which address "punching" shear in slabs and footings with the ϕ factor taken care of in the basic shear equation (Section CC-3521.2.1, Eqn. 10).

ASME B&PV CODE, SECTION III, DIVISION 2, 1980
(ACI 359-80) VS. ACI 318-63 CODE COMPARISON

Scale A (Cont.)

Referenced Subsection Sec. III 1980	ACI 318-63	Structural Elements Potentially Affected	Comments
CC-3421.6 (Cont.)			Previous code logic did not address the problem of punching shear as related to diagonal tension, but control was on the average uniform shear stress on a critical section. See case study 13 for details.
CC-3421.7	921	Regions subject to torsion	New defined limit on shear stress due to pure torsion. The equation relates shear stress from a biaxial stress condition (plane stress) to the resulting principal tensile stress and sets the principal tensile stress equal to $6\sqrt{f'_c}$. Previous code superimposed only torsion and transverse shear stresses.
CC-3421.8	---	Bracket and corbels	New provisions. No comparable section in ACI 318-63; therefore, any existing corbels or brackets may not meet these criteria, and failure of such elements could be non-ductile type failure. Structural integrity may be seriously endangered if the design fails to fulfill these requirements.

ASME B&PV CODE, SECTION III, DIVISION 2, 1980
(ACI 359-80) VS. ACI 318-63 CODE COMPARISON

Scale A (Cont.)

Referenced Subsection Sec. III 1980	ACI 318-63	Structural Elements Potentially Affected	Comments
CC-3532.1.2	---	Where biaxial tension exists	ACI 318-63 did not consider the problem of development length in biaxial tension fields.

12. SUMMARY

The table that follows provides a summary of the status of the findings from the Task III-7.B criteria comparison review of structural codes and loading requirements for Category I structures at the Palisades Nuclear Power Station.

The first and second columns of the table show the extent to which all Category I structures external to containment comply with current design criteria codes. The first column applies to the concrete portion of these structures; the second column applies to the portions which are of steel frame construction. The third column applies to concrete structures with regard to original and current specifications for structural concrete. The fourth column applies only to the containment building, including its liner.

The salient feature of this table is the limited number of code change impacts requiring a Scale A ranking. Consequently, resolution, at the structural level, of potential concerns with respect to changes in structural code requirements appears, at least for the Palisades plant, to be an effort of tractable size.

SUMMARY
NUMBER OF CODE CHANGE IMPACTS
FOR PALISADES CATEGORY I STRUCTURES

SCALE RANKING		ACI 318-63 VS. ACI 349-76	AISC 1963 VS. AISC 1980	ACI 301-63 VS. ACI 301-72 (1975 Rev.)	ACI 318-63 VS. ASME B&PV SEC.III Div. 2, 1980
Total Changes Found		82	33	37	39
Do Not Require Further Investigation	A or Ax Not Applicable to Palisades	2 + 4*	11	0	3*
	B	64	10	21	27
	C	6	4	16	4
To Be Further Investigated	A	6	8	0	5
	A_x	0	0	0	0

SCALE RATINGS:

Scale A Change - The new criteria have the potential to substantially impair margins of safety as perceived under the former criteria.

Scale A_x Change - The impact of the code change on margins of safety is not immediately apparent. Scale A_x code changes require analytical studies of model structures to assess the potential magnitude of their effect upon margins of safety.

Scale C Change - The new criteria will give rise to larger margins of safety than were exhibited under the former criteria.

*These changes are related to specified loads and load combinations. Loading criteria changes are separately considered elsewhere.

13. RECOMMENDATIONS

Potential concerns with respect to the ability of Seismic Category I buildings and structures in SEP plants to conform to current structural criteria are raised by the review at the code comparison level. These must ultimately be resolved by examination of individual as-built structures.

It is recommended that Consumers Power Company be requested to take three actions:

1. Review individually all Seismic Category I structures at the Palisades plant to see if any of the structural elements listed in the following table occur in their designs. These are the structural elements for which a potential exists for margins of safety to be less than originally computed, due to criteria changes since plant design and construction. For structures which do incorporate these features, assess the actual impact of the associated code changes on margins of safety.

2. Reexamine the margins of safety of Seismic Category I structures under loads and load combinations which correspond to current criteria. Only those load combinations assigned a Scale A or Scale A_x rating in Section 10 of this report need be considered in this review. If the load combination includes individual loads which have themselves been ranked A or A_x, indicating that they do not conform to current criteria, update such loads.

   Full reanalysis of these structures is not necessarily required. Simple hand computations or appropriate modifications of existing results can qualify as acceptable means of demonstrating structural adequacy.

3. Review Appendix A of this report to confirm that all items listed there have no impact on safety margins at the Palisades plant.

LIST OF STRUCTURAL ELEMENTS TO BE EXAMINED

Structural Elements to be Examined	Code Change Affecting These Elements: New Code	Old Code	Scale
Beams	AISC 1980	AISC 1963	
a. Composite Beams			
1. Shear connectors in composite beams	1.11.4	1.11.4	A
2. Composite beams or girders with formed steel deck	1.11.5	--*	A
b. Hybrid Girders			
Stress in flange	1.10.6	1.10.6	A
Compression Elements	AISC 1980	AISC 1963	
With width-to-thickness ratio higher than specified in 1.9.1.2	1.9.1.2 and Appendix C	1.9.1	A
Tension Members	AISC 1980	AISC 1963	
When load is transmitted by bolts or rivets	1.14.2.2	--	A
Connections	AISC 1980	AISC 1963	
a. Beam ends with top flange coped, if subject to shear	1.5.1.2.2	--	A
b. Connections carrying moment or restrained member connection	1.15.5.2 1.15.5.3 1.15.5.4	--	A

*Double dash (--) indicates that no provisions were provided in the older code.

LIST OF STRUCTURAL ELEMENTS TO BE EXAMINED (Cont.)

Structural Elements to be Examined	Code Change Affecting These Elements: New Code	Old Code	Scale
Members Designed to Operate in an Inelastic Regime	AISC 1980	AISC 1963	
Spacing of lateral bracing	2.9	2.8	A
Short Brackets and Corbels having a shear span-to-depth ratio of unity or less	ACI 349-76 11.13	ACI 318-63 --	A
Shear Walls used as a primary load-carrying member	ACI 349-76 11.16	ACI 318-63 --	A
Precast Concrete Structural Elements, where shear is not a member of diagonal tension	ACI 349-76 11.15	ACI 318-63 --	A
Concrete Regions Subject to High Temperatures	ACI 349-76	ACI 318-63	
Time-dependent and position-dependent temperature variations	Appendix A	--	A
Columns with Spliced Reinforcement	ACI 349-76	ACI 318-63	
subject to stress reversals; f_y in compression to 1/2 f_y in tension	7.10.3	805	A
Steel Embedments used to transmit load to concrete	ACI 349-76 Appendix B	ACI 318-63 --	A
Containment and Other Elements, transmitting In-plane shear	B&PV Code Section III, Div. 2, 1980 CC-3421.5	ACI 318-63 --	A
Region of shell carrying concentrated forces normal to the shell surface (see case study 13 for details)	B&PV Code, Section III, Div. 2, 1980 CC-3421.6	ACI 318-63 1707	A

LIST OF STRUCTURAL ELEMENTS TO BE EXAMINED (Cont.)

Structural Elements to be Examined	Code Change Affecting These Elements — New Code	Old Code	Scale
Region of shell under torsion	B&PV Code Section III, Div. 2, 1980 CC-3421.7	ACI 318-63 921	A
Elements Subject to Biaxial Tension	B&PV Code, Section III, Div. 2, 1980 CC-3532.1.2	ACI 318-63 --	A
Brackets and Corbels	B&PV Code, Section III, Div. 2, 1980 CC-3421.8	ACI 318-63 --	A

14. REFERENCES

1. NRC Standard Review Plan, NUREG-0800 (Formerly NUREG-75/087), Rev. 1, July 1981

2. AISC, "Specification for Design, Fabrication, and Erection of Structural Steel for Buildings," 1963

3. ACI 318-63, "Building Code Requirements for Reinforced Concrete" 1963

4. ACI 301-63, "Suggested Specifications for Structural Concrete for Buildings," 1963

5. NRC Docket No. 50-255, memorandum dated June 30, 1978 (A. J. Ignatonis to M. H. Fletcher (NRC), Subject: Palisades quality group and seismic design classification, TAC No. 07349)

APPENDIX A

SCALE A AND A_x CHANGES

DEEMED INAPPROPRIATE TO PALISADES PLANT

APPENDIX A-1

AISC 1963 VS. AISC 1980 CODE COMPARISON

(SCALE A OR A_x CHANGES DEEMED NOT APPLICABLE TO PALISADES OR CODE CHANGES RELATED TO LOADS OR LOAD COMBINATIONS AND THEREFORE TREATED ELSEWHERE)

AISC 1963 VS. AISC 1980 CODE COMPARISON

Referenced Subsection AISC 1980	AISC 1963	Structural Elements Potentially Affected		Comments
1.5.1.1	1.5.1.1	Structural members under tension, except for pin connected members		Structural steel used in Palisades Cat. I structures is A-36. Thus, $F_y < 0.83\ F_u$ Therefore, Scale C for Palisades.
		Limitations	**Scale**	
		$F_y \leq 0.833\ F_u$	C	
		$0.833\ F_u < F_y < 0.875\ F_u$	B	
		$F_y \geq 0.875\ F_u$	A	
2.4 1st Para.	2.3 1st Para.	Slenderness ratio for columns. Must satisfy: $\frac{l}{r} \leq \sqrt{\frac{2\pi^2 E}{F_y}}$		
			Scale	Scale C for Palisades. See case study 4 for details.
		$F_y \leq 40$ ksi	C	
		$40 < F_y < 44$ ksi	B	
		$F_y \geq 44$ ksi	A	
2.7	2.6	Flanges of rolled W, M, or S shapes and similar built-up single-web shapes subject to compression		Scale C for Palisades. See case study 6 for details.
			Scale	
		$F_y \leq 36$ ksi	C	
		$36 < F_y < 38$ ksi	B	
		$F_y \geq 38$ ksi	A	

AISC 1963 VS. AISC 1980 CODE COMPARISON

Referenced Subsection AISC 1980	AISC 1963	Structural Elements Potentially Affected	Comments
1.5.1.4.1 Subpara. 6	1.5.1.4.1	Box-shaped members (subject to bending) of rectangular cross section whose depth is not more than 6 times its width and whose flange thickness is not more than 2 times the web thickness New requirement in the 1980 Code	Box-shaped members not found to be used in Palisades Cat. I structures; therefore, not applicable
1.5.1.4.1 Subpara. 7	1.5.1.4.1	Hollow circular sections subject to bending New requirement in the 1980 Code	Hollow circular sections not found to be used in Palisades Cat. I structures; therefore, not applicable
1.5.1.4.4	--	Lateral support requirements for box sections whose depth is larger than 6 times their width New requirement in the 1980 Code	Box section members not found to be used in Palisades Cat. I structures; therefore; not applicable
1.5.2.2	1.7	Rivets, bolts, and threaded parts subject to 20,000 cycles or more	Cat. I structures are not subject to such cyclic loading; therefore, not applicable
1.7 and Appendix B	1.7	Members and connections subject to 20,000 cycles or more	Cat. I structures are not subject to such cyclic loading; therefore, not applicable

AISC 1963 VS. AISC 1980 CODE COMPARISON

Referenced Subsection AISC 1980	AISC 1963	Structural Elements Potentially Affected	Comments
1.9.2.3 and Appendix C	--	Circular tubular elements subject to axial compression New requirements added to the 1980 Code	Circular tubular elements are not found to be used in Palisades Cat. I structures; therefore, not applicable
1.13.3	--	Roof surface not provided with sufficient slope towards points of free drainage or adequate individual drains to prevent the accumulation of rain water (ponding)	
Appendix D	--	Web tapered members New requirement added in the 1980 Code	Web tapered members are not found to be used in Palisades Cat. I structures; therefore, not applicable

APPENDIX A-2

ACI 318-63 VS. ACI 349-76 CODE COMPARISON

(SCALE A OR A_x CHANGES DEEMED NOT APPLICABLE TO PALISADES
OR CODE CHANGES RELATED TO LOADS OR LOAD COMBINATIONS
AND THEREFORE TREATED ELSEWHERE)

ACI 318-63 VS. ACI 349-76 CODE COMPARISON

Referenced Section ACI 349-76	ACI 318-63	Structural Elements Potentially Affected	Comments
Chapter 9 9.1, 9.2, & 9.3 most specifically	Chapter 15	All primary load-carrying members or elements of the structural system are potentially affected. Definition of new loads not normally used in design of traditional buildings and redefinition of load factors and capacity reduction factors have altered the traditional analysis requirements.*	
10.1 and 10.10	--	All primary load-carrying members Design loads here refer to Chapter 9 load combinations.*	
11.1	--	All primary load-carrying members Design loads here refer to Chapter 9 load combinations.*	
18.1.4 and 18.4.2		Prestressed concrete elements New loadings here refer to Chapter 9 load combinations.*	No prestressed elements outside primary containment; therefore, not applicable.
Chapter 19	--	Shell structures with thickness equal to or greater than 12 in This chapter is completely new; therefore, shell structures designed by the general criteria of older codes may not satisfy all aspects of this chapter. This chapter also refers to Chapter 9 load provisions.	No shell structure except primary containment; therefore, not applicable.

*Special treatment of loads and load combinations is addressed in other sections of the report.

ACI 318-63 VS. ACI 349-76 CODE COMPARISON

Referenced Section		Structural Elements Potentially Affected	Comments
ACI 349-76	ACI 318-63		
Appendix C	--	All elements whose failure under impulsive and impactive loads must be precluded New appendix; therefore, consideration and review of older designs is considered important. Since stress analysis associated with these conditions is highly dependent on definition of failure planes and allowable stress for these special conditions, past practice varied with designers' opinions. Stresses may vary significantly from those thought to exist under previous design procedures.	

APPENDIX A-3

ACI 318-63 VS. ASME B&PV CODE, SECTION III,

DIVISION 2, 1980 (ACI 359-80) CODE COMPARISON

(SCALE A OR A_x CHANGES DEEMED NOT APPLICABLE TO PALISADES OR CODE CHANGES RELATED TO LOAD COMBINATIONS AND THEREFORE TREATED ELSEWHERE)

ACI 318-63 VS. AMSE B&PV CODE, SECTION III,
DIVISION 2, 1980 (ACI 359-80) CODE COMPARISON

Referenced Section: Sec. III 1980	Referenced Section: ACI 318-63	Structural Elements Potentially Affected	Comments
CC-3230	1506	Containment (load combinations and applicable load factor)*	Definition of new loads not normally used in design of traditional buildings.
Table CC-3230-1	1506	Containment (load combinations and applicable load factor)*	Definition of loads and load combinations along with new load factors have altered the traditional analysis requirements.
CC-3900 All sections in this chapter	---	Concrete containment*	New design criteria. ACI 318-63 did not contain design criteria for loading such as impulse or missile impact. Therefore, no comparison is possible for this section.

*Special treatment of loads and load combinations is addressed in other sections of the report.

APPENDIX B

SUMMARIES OF CODE COMPARISON FINDINGS

APPENDIX B-1

AISC 1963 VS. AISC 1980

SUMMARY OF CODE COMPARISON

AISC 1963 VS. AISC 1980
SUMMARY OF CODE COMPARISON

Scale A

Referenced Subsection AISC 1980	Referenced Subsection AISC 1963	Structural Elements Potentially Affected	Comments: Limitations	Comments: Scale
1.5.1.1	1.5.1.1	Structural members under tension, except for pin connected members		
			$F_y \leq 0.833\ F_u$	C
			$0.833\ F_u < F_y < 0.875\ F_u$	B
			$F_y \geq 0.875\ F_u$	A
1.5.1.2.2	--	Beam end connection where the top flange is coped and subject to shear, failure by shear along a plane through fasteners, or shear and tension along and perpendicular to a plane through fasteners	See case study 1 for details.	
1.5.1.4.1 Subpara. 6	1.5.1.4.1	Box-shaped members (subject to bending) of rectangular cross section whose depth is not more than 6 times their width and whose flange thickness is not more than 2 times the web thickness	New requirement in the 1980 Code	
1.5.1.4.1 Subpara. 7	1.5.1.4.1	Hollow circular sections subject to bending	New requirement in the 1980 Code	
1.5.1.4.4	--	Lateral support requirements for box sections whose depth is larger than 6 times their width	New requirement in the 1980 Code	
1.5.2.2	1.7	Rivets, bolts, and threaded parts subject to 20,000 cycles or more	Change in the requirements	

AISC 1963 VS. AISC 1980
SUMMARY OF CODE COMPARISON

Scale A (Cont.)

Referenced Subsection AISC 1980	Referenced Subsection AISC 1963	Structural Elements Potentially Affected	Comments
1.7 and Appendix B	1.7	Members and connections subject to 20,000 cycles or more	Change in the requirements
1.9.1.2 and Appendix C	1.9.1	Slender compression unstiffened elements subject to axial compression or compression due to bending when actual width-to-thickness ratio exceeds the values specified in subsection 1.9.1.2	New provisions added in the 1980 Code, Appendix C. See case study 10 for details.
1.9.2.3 and Appendix C	--	Circular tubular elements subject to axial compression	New requirements added in the 1980 Code
1.10.6	1.10.6	Hybrid girder - reduction in flange stress	New requirement added in the 1980 Code. Hybrid girders were not covered in the 1963 Code. See case study 9 for details.
1.11.4	1.11.4	Shear connectors in composite beams	New requirements added in the 1980 Code regarding the distribution of shear connectors (eqn. 1.11-7). The diameter and spacing of the shear connectors are also introduced.
1.11.5	--	Composite beams or girders with formed steel deck	New requirements added in the 1980 Code
1.15.5.2 1.15.5.3 1.15.5.4	--	Restrained members when flange or moment connection plates for end connections of beams and girders are welded to the flange of I or H shaped columns	New requirement added in the 1980 Code

AISC 1963 VS. AISC 1980
SUMMARY OF CODE COMPARISON

Scale A (Cont.)

Referenced Subsection AISC 1980	AISC 1963	Structural Elements Potentially Affected	Comments	
1.13.3	--	Roof surface not provided with sufficient slope towards points of free drainage or adequate individual drains to prevent the accumulation of rain water (ponding)		
1.14.2.2	--	Axially loaded tension members where the load is transmitted by bolts or rivets through some but not all of the cross-sectional elements of the members	New requirement added in the 1980 Code	
2.4 1st Para.	2.3 1st Para.	Slenderness ratio for columns must satisfy $\frac{l}{r} \leq \sqrt{\frac{2\pi^2 E}{F_y}}$	See case study 4 for details.	Scale
			$F_y \leq 40$ ksi	C
			$40 < F_y < 44$ ksi	B
			$F_y \geq 44$ ksi	A
2.7	2.6	Flanges of rolled W, M, or S shapes and similar built-up single-web shapes subject to compression	See case study 6 for details.	Scale
			$F_y \leq 36$ ksi	C
			$36 < F_y < 38$ ksi	B
			$F_y \geq 38$ ksi	A
2.9	2.8	Lateral bracing of members to resist lateral and torsional displacement	See case study 7 for details.	
Appendix D	--	Web tapered members	New requirements added in the 1980 Code	

AISC 1963 VS. AISC 1980
SUMMARY OF CODE COMPARISON

Scale B

Referenced Subsection AISC 1980	AISC 1963	Structural Elements Potentially Affected	Comments
1.9.2.2	1.9.2	Flanges of square and rectangular box sections of uniform thickness, of stiffened elements, when subject to axial compression or to uniform compression due to bending	The 1980 Code limit on width-to-thickness ratio of flanges is slightly more stringent than that of the 1963 Code.
1.10.1	--	Hybrid girders	Hybrid girders were not covered in the 1963 Code. Application of the new requirement could not be much different from other rational method.
1.11.4	1.11.4	Flat soffit concrete slabs, using rotary kiln produced aggregates conforming to ASTM C330	Lightweight concrete is not permitted in nuclear plants as structural members (Ref. ACI-349).
1.13.2	--	Beams and girders supporting large floor areas free of partitions or other source of damping, where transient vibration due to pedestrian traffic might not be acceptable	Lightweight construction not applicable to nuclear structures which are designed for greater loads
1.14.6.1.3	--	Flare type groove welds when flush to the surface of the solid section of the bar	
1.16.4.2	1.16.4	Fasteners, minimum spacing, requirements between fasteners	
1.16.5	1.16.5	Structural joints, edge distances of holes for bolts and rivets	

AISC 1963 VS. AISC 1980
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Subsection AISC 1980	Referenced Subsection AISC 1963	Structural Elements Potentially Affected	Comments
1.15.5.5	--	Connections having high shear in the column web	New insert in the 1980 Code
2.3.1 2.3.2	--	Braced and unbraced multi-story frame - instability effect	Instability effect on short buildings will have negligible effect.
2.4	2.3	Members subject to combined axial and bending moments	Procedure used in the 1963 Code for the interaction analysis is replaced by a different procedure. See case study 8 for details.

AISC 1963 VS. AISC 1980
SUMMARY OF CODE COMPARISON

Scale C

Referenced Subsection AISC 1980	AISC 1963	Structural Elements Potentially Affected	Comments
1.3.3	1.3.3	Support girders and their connections - pendant operated traveling cranes	
		The 1963 Code requires 25% increase in live loads to allow for impact as applied to traveling cranes, while the 1980 Code requires 10% increase.	The 1963 Code requirement is more stringent, and, therefore, conservative.
1.5.1.5.3	1.5.2.2	Bolts and rivets - projected area - in shear connections $F_p = 1.5\ F_u$ (1980 Code) $F_p = 1.35\ F_y$ (1963 Code)	Results using 1963 Code are conservative.
1.10.5.3	1.10.5.3	Stiffeners in girders - spacing between stiffeners at end panels, at panels containing large holes, and at panels adjacent to panels containing large holes	New design concept added in 1980 Code giving less stringent requirements. See case study 5 for details.
1.11.4	1.11.4	Continuous composite beams, where longitudinal reinforcing steel is considered to act compositely with the steel beam in the negative moment regions	New requirement added in the 1980 Code

APPENDIX B-2

ACI 318-63 VS. ACI 349-76

SUMMARY OF CODE COMPARISON

ACI 318-63 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale A

Referenced Section			
ACI 349-76	ACI 318-63	Structural Elements Potentially Affected	Comments
7.10.3	805	Columns designed for stress reversals with variation of stress from f_y in compression to 1/2 f_y in tension	Splices of the main reinforcement in such columns must be reasonably limited to provide for adequate ductility under all loading conditions.
Chapter 9 9.1, 9.2, & 9.3 most specifically	Chapter 15	All primary load-carrying members or elements of the structural system are potentially affected	Definition of new loads not normally used in design of traditional buildings and redefinition of load factors and capacity reduction factors has altered the traditional analysis requirements.*
10.1 and 10.10	--	All primary load-carrying members	Design loads here refer to Chapter 9 load combinations.*
11.1	--	All primary load-carrying members	Design loads here refer to Chapter 9 load combinations.*
11.13	--	Short brackets and corbels which are primary load-carrying members	As this provision is new, any existing corbels or brackets may not meet these criteria and failure of such elements could be non-ductile type failure. Structural integrity

*Special treatment of load and loading combinations is addressed in other sections of the report.

ACI 318-63 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale A (Cont.)

Referenced Section ACI 349-76	Referenced Section ACI 318-63	Structural Elements Potentially Affected	Comments
11.13 (Cont.)			may be seriously endangered if the design fails to fulfill these requirements.
11.15	--	Applies to any elements loaded in shear where it is inappropriate to consider shear as a measure of diagonal tension and the loading could induce direct shear-type cracks	Structural integrity may be seriously endangered if the design fails to fulfill these requirements.
11.16	--	All structural walls - those which are primary load carrying, e.g., shear walls and those which serve to provide protection from impacts of missile-type objects	Guidelines for these kinds of wall loads were not provided by older codes; therefore, structural integrity may be seriously endangered if the design fails to fulfill these requirements.
18.1.4 and 18.4.2	--	Prestressed concrete elements	New load combinations here refer to Chapter 9 load combinations.*
Chapter 19	--	Shell structures with thickness equal to or greater than 12 inches	This chapter is completely new; therefore, shell structures designed by the general criteria of older codes may not satisfy all aspects of this chapter.

*Special treatment of loads and loading combinations is addressed in other sections of the report.

ACI 318-63 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale A (Cont.)

Referenced Section ACI 349-76	Referenced Section ACI 318-63	Structural Elements Potentially Affected	Comments
Chapter 19 (Cont.)			Additionally, this chapter refers to Chapter 9 provisions.
Appendix A	--	All elements subject to time-dependent and position-dependent temperature variations and which are restrained such that thermal strains will result in thermal stresses	New appendix; older Code did not give specific guidelines on temperature limits for concrete. The possible effects of strength loss in concrete at high temperatures should be assessed.
Appendix B	--	All steel embedments used to transmit loads from attachments into the reinforced concrete structures	New appendix; therefore, considerable review of older designs is warranted.**
Appendix C	--	All elements whose failure under impulsive and impactive loads must be precluded	New appendix; therefore, considerations and review of older designs is considered important.**

**Since stress analysis associated with these conditions is highly dependent on definition of failure planes and allowable stress for these special conditions, past practice varied with designers' opinions. Stresses may vary significantly from those thought to exist under previous design procedures.

ACI 318-63 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B

Referenced Section ACI 349-76	Referenced Section ACI 318-63	Structural Elements Potentially Affected	Comments
1.3.2	103(b)	Ambient temperature control for concrete inspection - upper limit reduced 5° (from 100°F to 95°F) applies to all structural concrete	Tighter control to ensure adequate control of curing environment for cast-in-place concrete.
1.5	--	Requirement of a "Quality Assurance Program" is new. Applies to all structural concrete	Previous codes required inspection but not the establishment of a quality assurance program.
Chapter 3	Chapter 4	Any elements containing steel with $f_y > 60,000$ psi or lightweight concrete	Use of lightweight concrete in a nuclear plant not likely. Elements containing steel with $f_y > 60,000$ psi may have inadequate ductility or excessive deflections at service loads.
3.2	402	Cement	This serves to clarify intent of previous code.
3.3	403	Aggregate	Eliminated reference to lightweight aggregate.
3.3.1	403	Any structural concrete covered by ACI 349-76 and expected to provide for radiation shielding in addition to structural capacity	Controls of ASTM C637, "Standard Specifications for Aggregates for Radiation Shielding Concrete," closely parallel those for ASTM C33, "Standard Specification for Concrete Aggregates."

ACI 318-63 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Section ACI 349-76	Referenced Section ACI 318-63	Structural Elements Potentially Affected	Comments
3.3.3	403	Aggregate	To ensure adequate control.
3.4.2	404	Water for concrete	Improve quality control measures.
3.5	405	Metal reinforcement	Removed all reference to steel with f_y > 60,000 psi.
3.6	406, 407 & 408	Concrete mixtures	Added requirements to improve quality control.
4.1 and 4.2	501 & 502	Concrete proportioning	Proportioning logic improved to account for statistical variation and statistical quality control.
4.3	504	Evaluation and acceptance of concrete	Added provision to allow for design specified strength at age > 28 days to be used. Not considered to be a problem, since large cross sections will allow concrete in place to continue to hydrate.
5.7	607	Curing of very large concrete elements and control of hydration temperature	Attention to this is required because of the thicker elements encountered in nuclear-related structures.
6.3.3	--	All structural elements with embedded piping containing high temperature materials in excess	Previous codes did not address the problem of long periods of exposure to high temperature and

ACI 318-63 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Section ACI 349-76	Referenced Section ACI 318-63	Structural Elements Potentially Affected	Comments
6.3.3 (Cont.)		of 150°F, or 200°F in localized areas not insulated from the concrete	did not provide for reduction in design allowables to account for strength reduction at high (>150°F) temperatures.
7.5, 7.6, & 7.8	805	Members with spliced reinforcing steel	Sections on splicing and tie requirements amplified to better control strength at splice locations and provide ductility.
7.9	805	Members containing deformed wire fabric	New sections to define requirements for this new material.
7.10 & 7.11	--	Connection of primary load-carrying members and at splices in column steel	To ensure adequate ductility.
7.12.3 7.12.4	--	Lateral ties in columns	To provide for adequate ductility.
7.13.1 through 7.13.3	--	Reinforcement in exposed concrete	New requirements to conform with the expected large thick-nesses in nuclear related structures.
8.6	--	Continuous nonprestressed flexural members.	Allowance for redistri-bution of negative moments has been redefined as a function of the steel percentage.
9.5.1.1	--	Reinforced concrete members subject to bending - deflection limits	Allows for more stringent controls on deflection in special cases.

ACI 318-63 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Section ACI 349-76	Referenced Section ACI 318-63	Structural Elements Potentially Affected	Comments
9.4	1505	Reinforcing steel - design strength limitation	See comments in Chapter 3 summary.
9.5.1.2 through 9.5.1.4	--	Slab and beams - minimum thickness requirements	Minimum thickness generally would not control this type of structure.
9.5.2.4	909	Beams and one-way slabs	Affects serviceability, not strength.
9.5.3	--	Nonprestressed two-way construction	Immediate and long time deflections generally not critical in structures designed for very large live loadings; however, design by ultimate requires more attention to deflection controls.
9.5.4 & 9.5.5	--	Prestressed concrete members	Control of camber, both initial and long time in addition to service load deflection, requires more attention for designs by ultimate strength.
10.2.7	--	Flexural members - new limit on B factor	Lower limit on B of 0.65 would correspond to an f'_c of 8,000 psi. No concrete of this strength likely to be found in a nuclear structure.
10.3.6	--	Compression members, with spiral reinforcement or tied reinforcement, non-prestressed and pre-stressed	Limits on axial design load for these members given in terms of design equations. See case study 2

ACI 318-63 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Section ACI 349-76	Referenced Section ACI 318-63	Structural Elements Potentially Affected	Comments
10.6.1 10.6.2 10.6.3 10.6.4	1508	Beams and one-way slabs	Changes in distribution of reinforcement for crack control.
10.6.5	--	Beams	New insert
10.8.1 10.8.2 10.8.3	912	Compression members, limiting dimensions	Moment magnification concept introduced for compression members. Results using column reduction factors in ACI 318-63 are reasonably the same as using magnification.
10.11.1 10.11.2 10.11.3 10.11.4 10.11.5 10.11.5.1 10.11.5.2 10.11.6 10.11.7 10.12	915 916	Compression members, slenderness effects	For slender columns, moment magnification concept replaces the so-called strength reduction concept but for the limits stated in ACI 318-63 both methods yield equal accuracy and both are acceptable methods.
10.15.1 10.15.2 10.15.3 10.15.4 10.15.5 10.15.6	1404-1406	Composite compression members	New items - no way to compare; ACI 318-63 contained only working stress method of design for these members.
10.17	--	Massive concrete members, more than 48 in thick	New item - no comparison.

ACI 318-63 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Section ACI 349-76	ACI 318-63	Structural Elements Potentially Affected	Comments
11.2.1 11.2.2	--	Concrete flexural members	For nonprestressed members, concept of minimum area of shear reinforcement is new. For prestressed members, Eqn. 11-2 is the same as in ACI 318-63. Requirement of minimum shear reinforcement provides for ductility and restrains inclined crack growth in the event of unexpected loading.
11.7 through 11.8.6	--	Nonprestressed members	Detailed provisions for this load combination were not part of ACI 318-63. These new sections provide a conservative logic which requires that the steel needed for torsion be added to that required for transverse shear, which is consistent with the logic of ACI 318-63. This is not considered to be critical, as ACI 318-63 required the designer to consider torsional stresses; assuming that some rational method was used to account for torsion, no problem is expected to arise.

ACI 318-63 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Section ACI 349-76	Referenced Section ACI 318-63	Structural Elements Potentially Affected	Comments
11.9 through 11.9.6	--	Deep beams	Special provisions for shear stresses in deep beams is new. The minimum steel requirements are similar to the ACI 318-63 requirements of using the wall steel limits. Deep beams designed under previous ACI 318-63 criterion were reinforced as walls at the minimum and therefore no unreinforced section would have resulted.
11.10 through 11.10.7	--	Slabs and footings	New provision for shear reinforcement in slabs or footings for the two-way action condition and new controls where shear head reinforcement is used. Logic consistent with ACI 318-63 for these conditions and change is not considered major.

ACI 318-63 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Section ACI 349-76	Referenced Section ACI 318-63	Structural Elements Potentially Affected	Comments
11.11.1	1707	Slabs and footings	The change which deletes the old requirement that steel be considered as only 50% effective and allows concrete to carry 1/2 the allowable for two-way action is new. Also deleted was the requirement that shear reinforcement not be considered effective in slabs less than 10 in thick. Change is based on recent research which indicates that such reinforcement works even in thin slabs.
11.11.2 through 11.11.2.5	--	Slabs	Details for the design of shearhead is new. ACI 318-63 had no provisions for shearhead design. The requirements in this section for slabs and footings are not likely to have been used in older plant designs. If such devices were used, it is assumed a rational design method was used.
11.12	--	Openings in slabs and footings	Modification for inclusion of shearhead design. See above conclusion.

ACI 318-63 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Section ACI 349-76	Referenced Section ACI 318-63	Structural Elements Potentially Affected	Comments
11.13.1 11.13.2	--	Columns	No problem anticipated since previous code required design consideration by some analysis.
Chapter 12	--	Reinforcement	Development length concept replaces bond stress concept in ACI 318-63. The various l_d lengths in this chapter are based entirely on ACI 318-63 permissible bond stresses. There is essentially no difference in the final design results in a design under the new code compared to ACI 318-63.
12.1.6 through 12.1.63	918(C)	Reinforcement	Modified with minimum added to ACI 318-63, 918(C).
12.2.2 12.2.3	--	Reinforcement	New insert in ACI 349-76.
12.4	--	Reinforcement of special members	New insert. Gives emphasis to special member consideration.
12.8.1 12.8.2	--	Standard hooks	Based on ACI 318-63 bond stress allowables in general; therefore, no major change.

ACI 318-63 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Section ACI 349-76	Referenced Section ACI 318-63	Structural Elements Potentially Affected	Comments
12.10.1 12.10.2(b)	--	Wire fabric	New insert. Use of such reinforcement not likely in Category I structures for nuclear plants.
12.11.2	--	Wire fabric	New insert. Mainly applies to precast prestressed members.
12.13.1.4	--	Wire fabric	New insert. Use of this material for stirrups not likely in heavy members of a nuclear plant.
13.5	--	Slab reinforcement	New details on slab reinforcement intended to produce better crack control and maintain ductility. Past practice was not inconsistent with this in general.
14.2	--	Walls with loads in the Kern area of the thickness	Change of the order of the empirical equation (14-1) makes the solution compatible with Chapter 10 for walls with loads in the Kern area of the thickness.

ACI 318-63 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Section ACI 349-76	Referenced Section ACI 318-63	Structural Elements Potentially Affected	Comments
15.5	--	Footings - shear and development of reinforcement	Changes here are intended to be compatible with change in concept of checking bar development instead of nominal bond stress consistent with Chapter 12.
15.9	--	Minimum thickness of plain footing on piles	Reference to minimum thickness of plain footing on piles which was in ACI 318-63 was removed entirely.
16.2	--	Design considerations for a structure behaving monolithically or not, as well as for joints and bearings.	New but consistent with the intent of previous code.
17.5.3	2505	Horizontal shear stress in any segment	Use of Nominal Average Shear Stress equation (17-1) replaces the theoretical elastic equation (25-1) of ACI 318-63. It provides for easier computation for the designer.
18.4.1	--	Concrete immediately after prestress transfer	Change allows more tension, thus is less conservative but not considered a problem.

ACI 318-63 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Section ACI 349-76	ACI 318-63	Structural Elements Potentially Affected	Comments
18.5	2606	Tendons (steel)	Augmented to include yield and ultimate in the jacking force requirement.
18.7.1	--	Bonded and unbonded members	Eqn. 18-4 is based on more recent test data.
18.9.1 18.9.2 18.9.3	--	Two-way flat plates (solid slabs) having minimum bonded reinforcement	Intended primarily for control of cracking.
18.11.3 18.11.4	--	Bonded reinforcement at supports	New to allow for consideration of the redistribution of negative moments in the design.
18.13 18.14 18.15 18.16.1	--	Prestressed compression members under combined axial load and bending. Unbonded tendons. Post tensioning ducts. Grout for bonded tendons.	New to emphasize details particular to prestressed members not previously addressed in the codes in detail.
18.16.2	--	Proportions of grouting materials	Expanded definition of how grout properties may be determined.
18.16.4	--	Grouting temperature	Expanded definition of temperature controls when grouting.

ACI 318-63 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale C

Referenced Section			
ACI 349-76	ACI 318-63	Structural Elements Potentially Affected	Comments
7.13.4	--	Reinforcement in flexural slabs	
10.14	2306	Bearing - sections controlled by design bearing stresses	ACI 318-63 is more conservative, allowing a stress of $1.9(0.25\ f'_c) = 0.475\ f'_c < 0.6\ f'_c$
11.2.5	1706	Reinforcement concrete members without prestressing	Allowance of spirals as shear reinforcement is new. Requirement, where shear stress exceeds $6\phi\sqrt{f'_c}$, of 2 lines of web reinforcment was removed.
13.0 to end	--	Two-way slabs with multiple square or rectangular panels	Slabs designed by the previous criteria of ACI 318-63 are generally the same or more conservative.
13.4.1.5	--	Equivalent column flexibility stiffness and attached torsional members	Previous code did not consider the effect of stiffness of members normal to the plane of the equivalent frame.
17.5.4 17.5.5	--	Permissible horizontal shear stress for any surface, ties provided or not provided	Nominal increase in allowable shear stress under new code.

APPENDIX B-3

ACI 301-63 VS. ACI 301-72 (REVISED 1975)

SUMMARY OF CODE COMPARISON

ACI 301-63 VS. ACI 301-72 (REVISED 1975)
SUMMARY OF CODE COMPARISON

Scale B

Referenced Section ACI 301-72	ACI 301-63	Structural Elements Potentially Affected	Comments
3.8.2.1 3.8.2.3	309b	Lower strength concrete can be proportioned when "working stress concrete" is used	ACI 301-72 (Rev. 1975) bases proportioning of concrete mixes on the specified strength plus a value determined from the standard deviation of test cylinder strength results. ACI 301-63 bases proportioning for "working stress concrete" on the specified strength plus 15 percent with no mention of standard deviation. High standard deviations in cylinder test results could require more than 15 percent under ACI 301-72 (Rev. 1975)
3.8.2.2 3.8.2.3	309d	Mix proportions could give lower strength concrete	ACI 301-72 (Rev. 1975) requires more strength tests than ACI 301-63 for evaluation of strength and bases the strength to be achieved on the standard deviation of strength test results.
17.3.2.3	1704d	Lower strength concrete could have been used	ACI 301-72 (Rev. 1975) requires core samples to have an average strength at least 85 percent of the specified strength with no single result less than 75 percent of the specified strength. ACI 301-63 simply requires "strength adequate for the intended purpose." If "adequate for the intended purpose" is less than 85 percent of the specified strength, lower strength concrete could be used.

ACI 301-63 VS. ACI 301-72 (REVISED 1975)
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Section ACI 301-72	Referenced Section ACI 301-63	Structural Elements Potentially Affected	Comments
17.2	1702a 1703a	Lower strength concrete could have been used	ACI 301-72 (Rev. 1975) specifies that that no individual strength test result shall fall below the specified strength by more than 500 psi. ACI 301-63 specifies that either 20 percent (1702a) or 10 percent (1703a) of the strength tests can be below the specified strength. Just how far below is not noted.
15.2.6.1	1502b1	Weaker tendon bond possible	ACI 301-72 (Rev. 1975) requires fine aggregate in grout when sheath is more than four times the tendon area. ACI 301-63 requires fine sand addition at five times the tendon area.
15.2.2.1 15.2.2.2 15.2.2.3	1502e1	Prestressing may not be as good	ACI 301-72 (Rev. 1975) gives considerably more detail for bonded and unbonded tendon anchorages and couplings. ACI 301-63 does not seem to address unbonded tendons.
8.4.3	804b	Cure of concrete may not be as good	ACI-301-72 (Rev. 1975) provides for better control of placing temperature. This will give better initial cure.
8.2.2.4	802b4	Concrete may be more nonuniform when placed	ACI 301-72 (Rev. 1975) provides for a maximum slump loss. This gives better control of the characteristics of the placed concrete.

ACI 301-63 VS. ACI 301-72 (REVISED 1975)
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Section ACI 301-72	Referenced Section ACI 301-63	Structural Elements Potentially Affected	Comments
8.3.2	803b	Weaker columns and walls possible	ACI 301-72 (Rev. 1975) provides for a longer setting time for concrete in columns and walls before placing concrete in supported elements.
5.5.2	--	Poor bonding of reinforcement to concrete possible	ACI 301-72 (Rev. 1975) provides for cleaning of reinforcement. ACI 301-63 has no corresponding section.
5.2.5.3	--	Reinforcement may not be as good	ACI 301-72 (Rev. 1975) provides for use of welded deformed steel wire fabric for reinforcement. ACI 301-63 has no corresponding section.
5.2.5.1 5.2.5.2	503a	Reinforcement may not be as good when welded steel wire fabric is used	ACI 301-72 (Rev. 1975) provides a maximum spacing of 12 in for welded intersection in the direction of principal reinforcement.
5.2.1	--	Reinforcement may not have reserve strength and ductility	ACI 301-72 (Rev. 1975) has more stringent yield requirements.
4.6.3	406c	Floors may crack	ACI 301-72 (Rev. 1975) provides for placement of reshores directly under shores above, while ACI 301-63 states that reshores shall be placed "in approximately the same pattern."

ACI 301-63 VS. ACI 301-72 (REVISED 1975)
SUMMARY OF CODE COMPARISON

Scale B (Cont).

Referenced Section ACI 301-72	ACI 301-63	Structural Elements Potentially Affected	Comments
4.6.2	--	Concrete may sag or be lower in strength	ACI 301-72 (Rev. 1975) provides for reshoring no later than the end of the working day when stripping occurs.
4.6.4	--	Concrete may sag or be lower in strength	ACI 301-72 (Rev. 1975) provides for load distribution by reshoring in multistory buildings.
4.2.13	--	Low strength possible if reinforcing steel is distorted	ACI 301-72 (Rev. 1975) requires that equipment runways not rest on reinforcing steel.
3.8.5	--	Possible to have lower strength floors	ACI 301-72 (Rev. 1975) places tighter control on the concrete for floors.
3.7.2 3.4.4	--	Embedments may corrode and lower concrete strength	ACI 301-72 (Rev. 1975) requires that it be demonstrated that mix water does not contain a deleterious amount of chloride ion.
3.4.2 3.4.3	--	Possible lower strength	ACI 301-72 (Rev. 1975) places tighter control on water-cement ratios for watertight structures and structures exposed to chemically aggressive solutions.
1.2	--	Possible damage to green or underage concrete resulting in lower strength	ACI 301-72 (Rev. 1975) provides for limits on loading of emplaced concrete.

ACI 301-63 VS. ACI 301-72 (REVISED 1975)
SUMMARY OF CODE COMPARISON

Scale C

Referenced Section ACI 301-72	Referenced Section ACI 301-63	Structural Elements Potentially Affected	Comments
3.5	305	Better strength resulting from better placement and consolidation	ACI 301-63 gives a minimum slump requirement. ACI 301-72 (Rev. 1975) omits minimum slump which could lead to difficulty in placement and/or consolidation of very low slump concrete. A tolerance of 1 in above maximum slump is allowed provided the average slump does not exceed maximum. Generally the placed concrete could be less uniform and of lower strength.
3.6	306b	Better strength resulting from better placement and consolidation	ACI 301-63 provides for use of single mix design with maximum nominal aggregate size suited to the most critical condition of concreting. ACI 301-72 (Rev. 1975) allows waiver of size requirement if the architect-engineer believes the concrete can be placed and consolidated.
3.8.2.1	309b	Higher strength from better proportioning	ACI 301-63 bases proportioning for "ultimate strength" concrete on the specified strength plus 25%. ACI 301-72 (Rev. 1975) bases proportioning on the specified strength plus a value determined from the standard deviation of test cylinder strengths. The requirement to exceed the specified strength by 25% gives higher strengths than the standard deviation method.

ACI 301-63 VS. ACI 301-72 (REVISED 1975)
SUMMARY OF CODE COMPARISON

Scale C (Cont.)

Referenced Section ACI 301-72	ACI 301-63	Structural Elements Potentially Affected	Comments
4.4.2.2	404c	Better bond to reinforcement gives better strength	ACI 301-63 provides that form coating be applied prior to placing reinforcing steel. ACI 301-72 (Rev. 1975) omits this requirement. If form coating contacts the reinforcement, no bond will develop.
4.5.5	405b	Better strength and less chance of cracking or sagging	ACI 301-63 provides for keeping forms in place until the 28-day strength is attained. ACI 301-72 (Rev. 1975) provides for removal of forms when specified removal strength is reached.
4.6.2	406b	Better strength and less chance of cracking or sagging	Same as above but applied to reshoring.
4.7.1	407a	Better strength by curing longer in forms	ACI 301-63 provides for cylinder field cure under most unfavorable conditions prevailing for any part of structure. ACI 301-72 (Rev. 1975) provides only that the cylinders be cured along with the concrete they represent. Cure of cylinders could give higher strength than the in-place concrete and forms could be removed too soon.

ACI 301-63 VS. ACI 301-72 (REVISED 1975)
SUMMARY OF CODE COMPARISON

Scale C (Cont.)

Referenced Section ACI 301-72	Referenced Section ACI 301-63	Structural Elements Potentially Affected	Comments
5.2.2.1 5.2.2.2	--	Better strength, less chance of cracked reinforcing bars	ACI 301-72 (Rev. 1975) has less stringent bending requirement for reinforcing bars than does ACI 318-63.
5.5.4 5.5.5	505b	Better strength from reinforcement	ACI 301-63 provides for more overlap in welded wire fabric.
12.2.3	1201d	Better strength from better cure of concrete	ACI 301-63 provides for final curing for 7 days with air temperature above 50°F. ACI 301-72 (Rev. 1975) provides for curing for 7 days and compressive strength of test cylinders to be 70 percent of specified strength. This could allow termination of cure too soon.
14.4.1	1404	Better strength resulting from better uniformity	ACI 301-63 provides for a maximum slump of 2 in. ACI 301-72 (Rev. 1975) gives a tolerance on the maximum slump which could lead to nonuniformity in the concrete in place.
15.2.1.1	1502-clb	Higher strength from higher yield prestressing bars	ACI 301-63 requires higher yield stress than does ACI 301-72 (Rev. 1975)
15.2.1.2	1502-c2	Higher strength from better prestressing steel	ACI 301-63 requires that stress curves from the production lot of steel be furnished. ACI 301-72 (Rev. 1975) requires that a typical stress-strain curve be submitted. The use of the typical curve may miss lower strength material.

ACI 301-63 VS. ACI 301-72 (REVISED 1975)
SUMMARY OF CODE COMPARISON

Scale C (Cont.)

Referenced Section ACI 301-72	Referenced Section ACI 301-63	Structural Elements Potentially Affected	Comments
16.3.4.3	1602-4c	Better strength resulting from better cylinder tests	ACI 301-63 requires 3 cylinders to be tested at 28 days; if a cylinder is damaged, the strength is based on the average of two. ACI 301-72 (Rev. 1975) requires only two 28-day cylinders; if one is damaged, the strength is based on the one survivor.
16.3.4.4	1602-4d	Better strength, less chance of substandard concrete	ACI 301-63 requires that less than 100 yd^3 of any class of concrete placed in any one day be represented by 5 tests. ACI 301-72 (Rev. 1975) allows strength tests to be waived on less than 50 yd^3.
17.3.2.3	1704d	Better strength could be developed	ACI 301-63 requires core strengths "adequate for the intended purposes." ACI 301-72 (Rev. 1975) requires an average strength at least 85 percent of the specified strength with no single result less than 75 percent of the specified strength. If "adequate for the intended purpose" is higher than 85 percent of the specified strength, the concrete is stronger.

APPENDIX B-4

ACI 318-63 VS. ASME B&PV CODE, SECTION III, DIVISION 2, 1980

SUMMARY OF CODE COMPARISON

ACI 318-63 VS. ASME B&PV CODE, SECTION III,
DIVISION 2, 1980 (ACI 359-80) CODE COMPARISON

Scale A

Referenced Subsection Sec. III 1980	Referenced Subsection ACI 318-63	Structural Elements Potentially Affected	Comments
CC-3230	1506	Containment (load combinations and applicable load factor)*	Definition of new loads not normally used in design of traditional buildings.
Table CC-3230-1	1506	Containment (load combinations and applicable load factor)*	Definition of loads and load combinations along with new load factors has altered the traditional analysis requirements.
CC-3421.5	---	Containment and other elements transmitting in-plane shear	New concept. There is no comparable section in ACI 318-63, i.e., no specific section addressing in-plane shear. The general concept used here (that the concrete, under certain conditions, can resist some shear, and the remainder must be carried by reinforcement) is the same as in ACI 318-63. Concepts of in-plane shear and shear friction were not addressed in the old codes and therefore a check of old designs could show some significant decrease in overall prediction of structural integrity.

*Special treatment of load and load combinations is addressed in other sections of the report.

ACI 318-63 VS. ASME B&PV CODE, SECTION III,
DIVISION 2, 1980 (ACI 359-80) CODE COMPARISON

Scale A (Cont.)

Referenced Subsection Sec. III 1980	ACI 318-63	Structural Elements Potentially Affected	Comments
CC-3421.6	1707	Peripheral shear in the region of concentrated forces normal to the shell surface	These equations reduce to $V_c = 4\sqrt{f'_c}$ when membrane stresses are zero, which compares to ACI 318-63, Sections 1707 (c) and (d) which address "punching" shear in slabs and footings with the ϕ factor taken care of in the basic shear equation (Section CC-3521.2.1, Eqn. 10). Previous code logic did not address the problem of punching shear as related to diagonal tension, but control was on the average uniform shear stress on a critical section. See case study 12 for details.
CC-3421.7	921	Torsion	New defined limit on shear stress due to pure torsion. The equation relates shear stress from a biaxial stress condition (plane stress) to the resulting principal tensile stress and sets the principal tensile stress equal to $6\sqrt{f'_c}$. Previous code superimposed only torsion and transverse shear stresses. See case study 13 for details.

ACI 318-63 VS. ASME B&PV CODE, SECTION III,
DIVISION 2, 1980 (ACI 359-80) CODE COMPARISON

Scale A (Cont.)

Referenced Subsection Sec. III 1980	Referenced Subsection ACI 318-63	Structural Elements Potentially Affected	Comments
CC-3421.8	---	Bracket and corbels	New provisions. No comparable section in ACI 318-63; therefore, any existing corbels or brackets may not meet these criteria and failure of such elements could be non-ductile type failure. Structural integrity may be seriously endangered if the design fails to fulfill these requirements.
CC-3532.1.2	---	Where biaxial tension exists	ACI 318-63 did not consider the problem of development length in biaxial tension fields.
CC-3900 All sections in this chapter	---	Concrete containment*	New design criteria. ACI 318-63 did not contain design criteria for loading such as impulse or missile impact. Therefore, no comparison is possible for this section.

*Special treatment of load and load combinations is addressed in other sections of the report.

ASME B&PV CODE, SECTION III, DIVISION 2, 1980
(ACI 359-80) VS. ACI 318-63 CODE COMPARISON

Scale B

Referenced Subsection Sec. III 1980	ACI 318-63	Structural Elements Potentially Affected	Comments
CC-3320	---	Shells	Added explicit design guidance for concrete reactor vessels not stated in the previous code. Acceptance of elastic behavior as the basis for analysis is consistent with the logic of the older codes.
CC-3340	---	Penetrations and openings	Added to ensure the consideration of special conditions particular to concrete reactor vessels and containments. These conditions would have been considered in design practice even though not specifically referred to in the old code.
Table CC-3421-1	1503(c)	Containment-allowable stress for factored compression loads	ACI 318-63 allowable concrete compressive stress was 0.85 f'_c if an equivalent rectangular stress block was assumed; also ACI 318-63 made no distinction between primary and secondary stress. ACI 318-63 used 0.003 in/in as the maximum concrete compressive strain at ultimate strength.
CC-3421.4.1	1701	Containment and any section carrying transverse shear	Modified and amplified from ACI 318-63, Section 1701.1. 1. ϕ factors removed from all equations and included in CC-3521.2.1, Eqn. 17.

ASME B&PV CODE, SECTION III, DIVISION 2, 1980
(ACI 359-80) VS. ACI 318-63 CODE COMPARISON

Scale B (Cont.)

Referenced Subsection Sec. III 1980	ACI 318-63	Structural Elements Potentially Affected	Comments
CC-3421.4.1 (Cont.)			2. Separation of equations applicable to sections under axial compression and axial tension. New equations added. 3. Equations applicable to cross sections with combined shear and bending modified for case where $\rho < 0.015$. 4. Modification for low values of ρ will not be a large reduction; therefore, change is not deemed to be major.
CC-3421.4.2	2610(b)	Prestressed concrete sections	ACI 318-63, Eqn. 26-13 is a straight line approximation of Eqn. 8 (the "exact" Mohr's circle solution) with the prestress force shear component "Vp" added. (Ref. ACI 426 R-74) ACI 318-63, Eqn. 26-12 modified to include members with axial load on the cross section and modified to reflect steel percentage. Remaining logic similar to ACI 318-63, Section 2610. Both codes intend to control the principal tensile stress.
CC-3422.1	1508(b)	Reinforcing steel	ACI 318-63 allowed higher f_y if full scale tests show adequate crack control.

ASME B&PV CODE, SECTION III, DIVISION 2, 1980
(ACI 359-80) VS. ACI 318-63 CODE COMPARISON

Scale B (Cont.)

Referenced Subsection Sec. III 1980	Referenced Subsection ACI 318-63	Structural Elements Potentially Affected	Comments
CC-3422.1 (Cont.)			The requirement for tests where $f_y > 60$ ksi was used would provide adequate assurance, in old design, that crack control was maintained.
CC-3422.1	1503(d)	All ordinary reinforcing steel	ACI 318-63-allowed stress for load resisting purposes was f_y. However, a capacity reduction factor ϕ of 0.9 was used in flexure. Therefore, allowable tensile stress due to flexure could be interpreted as limited to some percentage of f_y less than 1.0 f_y and greater than 0.9 f_y. Limiting the allowable tensile stress to 0.9 f_y is in effect the same as applying a capacity reduction factor ϕ of 0.9 to the theoretical equation.
CC-3422.1		All ordinary reinforcing steel	ACI 318-63 had no provision to cover limiting steel strains; therefore, this section is completely new. Traditional concrete design practice has been directed at control of stresses and limiting steel percentages to control ductility.

ACI 318-63 VS. ASME B&PV CODE, SECTION III,
DIVISION 2, 1980 (ACI 359-80) CODE COMPARISON

Scale B (Cont.)

Referenced Subsection Sec. III 1980	ACI 318-63	Structural Elements Potentially Affected	Comments
CC-3422.1 (Cont.)			The logic of providing a control of design parameters at the centroid of all the bars in layered bar arrangement is consistent with older codes and design practice.
CC-3422.2	1503(d)	Stress on reinforcing bars	ACI 318-63 allowed the compressive steel stress limit to be f_y; however, the capacity reduction factor for tied compression members was $\phi = 0.70$ and for spiral ties $\phi = 0.75$, applied to the theoretical equation. As this overall reduction for such members is so large, part of the reduction could be considered as reducing the allowable compressive stress to some level less than f_y; therefore, the $0.9\ f_y$ limit here is consistent with and reasonably similar to the older code.
CC-3423	2608	Tendon system stresses	ACI 318-63, Section 2608 is generally less conservative.
CC-3431.3	---	Shear, torsion, and bearing	ACI 318-63 does not have a strictly comparable section; however, the 50% reduction of the utimate strength requirements on shear and bearing stresses to get the working stress limits is identical to the ACI 318-63 logic and requirements.

ACI 318-63 VS. ASME B&PV CODE, SECTION III,
DIVISION 2, 1980 (ACI 359-80) CODE COMPARISON

Scale B (Cont.)

Referenced Subsection: Sec. III 1980	Referenced Subsection: ACI 318-63	Structural Elements Potentially Affected	Comments
Table CC-3431-1	---	Allowable stresses for service compression loads	Allowable concrete compressive stresses are less conservative than or the same as the ACI 318-63 equivalent allowables.
CC-3432.2	1003(b)	Reinforcing bar (compression)	ACI 318-63 is slightly more conservative in using 0.4 f_y up to a limit of 30 ksi. The upper limit is the same, since ACI 359-80 stipulates max f_y = 60 ksi.
CC-3432.2 (b), (c)	1004	Reinforcing bar (compression)	Logic similar to older codes. Allowance of 1/3 overstress for short duration loading.
CC-3433	2606	Tendon system stress	Limits here are essentially the same as in ACI 318-63 or slightly less conservative; ACI 318-63 limits effective prestress to 0.6 of the ultimate strength or 0.8 of the yield strength, whichever is smaller.
CC-3521	---	Reinforced concrete	Membrane forces in both horizontal and vertical directions are taken by the reinforcing steel, since concrete is not expected to take any tension. Tangential shear in the inclined direction is taken, up to V_c, by the concrete, and the rest by the reinforcing steel. In all cases, the ACI concept of ϕ is incorporated

ACI 318-63 VS. ASME B&PV CODE, SECTION III,
DIVISION 2, 1980 (ACI 359-80) CODE COMPARISON

Scale B (Cont).

Referenced Subsection Sec. III 1980	ACI 318-63	Structural Elements Potentially Affected	Comments
CC-3521 (Cont.)			in the equation as 0.9. While not specifically indicating how to design for membrane stresses, ACI 318-63 indicated the basic premises that tension forces are taken by reinforcing steel (and not concrete) and that concrete can take some shear, but any excess beyond a certain limit must be taken by reinforcing steel.
CC-3521.2.1	1701	Nominal shear stress	Similar to ACI 318-63, with the exception of ϕ, which equals 0.85, being included in the Eqn. 17. Placing ϕ in the stress formula, rather than in the formulae for shear reinforcement, provides the same end result.
CC-3532	---	Where bundled bars are used	Bundled bars were not commonly used prior to 1963; therefore, no criteria were specified in ACI 318-63. In more recent codes, identical requirements are specified for bundled bars.

ASME B&PV CODE, SECTION III, DIVISION 2, 1980
(ACI 359-80) VS. ACI 318-63 CODE COMPARISON

Scale B (Cont.)

Referenced Subsection Sec. III 1980	ACI 318-63	Structural Elements Potentially Affected	Comments
CC-3532.1.2	918(c)	Where tensile steel is terminated in tension zones	Similar to older code, but maximum shear allowed at cutoff point increased to 2/3, as compared to 1/2 in ACI 318-63, over that normally permitted. Slightly less conservative than ACI 318-63. This is not considered critical since good design practice has always avoided bar cutoff in tension zones.
CC-3532.1.2	1801	Where bars carrying stress are to be terminated	Development lengths derived from the basic concept of ACI 318-63 where: bond strength = tensile strength $\Sigma_o \mu L = A_b f_y$ $L = A_b f_y / (\mu \pi D)$ If $\mu = 9.5\sqrt{f'_c}/D$ then $L = 0.0335\ A_b f_y/\sqrt{f'_c}$ With $\phi = 0.85$ $L = 0.0394\ A_b f_y/\sqrt{f'_c}$ No change in basic philosophy for #11 and smaller bars.
CC-3532.3	918(h) 801	Hooked bars	Change in format. New values are similar for small bars and more conservative for large bars and higher yield strength bars. Not considered critical since prior to 1963 the use of f_y > 40 ksi steel was not common.

ASME B&PV CODE SECTION III DIV. 2
1980 (ACI 359-80) VS. ACI 318-63 CODE COMPARISON

Scale B (Cont.)

Referenced Subsection Sec. III 1980	ACI 318-63	Structural Elements Potentially Affected	Comments
CC-3533	919	Shear reinforcement	Essentially the same concepts. Bend of 135° now permitted (versus 180° formerly) and two-piece stirrups now permitted. These are not considered as sacrificing strength. Other items here are identical.
CC-3534.1	---	Bundled bars - any location	Provisions for bundled bars were not considered in ACI 318-63. Bundled bars were not commonly used before the early 1960s. Later codes provide identical provisions.
CC-3536	---	Curved reinforcement	Early codes did not provide detailed information, but good design practice would consider such conditions.
CC-3543	2614	Tendon end anchor reinforcement	Similar to concepts in ACI 318-63, Section 2614 but new statement is more specific. Basic requirements are not changed.
CC-3550	--	Structures integral with containment	Statement here is specific to concrete reactor vessels. The logic of this guideline is consistent with the design logic used for all indeterminate structures.

ASME B&PV CODE SECTION III DIV. 2
1980 (ACI 359-80) VS. ACI 318-63 CODE COMPARISON

Scale B (Cont.)

Referenced Subsection Sec. III 1980	Referenced Subsection ACI 318-63	Structural Elements Potentially Affected	Comments
CC-3550 (Cont.)			ACI 318-63 did not specifically state any guideline in this regard.
CC-3560		Foundation requirements	There is no comparable section in ACI 318-63. These items were assumed to be controlled by the appropriate general building code of which ACI 318-63 was to be a referenced inclusion. All items are considered to be part of common building design practice.

ASME B&PV CODE SECTION III DIV. 2
1980 (ACI 359-80) VS. ACI 318-63 CODE COMPARISON

Scale C

Referenced Subsection Sec. III 1980	ACI 318-63	Structural Elements Potentially Affected	Comments
CC-3421.9	2306(f) and (g)	Bearing	ACI 318-63 is more conservative, allowing a stress of 1.9 (0.25 f'_c) = 0.475 $f'_c < 0.6 f'_c$
CC-3431.2	2605	Concrete (allowable stress in concrete)	Identical to ACI 318-63 logic.
Appendix II	--	Concrete reactor vessels	ACI 318-63 did not contain any criteria for compressive strength modification for multiaxial stress conditions. Therefore, no comparison is possible for Section II-1100. Because of this, ACI 318-63 was more conservative by ignoring the strength increase which accompanies triaxial stress conditions. This section probably does not apply to concrete containment structures.
CC-3531	---	All	Rather conservative for service loads. Using ϕ of 0.9 for flexure, $\frac{U}{\phi} = \frac{1.5}{0.9}$ to $\frac{1.8}{0.9} = 1.67$ to 2.0 for ACI 318-63. By using the value of 2.0, the upper limit of the ratio of factored to service loads is employed.

APPENDIX C

COMPARATIVE EVALUATIONS AND MODEL STUDIES

CASE STUDY -1-

The allowable stress for structural steel subject to shear is specified in section 1.5.1.2 of the AISC code both in the 1963 and 1980 editions as

$F_v = 0.40 \ F_y$ ——— (1) based on the sectional area effective in resisting shear

However, in the 1980 Code a new section 1.5.1.2.2 is introduced stating that;

"AT beam end connections where the top flange is coped, and in similar situations where failure might occur by shear along a plane through the fasteners, or by a combination of shear along a plane through the fasteners plus tension along a perpendicular plane, on the area effective in resisting tearing failure: $F_v = 0.30 \ F_u$ where the effective area is the minimum net failure surface, bounded by the bolt holes."

Referring to the 1980 Commentary and Fig. C.1.5.1.2 The connection allowable capacity in the tearing failure mode can be taken as

$0.30 \ A_v \ F_u + 0.50 \ A_t \ F_u$ ——— (2)

where A_v and A_t are the net shear and net tension areas respectively.

In order to evaluate the effect of the code change, 3 sets of each; Material, beam size & coefficients for web tear out (Table 1-G page 4-11 of the AISC Steel Manual) were used.

The results obtained by using equations (1) & (2) above indicate that the 1980 Code gives less conservative results as shown on the following tabulation.

Therefore, Scale -A-

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project C5257			Page C- 3	
	By M.D	Date OCT. '81	Ch'k'd [illegible] Date 10/81	Rev.	Date

BEAM END CONNECTION WHERE TOP FLANGE IS COPED, CASE STUDY -1-

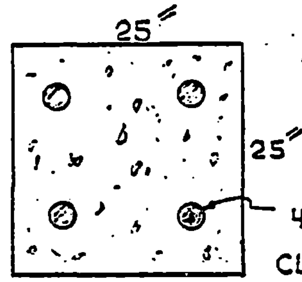
FY,PSI	FU,PSI	H,IN	C1	C2	ALLOWABLE LOAD,LB 1963 CODE	1980 CODE	PCT.
36000.	60000.	12.00	1.00	0.74	172800.	104400.	40.
36000.	60000.	12.00	1.50	0.74	172800.	134400.	22.
36000.	60000.	24.00	1.00	0.74	345600.	104400.	70.
36000.	60000.	24.00	1.00	2.48	345600.	208800.	40.
36000.	60000.	24.00	1.50	0.74	345600.	134400.	61.
36000.	60000.	24.00	1.50	2.48	345600.	238800.	31.
36000.	60000.	24.00	2.25	0.74	345600.	179400.	48.
36000.	60000.	24.00	2.25	2.48	345600.	283800.	18.
36000.	60000.	36.00	1.00	2.48	518400.	208800.	60.
36000.	60000.	36.00	1.00	4.81	518400.	348600.	33.
36000.	60000.	36.00	1.50	2.48	518400.	238800.	54.
36000.	60000.	36.00	1.50	4.81	518400.	378600.	27.
36000.	60000.	36.00	2.25	2.48	518400.	283800.	45.
36000.	60000.	36.00	2.25	4.81	518400.	423600.	18.
50000.	70000.	12.00	1.00	0.74	240000.	121800.	49.
50000.	70000.	12.00	1.50	0.74	240000.	156800.	35.
50000.	70000.	12.00	2.25	0.74	240000.	209300.	13.
50000.	70000.	24.00	1.00	0.74	480000.	121800.	75.
50000.	70000.	24.00	1.00	2.48	480000.	243600.	49.
50000.	70000.	24.00	1.50	0.74	480000.	156800.	67.
50000.	70000.	24.00	1.50	2.48	480000.	278600.	42.
50000.	70000.	24.00	2.25	0.74	480000.	209300.	56.
50000.	70000.	24.00	2.25	2.48	480000.	331100.	31.
50000.	70000.	36.00	1.00	2.48	720000.	243600.	66.
50000.	70000.	36.00	1.00	4.81	720000.	406700.	44.
50000.	70000.	36.00	1.50	2.48	720000.	278600.	61.
50000.	70000.	36.00	1.50	4.81	720000.	441700.	39.
50000.	70000.	36.00	2.25	2.48	720000.	331100.	54.
50000.	70000.	36.00	2.25	4.81	720000.	494200.	31.
65000.	80000.	12.00	1.00	0.74	312000.	139200.	55.
65000.	80000.	12.00	1.50	0.74	312000.	179200.	43.
65000.	80000.	12.00	2.25	0.74	312000.	239200.	23.
65000.	80000.	24.00	1.00	0.74	624000.	139200.	78.
65000.	80000.	24.00	1.00	2.48	624000.	278400.	55.
65000.	80000.	24.00	1.50	0.74	624000.	179200.	71.
65000.	80000.	24.00	1.50	2.48	624000.	318400.	49.
65000.	80000.	24.00	2.25	0.74	624000.	239200.	62.
65000.	80000.	24.00	2.25	2.48	624000.	378400.	39.
65000.	80000.	36.00	1.00	2.48	936000.	278400.	70.
65000.	80000.	36.00	1.00	4.81	936000.	464800.	50.
65000.	80000.	36.00	1.50	2.48	936000.	318400.	66.
65000.	80000.	36.00	1.50	4.81	936000.	504800.	46.
65000.	80000.	36.00	2.25	2.48	936000.	378400.	60.
65000.	80000.	36.00	2.25	4.81	936000.	564800.	40.

NOTES:

1- ALLOWABLE LOADS ARE GIVEN PER INCH OF WEB THICKNESS
2- PCT= PERCENT OF THE REDUCTION OF PERCEIVED MARGIN OF SAFETY

CASE STUDY 2

COMPARING SHORT COLUMNS
BY 318-63 AND 349-76



$A_g = 625\ IN^2$
$\underline{X .01} = P$
$6.25\ IN^2$

$4-\#11 = 6.24\ IN^2$
CLOSE TO 1% P_g

SHORT COLUMNS

SEC. 1403 @ (AND 1402 ACI 318-63)

$$P = .85\left[(A_g(.25 f_c' + f_s \cdot P_g)\right] \qquad \left(\begin{array}{l} f_c = 3{,}000 \pm 1 \ \ PSI \\ f_s = .4 \times 40{,}000 = 16{,}000\ PSI \end{array}\right)$$

$$= .85\left[625\ IN^2(.25(3{,}000) + 16{,}000(.01)\right]$$

$$= .85\left[625(750 + 160)\right] = \underline{483{,}000}^{\#} \text{ (SERVICE LOAD)}$$

BY 349-76 SEC. 10.3.6

$$P_u = \phi\ .80[.85 f_c'(A_g - A_{st}) + f_y A_{st}]$$

$$= .7(.8)\left[(.85)(3{,}000)(625 - 6.24) + 40{,}000(6.24)\right]$$

$$= .56\left[1578{,}000 + 249{,}600\right] = \underline{\underline{1{,}023{,}000}} \text{ (ULT. LOAD)}$$

USING LOAD FACTORS OF D.L. = L.L.

$$\frac{1.4 + 1.7}{2} = 1.55$$

THEN SERVICE LOAD = $\dfrac{1{,}023{,}000}{1.55} = \underline{\underline{660{,}000}}^{\#}$

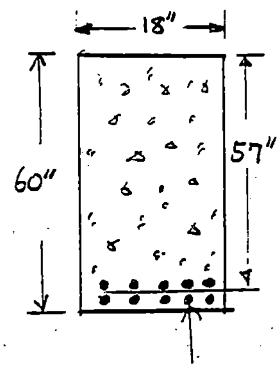
INCREASE OF $\dfrac{660 - 483}{483} \times 100\% = \underline{\underline{36.6\%}}$

CONCLUSION: FOR "SHORT COLUMNS" THE PREVIOUS CODES WERE MUCH MORE CONSERVATIVE

CASE STUDY -3-

Sample Comparison Between Strength (Ultimate) and Alternate (Working Stress) Designs

Sample Section



Allowable Stresses

Concrete : 3000 lb/in^2 grade
($f'_c = 3{,}000$, $f_c = 1350$, $n = 9$)

Reinforcing steel : Grade 40
($f_y = 40{,}000\ lb/in^2$; $f_s = 20{,}000\ lb/in^2$)

$A_s = 10 - \#10$ bars $= 12.66\ in^2$

I. By Strength Design

$$\rho = \frac{12.66}{18 \times 57} = .01234$$

(There is a limit of .0278, But a "reasonable" design is half of this.)

$$q = .01234\left(\frac{40}{3}\right) = .1645$$

$$M_u = .9\left[(18'')(57'')^2(3^k/in^2)(.1645)(1-.59(.1645))\right] = 23{,}450''^k$$

Assuming L.L. = D.L , $U = \frac{1.4+1.7}{2} = 1.55(D+L)$

The moment then is equivalent to a "service" moment of $23{,}450''^k / 1.55 = 15{,}130''^k$

II. BY Alternate Design

Finding the location of the neutral axis $X (= kd)$

$$18\,x\left(\tfrac{x}{2}\right) = 9\,(12.66)(57 - x)$$

solving, $x = kd = 21.27''$

the moment arm $= jd = 57 - \frac{21.27}{3} = 49.91''$

Then $M_c = \frac{1}{2}(1.35\ ^{k}/in^2)(18'')(21.27'')(49.91'') = 12{,}900''^{k}$

and $M_s = 12.66\ in^2 (20\ ^{k}/in^2)(49.91'') = 12{,}640''^{k}$

(Governs)

III Comparison:

$$\frac{15{,}130''^{k} - 12{,}640''^{k}}{12{,}640''^{k}} \times 100\,\% = \underline{19.7\,\%}$$ ADVANTAGE

Conclusion: For Rectangular Beams, The working Stress Designs (commonly used when following the earlier ACI 318 codes) were considerably more conservative.

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project C5257			Page C- 7	
	By M.D	Date SEPT. '81	Ch'k'd [illegible] Date 10/81	Rev.	Date

CASE STUDY -4-

Ref AISC 1980 CODE

Subsection 2.4 Columns

" In the plane of bending of columns which would develop a plastic hinge at ultimate loading, the slenderness ratio $\frac{l}{r}$ shall not exceed Cc, ... "

where $C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$

$E = 29 \times 10^3$ KSI

F_y = yield stress

Therefore $\frac{l}{r} \leq \frac{756.6}{\sqrt{F_y}}$

Ref AISC 1963 Code

Subsection 2.3 Columns

" In the plane of bending of columns which would develop a plastic hinge at ultimate loading, the slenderness ratio shall not exeed 120, ... "

$\frac{l}{r} \leq 120$

which of the two codes is the more restrictive on l/r ratio depends on the yield strength of the steel used for the columns.

1) Both codes give $\frac{l}{r} = 120$ when

$$C_c = \frac{756.6}{\sqrt{F_y}} = 120$$

then,

$$F_y = 40 \text{ KSI}$$

2) The 1980 Code is 5% more conservative when

$$\frac{l}{r} = 114 = \frac{756.6}{\sqrt{F_y}}$$

then, $F_y = 44$ KSI

Conclusion:

	Scale
$F_y \leq 40$ KSI	C
$40 < F_y < 44$	B
$F_y > 44$	A

CASE STUDY -5-

Ref AISC 1980 Code

Subsection 1.10.5.3

" In girders designed on the basis of tension field action, the spacing between stiffeners at end panels, at panels containing large holes, and at panels adjacent to panels containing large holes shall be such that f_v does not exceed the value given" below.

$$F_v = \frac{F_y}{2.89} C_v \leq 0.4 F_y$$

Where

$$C_v = \frac{45000 k}{F_y (h/t)^2} \quad \text{when } C_v < 0.8$$

$$k = 4 + \frac{5.34}{(a/h)^2} \quad \text{when } a/h < 1.0$$

$$= 5.34 + \frac{4}{(a/h)^2} \quad \text{when } a/h > 1.0$$

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	By: MD / Date: SEPT. '81	Ch'k'd: E.M.D. / Date: 10/81	Rev. / Date

Ref AISC 1963 Code

Subsection 1.10.5.3

" The spacing between stiffeners at end panels and panels containing large holes shall be such that the smaller panel dimension a or h shall not exceed

$$\frac{1100t}{\sqrt{f_v}} \text{ "}$$

REF AISC sub section 1.10.5.3

V=240 Kips

EXAMPLE

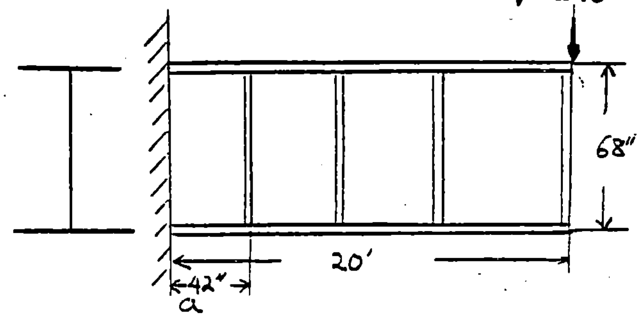
$h = 68''$

$t = .375''$

$A_w = 68 \times \frac{3}{8} = 25.5 \ \text{in}^2$

$V = 240$ Kips

$f_v = \frac{240}{25.5} = 9.06$ KSI



from 1.10.5.3 1963 Code

$$a \text{ or } h \ngtr \frac{11000t}{\sqrt{f_v}} = \frac{11000 \times 3/8}{\sqrt{9.06 \times 1000}} = 43 \text{ in}$$

Which is the distance from the end of the girder to the first transverse stiffener.

By considering the tension field action as specified in 1980 Code subsection 1.10.5.3

$f_v = 9.06$ KSI $\quad \frac{h}{t} = \frac{68}{.375} = 181 \ \ \& \ \frac{a}{h} = \frac{42}{68} = .618$

$$k = 4 + \frac{5.34}{(a/h)^2} = 4 + \frac{5.34}{(.618)^2} = 17.98$$

$$C_v = \frac{45000k}{F_y(h/t)^2} = \frac{45000 \times 17.98}{36\,(181)^2} = .686$$

$$F_v = \frac{F_y}{2.89} C_v \leq .4F_y$$

$= \frac{36}{2.89} \times .686 = 8.54$ KSI & from table 10.36 the

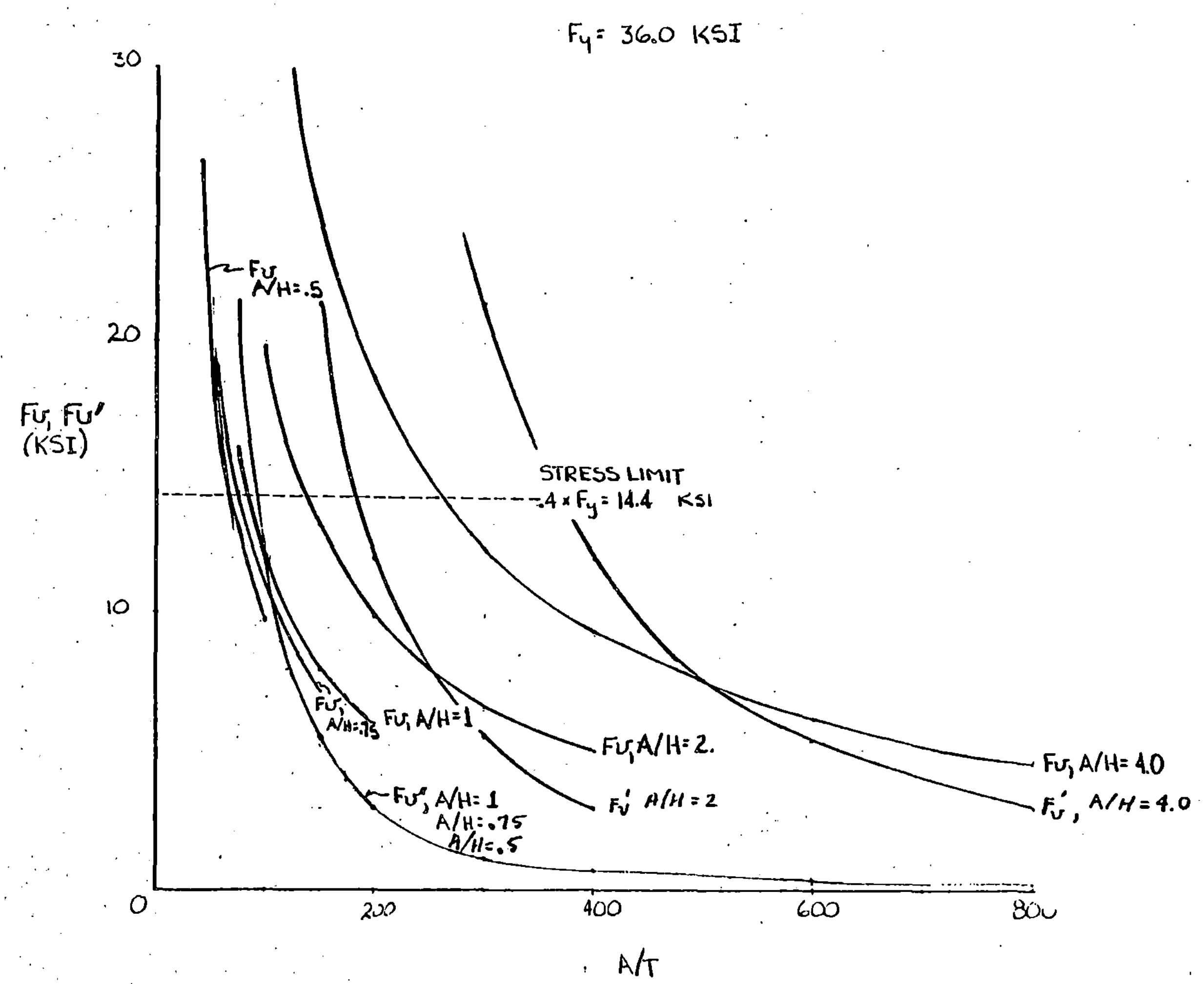
Allowable shear stress $\approx$ 8.6 Ksi (checks Computed Value)

however, lower than f_v of 9.06 Ksi

$\therefore$ Scale B for this example

Remarks

The following two figures show Fv vs. A/T for various values of A/H and Fy.
By knowing the shear stress Fv or Fv′ the A/T value can be abtained and compared with the design A/T. Thus comparison should be examined on a case by case basis.



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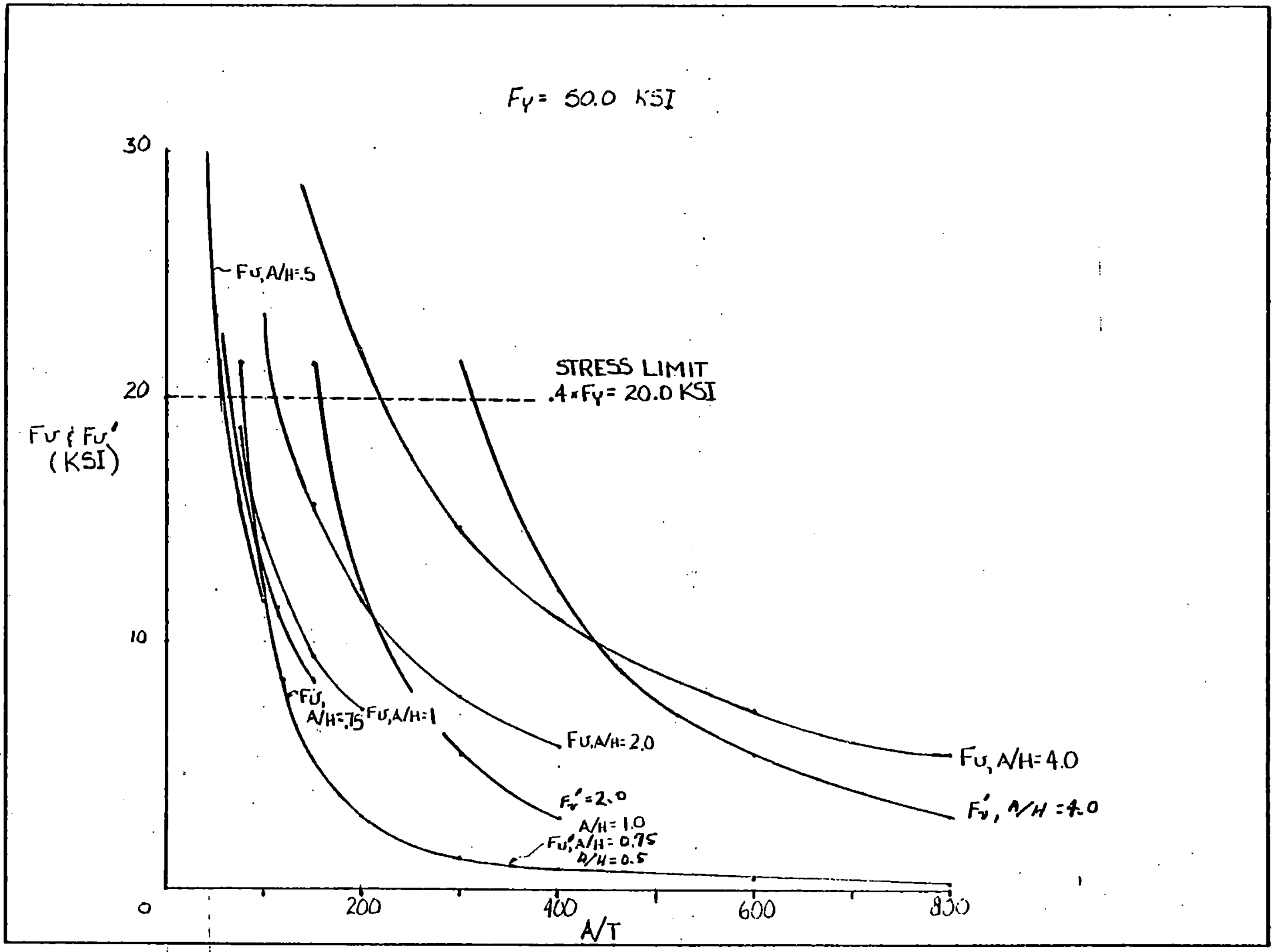
Rev. | Date



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CASE STUDY -6-

Ref AISC 1980 Code
Section 2.7

"The width-thickness ratio for flange of rolled W, M, or S shapes and similar built-up single-web shapes that would be subjected to compression involving hinge rotation under ultimate loading shall not exceed the following values:"

F_y, KSI	$b_f/2t_f$
36	8.5
42	8.0
45	7.4
50	7.0
55	6.6
60	6.3
65	6.0

"The width-thickness ratio of similarly compressed flange plates in box sections and cover plates shall not exceed $190/\sqrt{F_y}$"

Example

$$\frac{b}{t} = \frac{190}{\sqrt{F_y}}$$

F_y KSI	b/t
36	31.7
50	26.9
75	22
100	19

"The depth-thickness ratio of webs of members subjected to plastic bending shall not exceed ...."

$$d/t = \frac{412}{\sqrt{F_y}}\left(1 - 1.4\frac{P}{P_y}\right) \quad \text{when } \frac{P}{P_y} \leq 0.27$$

For $\frac{P}{P_y} = 0.0$

F_y	d/t
36	68.7
50	58.3
75	47.6
100	41.2

$$d/t = \frac{257}{\sqrt{F_y}} \quad \text{when } \frac{P}{P_y} > 0.27$$

F_y	d/t
36	42.8
50	36.3
75	30
100	25.7

Ref AISC 1963 Code
Section 2.6

" Projecting element, that would be subjected to compression involving plastic hinge rotation under ultimate loading shall have width-thickness ratio no greater than the following: "

$b_f/2t_f \leq 8.5$ Rolled Shapes

$b_f/t_f \leq 32$ Box Sections

" The depth-thickness ratio of beam and girder webs subjected to plastic bending " is given by the following formula

$$43 \leq d/w \leq 70 - 100\frac{P}{P_y}$$

Remarks

The 1963 Code take into account material for A36 of $F_y = 36$ KSI or less (note that the two codes are the same for $F_y = 36$).
If the structure was designed using material having higher yield, the design might not be acceptable under present requirements.

$F_y \leq 36$ KSI C

$36 < F_y < 38$ KSI B

$F_y \geq 38$ KSI A

CASE STUDY -7-

Ref AISC 1980 Code
Section 2.9 Lateral Bracing

" Members shall be adequately braced to resist lateral and torsional displacements .... The laterally unsupported distance, l_{cr}, ... shall not exceed the value determined from "

$$\frac{l_{cr}}{r_y} = \frac{1375}{F_y} + 25 \quad \text{when } 1.0 > \frac{M}{M_p} > -0.5$$

or $$\frac{l_{cr}}{r_y} = \frac{1375}{F_y} \quad \text{when } -0.5 \geq \frac{M}{M_p} > -1.0$$

example

l_{cr}/r_y	F_y = 36 KSI	50	75	100
$1 > \frac{M}{M_p} > -.5$	63.2	52.5	43.3	38.75
$-.5 \geq \frac{M}{M_p} > -1.0$	38.2	27.5	18.3	13.75

Ref AISC 1963 Code

Section 2.8 Lateral Bracing

When the moment definition is compatible with the 1980 code, the formula for l_{cr}/r_y becomes:

$$35 < \frac{l_{cr}}{r_y} = 60 + 40\frac{M}{M_p}$$

example

$\frac{M}{M_p}$	$\frac{l_{cr}}{r_y}$
1	100
0	60
-.5	40

CONCLUSIONS

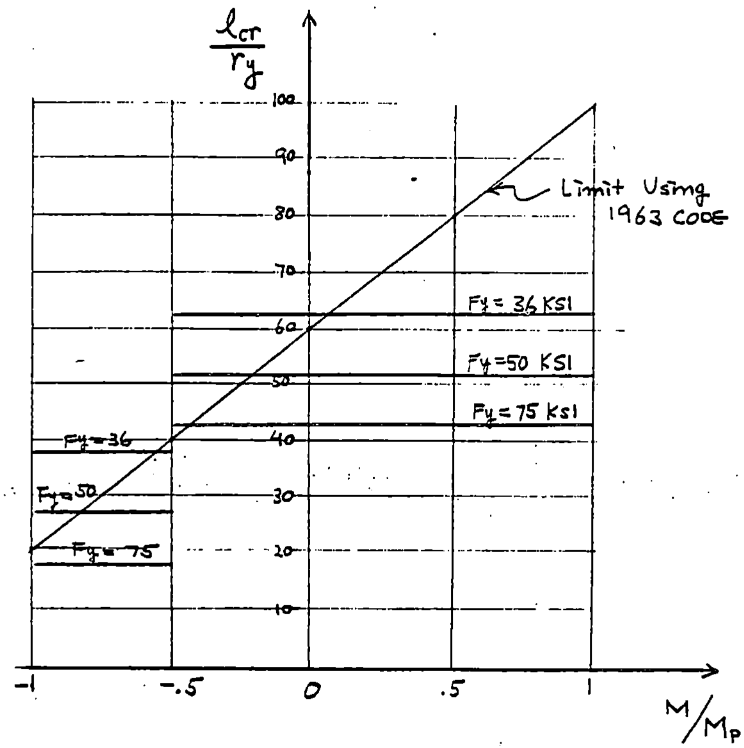
The figure which follows (l_{cr}/r_y vs. M/M_p) indicates that for A-36 steel ($F_y = 36$ ksi)

Scale

$0 < \frac{M}{M_p} < 1$ ——— (A)

$0 > \frac{M}{M_p} > -1$ ——— (C)

Note: The summary is based on material with $F_y = 36$, other material should be examined on a case by case basis.



l_{cr}/r_y
100
90
80
70
60
50
40
30
20
10
Limit Using 1963 CODE
Fy = 36 KSI
Fy = 50 KSI
Fy = 75 ksi
Fy = 36
Fy = 50
Fy = 75
-1
-.5
0
.5
1
M/Mp

CASE STUDY - 8 -

Comparison of Section 2.3, Columns (AISC, 1963) with Section 2.4, Columns (AISC, 1980)

AISC 1963	AISC 1980
1. Slenderness ratio for columns in continuos frames where sideway is not prevented, is limited by Formula (20) $\frac{2P}{P_y} + \frac{\ell}{70r} \leq 1.0$ This limits slenderness Ratio $\frac{\ell}{r} \leq 70$ and axial load not to exceed $0.5 P_y$ for $\frac{\ell}{r} = 0$. Also limited by Formula (26) given below.	1. Slenderness ratio for Columns in continuos frames where Sidesway is not prevented, not limited to only 70. But limited by Formulas (2.9 - 1a) and (2.9 - 1b) given below and $\frac{\ell}{r}$ not to exceed C_c, as given below
2. For columns in braced frames the maximum axial load P shall not exceed $0.6 P_y$.	2. The axial load in columns in braced frames not to exceed $0.85 P_y$

(See Case Study 4 also, for Slenderness ratio)

3. a) Slenderness ratio $\frac{l}{r}$ not to exceed 120

b) The allowable laterally unsupported distance

$$l_{cr} = (60-40)\frac{M}{M_p} r_y,$$

Formula (26) But $l_{cr} \nless 35 r_y$

c) $\frac{Kl}{r_{min}}$ not to exceed 200 in any case

3a. a Slenderness ratio $\frac{l}{r}$ not to exceed C_c

where $C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$

and for $F_y = 36$ Ksi,
$C_c = 126.1$

3 b. The laterally unsupported distance l_{cr} not to exceed the following

$$\frac{l_{cr}}{r_y} = \frac{1375}{F_y} + 25 \quad (2.9-1a)$$

when $+1.0 > \frac{M}{M_p} > -0.5$

And

$$\frac{l_{cr}}{r_y} = \frac{1375}{F_y} \quad (2.9-1b)$$

when $-0.5 \geq \frac{M}{M_p} > -1.0$

3c. $\frac{Kl}{r_{min}}$ not to exceed 200 in any case.

4(a) Interaction formulas for single curvature are

Formula (22)

$$\frac{M}{M_p} \leq B - G\left(\frac{P}{P_y}\right) \leq 1.0$$

$$M \leq M_p$$

and Formula (23)

$$\frac{M}{M_p} \leq 1.0 - H\left(P/P_y\right) - J\left(P/P_y\right)^2$$

Values of B, G, H and J listed in tables as a function of slenderness ratio and F_y

(b) Interaction formulas for double curvature are

Formula (21)

$M \leq M_p$ for $P/P_y \leq 0.15$

$$\frac{M}{M_p} \leq 1.18 - 1.18\left(P/P_y\right) \leq 1.0$$

for $P/P_y \geq 0.15$

and Formula (22)

$$\frac{M}{M_p} \leq B - G\left(\frac{P}{P_y}\right) \leq 1.0 \; ;$$

$$M \leq M_p$$

4. Interaction formulas are

Formula (2.4-2)

$$\frac{P}{P_{cr}} + \frac{C_m M}{\left(1 - \frac{P}{P_e}\right) M_m} \leq 1.0$$

and Formula (2.4-3)

$$\frac{P}{P_y} + \frac{M}{1.18 M_p} \leq 1.0 \; ; \; M \leq M_p$$

where $P_{cr} = 1.7 A F_a$

$$P_e = \frac{23}{12} A F_e'$$

F_a given by (1.5-1) and

F_e' given in Section 1.6.1

$M_m = M_p$ (braced in the weak direction)

$$= \left[1.07 - \frac{(\ell/r_y)\sqrt{F_y}}{3160}\right] M_p \leq M_p$$

(unbraced in weak direction)

a) For single curvature

$$0.6 \leq C_m \leq 1.0$$

b) For double curvature

$$0.4 \leq C_m \leq 0.6$$

For comparison of these specifications, graphs of P/P_y vs M/M_p are drawn for slenderness ratio of 30, 70 and 100. Typical Column 14 WF 150 with $F_y = 36$ ksi has been taken as an example for our purposes Separate graphs are drawn for single curvature ($0.6 \le C_m \le 1.0$) and double Curvature ($0.4 \le C_m \le 0.6$) cases.

For frames with sidesway ($C_m = 0.85$) allowed, graphs of P/P_y vs M/M_p are drawn for two types of columns 14 WF 150 and 12 WF 45, with $F_y = 36$ ksi. Columns assumed to be braced in the weak direction.

It can be inferred from the graphs that in all cases, the major change is the limit of allowable axial load, which is increased from $0.5 P_y$ to $0.75 P_y$ for unbraced columns (Sidesway allowed) and $0.6 P_y$ to $0.85 P_y$ for braced columns. But the acceptable design region in both codes is almost same. For single curvature we notice for $\frac{K\ell}{r} = 30$ the Formula (2.4-2) line for $C_m = 1.0$ is below the formula (23) line, but for $\frac{K\ell}{r} = 70$, they overlap and for $\frac{K\ell}{r} = 100$, the Formula (2.4-2) for $C_m = 1.0$ is above the formula (23) line. Thus for $\frac{K\ell}{r} = 30$ 1980 code being more conservative, while for $\frac{K\ell}{r} = 100$, 1963 code seems to be <u>more conservative</u>. This change can thus be classified best as a <u>**B**</u> change.

F_y = 36 ksi $\quad \frac{kl}{r} = 30 \quad$ 14 w= 150 $\quad$ SINGLE CURVATURE

<u>1963 Code</u>

Formula (22) $\frac{M}{M_p} \leq B - G(P/Py) \leq 1.0$

$M \leq M_p$

Formula (23) $\frac{M}{M_p} \leq 1.0 - H(P/Py) - J(P/Py)^2$

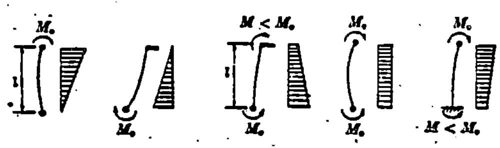
<u>1980 Code</u>

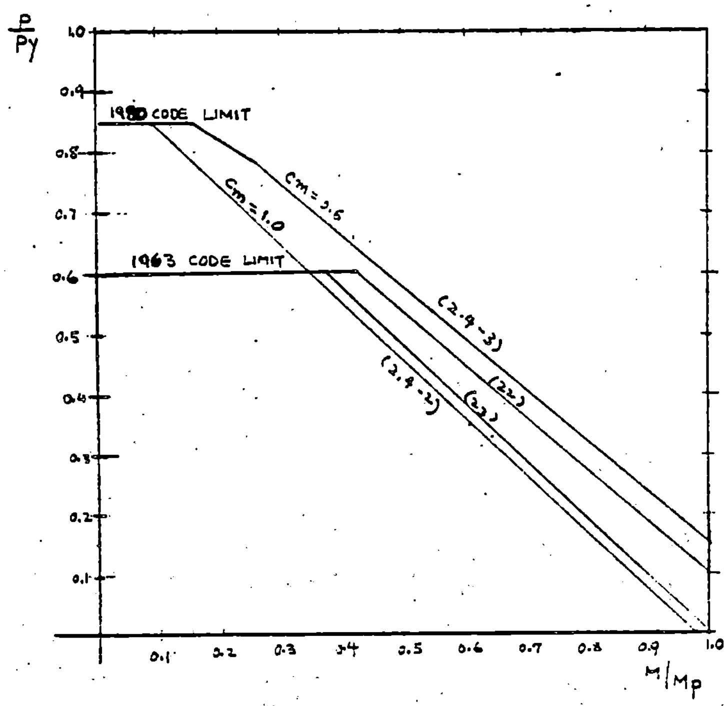
(2.4-2) $\frac{P}{P_{cr}} + \frac{C_m M}{(1 - \frac{P}{P_e})M_p} \leq 1.0$

$0.6 \leq C_m \leq 1.0$

(2.4-3) $\frac{P}{P_y} + \frac{M}{1.18M_p} \leq 1.0, \; M \leq M_p$

<u>TYPICAL EXAMPLES</u>





Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project C5257				Page C- 26
	By RA	Date SEPT '81	Ch'k'd [illegible]	Date 10/81	Rev. Date

$F_y = 36$ ksi $\quad \frac{kl}{r} = 30 \quad$ 14 WF 150 $\quad$ DOUBLE CURVATURE

<u>1963 Code</u>

Formula (21) $M = M_p$ when $P/Py \le 0.15$

$$\frac{M}{M_p} \le 1.18 - 1.18(P/Py) \le 1.0$$

Formula (22) $\frac{M}{M_p} \le B\text{-}G(P/Py) \le 1.0$

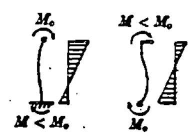
$$M \le M_p$$

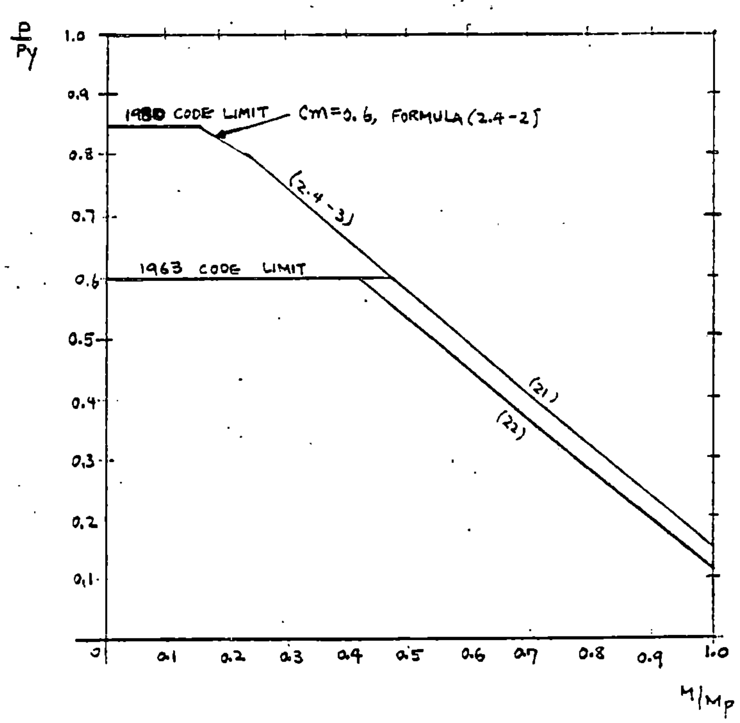
<u>1980 Code</u>

(2.4-2) $\frac{P}{P_{cr}} + \frac{C_m M}{(1 - \frac{P}{P_e})M_p} \le 1.0 \qquad 0.4 \le C_m \le 0.6$

(2.4-3) $\frac{P}{P_y} + \frac{M}{1.18M_p} \le 1.0, \; M \le M_p$

<u>TYPICAL EXAMPLES</u>





F_y = 36 ksi $\frac{kl}{r}$ = 70 14w 150 SINGLE CURVATURE

1963 Code

Formula (22) $\frac{M}{M_p} \leq B-G(P/Py) \leq 1.0$

$M \leq M_p$

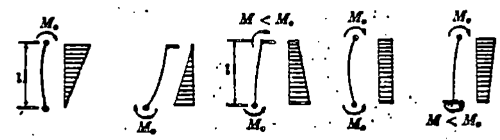
Formula (23) $\frac{M}{M_p} \leq 1.0 - H(P/Py) - J(P/Py)^2$

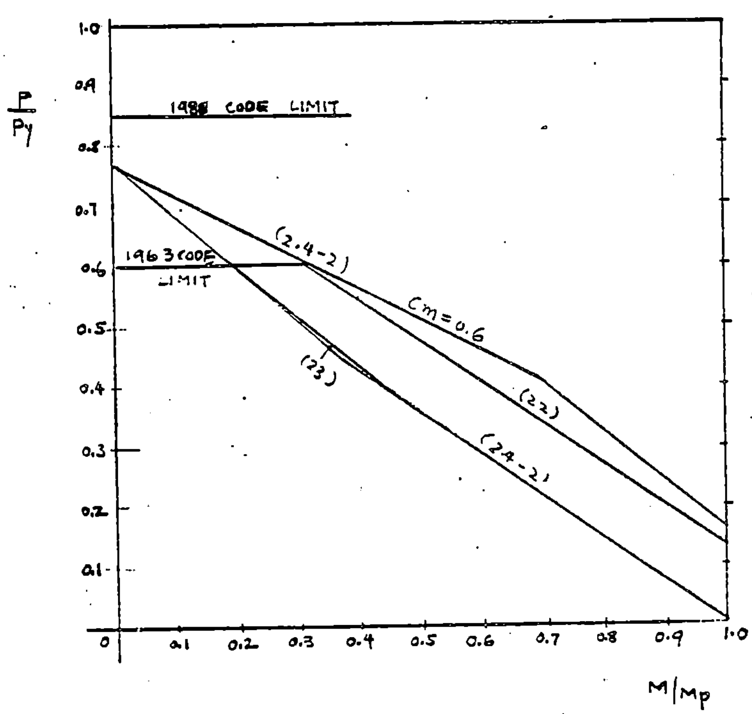
1980 Code

(2.4-2) $\frac{P}{P_{cr}} + \frac{C_m M}{(1 - \frac{P}{P_e})M_p} \leq 1.0$ $0.6 \leq C_m \leq 1.0$

(2.4-3) $\frac{P}{P_y} + \frac{M}{1.18M_p} \leq 1.0$, $M \leq M_p$

TYPICAL EXAMPLES





Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project C5257		Page C- 28
	By RA Date SEPT '81	Ch'k'd [illegible] Date [illegible]	Rev. Date

$F_y = 36$ ksi $\quad \frac{kl}{r} = 70 \quad$ 14 WF 150 $\quad$ DOUBLE CURVATURE

<u>1963 Code</u>

Formula (21) $M = M_p$ when $P/Py \leq 0.15$

$$\frac{M}{M_p} \leq 1.18 - 1.18(P/Py) \leq 1.0$$

Formula (22) $\frac{M}{M_p} \leq B\text{-}G(P/Py) \leq 1.0$

$$M \leq M_p$$

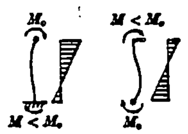
<u>1980 Code</u>

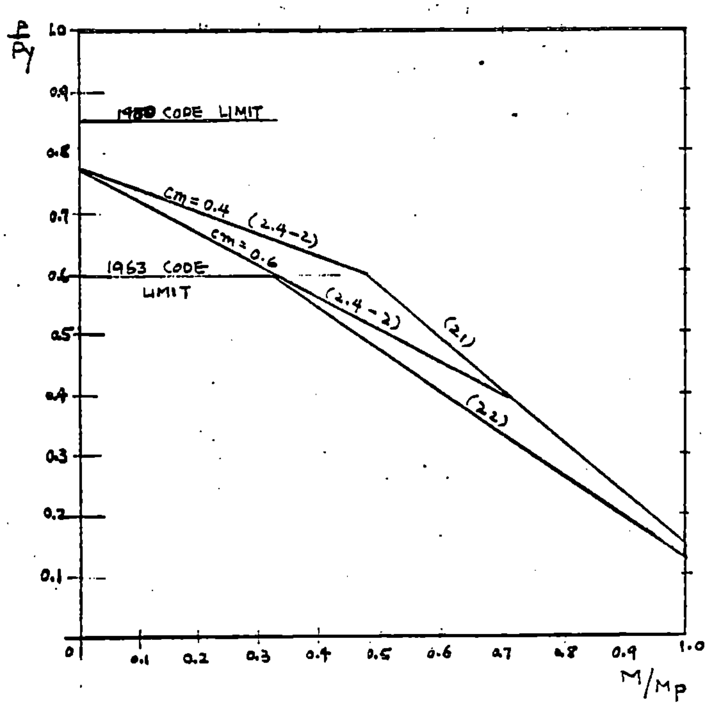
(2.4-2) $\frac{P}{P_{cr}} + \frac{C_m M}{(1 - \frac{P}{P_e})M_p} \leq 1.0$

$$0.4 \leq C_m \leq 0.6$$

(2.4-3) $\frac{P}{P_y} + \frac{M}{1.18M_p} \leq 1.0, \; M \leq M_p$

<u>TYPICAL EXAMPLES</u>





$F_y = 36$ ksi $\quad \frac{kl}{r} = 100$ 14 W 50 $\quad$ SINGLE CURVATURE

1963 Code

Formula (22) $\frac{M}{M_p} \le B\text{-}G(P/Py) \le 1.0$

$M \le M_p$

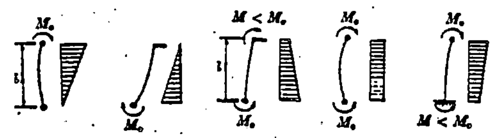
Formula (23) $\frac{M}{M_p} \le 1.0 - H(P/Py) - J(P/Py)^2$

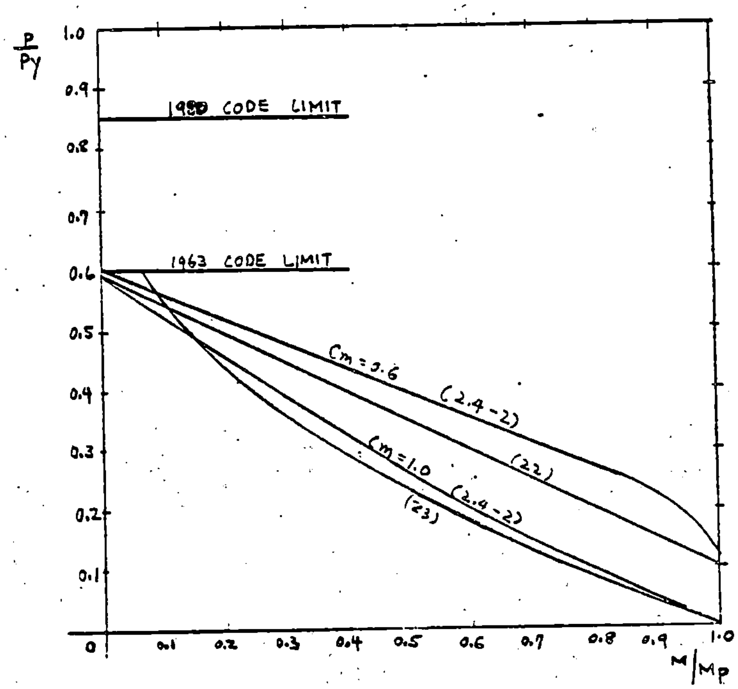
1980 Code

(2.4-2) $\frac{P}{P_{cr}} + \frac{C_m M}{(1 - \frac{P}{P_e})M_p} \le 1.0 \quad 0.6 \le C_m \le 1.0$

(2.4-3) $\frac{P}{P_y} + \frac{M}{1.18M_p} \le 1.0,\ M \le M_p$

TYPICAL EXAMPLES





$F_y = 36$ ksi $\frac{kl}{r} = 100$ 14W 150 DOUBLE CURVATURE

1963 Code

Formula (21) $M = M_p$ when $P/Py \leq 0.15$

$$\frac{M}{M_p} \leq 1.18 - 1.18(P/Py) \leq 1.0$$

Formula (22) $\frac{M}{M_p} \leq B - G(P/Py) \leq 1.0$

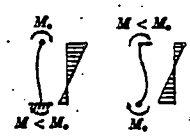
$$M \leq M_p$$

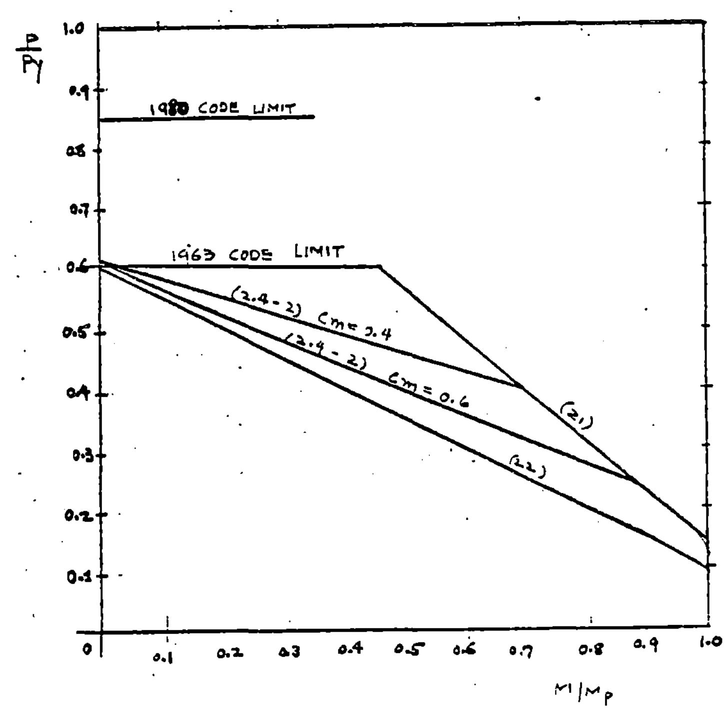
1980 Code

(2.4-2) $\frac{P}{P_{cr}} + \frac{C_m M}{(1 - \frac{P}{P_e})M_p} \leq 1.0$ $0.4 \leq C_m \leq 0.6$

(2.4-3) $\frac{P}{P_y} + \frac{M}{1.18M_p} \leq 1.0,\ M \leq M_p$

TYPICAL EXAMPLES





$F_y = 36$ ksi $\quad \frac{kl}{r} = 30 \quad$ 12 WF 45 $\quad$ SIDESWAY ALLOWED

1963 Code

Formula (21) $M = M_p$ when $P/Py \le 0.15$

$$\frac{M}{M_p} \le 1.18 - 1.18(P/Py) \le 1.0$$

Formula (22) $\frac{M}{M_p} \le B\text{-}G(P/Py) \le 1.0$

$$M \le M_p$$

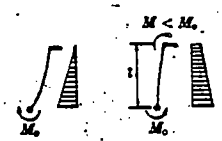
Formula (23) $\frac{M}{M_p} \le 1.0 - H(P/Py) - J(P/Py)^2$

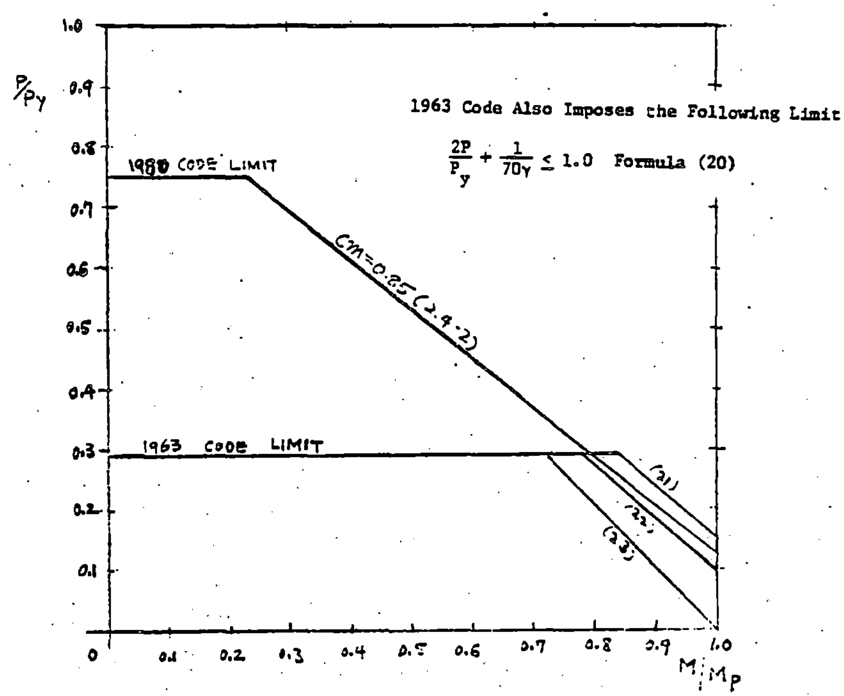
1980 Code

(2.4-2) $\frac{P}{P_{cr}} + \frac{C_m M}{(1 - \frac{P}{P_e})M_p} \le 1.0$ $\quad C_m = 0.85$

(2.4-3) $\frac{P}{P_y} + \frac{M}{1.18M_p} \le 1.0,\ M \le M_p$

TYPICAL EXAMPLES





$F_y = 36$ ksi $\frac{kl}{r} = 30$ 14 WF 150 SIDESWAY ALLOWED

1963 Code

Formula (21) $M = M_p$ when $P/Py \leq 0.15$

$\frac{M}{M_p} \leq 1.18 - 1.18(P/Py) \leq 1.0$

Formula (22) $\frac{M}{M_p} \leq B\text{-}G(P/Py) \leq 1.0$

$M \leq M_p$

Formula (23) $\frac{M}{M_p} \leq 1.0 - H(P/Py) - J(P/Py)^2$

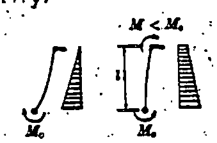
1980 Code

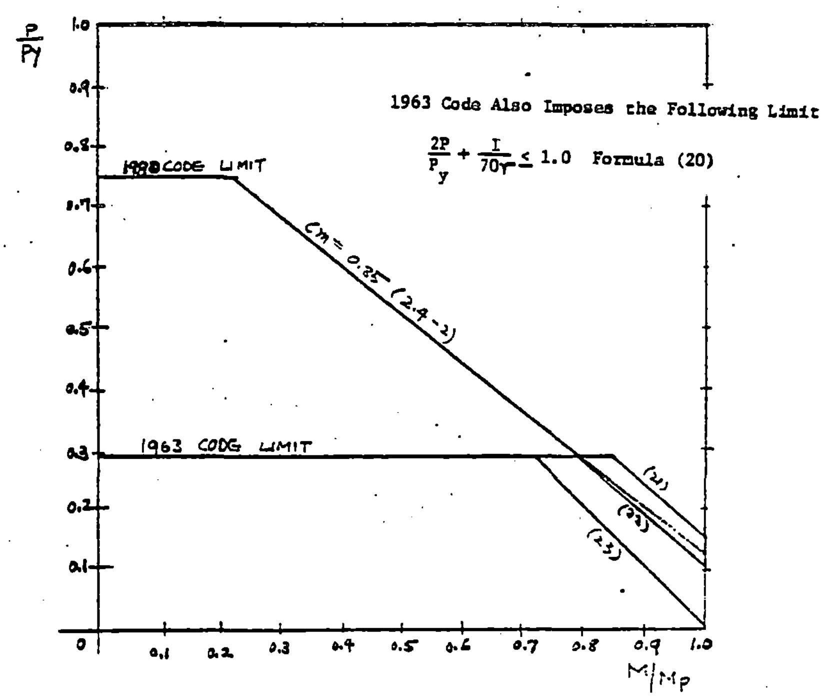
(2.4-2) $\frac{P}{P_{cr}} + \frac{C_m M}{(1 - \frac{P}{P_e})M_p} \leq 1.0$

$C_m = 0.85$

(2.4-3) $\frac{P}{P_y} + \frac{M}{1.18M_p} \leq 1.0,\ M \leq M_p$

TYPICAL EXAMPLES





CASE STUDY -9-

Comparison of AISC-1980 Section 1.10.6 with AISC-1963 Section 1.10.6, Reduction in Flange Stress, HYbrid Girders only.

The only change between the two codes is the introduction of Formula (1.10-6) for case of hybrid girder, in the 1980 code. Formula (1.10-5) of 1980 Code with F_b in Ksi is identical to Formula (12) of 1963 with F_b in Psi. Hybrid girder designed in 1963 would be designed in accordance with Formula (12) which is identical to (1.10-5) in 1980 Code. But a hybrid girder designed in accordance with 1980 has to conform to both Formulas (1.10-5) and (1.10-6). For $F_b = 25$ Ksi and 50 ksi, we draw graphs of reduction Factor $\left(\frac{F_b'}{F_b}\right)$ Vs. Area of web to Area of Flange ratio (A_w/A_f), using Formulas (1.10-5) and (1-10-6) for given $\alpha = 0.3$, 0.6, and 0.9 and for given h/t ratios (162, 172 & 182, for $F_b = 25$ ksi and 117, 127 & 137 for $F_b = 50$ Ksi). We find in all six cases depending on A_w/A_f ratio for $\alpha = 0.45$, Formula (1.10-6) in the 1980 code is quite conservative.

But for $0.45 < \alpha \leq 0.75$, Formula (1.10-6) or Formula (1.10-5) could be conservative as compared to each other depending on h/t ratio for given Fb. But for $\alpha > 0.75$, in any case, Formula (1.10-5) is more conservative. Thus we can make the following judgment on them.

<u>OLD Formulas</u>

a) Formula (12), 1963 Code

$$F_b' \leq F_b \left[1.0 - 0.0005 \frac{A_w}{A_f} \left(\frac{h}{t} - \frac{24000}{\sqrt{F_b}} \right) \right]$$

with Fb in Psi.

b) Formula (1.10-5) 1980 code

$$F_b' \leq F_b \left[1.0 - 0.0005 \frac{A_w}{A_f} \left(\frac{h}{t} - \frac{760}{\sqrt{F_b}} \right) \right],$$

with Fb in ksi

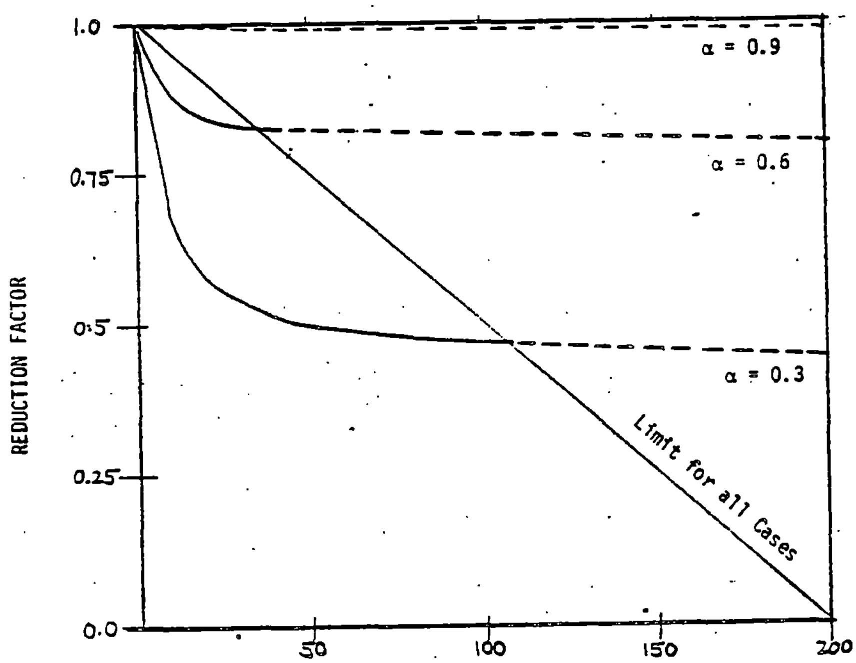
<u>New Formula</u>

Formula (1.10-6) 1980 code

$$F_b' \leq F_b \left[\frac{12 + \left(\frac{A_w}{A_f}\right)(3\alpha - \alpha^3)}{12 + 2\left(\frac{A_w}{A_f}\right)} \right]$$

α	Scale
≤ 0.45 and low A_w/A_f ratio	A
0.45 to 0.75	B
≥ 0.75	C

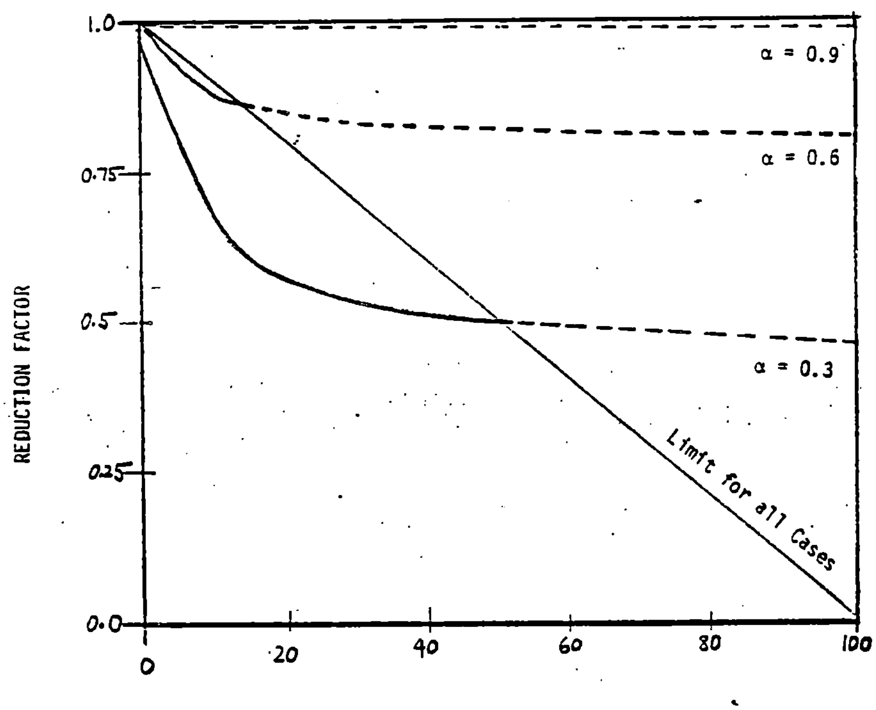
AISC 1.10.6 1963/1980 CODE COMPARISON



WEB/FLANGE AREA RATIO

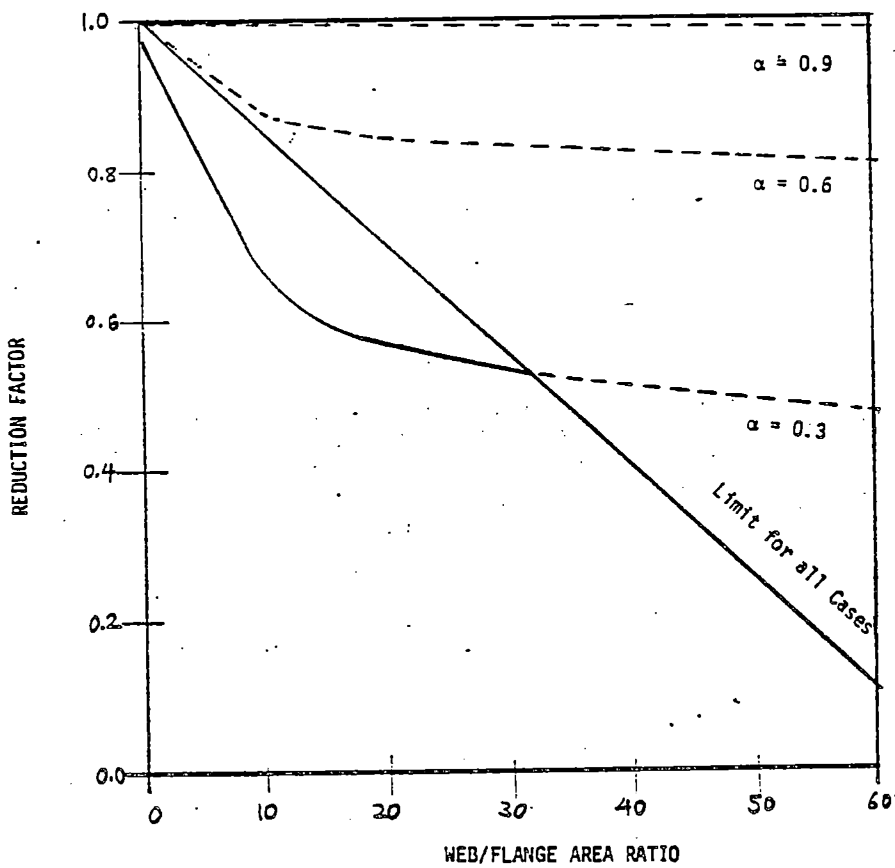
BENDING STRESS = 25KSI ALPHA=0.3, 0.6, 0.9, H/T RATIO = 162

AISC 1.10.6 1963/1980 CODE COMPARISON



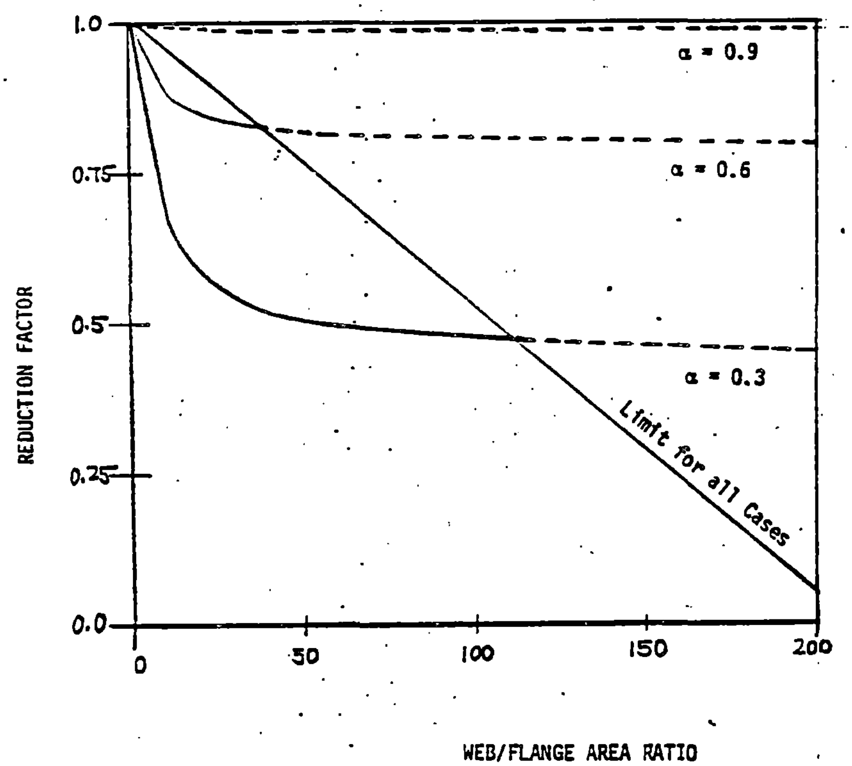
BENDING STRESS = 25KSI   ALPHA=0.3, 0.6, 0.9, H/T RATIO = 172

AISC 1.10.6 1963/1980 CODE COMPARISON



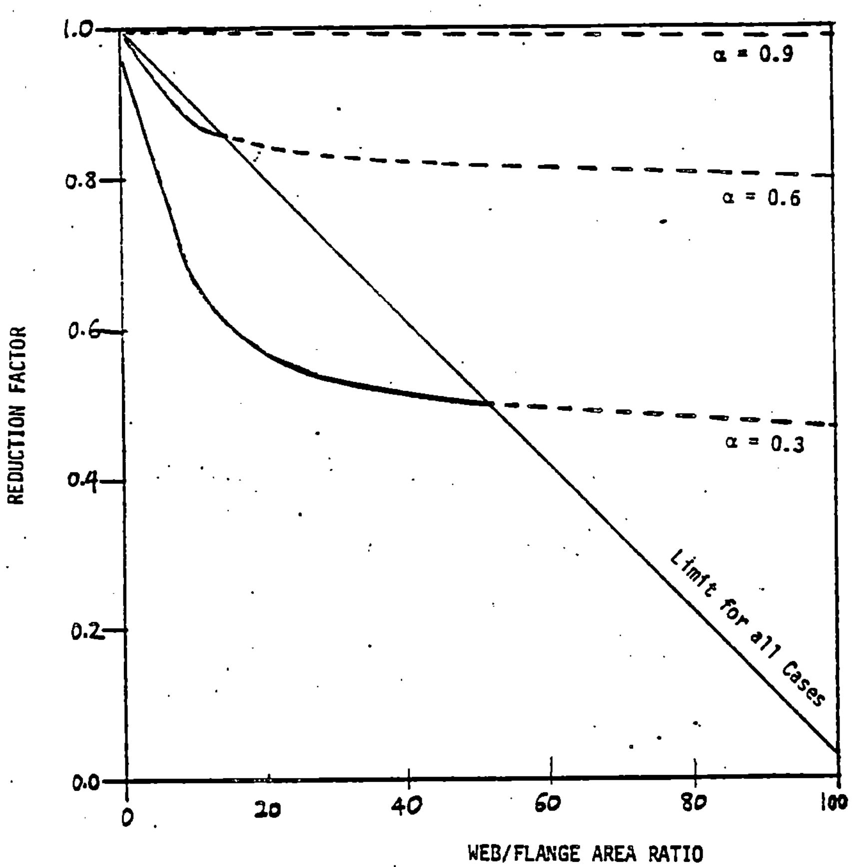
BENDING STRESS = 25KSI ALPHA=0.3, 0.6, 0.9, H/T RATIO = 182

AISC 1.10.6 1963/1980 CODE COMPARISON



BENDING STRESS = 50KSI ALPHA=0.3, 0.6, 0.9, H/T RATIO = 117

AISC 1.10.6 1963/1980 CODE COMPARISON



BENDING STRESS = 50KSI   ALPHA=0.3, 0.6, 0.9, H/T RATIO = 127

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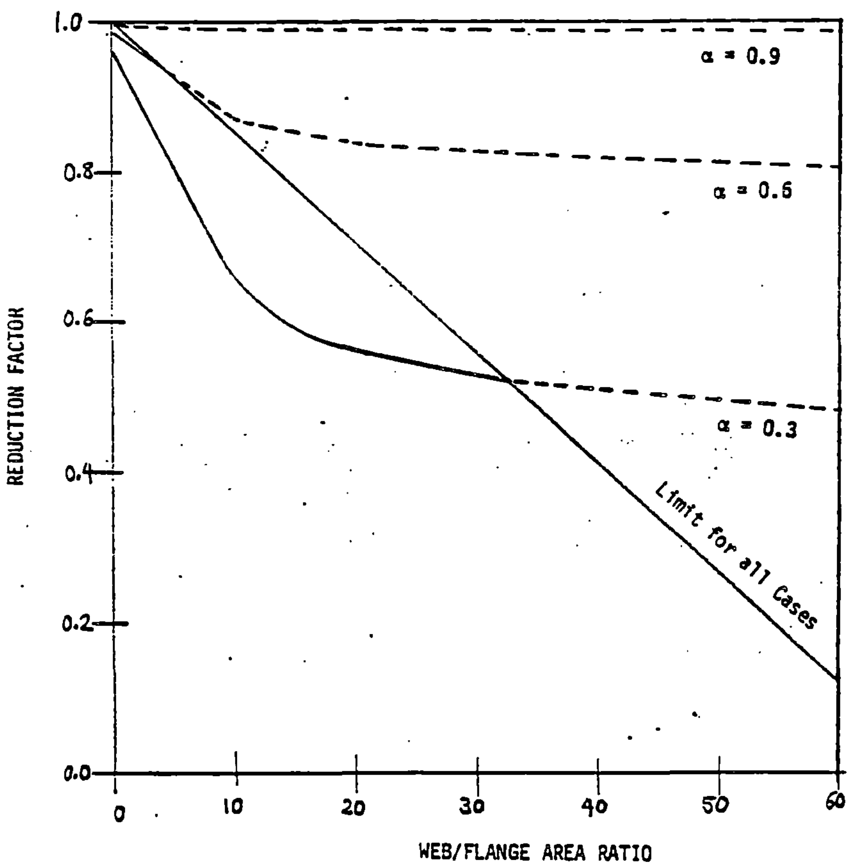
Project				Page
C5257				C- 40
By RA	Date OCT '81	Ch'k'd EMW	Date 11/71	Rev. Date

AISC 1.10.6 1963/1980 CODE COMPARISON



BENDING STRESS = 50KSI ALPHA=0.3, 0.6, 0.9, H/T RATIO = 137

CASE STUDY -10-

Comparison of Section (1.9.1.2) and Appendix C (AISC 1980) with Section 1.9.1 (AISC, 1963); width-thickness ratio of unstiffened elements Subject to axial Compression and compression due to bending.

In both sections the limit of width-thickness ratio is given for the following various cases.

CASE I: single-angle struts; double-angle struts with separators

CASE II: Struts comprising double angles in contact; angles or plates projecting from girders, Columns, or other compression members; compression flanges of beams; Stiffeners on plate girders

CASE III: Stems of tees

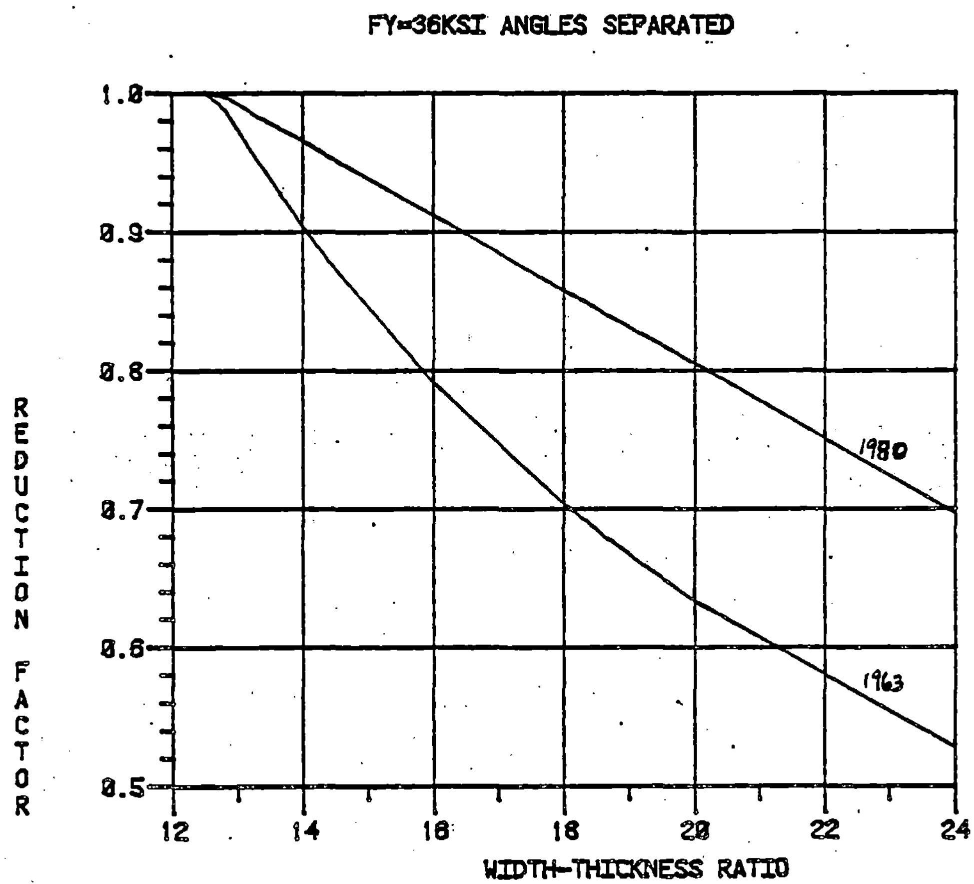
In AISC, 1980, according to the specifications for the above cases, when compression members exceed the allowable width-thickness ratio, the allowable stresses are reduced by a factor based on formulas given in appendix C which depends on yield stress (F_y) and the width-thickness ratio.

But according to AISC, 1963 Specifications, when compression members exceed the allowable width - thickness ratio, the member is acceptable if it satisfies the allowable stress requirements with a portion of width ie. effective width meets stress requirements.

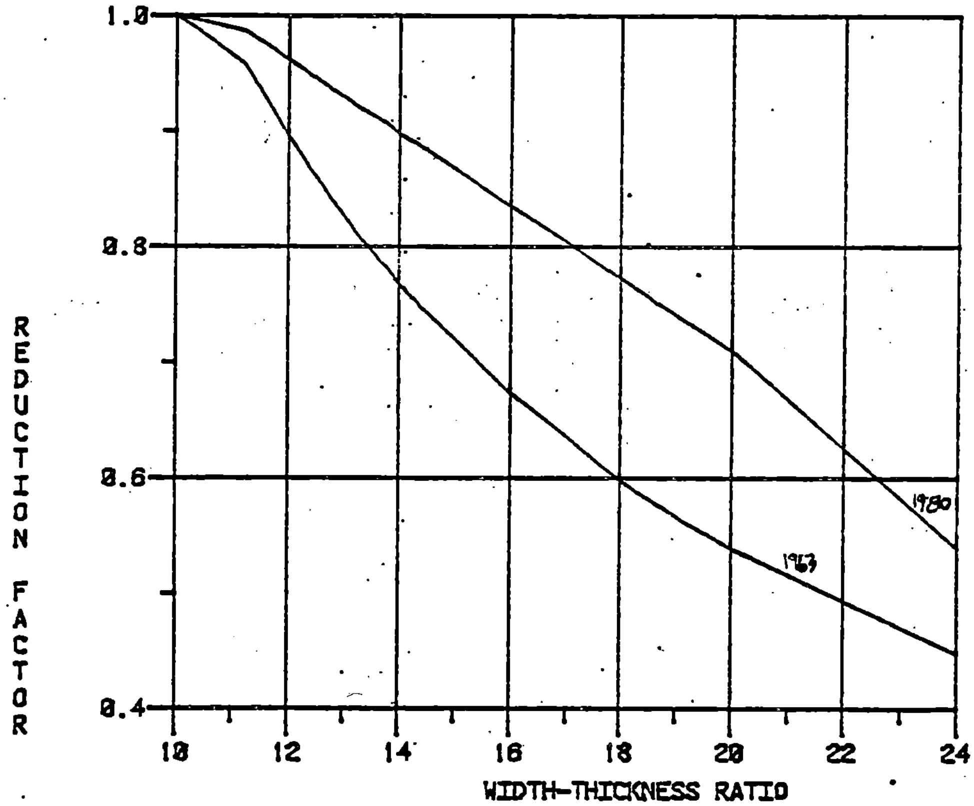
For the case study, two values of Fy 36 ksi and 50 ksi are chosen. For the two values for typical angle section and T sections given in AISC Manual graphs have been plotted for Reduction Factor VS Width - thickness ratio.
Reduction Factor for AISC, 1980 Code is based on formulas given in appendix C and for AISC, 1963, reduction factor is the ratio of effective width to actual width of the section.

Based on the graphs, the change for case I and Case II at higher width/thickness ratio would be a C change, as Specifications were more conservative in 1963 code. But for Case III the change in Specification is A change as it is more conservative in 1980 Code, at higher width - thickness ratio.

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project C5257				Page C- 43	
	By RA	Date SEPT '81	Ch'k'd E.M.D.	Date 10/81	Rev.	Date

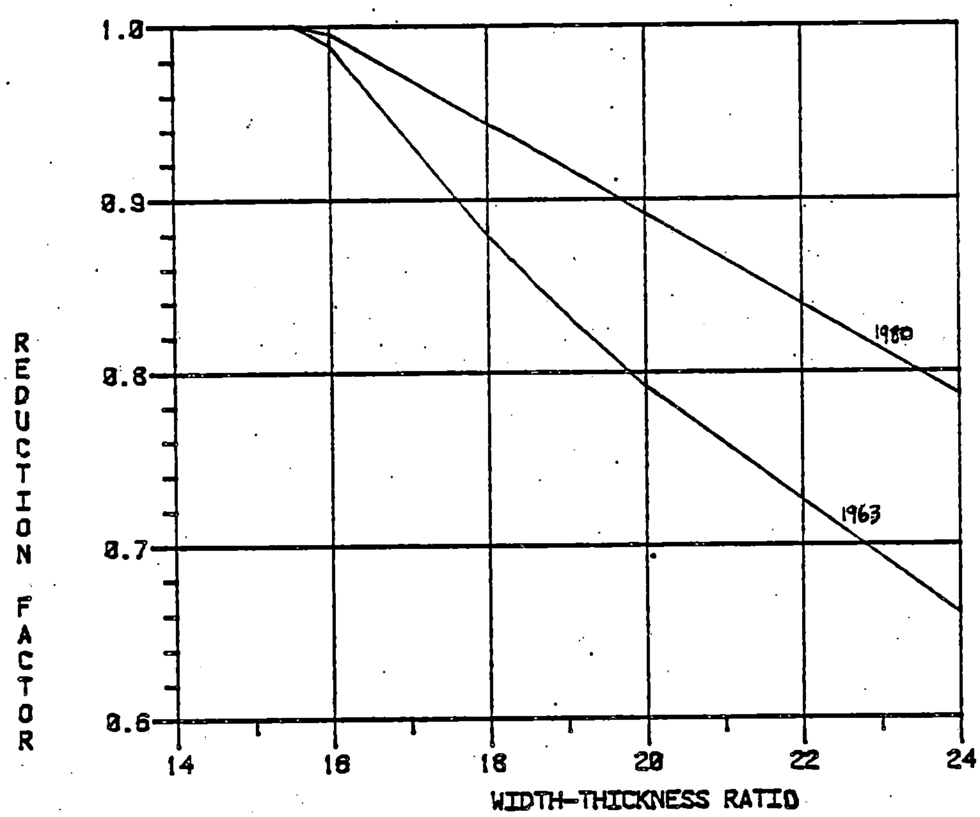


FY=50KSI ANGLES SEPARATED



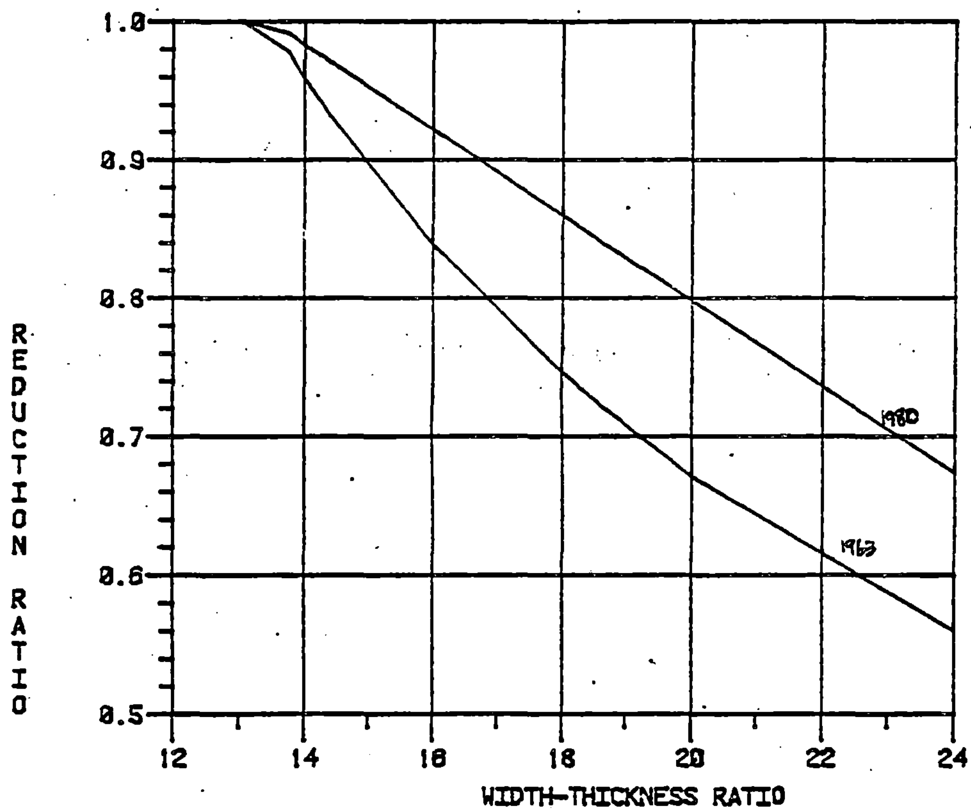
Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project C5257				Page C- 45	
	By RA	Date SEPT '81	Ch'k'd EMW	Date 10/81	Rev.	Date

FY=36KSI ANGLES IN CONTACT

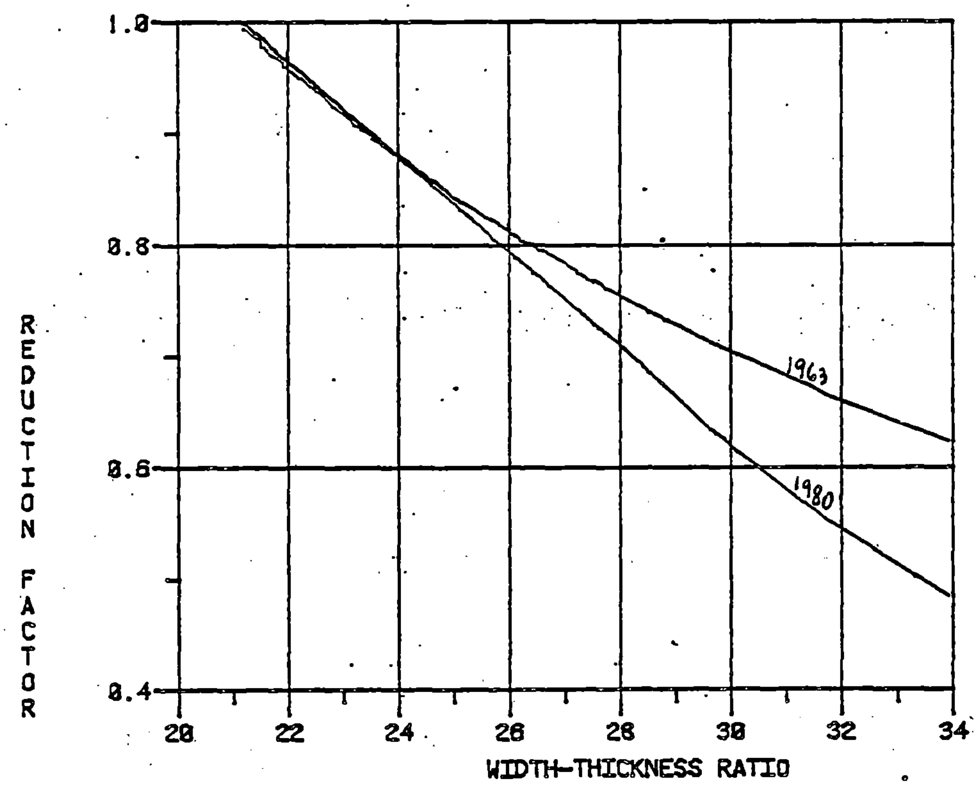


Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project C5257				Page C- 46	
	By RA	Date SEPT'81	Ch'k'd	Date 11/81	Rev.	Date

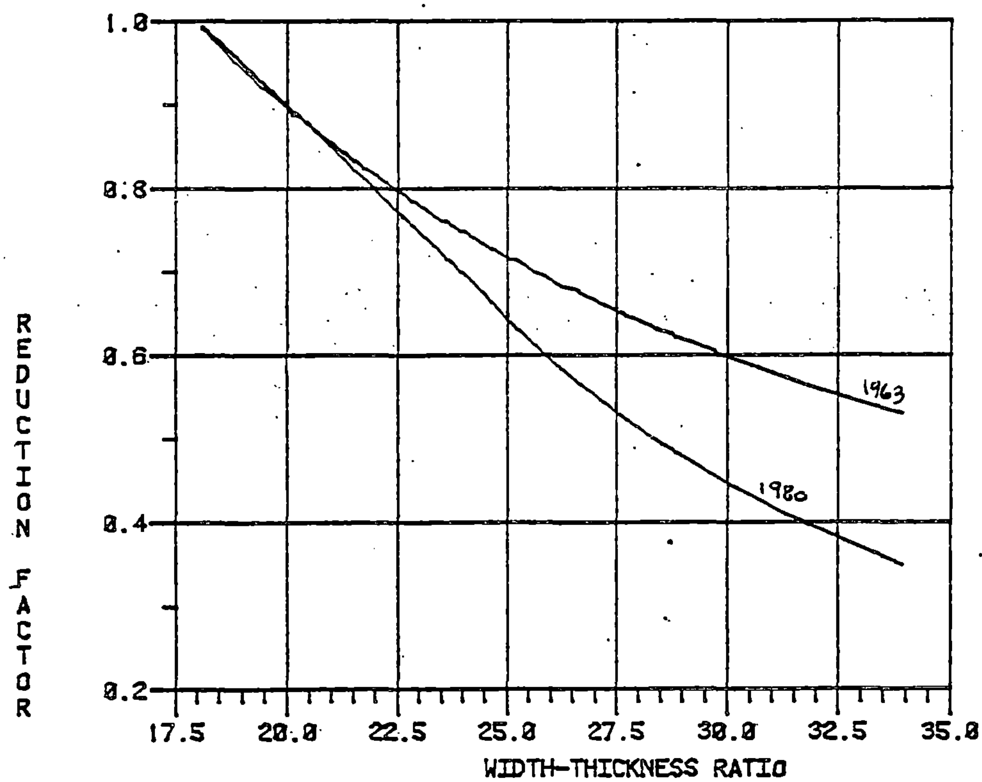
FY=50KSI ANGLES IN CONTACT



FY=36KSI T SHAPES



FY=50KSI T SHAPES



CASE STUDY -11-

Comparison of AISC 1980 Section 1.11.4 with AISC 1963 Section 1.11.4; Shear connectors for composite beams, where longitudinal reinforcing steel acts with beam.

According to AISC 1980, Formula (1.11-5)

$$V_h = A_{sr} F_{yr}/2 \quad (1.11\text{-}5)$$

is given for continuous composite beam where longitudinal reinforcing steel is considered to act compositely with the steel beam in the negative moment regions, to calculate the total horizontal shear to be resisted by shear connectors between an interior support and each adjacent point of contraflexure.

Whereas in AISC 1963 specifications, the total horizontal shear to be resisted between the point of maximum positive moment and each end or a point of contraflexure in continuous beams is given as the smaller value of Formula (18) and (19)

$$V_h = 0.85 \frac{f'_c A_c}{2} \quad (18)$$

and

$$V_h = \frac{A_s F_y}{2} \quad (19)$$

There is no separate formula for negative moment region in AISC, 1963. The above formulas are the same in AISC, 1980; Formula (1.11-3) and (1.11-4) for the positive moment region. Moreover in AISC, 1963, there is no consideration of reinforcing steel in concrete acting compositely with the steel beam in negative moment regions.

This implies that in computing the section modulus at the points of negative bending, reinforcement parallel to the steel beam, and lying within the effective width of slab may be included according to AISC, 1980. But it is not allowed to include reinforcing steel in computing the section modulus for the above case as per the specifications of AISC 1963. Thus design criteria is being liberalized in AISC 1980. Since the quantification of this liberal criteria is unknown, this change can best be classified as C. Any composite beam designed as per AISC 1963 specifications will show more moment capacity when calculated according to AISC, 1980 Specifications.

CASE STUDY -12-

The allowable peripheral shear stress (Punching Shear Stress) as stated in the B & PV ASME Code Section III Div. 2, 1980 (ACI 359-80) Para. CC-3421.6 is limited to v_c where v_c shall be calculated as the weighted average of v_{ch} and v_{cm}

$$v_{ch} = 4\sqrt{f'_c}\sqrt{1 + (f_m/4\sqrt{f'_c})}$$

$$v_{cm} = 4\sqrt{f'_c}\sqrt{1 + (f_h/4\sqrt{f'_c})}$$

The ACI 318-63 Code Section 1707 states that the ultimate shear strength v_u shall not exceed $v_c = 4\sqrt{f'_c}$.

Comparing the above two cases the following is concluded;

When: **Scale**

1. Membrane stresses are compressive
   318-63 is more conservative (C)
2. Membrane stresses are tensile
   318-63 is less conservative (A)

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project C5257			Page C- 52	
	By E.M.L.	Date 10/81	Ch'k'd RK/MD	Date 10/81	Rev. / Date

Scale

3. Membrane stresses are zero
   318 - 63 is identical — No rating

4. Membrane stresses are opposite in sign
   318 - 63 could be less conservative — (A)

CASE STUDY -13-

The B & PV ASME Code Section III Division 2, 1980 (ACI 359-80) Para. CC-3421.7 states that the shear stress taken by the concrete resulting from pure torsion shall not exceed v_{ct} where

$$v_{ct} = 6\sqrt{f'_c}\sqrt{1 + \frac{f_h + f_m}{6\sqrt{f'_c}} + \frac{f_m f_h}{(6\sqrt{f'_c})^2}}$$

While the ACI 318-63 Code Section 1707 limits the ultimate Shear Strength v_u to

$$v_c = 4\sqrt{f'_c}$$

From the above two cases the following is concluded;

When : **Scale**

1. Membrane stresses are compressive
   318-63 is more conservative (C)

2. Membrane stresses are tensile
   318-63 is less conservative (A)

Scale

3. Membrane stresses are zero
   318-63 is more conservative (C)

4. Membrane stresses are opposite in sign
   318-63 could be less conservative (A)

APPENDIX D

ACI CODE PHILOSOPHIES

ACI CODE PHILOSOPHIES

The American Concrete Institute (ACI) Building Code Requirements for Reinforced Concrete delineate two philosophies of design which have long been in use: the so-called working stress method, which was in general acceptance and predominant use from early in this century to the early 1960's, and the ultimate strength method, which has been rapidly replacing working stress since about 1963.

Working Stress Method

The working stress method of design is referred to as the "alternate design method" by the most recent ACI code. By this method, the designer proportions structural elements so that internal stresses, which result from the action of service loads* and are computed by the principles of elastic mechanics, do not exceed allowable stress values prescribed by the code.

The allowable stresses as prescribed by the ACI code are set such that the stresses under service load conditions will be within the elastic range of behavior for the materials involved. As a result of this, the assumption of straight line stress-strain behavior applies reasonably for properly designed structural members. The member forces used in design by this method are those which result from an elastic analysis of the structure under the action of the service loads.

Ultimate Strength Design

The ultimate strength method is referred to as the "strength method" in the most recent ACI code. By this method, the proportioning of the members is based on the total theoretical strength of the member, satisfying equilibrium and compatibility of stress and strain, at failure. This theoretical strength is modified by capacity reduction factors which attempt to assess the variations to be encountered in material, construction tolerances, and calculation approximation.

*Service loads are defined as those loads which are assumed to occur during the service life of the structure.

Strength Reduction Factor

In the present code, the capacity reduction factor (ϕ) varies for the type of member and is considered to account for the relative seriousness of the member failure as regards the overall integrity of the structure.

Load Factors

Also, by this method, the designer increases the service loads by applying appropriate load factors to obtain the ultimate design loads in an attempt to assess the possibility that the service loads may be exceeded in the life of the structure. The member forces used to proportion members by this method are based on an elastic analysis of the structure under the action of the ultimate design loads.

Importance of Ductility

A critical factor involved in the logic of ultimate strength design is the need to control the mode of failure. The present ACI code, where possible, has incorporated a philosophy of achieving ductility in reinforced concrete designs. Ductility in a structural member is the ability to maintain load carrying capacity while significant, large deformations occur. Ductility in members is a desired quality in structures. It permits significant redistribution of internal loads allowing the structure to readjust its load resistance pattern as critical sections or members approach their limiting capacity. This deformation results in cracking and deflections which provide a means of warning in advance of catastrophic collapse. Under conditions of loading where energy must be absorbed by the structure, member ductility becomes very important.

This concern for preserving ductility appears in the present code in many ways and has guided the changes in code requirements over the recent decades. Where research results have confirmed analysis and intuition, the code has provided for limiting steel percentages, reinforcing details, and controls--all directed as guaranteeing ductility. In those aspects of design where ductility cannot be achieved or insured, the code has required added strength to insure potential failure at the more ductile sections of structures. Examples of this are evident in the more conservative capacity reduction factors for columns and in the special provisions required for seismic design.

Strength and Serviceability in Design

There are many reasons for the recent trend in reinforced concrete codes toward ultimate strength rather than working stress concepts. Research in reinforced concrete has indicated that the strain distributions predicted by working stress computations in general do not exist in the members under load. There are many reasons for this lack of agreement. Concrete is a brittle, non-linear material in its stress-strain behavior, exhibiting a down trend beyond its ultimate stress and characterized by a tensile stress-strain curve which in all its features is approximately on the order of one tenth smaller than its compressive stress-strain curve.

Time-dependent shrinkage and creep strains are often of significant magnitude at service load levels and are difficult to assess by working stress methods. While ultimate strength methods do not eliminate these factors, they become less significant at ultimate load levels. In addition, ultimate strength methods allow for more reasonable approximations to the non-linear concrete stress-strain behavior.

In the analyses of structures, the designer must, by necessity, make certain assumptions which serve to idealize the structures. The primary assumptions are that the structure behaves in a linearly elastic manner, and that the idealized member stiffness is constant throughout each member and constant in time.

Working stress logic does not lend itself well to accounting for variations in stiffness caused by cracking and variations in material properties with time. Although the ultimate strength method in the present code requires an elastic structural analysis to determine member forces for design, it recognizes these limitations and, in concept, anticipates the redistribution resulting from ductile deformation at the most critically stressed sections and in fact proportions members so that redistribution will occur.

In addition to strength, a design must satisfy serviceability requirements. In some designs, serviceability factors (such as excessive deflection, cracking, or vibration at service load) may prove to be more important than strength. Computations of the various serviceability factors are generally at service load levels; therefore, the present code uses elastic concepts in its controls of serviceability.

Factors of Safety

Factors of safety* are subjects of serious concern in this review. For working stress, the definition of the factor of safety is often considered to be the ratio of yield stress to service load stress. This definition becomes suspect or even incorrect where nonlinear response is involved. For ultimate strength, one definition of factors of safety is the ratio of the load that would cause collapse to the service or working load. As presented in the present code, a factor of safety is included for a variety of reasons, each of which is important but has no direct interrelation with the other.

The present ACI code has divided the provisions for safety into two factors; the overload factors and the capacity reduction factors (considered separately by the code) are both provisions to insure adequate safety but for distinctly different reasons. The code provisions imply that the total theoretical strength to be designed for is the ratio of the overload factor (U) over the capacity reduction factor (ϕ). The present ACI code has assigned values to the above factors such that the ratio U/ϕ ranges from about 1.5 to 2.4 for reinforced concrete structural elements.

*Factors of safety (FS) are related to margins of safety (MS) through the relation MS = FS - 1.

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Structural Review of the Palisades Nuclear Power Plant Unit 1 Containment Structure Under Combined Loads for the Systematic Evaluation Program

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FOREWORD

The U.S. Nuclear Regulatory Commission (NRC) is conducting the Systematic Evaluation Program (SEP). The Program is a plant-by-plant reassessment of the safety of eleven operating nuclear reactors that received construction permits between 1956 and 1967. Many safety criteria have changed since these plants were licensed. The purpose of the SEP is to develop a current, documented basis for the safety of older facilities.

For the Palisades Unit 1, seismic analyses for a Safe Shutdown Earthquake (SSE) had been performed in a previous study for selected plant structures and components from generic groups of equipment. The results were reported in an earlier SEP report, NUREG/CR-1833. The SSE was considered to be the Extreme Environmental condition. In the study reported here, the containment structure was selected for further evaluation of the Abnormal/Extreme Environment.

This report is a collective effort by the following people:

Nelson, T. A., and Lo, T. [Lawrence Livermore National Laboratory (LLNL)], who provided project management support and reviewed the report.

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Tsai, N. C. (NCT Engineering, Inc.), who conducted the structural reevaluation of the steel liner plate system.

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ABSTRACT

A structural reassessment of the containment structure of the Palisades Nuclear Power Plant Unit 1 was performed for the Nuclear Regulatory Commission as part of the Systematic Evaluation Program. Conclusions about the ability of the containment structure to withstand the Abnormal/Extreme Environment are presented.

The reassessment focused mainly on the overall structural integrity of the containment building for the Abnormal/Extreme Environment. In this case, the Abnormal Environmental condition is caused by the worst case of either a Loss-of-Coolant Accident or a main steam line break. The Extreme Environmental condition is the Safe Shutdown Earthquake.

CONTENTS

LIST OF ILLUSTRATIONS

LIST OF TABLES

CHAPTER 1: INTRODUCTION

1.1 SCOPE OF WORK

Structural reassessment of nuclear power plants is one facet of the Systematic Evaluation Program (SEP), conducted by the Nuclear Regulatory Commission (NRC). This report is a structural review of the containment building of the Palisades Nuclear Power Plant Unit 1. We evaluated the overall structural integrity of the containment building for the Abnormal/Extreme Environmental condition as defined in the ASME Boiler and Pressure Vessel Code, Section III (ASME code). In this instance, the Abnormal load case is that induced by a Loss of Coolant Accident (LOCA), and the Extreme Environmental load case is induced by the Safe Shutdown Earthquake (SSE). It is important to point out that, in this report, LOCA includes both the primary and secondary loop break cases.

Two previous SEP reports served as the basis for this work: <u>SEP Containment Analysis and Evaluation for the Palisades Power Plant</u>,[1] which defined the LOCA Loading, and <u>Seismic Review of the Palisades Nuclear Power Plant Unit 1 as Part of the SEP</u>,[2] in which the plant was analyzed for SSE Load.

We based our analysis on the LOCA discussed in Ref. 1: pipe breaks in the primary and secondary systems. The seismic event we used is described in <u>Site-Specific Ground Response Spectra for SEP Plants Located in the Eastern United States</u>.[5,11] Our reassessment combined the accident and seismic event with existing load conditions on the containment building. We then evaluated the containment building's and its steel liner's ability to withstand the Abnormal/Extreme environmental condition. We also evaluated the steel liner system for the Extreme environmental condition, which can be a more critical loading combination. Because the primary purpose of this analysis is to evaluate the overall structural integrity of the containment building, no local load effects are considered.

1.2 STRUCTURE DESCRIPTION

The reactor containment building of Palisades Plant Unit 1 houses the nuclear steam supply system. This building is a vertical, cylindrical, prestressed concrete structure (Fig. 1.1). The inside diameter is 116 ft; the inside height is 189 ft. The containment walls are 3.5 ft thick, the dome is 3 ft thick, and the base slab varies in thickness between 8 ft. and 13 ft. The dome has a radius of 89 ft. 2-1/4 in. The containment building was the first in the United States to be post-tensioned, in both directions, with fully prestressed walls and dome. Each of the 845 tendons is stressed to about 800,000 lb., and each contains ninety 1/4-in.-diameter, high-tensile steel wires.

The post-tensioning system consists of:

1) Three groups of 55 dome tendons oriented at 120^{0} to each other for a total of 165 tendons anchored at the vertical face of the dome ring girder.
2) 180 vertical tendons anchored at the top surface of the ring girder and at the bottom of the base slab.
3) Six groups of 87 hoop tendons enclosing 120^{0} of arc for a total of 522 tendons anchored at the six vertical buttresses.

The design strengths of the concrete are 5,000 psi at 28 days for the shell and 4,000 psi at 90 days for the base slab. The prestressed concrete dome has reinforcing steel bars on both outside and inside surfaces. The reinforcing bars on the outside surface are #9 (12 in. square mesh). The inside reinforcing bars are #6 (18 in. square mesh).

The prestressed concrete cylindrical wall is reinforced on the outside surface in both vertical and hoop directions. The bottom 13 ft. of the inside surface of the wall is also reinforced in both vertical and hoop directions.

Access to the structure for personnel and equipment is through a double-locked door and a 12 ft. 0 in. clear-diameter, double-gasketed single door. An emergency personnel escape is also provided by a double-locked door.

The massive reinforced concrete foundation of the containment building sits on compact glacial deposits and very dense, fine sand. The bedrock is at an elevation of about 440 ft. The grade elevation of the soil surface is 590 ft.

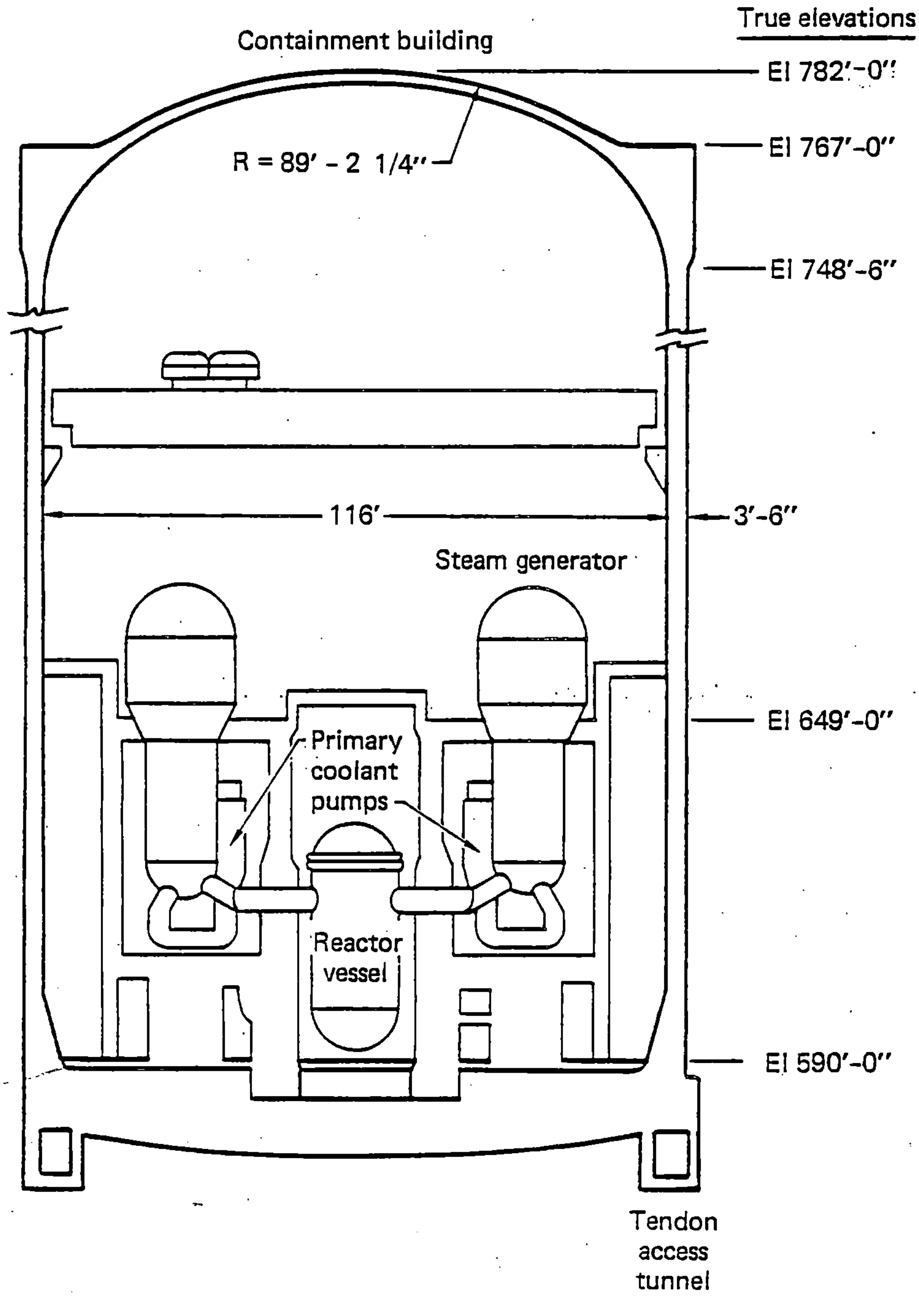


Fig. 1.1. Containment building.

The interior surface of the concrete shell is lined with a 1/4 in. thick ASTM A-442 carbon steel plate. The liner plate functions as a gas barrier to prevent uncontrolled release of fission products from the reactor building during operation and also during a large Loss of Coolant Accident (LOCA). The liner is not relied upon to help the concrete maintain its structural integrity. Figure 1.2 shows the arrangement of the liner system. Figure 1.3 shows some typical details of the system.

At the cylinder portion of the liner, ASTM A-36 stiffener angles are welded longitudinally to the liner at 15 in. intervals. The stiffener angles are, typically, L3x2x1/4. An intermittent fillet weld is used between the liner and the anchors. The typical weld dimensions are 3/16 in. x 4 in. at 12 in. spacing. Horizontal ASTM A-36 channels, angles, and flat bars are attached to the liner plate as well as to the longitudinal angles. The rolled structural shapes and flat bars stiffen the liner plate during the erection and placement of the concrete, and they anchor the liner to the hardened concrete.

Construction of the liner system at the dome is similar to that at the cylinder. The exception are the angles, which are used both as stiffeners and as anchors. The angles are oriented in the hoop direction.

On the base slab, the liner plate is welded to embedded beams. An 18 in.-thick concrete slab is placed over the liner, and a leak-chase system is placed over the liner plate weld seams, which are composed of 1/4 in.-butt welds.

At the junctions where the cylinder intersects the dome and the base slab, horizontal channels or angles are attached as anchors near the locations of maximum change in meridional curvature. The details of typical liner construction at penetrations and polar-crane brackets are not described here because they were not evaluated.

1.3 LOADS AND LOAD COMBINATIONS

Two LOCA conditions were considered in this analysis: 1) the primary system pipe break, and 2) the secondary pipe break. To evaluate the containment building for the Abnormal/Extreme Environmental conditions, the following load conditions were analyzed:

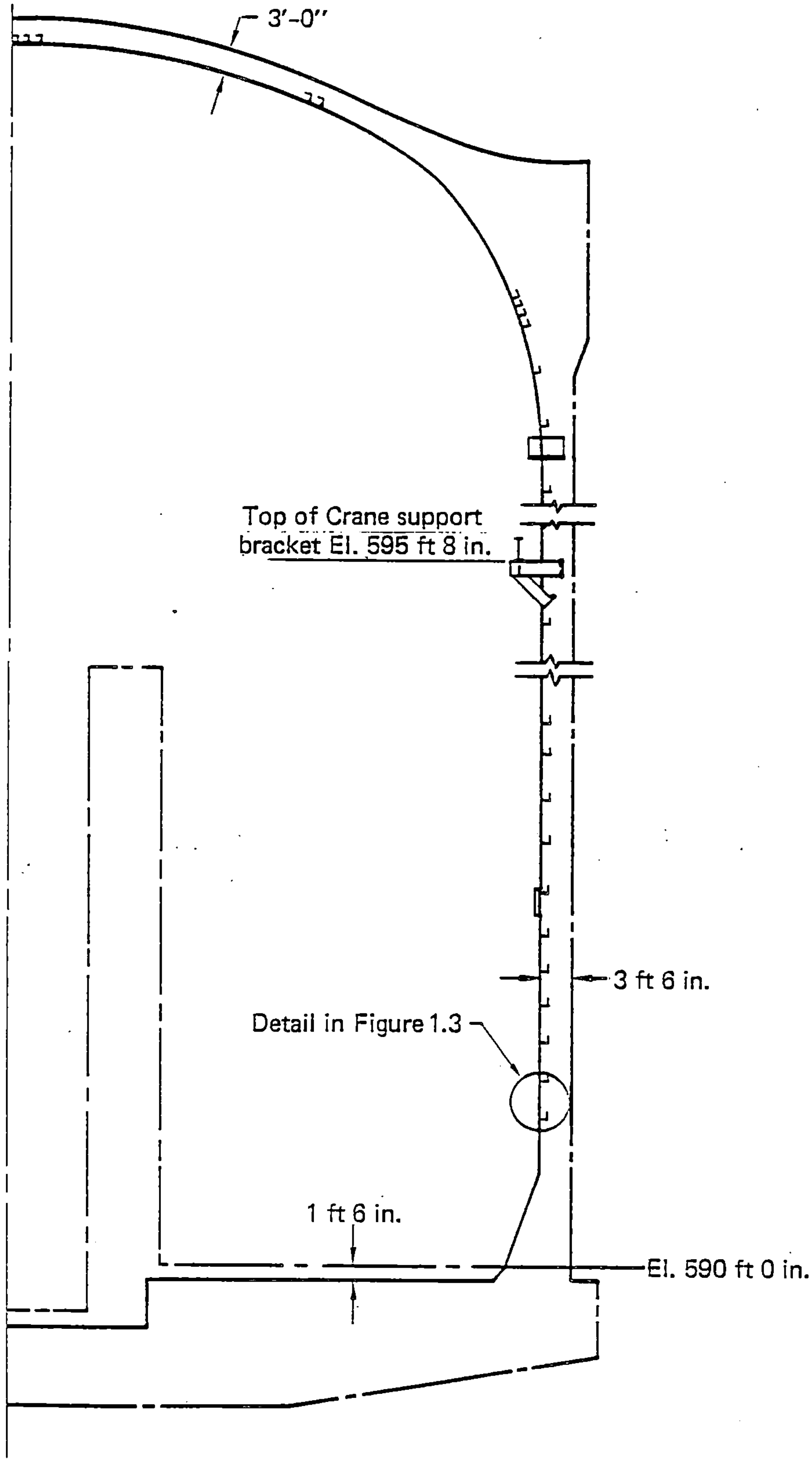


Fig. 1.2. General arrangement of liner system.

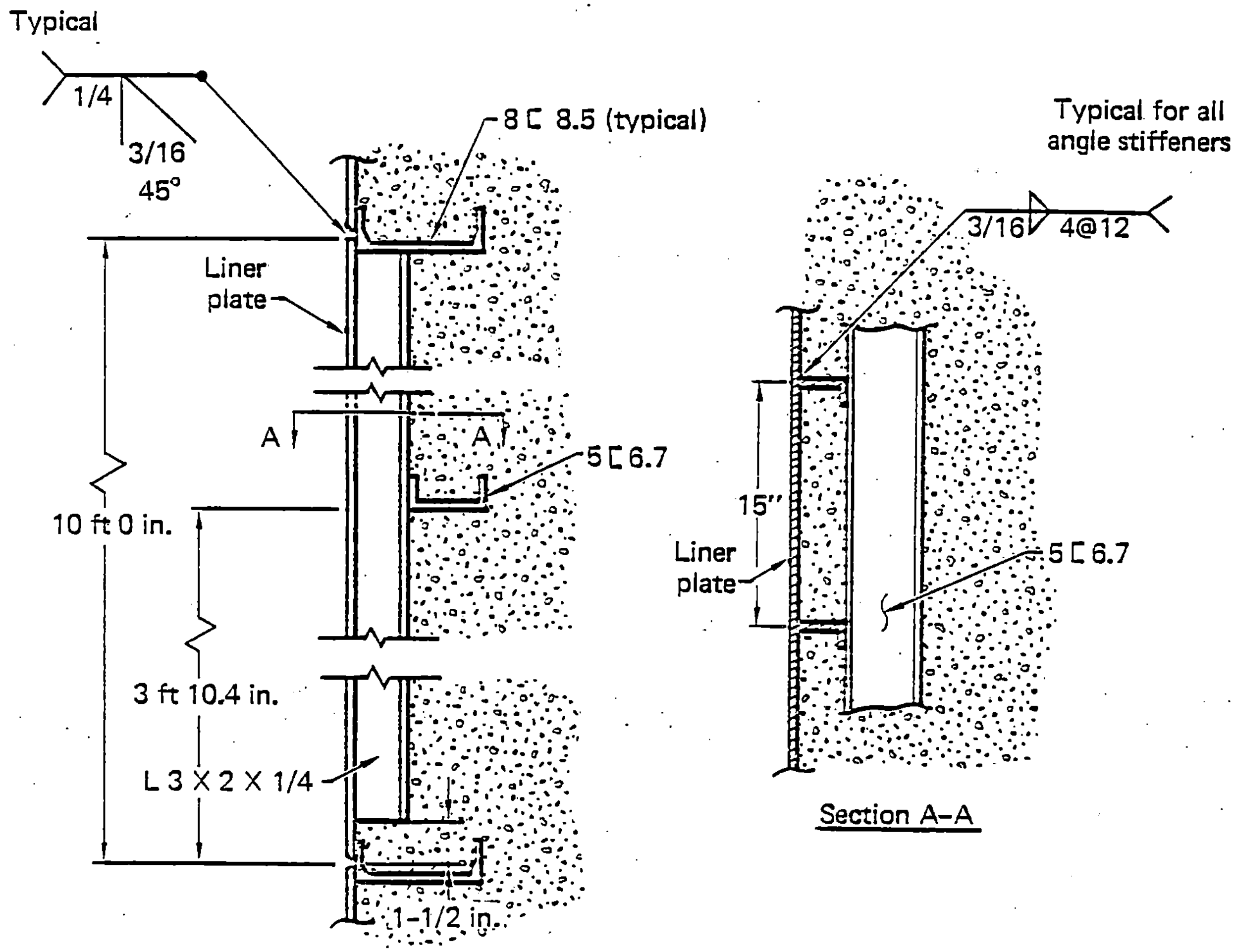


Fig. 1.3. Typical liner anchor details in the cylindrical portion of the containment.

a. Deadweight loads (D)

Deadweight loads were generated by multiplying concrete weight density ($150\ lb/ft^3$) by the structural volume. The dimensions used to determine the structural volume were based on the structural drawings supplied by Consumer Power Company (CPCO).

b. Prestress loads (F)

The response to prestress loads was extracted from information contained in Ref. 3, The Palisades Plant Preliminary Description and Safety Analysis Report (PDSAR), Amendment 1, Figure 2.12.2.3. Figure 3.3 of Sec. 3 illustrates the values used. No new analysis was performed for this load case. For the liner system, we assumed shrinkage of the concrete (prior to prestressing) of 100 μ, where μ represents strain in micro-inch/inch.

c. Pressure loads (P)

According to Ref. 1, the peak post-accident containment pressure (Pa) for both the primary and secondary system pipe breaks is 68 psia (Figs. 1.4 and 1.5). This pressure is very close to the original design pressure of 55 psig (or 69.7 psia) given in the FSAR.[4] Therefore, a relative pressure of 55 psi was applied to the structure for the pressure load case. For the liner system evaluation under the Extreme condition, we assumed a vacuum pressure (Pv) of -3 psig (11.7 psia) inside the containment, as given in the FSAR.[4]

d. Thermal Loads (T)

Figure 1.6 (Fig. 3.12T of Reference 1) gives a peak containment atmosphere temperature of 292^{0} F for the primary system pipe break case. A temperature of about 410^{0} F is given for the single-steam-generator blowdown of the secondary system pipe break case (Fig. 1.7). Accurate information about the temperature gradient in the concrete wall and dome is not available. It is estimated that the steel liner inside surface

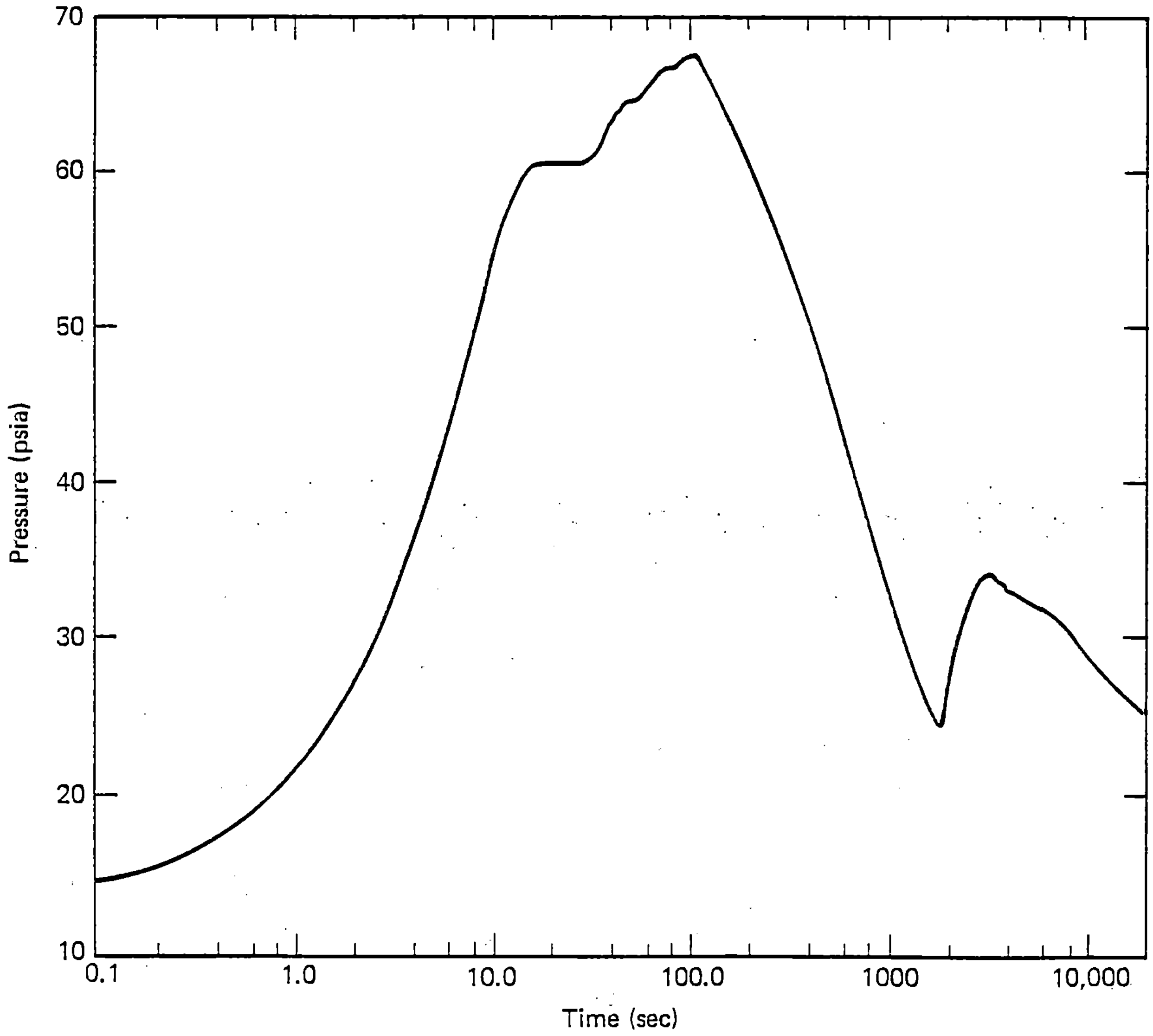


Fig. 1.4. Containment pressure response for primary loop break.

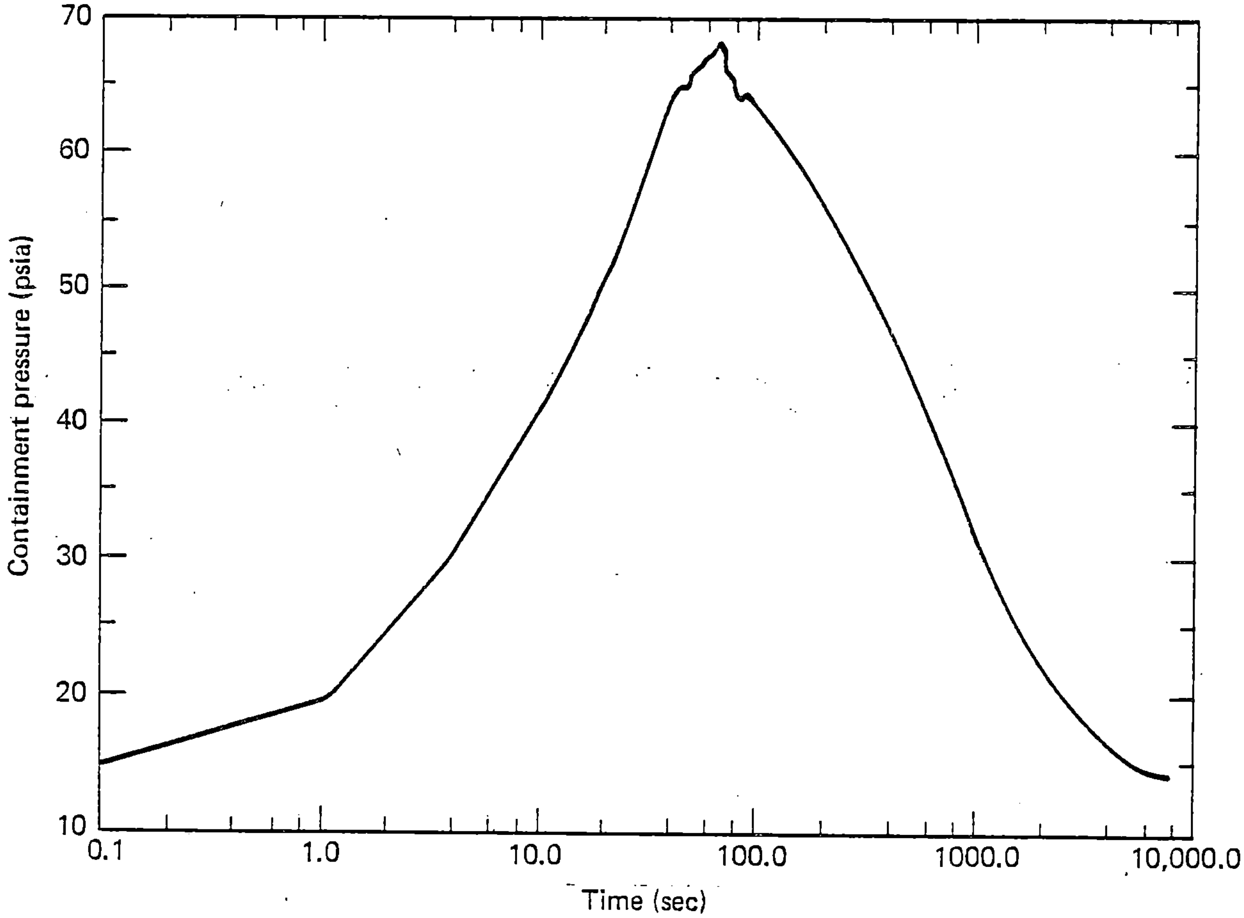


Fig. 1.5. Containment pressure response for secondary loop break.

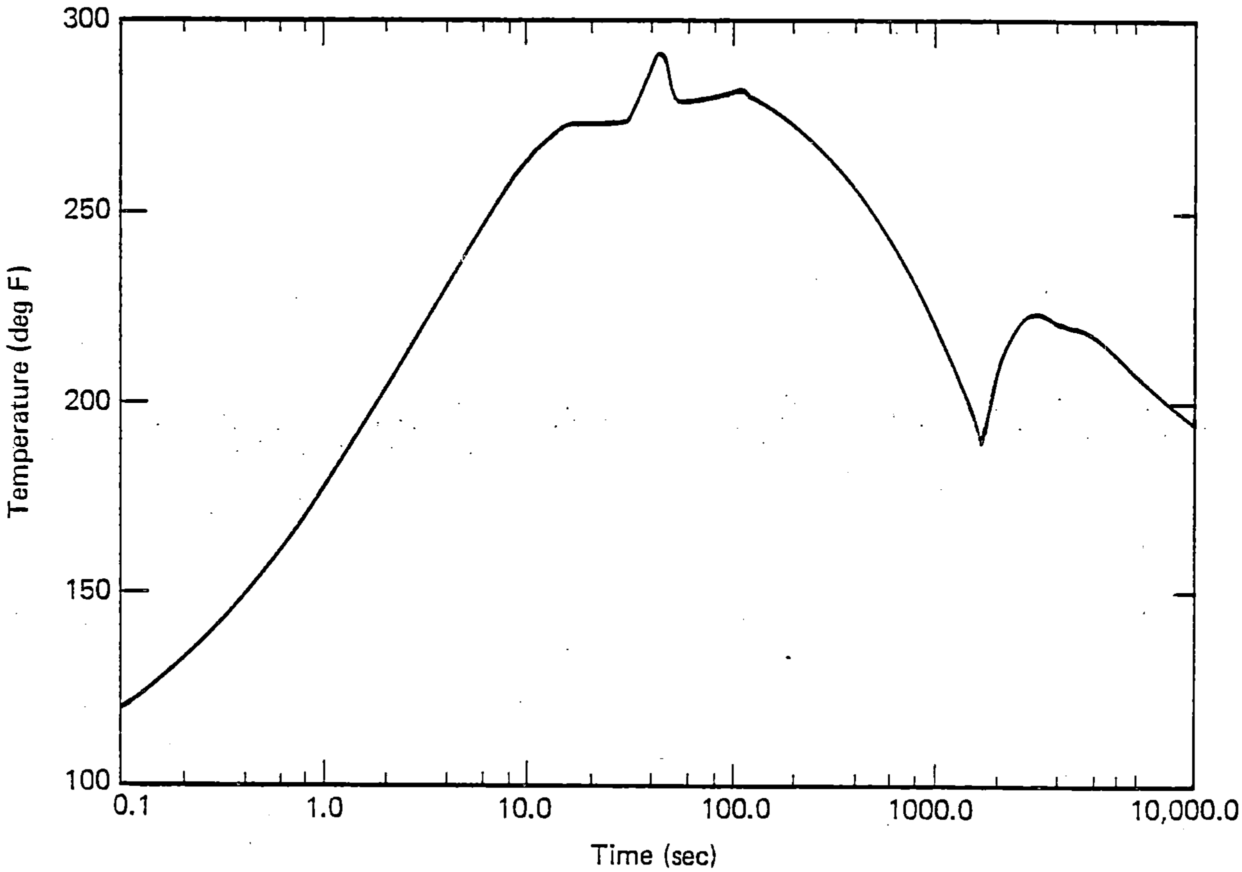


Fig. 1.6. Containment temperature response for primary loop break.

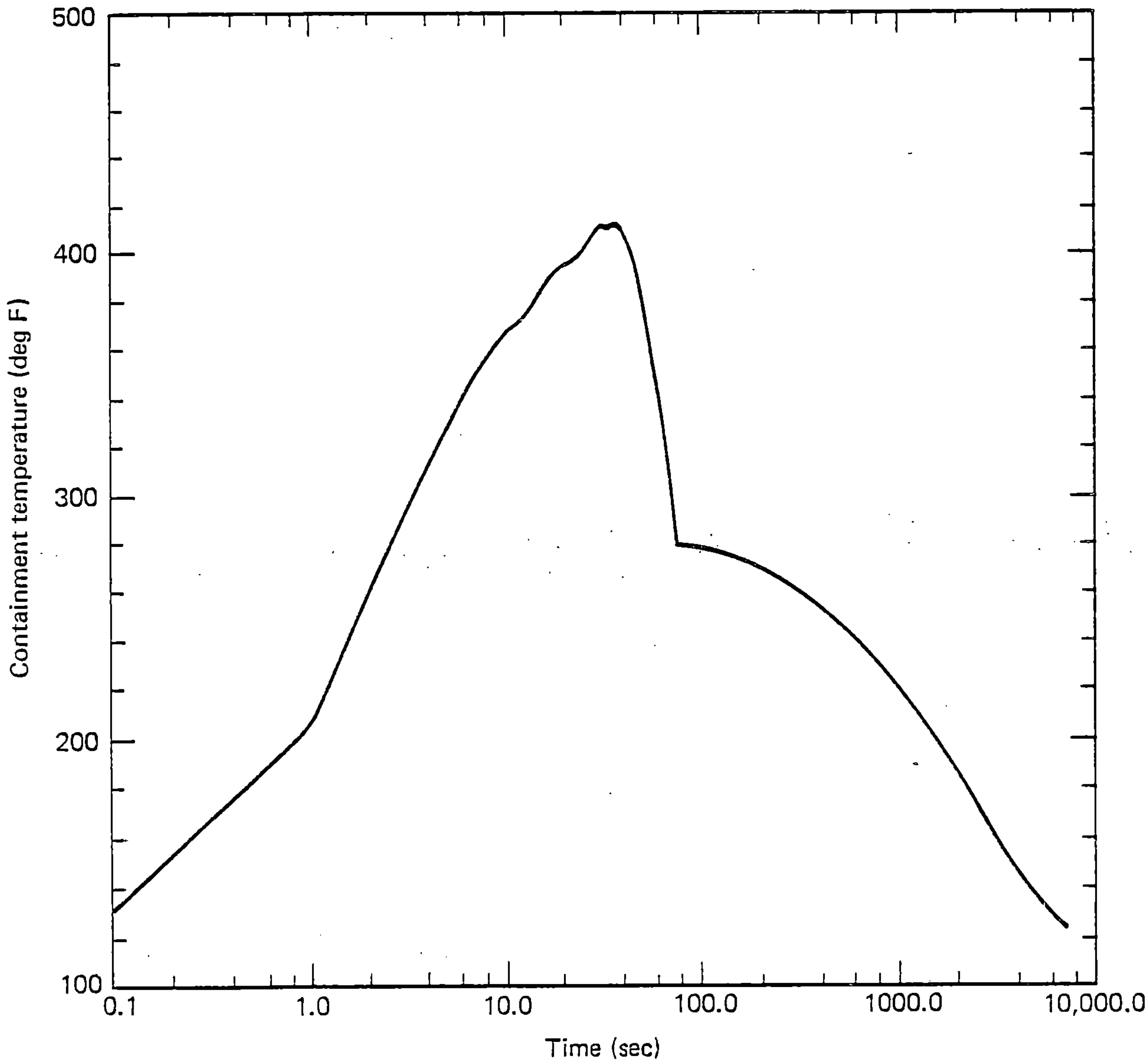


Fig. 1.7. Containment temperature response for secondary loop break.

will be heated to a temperature nearly equal to the high atmosphere-temperature, and only a small portion of the concrete wall will actually "see" a high temperature gradient. Because of the short duration of the high accident-temperature, it was decided that the following operating-condition temperatures be used for the concrete:[8] summer--73^{0} F at the outside containment wall and 123^{0} F at the inside containment wall; winter--minus 1^{0} F at the outside containment wall and 85^{0} F at the inside containment wall. The stress-free temperature in the concrete was assumed to be 70^{0} F.

The steel liner plate is only 1/4 in. thick and has a much higher thermal conductivity than the concrete wall; the liner plate was therefore assumed to have the same temperature as the containment atmosphere. The containment atmosphere temperature of the secondary system pipe break case (410^{0} F) is much higher than the containment atmosphere temperature of the primary system pipe break case (292^{0} F). The liner temperature was assumed, conservatively, to be 410^{0} F for the thermal load calculation.

The following thermal load component usually does not need to be considered separately if a composite liner-concrete section is used in evaluating loads on the section: the additional equivalent pressure between the concrete wall surface and the liner due to the different thermal expansion in the liner and concrete. However, this load had to be included in the mathematical model of this analysis for evaluating the concrete structure because the concrete wall was modeled without the liner. The additional equivalent pressure due to differential concrete and liner expansion was estimated to be 23 psi on the wall surface under these average winter wall temperatures: 410^{0} F in the liner and 43^{0} F in the concrete. The stress responses to this additional equivalent pressure can be obtained by multiplying the response-to-pressure-load case by the factor 23/55.

e. Seismic loads

Reference 5 suggested a set of site-specific SSE horizontal ground response spectra for SEP plants, including the Palisades site. In a subsequent study (Ref. 11) the original spectra developed for the Palisades site were modified to account for the site amplification

Table 1.1. Horizontal site-specific spectral accelerations.

	Pseudo acceleration (cm/sec^2) (5% damping)	
Period	Without Site Amplification[5]	With Site Amplification[11]
0.03	102.50	205.0
0.04	122.29	244.58
0.05	130.19	260.38
0.08	152.05	304.10
0.10	179.69	359.38
0.20	214.77	429.54
0.30	224.41	448.82
0.40	218.32	430.09
1.00	174.57	174.57

effects. The vertical SSE response spectra are two-thirds of the horizontal response spectra.[10]

We made a seismic reanalysis of the Palisades containment building using the same structural model reported in Ref. 2, but with the site-specific spectra, including site amplication. The spectral values are shown in Table 1.1. The reanalysis was necessary because the seismic responses reported in Ref. 2 were based on 0.2g R.G. 1.60 spectra, rather than the site-specific spectra which were developed later. The structural damping of the containment shell structure was increased to 7% to account for the significant cracks expected to develop under LOCA conditions. Figures 1.8 and 1.9 present the results of the reanalysis for the site-specific spectrum (SSSP). Three soil cases were considered, following the approach of Ref. 2. The results due to the site-specific spectrum are significantly lower than both those of the 0.2g R.G. 1.60 spectrum and the licensee's original design seismic loads. The vertical response throughout the containment building is 0.24g from the present analysis. This response results primarily from a mode where the structure acts as a rigid mass on the vertical soil spring.

Because the primary purpose of this analysis is to evaluate the overall structural integrity of the containment building, no local load effects are considered. We assumed that the areas around the penetrations are

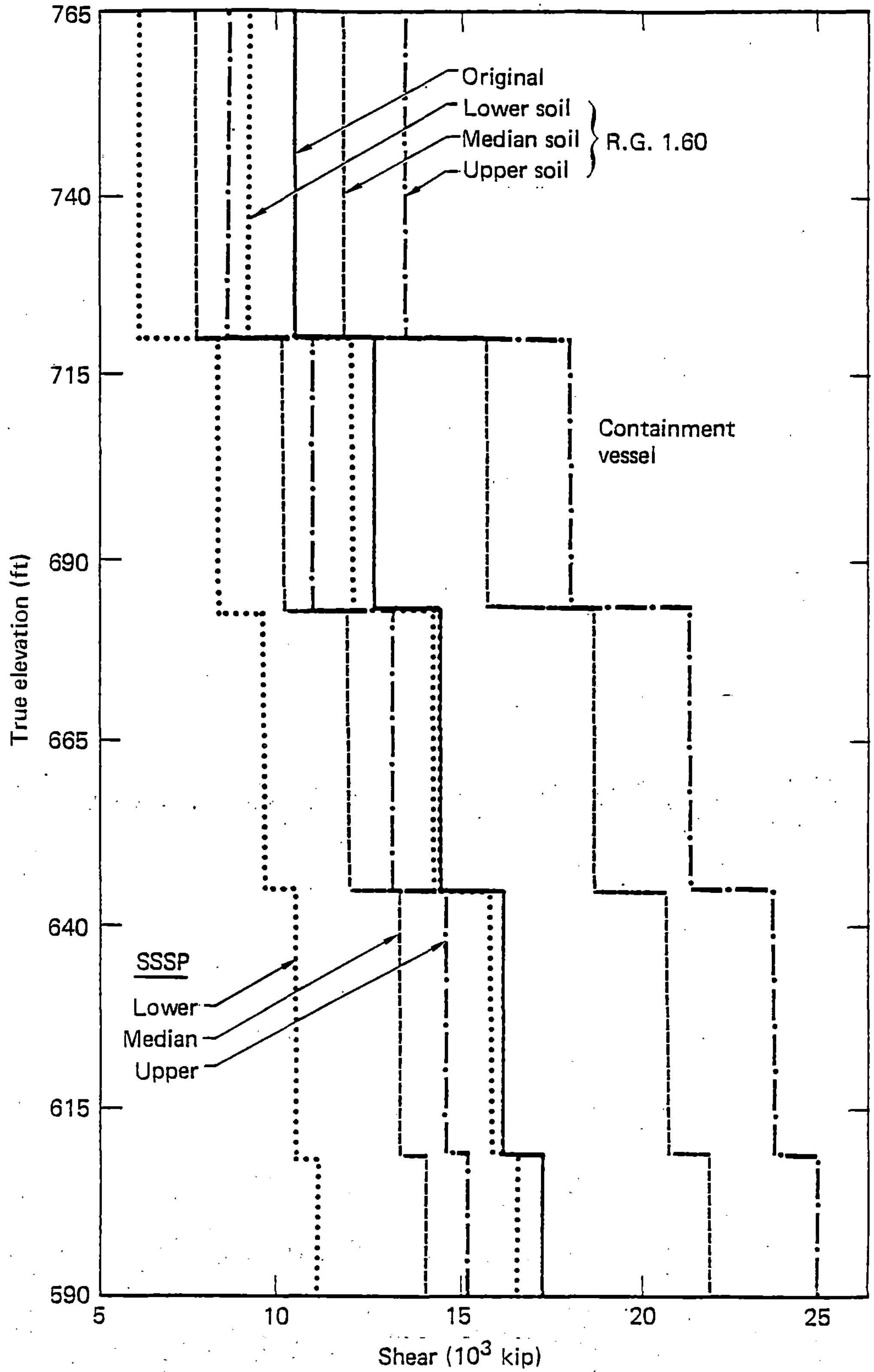


Fig. 1.3. Shear distribution in the containment building for the lower, median, and upper soil cases due to site-specific spectrum (SSSP), R.G. 1.60 (0.2g) spectrum, and the original seismic loads.

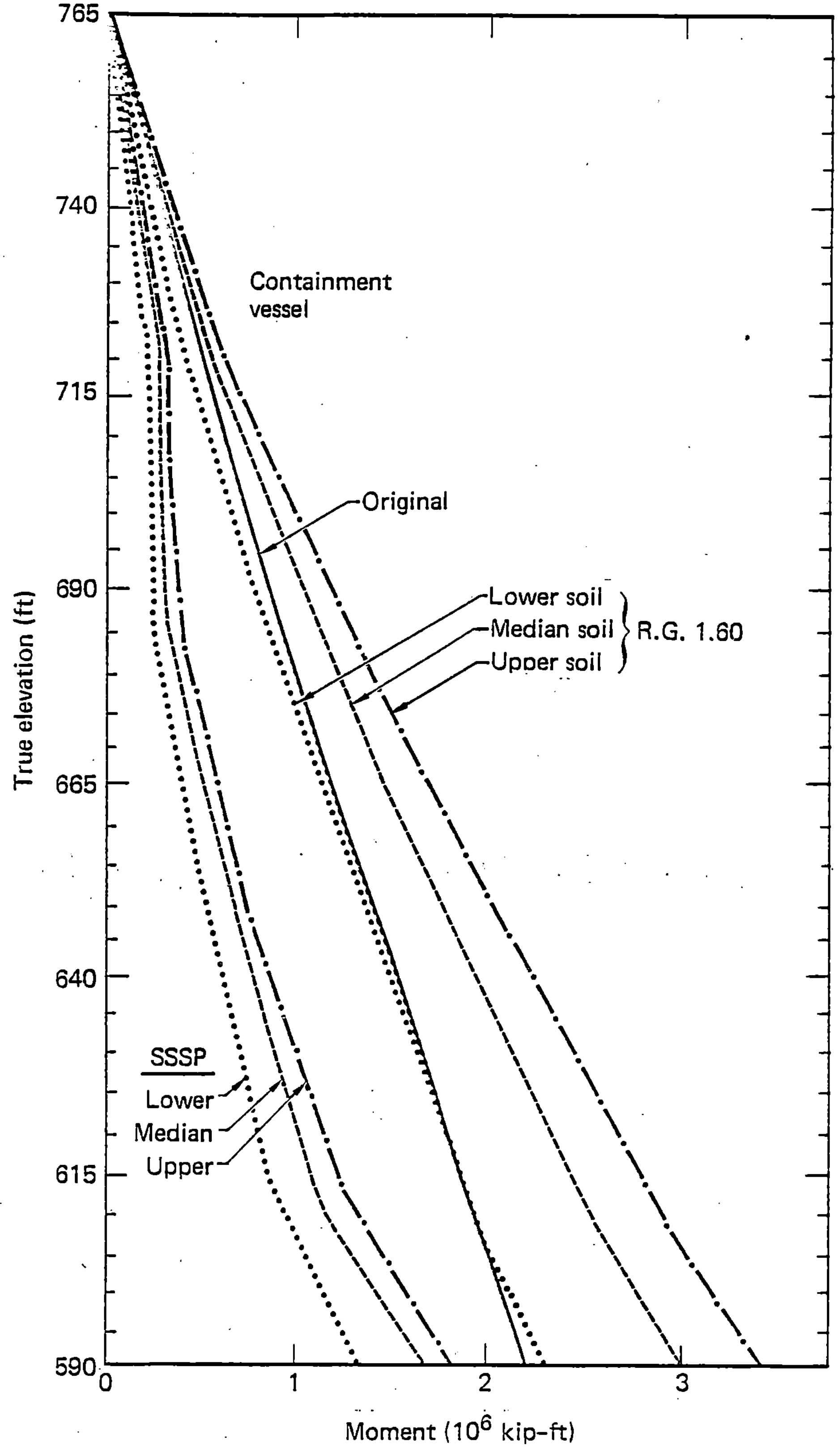


Fig. 1.9. Moment distribution in the containment building for the lower, median, and upper soil cases due to site-specific spectrum (SSSP), R.G. 1.60 (0.2g) spectrum, and the original seismic loads.

sufficiently strengthened so that they are stronger than the remainder of the structure. Therefore, we did no structural evaluation for these local areas and considered the structure to be axisymmetrical. The effects of live-loads, such as snow load, are considered small, and are therefore negelected. The total combined Abnormal/Extreme Environmental load for the containment building is the sum of all the load cases discussed above.

1.4 MATERIAL PROPERTIES

All material property values used in the analysis were extracted from section 5.1.3 of Ref. 4. Following is a list of these values for various loading conditions.

D = Dead load
F = Prestress
T = Thermal
P = Pressure
E = Earthquake

Materials	Loading conditions	
	D, F, T	P, E
Concrete walls		
E (Young's modulus) (psi)	2.7×10^{6}	5.5×10^{6}
ν (Poisson's ratio)	0.17	0.17
α (coefficient of thermal expansion) (in./in./^{o}F)	5.0×10^{-6}	-
f'_c (compressive strength) (psi)	5000	5000
Steel Liner (ASTM A-442)		
E (Young's Modulus) (psi)	30×10^{6}	30×10^{6}
f_y (minimum yield stress) (psi) 34,000	34,000	
ν (Poisson's ratio)	0.30	0.30
α (coefficient of thermal expansion) (in./in./^{o}F)	6.5×10^{-6}	-
Liner anchor (ASTM A-36)		
E (Young's Modulus) (psi)	30×10^{6}	30×10^{6}
ν (Poisson's ratio)	0.30	0.30

1.5 PREVIOUS ANALYSES OF CONTAINMENT BUILDING

Many analyses have been performed for various load conditions. It is not our purpose to review all earlier work. We discuss only those analyses which dealt with load combinations similar to those considered in this report.

Amendment 1 of the Preliminary Description and Safety Analysis Report[3] discussed a design-accident condition including dead load, prestress, thermal load, internal pressure, wind, and earthquake. The actual thermal gradients used in this analysis are not clear from the information availabe in Ref. 3, but they are presumably the same as those given in the FSAR (Ref. 4): i.e., 283°F inside and 10°F outside. The internal pressure load is 55 psig, as given in FSAR. The earthquake load is based on Housner's spectra with 0.2g peak ground acceleration.

The containment-building shell was modeled using an axisymmetrical solid-finite-element system. The axial force and the shear and moment distribution along the wall and dome were presented in Figure 2.12 of Reference 3. Amendment 4 of Reference 3 gave stress results for the liner plate and tendon and mat reinforcing bars, but this amendment did not give additional information on how the stresses were computed.

The FSAR of Palisades[4] described the load combinations used in designing the containment liner. The FSAR also furnished the computed concrete and reinforcing steel stresses at several sections of the wall and dome. A separate supplement to the "Response to NRC Seismic Question" (Item 2.A of Ref. 7) presented a calculation for a cracked concrete section of a typical wall section. The calculation seems to indicate that the force and moment for the section were obtained from a finite-element model which included both the concrete and the liner. The stresses were calculated using a technique in which the stress distribution is first determined for the uncracked concrete section. The concrete is then assumed to crack, and the neutral axis therefore shifts until force equilibrium is achieved between the concrete in compression and the reinforcing steel in tension. This technique was applied in Reference 7 to the combined loads, which included thermal and other nonthermal loads.

CHAPTER 2: SUMMARY AND CONCLUSION

2.1 CONCRETE STRUCTURE

The concrete containment shell structure was analyzed for these combined load conditions: dead weight, prestress, accident pressure (55 psig), thermal loads (410^0 F in the liner and operating temperatures in the concrete), and seismic loads of 0.21g, site-specific spectra. The structure was first analyzed for the above load cases on the assumption that the concrete section was uncracked. The stresses in the concrete and reinforcing steel were then evaluated, based on cracked concrete sections. In considering the cracked concrete section, the self-relieving effects of thermal loads were included. The results indicate that the highest stress in reinforcing steel is 15 ksi, which occurs about 14 ft above the base. The compressive concrete stresses are below 1000 psi. The maximum shear stress in the concrete is less than 220 psi. All concrete and reinforcing steel stresses are within the allowable ranges given by ASME Boiler and Pressure Vessel Code Section IV, Division 2, Articles CC-3420 and CC-3520. The concrete structure is therefore considered adequate to withstand the Abnormal/Extreme loads of a primary or secondary system pipe break which is combined with a SSE event.

2.2 LINER PLATE SYSTEM

The liner system was evaluated for the Extreme Environmental and Abnormal/Extreme Environmental conditions. Both conditions include the SSE seismic loads. The Abnormal/Extreme conditions also includes the accident pressure and temperature from the blowdown of one steam generator.

The liner system near the cylinder-to-base junction was evaluated because it is the most critically loaded point. The liner strains and the anchor movement and forces were computed and then compared with the allowables specified in ASME Code Section III, Division 2, Subsection CC. From the evaluation, we concluded that the existing design of the liner plate system possesses sufficient capacity against failure in the event of an SSE or an SSE plus LOCA.

CHAPTER 3: ANALYSIS OF CONTAINMENT BUILDING

3.1 ASSUMPTIONS

The containment building was modeled by a finite-element system for all load cases, with the exception of the seismic analysis. Seismic responses were calculated from an analysis using the stick model from Ref. 2, which included soil structure interaction effects. The following assumptions were made in constructing the finite-element model.

1. Only the containment shell structure was modeled. The structure was assumed to be axisymmetrical. No internal structure was included in the model because the interaction between the containment shell and the internal structure is expected to have a minimal effect on the containment shell structure.

2. Because the model was not used for the seismic analysis, the foundation (including the building base) was assumed to be completely rigid. It was therefore not necessary to include the foundation in the model. This is a conservative assumption for concrete stresses near the cylinder and base junction, which are caused by loads other than seismic loads.

3. In computing the section loads, the concrete section was assumed to remain elastic (no cracking of the concrete). After the force and moment of the section were obtained from the elastic analysis, a cracked-section analysis was performed. The cracked-section analysis took into account the self-limiting nature of the thermal load.

4. In evaluating the section loads, this conservative assumption was made: the liner made no contribution to the structural stiffness .

5. During a LOCA, the temperature of the liner plate was assumed to be the same as the containment atmosphere-temperature. The concrete wall and dome remained at the operating temperature and had a linear gradient throughout their thickness. This is a reasonable assumption, because previous thermal transient analyses (such as those shown in the FSAR) indicate that only very small portions of concrete near the inside liner will experience the highest temperatures. The major portion of the concrete wall will remain at the operating temperature throughout the accident.

3.2 MATHEMATICAL MODEL

The load analysis of the containment building was performed using a finite-element mathematical model to depict the structure and the computer code SAP4. Figure 3.1 illustrates the model which utilized 2-D axisymmetric elements. Four layers of elements were used through the thickness. The following constraints were present: fixed footing-nodes in both the horizontal and vertical directions. The horizontal direction was constrained for the center-line nodes at the top of the dome for all nonseismic loads.

As mentioned previously, for the purpose of determining section loads in this mathematical model, cracking was assumed not to occur. The analysis was performed for each of the load cases, using a linear elastic approach. We performed a verification analysis, using published shell stress equations. The verification analysis compared favorably with values predicted by SAP4.

3.3 METHOD OF ANALYSIS

The SAP4 finite element analysis generates only radial, meridian, hoop, and shear stresses for each element. It is necessary to determine the bending moment and axial force across the thickness of the shell in order to perform the cracked-section stress analysis. This was accomplished by calculating the appropriate meridian and hoop forces acting on each element, and then using this force distribution across the thickness to determine the hoop and meridian bending moments. To combine the loads we summed dead weight, prestress, pressure, thermal, and seismic loads. We included the additional pressure due to thermal expansion of the liner plate by increasing the pressure load response with the factor 23/55. This procedure was discussed in Section 1.3.

After the combined loads of a section were determined, an elastic bending-section analysis was performed to determine whether or not the section cracked. If the section did not crack, the concrete and reinforcement stresses were computed from the simple bending-section analysis. If cracking occured within the section, the stresses were calculated using the following approach.

Location	Elevation (ft) Grade + relative
A	590 + 190
B	590 + 189
C	590 + 185
D	590 + 181
E	590 + 175
F	590 + 170
G	590 + 164
H	590 + 156.9
I	590 + 149.1
J	590 + 129.8
K	590 + 87.7
L	590 + 28.8
M	590 + 13.6
N	590 + 9.6
O	590 + 6.0
P	590 + 0.5

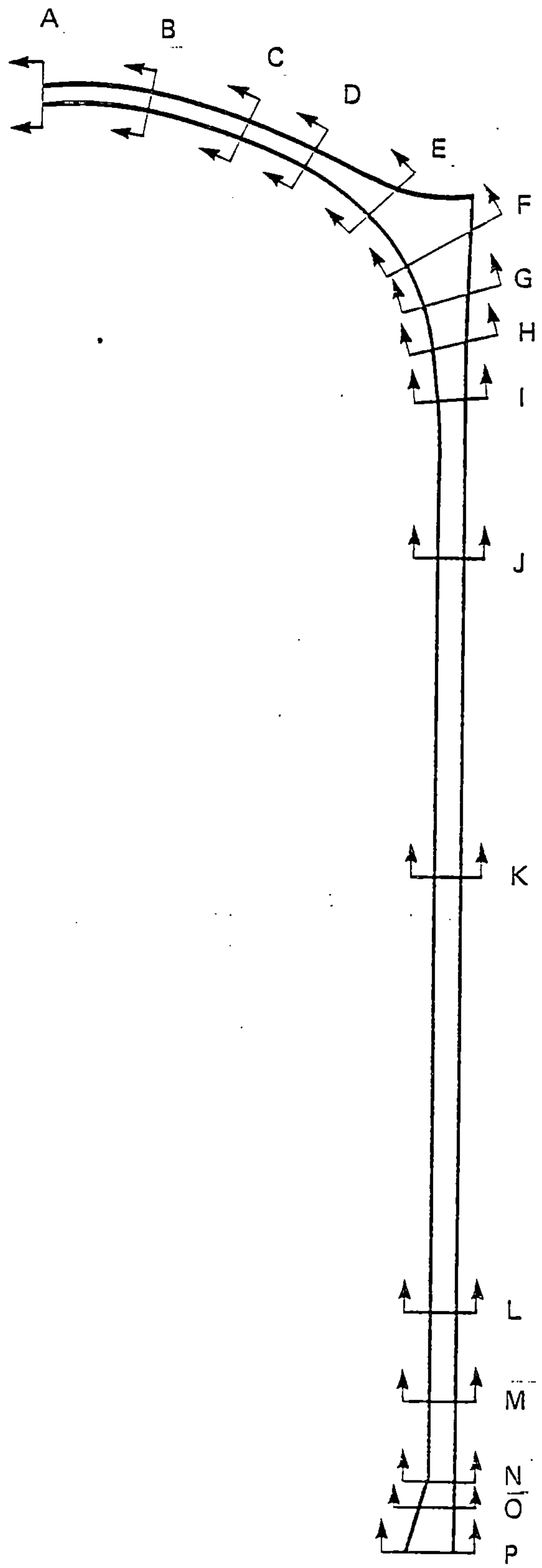


Fig. 3.1. Mathematical model of the containment building.

1. The total axial load on a section was divided into three groups.
   a. P_a, This includes loads which always act at the same location of the section, regardless of whether or not the section is cracked. An example is dead load.
   b. P_c, This includes loads which always act at the center of the uncracked portion of the cracked section. An example is pressure load.
   c. P_t, This is the thermal load. It acts at the center of the cracked section and is proportional to the effective area (A_c) of the cracked section, i.e.:

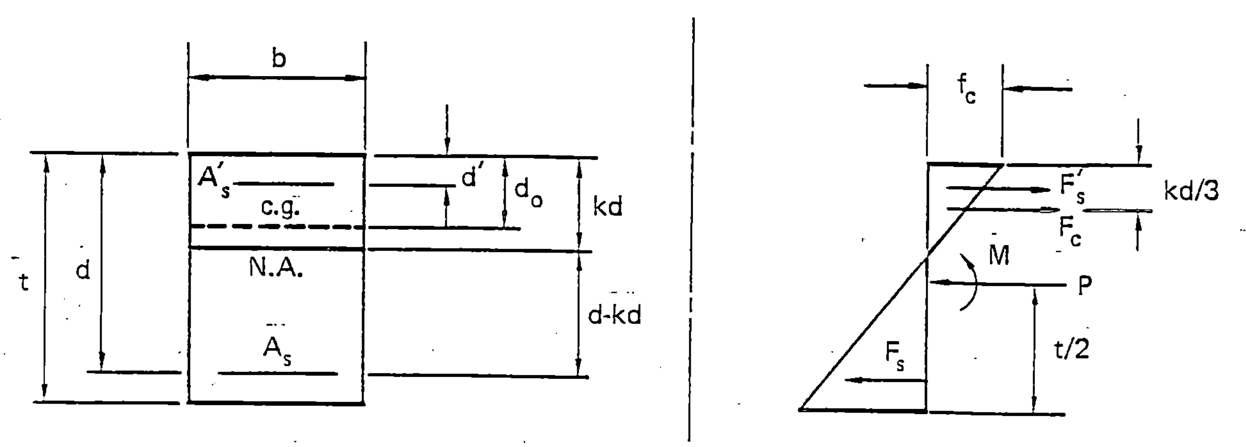
$$P_t = (A_c/A_o) \quad P_{to}$$

   where A_o is the sectional area and P_{to} is the thermal axial load of the uncracked section.

2. The axial loads just discussed cause the total axial moment about the midsection. Following is a discussion of the total axial moment.
   a. M_a, the moment due to Pa.
   b. M_c, the moment due to Pc. Its value varies with the location of the center of the cracked section

$$M_c = P_c(t/2 - d_o)$$

   where t is the thickness of the section and d_o is the distance from the compressive fiber to the center of gravity of the cracked section. These relationships are shown in the following diagram.



c. M_t, the moment due to thermal load. This includes two parts. One part is caused by the thermal gradient and is proportional to the cracked moment of inertia; the other part is caused by P_t. Therefore

$$M_t = (I_c/I_o)\, M_{to} + (t/2 - d_o)\, P_t$$

where I_c is the moment of inertia of the cracked section, and I_o is the moment of inertia of the uncracked section.

3. The properties of the section give us the following relationships.

$$n = E_s/E_c$$

$$A_o = bt$$

$$I_o = bt^3/12$$

$$A_c = bkd + (n-1)\, A_s' + nA_s$$

$$d_o = \left[1/2b\, (kd)^2 + (n-1)\, A_s'd' + nA_sd\right]/A_c$$

$$I_c = b(kd)^3/12 + bkd\, (d_o - kd/2)^2 + (n-1)\, A_s'\, (d_o - d')^2 + nA_s\, (d-d_o)^2$$

By strain compatibility

$$F_c = f_c\, kdb/2$$

$$F_s = A_sn\, \left[(d-kd)/kd\right]\, f_c$$

$$F_s' = \left[(kd-d')/kd\right]\, f_c\, (n-1)\, A_s'$$

By static equilibrium

$$P_a + P_c + P_t = F_c + F'_s - F_s$$

$$M_a + M_c + M_t = F_c\ (t/2 - kd/3) + F'_s\ (t/2 - d) + F_s\ (d - t/2)$$

By substituting the above expressions into these two equilibrium equations and solving simultaneously for k, a seventh-order polynomial expression is obtained. The polynomial equation can be solved numerically for k. Subsequently, solutions for f_c and f_s can be obtained.

3.4 RESULTS OF ANALYSIS

Figures 3.2 through 3.6 illustrate the calculated forces and moments for each of the load cases, except for seismic loads. The prestress values were extracted from Ref. 3. The seismic loads are listed in Table 3.1.

The SAP4 results of finite element analysis show good agreement with closed-form solutions from shell analysis at locations where such solutions are applicable. For instance, SAP4 hoop forces due to pressure loading are 450 kip/ft in the cylinder and 340 kip/ft in the dome. The values of the closed-form shell solution are 459 kip/ft for the cylinder and 359 kip//ft for the dome. The meridian forces for the cylinder are 232 kip/ft from SAP4 and 230 kip/ft from the shell solution. The meridian moment at the base due to pressure is 490 kip/ft from SAP4 and 470 kip/ft from the shell solution. The thermal moment at the cylinder in the meridian direction is 192 kip//ft from SAP4 and 205 kip/ft from the shell solution.

To evaluate the concrete and reinforcement stresses, 16 cross sections were taken along the dome and cylinder. At each of the sections the cracked-section analyses described in Section 3.3 were performed for both meridian and hoop directions. Table 3.2 gives the concrete and reinforcement stresses for all sections. The results for the winter thermal case are given in Table 3.2. The results for the summer thermal case are a little lower

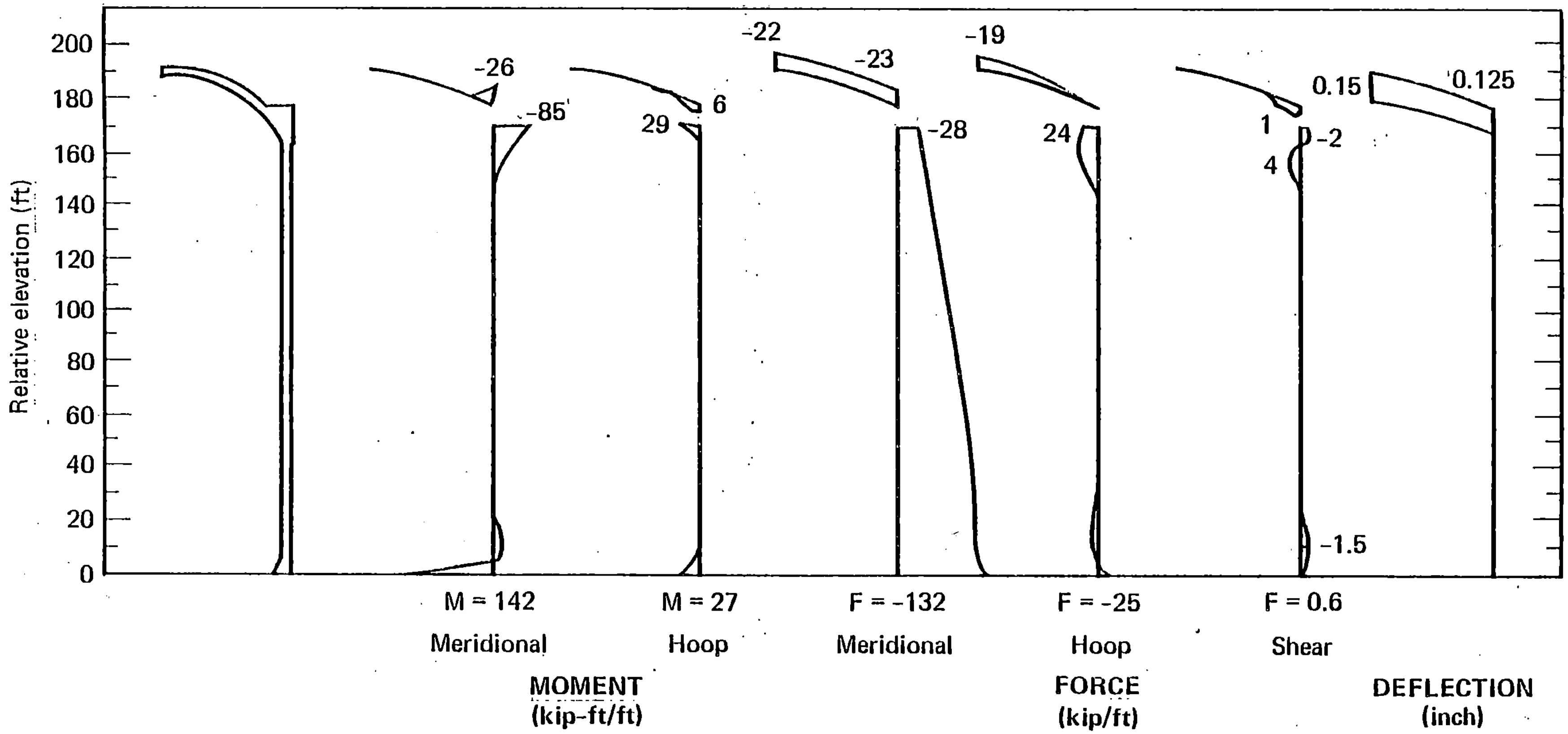


Fig. 3.2. Dead load.

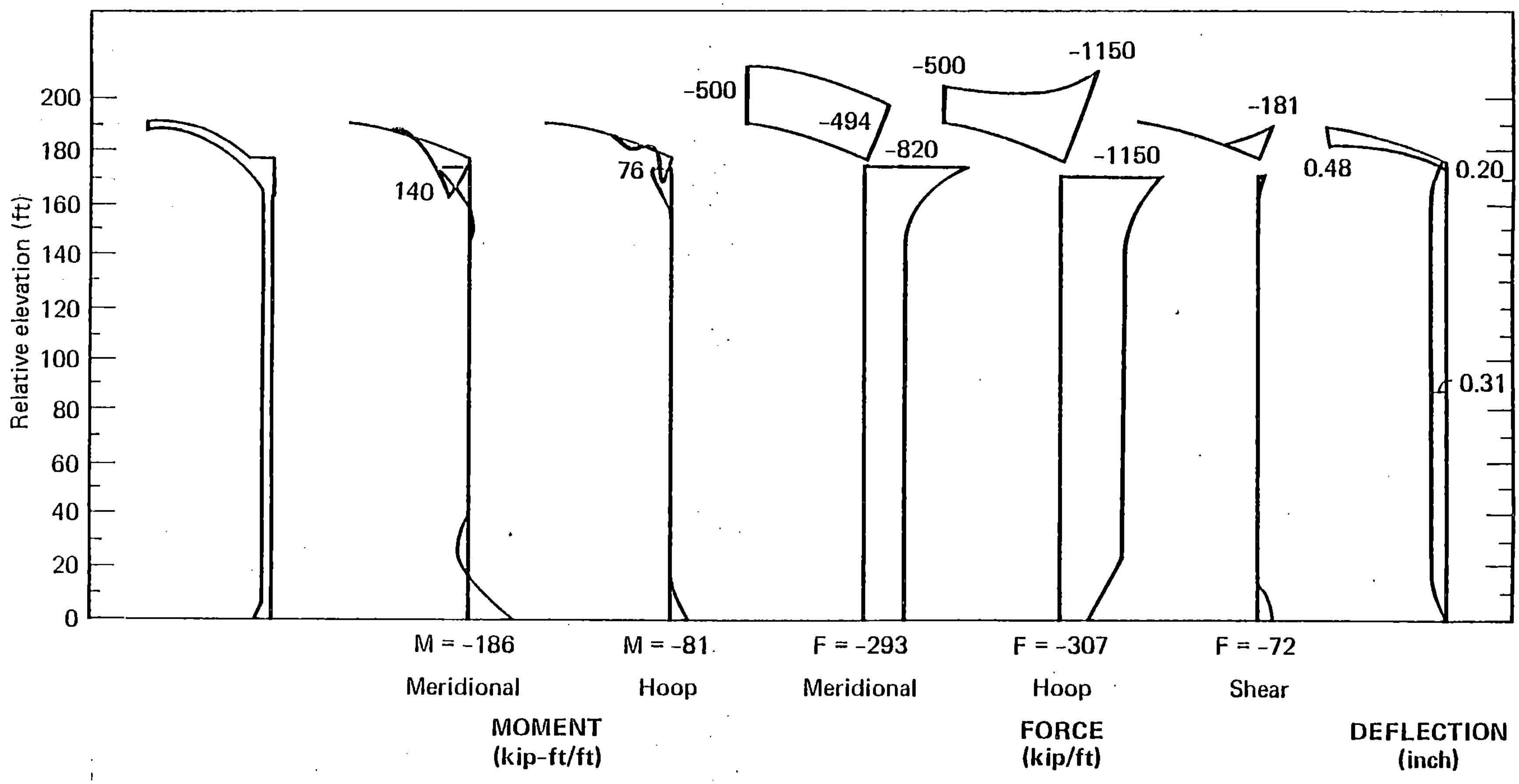


Fig. 3.3. Prestress load.

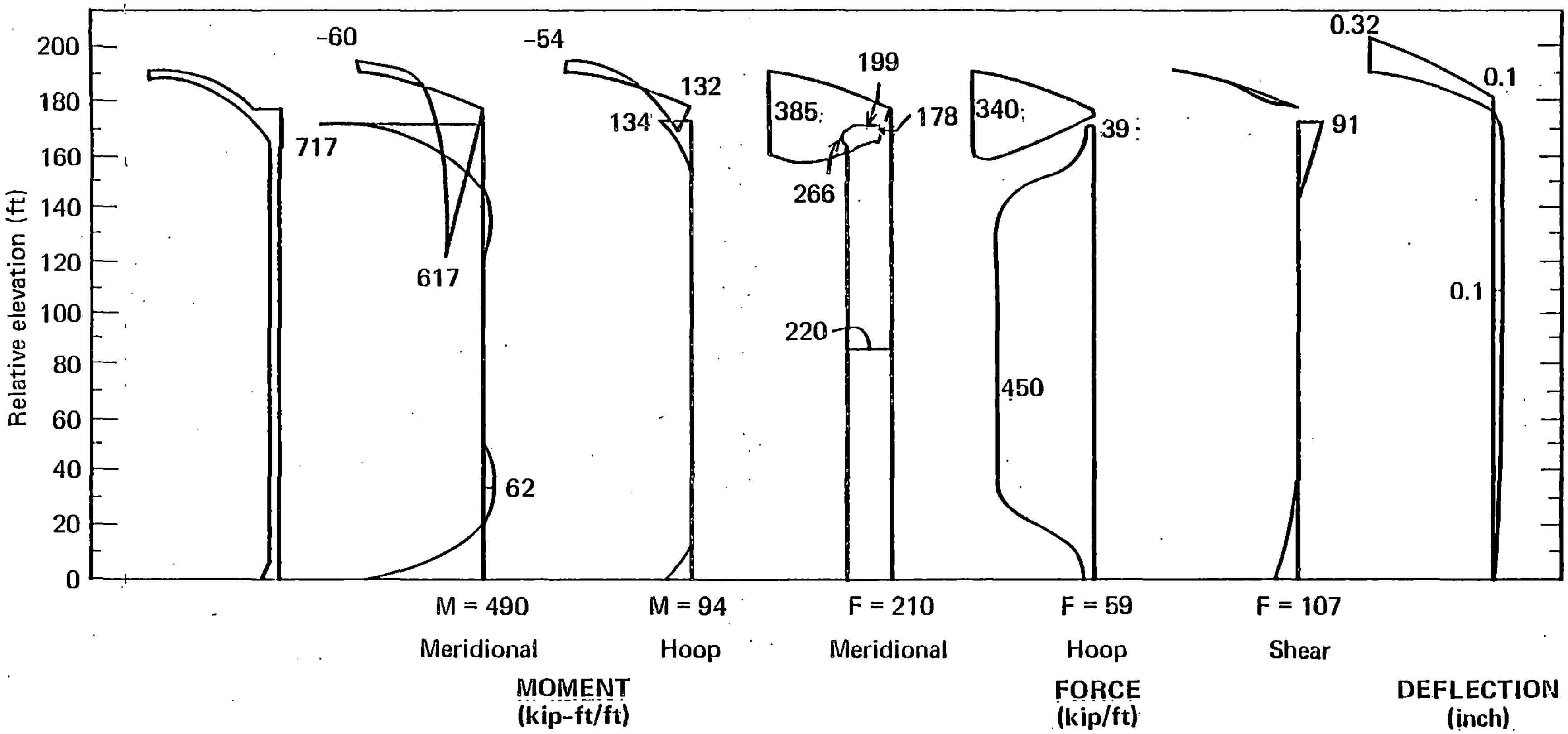


Fig. 3.4. Pressure load.

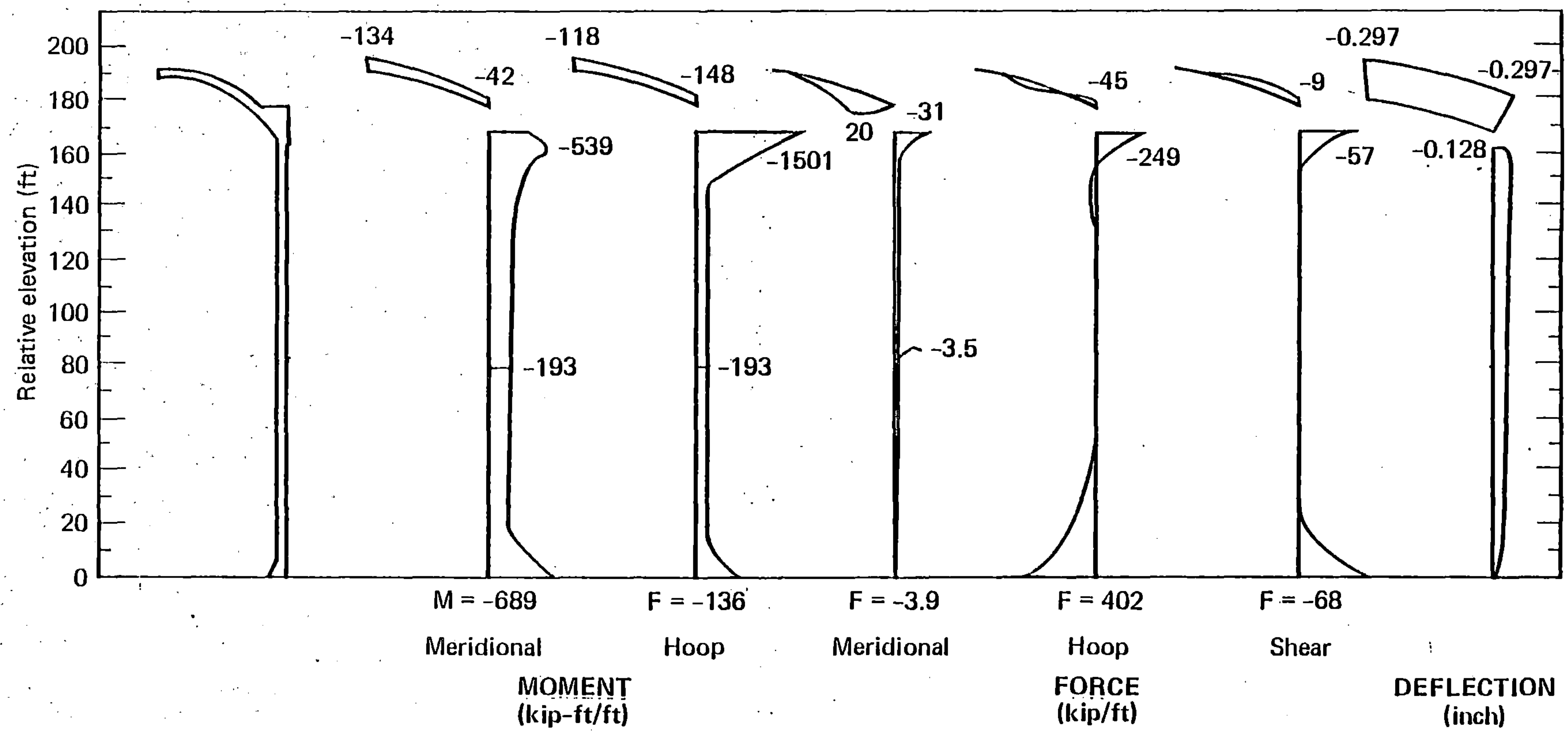


Fig. 3.5. Winter thermal load.

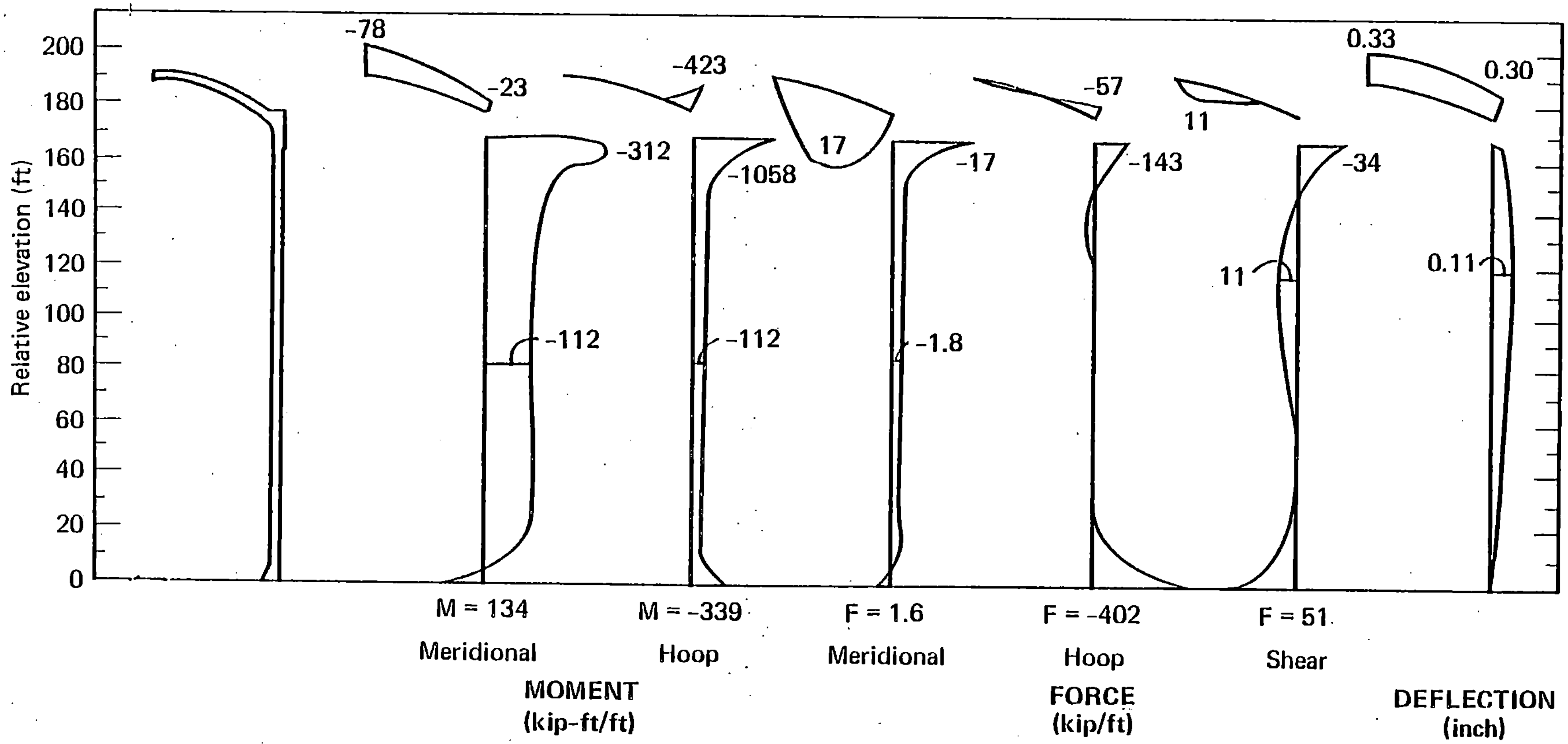


Fig. 3.6. Summer thermal load.

than those for winter. According to ASME Boiler and Pressure Vessel Code, Section III, the allowable concrete stress is 0.85 f'c or 4250 psi, and the allowable stress for reinforcing steel is 36 ksi. The concrete flexural stresses are all less than 1000 psi. The maximum steel stress is about 15 ksi. This stress is located about 14 ft above the base. The shear stresses were evaluated according to ASME code articles CC-3420 and CC-3520. Among the 16 sections, the more critical are those near the base and the ring girder. In comparison with the code allowable, the lowest factor of safety near the ring girder is 1.3, and the lowest factor of safety near the base is 1.7.

Table 3.1. Section forces due to horizontal seismic load.

Section	True Elevation ft	Global Moment 10^6 kip-ft	Global Shear 10^3 kip	Force: Meridional kip/ft	Force: Shear kip/ft
E	765	0	8.4	0	22.4
F	760	0.04	8.4	3.6	22.4
G	754	0.08	8.4	7.1	22.4
H	747	0.12	8.4	10.7	22.4
I	739	0.17	8.4	15.2	22.4
J	720	0.30	8.4	26.7	22.4
K	678	0.45	13.2	40.1	35.2
L	619	1.17	14.5	104.3	38.6
M	604	1.50	15.2	133.7	40.5
N	600	1.60	15.2	142.7	40.5
O	596	1.70	15.2	151.6	40.5
P	591	1.82	15.2	162.3	40.5

Table 3.2. Stresses of cracked section in concrete and steel (winter thermal case).

Section	Meridian stress, ksi		Hoop stress, ksi		Shear stress, ksi
	f_c	f_s	f_c	f_s	v_c
A	0.455	3.611	0.627	2.712	0.008
B	0.545	4.284	1.964	0.866	0.017
C	0.536	2.255	N/C[a]	N/C	0.010
D	0.565	0.271	N/C	N/C	0.130
E	N/C	N/C	N/C	N/C	0.217
F	N/C	N/C	N/C	N/C	0.136
G	N/C	N/C	N/C	N/C	0.139
H	0.614	1.805	N/C	N/C	0.127
I	0.672	6.977	N/C	N/C	0.095
J	0.420	3.899	0.810	4.483	0.054
K	0.415	3.265	0.884	3.675	0.070
L	0.495	8.348	0.805	4.414	0.085
M	0.572	14.933	1.207	0.142	0.121
N	0.602	12.039	N/C	N/C	0.098
O	0.427	8.881	1.849	4.989	0.068
P	0.205	4.893	0.469	7.086	0.058

[a]Not cracked.

f_c = normal stress in concrete
f_s = normal stress in steel
v_c = shear stress in concrete

CHAPTER 4: ANALYSIS OF LINER PLATE SYSTEM

4.1 METHOD OF ANALYSIS

Most of the loads imposed on the liner plate result from the shortening of the concrete shell relative to the liner plate. The relative strain causes compressive membrane-loads on the plate. The anchors will not be loaded if all the liner plates are perfectly fabricated and are erected so that they are either perfectly flat or have outward curvature. When one panel has an inward curvature, caused by a fabrication or construction imperfection, it will deform inwardly because it has lower in-plane stiffness than the other panels. A panel with inward curvature is illustrated in Figure A.1. The anchor system is then subjected primarily to a shear load, which is largest at the two anchors adjacent to the bent plate and diminishes rapidly away from them. The anchors will also be subjected to radial force, longitudinal force, etc.; these are minor when compared with the shear load.

For the liner system, there are several possible modes of failure. Examples are:

a. Excessive strain in the liner.

b. Shearing failure of anchors in the hoop direction.

c. Radial pullout of an anchor adjacent to a bent plate with an inward curvature.

d. Longitudinal buckling of the liner plate.

We considered the possibility of pullout of the anchor. Reference 9 has demonstrated that the concrete and anchorage have a capacity of about 1500 lb/in. against pullout. This capacity arises from the shearing and bonding of the anchor and concrete, which has been shown to be much greater than the pullout force that can be developed adjacent to a bent plate. Therefore, a pullout failure of the anchor is not a concern. This leads to the further conclusion that longitudinal buckling of the liner plate is also highly unlikely, unless an anchor pullout does take place. Evaluation of the liner system can thus be concentrated on the liner strains and the shearing movement of the anchor adjacent to a bent plate.

The following analysis was made for both Extreme Environmental and Abnormal/Extreme Environmental conditions. The Extreme Environmental condition was considered because, as will be seen later, under mechanical loads it produces a more severe anchor load, in comparison with code allowables, than does the Abnormal/Extreme Environmental condition.

4.2 ANALYSIS MODEL

Based on the load combination, the liner system was most critically loaded near the junction of the cylinder and the base slab (Section N, relative el. 9.6 ft). Therefore, the analysis considered a 1-inch-wide strip of the liner system that runs in the hoop direction. One of the panels was given an initial inward curvature corresponding to a radial deflection of Δ = 1/8 in. at the center of the panel. The remaining liner was treated as flat plate. The 2 in. x 3 in. x 1/4 in. angles anchor the plate to the concrete at 15 in. intervals along the hoop direction. The resulting model is shown in Fig. A.2(a) of the Appendix. This model can be further reduced to the spring system illustrated in Fig. A.2(b).

The spring system consists of three types of springs: K_{BP}, K_C, and K_{FP}. The spring K_{BP} represents the in-plane stiffness of the bent plate panel; it is nonlinear in nature and its property was adopted from Ref. 9. These parameters were based on an in-plane compression test on a bent plate having similar material properties. The K_{BP} curve is shown in Fig. A.3. The linear portion of the curve has a slope of 130 kip/in./in.

The stiffness of the anchorage against shear movement is represented by K_C. This is also nonlinear in nature, as shown in Fig. A.4. It was adopted from the tests described in Ref. 9. These tests were performed on 3 x 2 x 1/4 angles embedded in concrete, which had a Young's modulus of 5400 ksi. The linear portion of the K_C curve had a slope of 270 kip/in./in.

The in-plane stiffness of the flat plate is represented by K_{FP}. This is equal to 500 kip/in./in., as computed in the Appendix.

4.3 ANALYSIS AND RESULTS

The procedure that follows is outlined in the Appendix. Strains in concrete on the inside face of the concrete shell were first computed from stresses due to mechanical loads. The total compressive strain in the liner plate was determined by combining the strains on the concrete due to mechanical loads, concrete shrinkage, and the differential strain between the liner and the concrete resulted from thermal loads. This liner strain was converted to the unbalanced membrane force N_h, which was combined with the unbalanced membrane force N_h' (due to pressure acting on the bent plate) and then applied to the anchors adjacent to the assumed bent plate to determine the shear force and movement of the anchor.

STEP 1: Compute Liner Plate Membrane Strains

Concrete strains due to mechanical loads were first computed at Section N (El. 9.6 ft) of the concrete cylinder near the base, where thickening of the concrete section begins. For the location of Section N, refer to the axisymmetric finite-element stress-analysis model of the concrete shell shown in Fig. 3.1. As stated previously, the concrete strain due to initial shrinkage prior to prestressing of the containment wall was assumed to be -100 μ. Otherwise, concrete strains due to dead load (D), vertical and horizontal seismic loads (E_V and E_H), pressure load (P_v and P_a), and prestressing load (F) were converted from the forces/moments generated by the finite-element stress-analysis of the concrete shell. Note that the stress results for the prestressing load are adopted from the PDSAR of Palisades Unit 1.

Table 4.1 lists the meridional force, f_z; meridional moment, M_z; hoop force, f_h; and hoop moment, M_h, due to D, E, P, and F. A positive force signifies tension. A positive moment is one which causes a tensile bending-stress on the inside face of the concrete shell. The combined forces and moments are also shown for the Extreme and Abnormal/Extreme conditions. The load combination for the Extreme condition was $D + 0.4E_V + E_H + F + P_V$. For the Abnormal/Extreme condition, the load combination was $D + 0.4E_V + E_H + F + P_a$. To simulate the equivalent effect of the square root of sum of squares (SSRS) combination between the vertical and horizontal seismic loads, the factor 0.4 was applied to the vertical seismic load.[10] Since E_v = 0.24g, upon application of the factor 0.4 the value of 0.4 E_v becomes 0.096g.

Table 4.1. Force and moment at Section N of concrete shell.

	$D+0.4E_V=$ 1.096D	E_H	P_a	P_v	F	Extreme $D+E+P_v+F$	Abnormal/ Extreme $D+E+P_a+F$
f_z (kip/ft)	-125.8	-142.7	216.9	-11.8	-293.0	-573.3	-344.6
M_z (kip-ft/ft)	- 14.3	--	131.8	- 7.2	- 50.0	- 71.5	67.5
f_h (kip/ft)	- 5.3	--	123.0	- 6.7	-715.0	-727.0	-597.6
M_h (kip-ft/ft)	2.8	--	24.1	- 1.3	- 20.0	- 18.5	6.9

Normal stresses at an element on the inside face of the concrete shell were then computed:

$$s_z = f_z/(42 \times 12) + 6M_z/42^2$$
$$= f_z/504 + M_z/294$$
$$s_h = f_h/504 + M_h/294$$

The concrete strains were related to the stresses in the following manner:

$$e_z = (s_z - \nu_c s_h)/E_c = (s_z - 0.17s_h) \times 10^3\ \mu/5.5$$
$$e_h = (s_h - 0.17s_z) \times 10^3\ \mu/5.5$$

When the concrete strains computed above were combined with the assumed strain of -100 μ, caused by the initial concrete shrinkage, we obtained the total strain for mechanical loads. This total included the relative membrane strain induced in the liner plate by the mechanical loads.

Thermal loads cause additional compressive strain in the liner because they produce larger expansion in the liner than in the concrete. According to the calculation procedures described in the Appendix, the thermal-induced relative strain in the liner was conservatively computed as follows:

Extreme Condition: (Winter) . for an inside temperature of 85^{o} F and outside temperature of -1^{o} F,

$$e_z = e_h = -6.5\ \mu\ (85+1)/2 = -280\ \mu.$$

Abnormal/Extreme: (Winter) . for a peak accident temperature of 410^{o} F,

$$e_z = e_h = -6.5\ \mu\ \left[410-(85-1)/2\right] = -2392\ \mu.$$

Table 4.2 lists the computed relative liner strains due to the mechanical loads, the thermal loads, and the combined effect of both.

STEP 2: Compute the unbalanced force, N.

The unbalanced membrane force is applied to the anchor point at the edge of the bent plate. This force is composed of two parts: N_h and $N_h^{'}$. N_h is due to the liner strains shown in Table 4.2. $N_h^{'}$ is due to the pressure, P_v or P_a, acting on the bent plate. For a 1/4 in. thick plate, Eq. (A-5) from the Appendix gave

$$N_h = 0.25 \times 30000\ (e_h + 0.3e_z)/(1-03^2) = 8242\ (e_h + 0.3e_z) \qquad \text{(A-5)}$$

From Eq. (A-6) of the Appendix,

$$N_h^{'} = P \times 15^2/(2\pi^2 \times 1000 \times 1/8) = 0.0912P \qquad \text{(A-6)}$$

and

$$N = N_h + N_h^{'}$$

Table 4.3 summarizes the values of N_h, $N_h^{'}$, and N for both the Extreme and Abnormal/Extreme conditions. Note that N was not computed for the mechanical loads under the Abnormal/Extreme condition because the mechanical loads under the Extreme condition were more critical. This is shown by Table 4.2.

Table 4.2. Relative liner plate membrane strains.

Condition	Loads	s_z (kip/in.^{2})	s_h (kip/in.^{2})	e_z	e_h
Extreme Condition	D + E + F + Pv	- 1.38	- 1.50	- 205 μ	- 230 μ
	So	N/A	N/A	- 100 μ	- 100 μ
	Thermal	N/A	N/A	- 280 μ	- 280 μ
	Total I			- 585 μ	- 610 μ
	Total II (mechanical loads)			- 305 μ	- 330 μ
Abnormal/ Extreme Condition	D + E + F + Pa	- 0.45	- 1.17	- 46 μ	- 199 μ
	So	N/A	N/A	- 100 μ	- 100 μ
	Thermal	N/A	N/A	-2392 μ	-2392 μ
	Total I			-2538 μ	-2691 μ
	Total II (mechanical loads)			- 146 μ	- 299 μ

So = shrinkage

Table 4.3. Unbalanced force, N, and equivalent force, $\tilde{N}$.

	Extreme Environment w/thermal kip/in.	Extreme Environment mech. load kip/in.	Abnormal/Extreme Environment w/thermal kip/in.
N_h	-6.48	-3.48	-28.4
N_h'	-0.28	-0.28	5.1
N	-6.76	-3.72	-23.3
$\tilde{N}$	-9.26	-5.15	-31.9

STEP 3: Compute anchor movement and force.

The analysis given in Appendix A demonstrated that the liner system model can be replaced by the 3-spring system shown in Fig. A.5. For this system, according to Eq. (A-14) the equivalent force, $\tilde{N}$, is

$$\tilde{N} = (1 + D)N \qquad \text{(A-14)}$$

where

$$D = a_1 + a_1 a_2 + \ldots. \qquad \text{(A-12)}$$

Using

K_C (linear) = 270 kip/in./in.
K_{BP} (linear) = 130 kip/in./in.
K_{FP} = 500 kip/in./in.

Eq. (A-12) also gave

$a_1 = 0.280$, $K'_2 = 222$ kip/in./in.
$a_2 = 0.248$, $K'_3 = 248$ kip/in./in.
$a_3 = 0.240$, $K'_4 = 254$ kip/in./in.
$a_4 = 0.238$,

Thus,

$$D = 0.280 + 0.069 + 0.017 + 0.004 + \ldots = 0.370$$

and

$$\tilde{N} = 1.37N$$

The value of $\tilde{N}$ is also listed in Table 4.3.

To compute the anchor movement, first try a linear solution.

$$\delta\ (\text{linear}) = \frac{\tilde{N}}{K_{FP} + K_C(\text{linear}) + K_{BP}(\text{linear})}$$

$$= \tilde{N}/900$$

When the linear solution for the anchor movement exceeds the elastic limit of K_C or K_{BP}, a nonlinear solution becomes necessary. This can be done by trial and error until equilibrium is reached. The results are shown in Table 4.4. For the Extreme Environmental condition the anchor shear force, V, is also computed. V is caused by the mechanical loads.

Table 4.4. Computed results vs. ASME code allowables.

		Extreme Environment		Abnormal/Extreme Environment
		w/thermal	mech. load	w/thermal
Liner	$e_{max.}$	-610 μ	N/A	-2691 μ
Plate	$e_{allow.}$	-2000 μ		-5000 μ
Liner	δ	0.0103 in.	N/A	0.0516 in.
Anchor	$\delta_{allow.}$	0.0350 in. (= $\delta_u/4$)		0.0700 in.(= $\delta_u/2$)
	V	N/A	1.54 kip/in.	N/A
	$V_{allow.}$		2.22 kip/in. (=$V_u/3$)	

<u>STEP 4</u>: Evaluate the liner plate and anchor.

For the liner plate, the calculated membrane strain was compared with the allowables specified in Division 2 of ASME Boiler & Pressure Vessel Code, Section III, Subsection CC, Article CC-3720. The liner anchor was evaluated against the allowable shear force (under mechanical loads only) and the displacement specified in Article CC-3730, the subsection of the ASME code given above. Both the allowable anchor force and displacement are specified as a fraction of test-determined ultimate capacity.

The test results for the case of no gap between the liner plate and concrete are tabulated in Figs. 5 through 19 of the FSAR. The minimum ultimate load is shown to be V_u = 6.67 kip/in. The ultimate displacement is δ_u = 0.14 in.

The analysis results are compared with the applicable allowables in Table 4.4. All computed results are within the code allowables, and it may be stipulated that the liner system possesses a sufficient margin of capacity under both the Extreme and Abnormal/Extreme Environmental conditions.

APPENDIX

PROCEDURES FOR CALCULATING LINER MEMBRANE STRAIN AND ANCHOR MOVEMENT

A.1 INTRODUCTION

In the Palisades Plant Unit 1 containment building, the liner plate is typically 1/4-inch thick and liner anchors in the cylindrical wall are typically L3x2x1/4 steel angles installed 15 in. apart in the hoop direction (Fig. A.1). For the purpose of analysis, all liner panels except one are assumed to be flat plates. The exception liner panel is assumed to have initial inward curvature. The maximum initial inward deflection at the center of the panel is assumed to be 1/8 in. (Ref. 4). The physical model thus described is illustrated in Fig. A.2(a), which represents a 1-inch-wide strip of the liner system. Analysis results based on this one-way physical model will be conservative because the benefit of the bi-axial stiffening of the plate is not taken into account. The corresponding analysis model may be represented by the spring system illustrated in Fig. A.2(b). The spring properties and the analysis method are based on Ref. 9, with some minor modifications to the analysis procedure. The stiffness properties of the anchor and the concrete spring, K_C, and of the bent plate spring, K_{BP}, were established from test data.[9] These stiffness properties are applicable to the present study because the materials and configurations of the bent plate and liner anchor test models are similar to those used in the construction of the Palisades containment liner system.

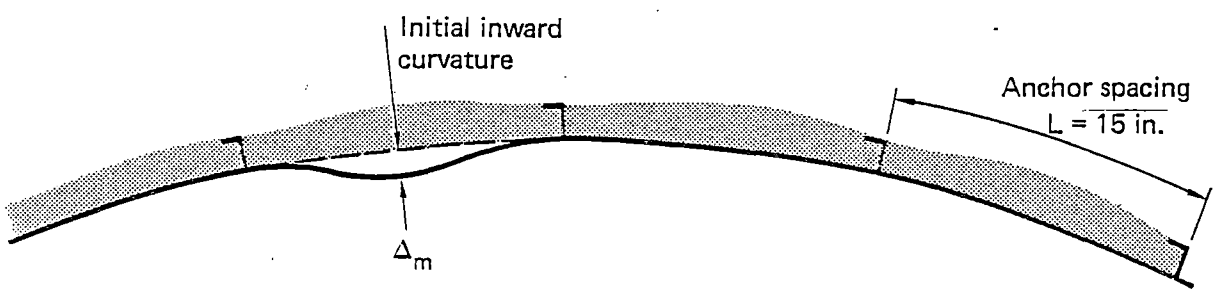


Fig. A.1. Circumferential section of the cylinder liner with one bent panel.

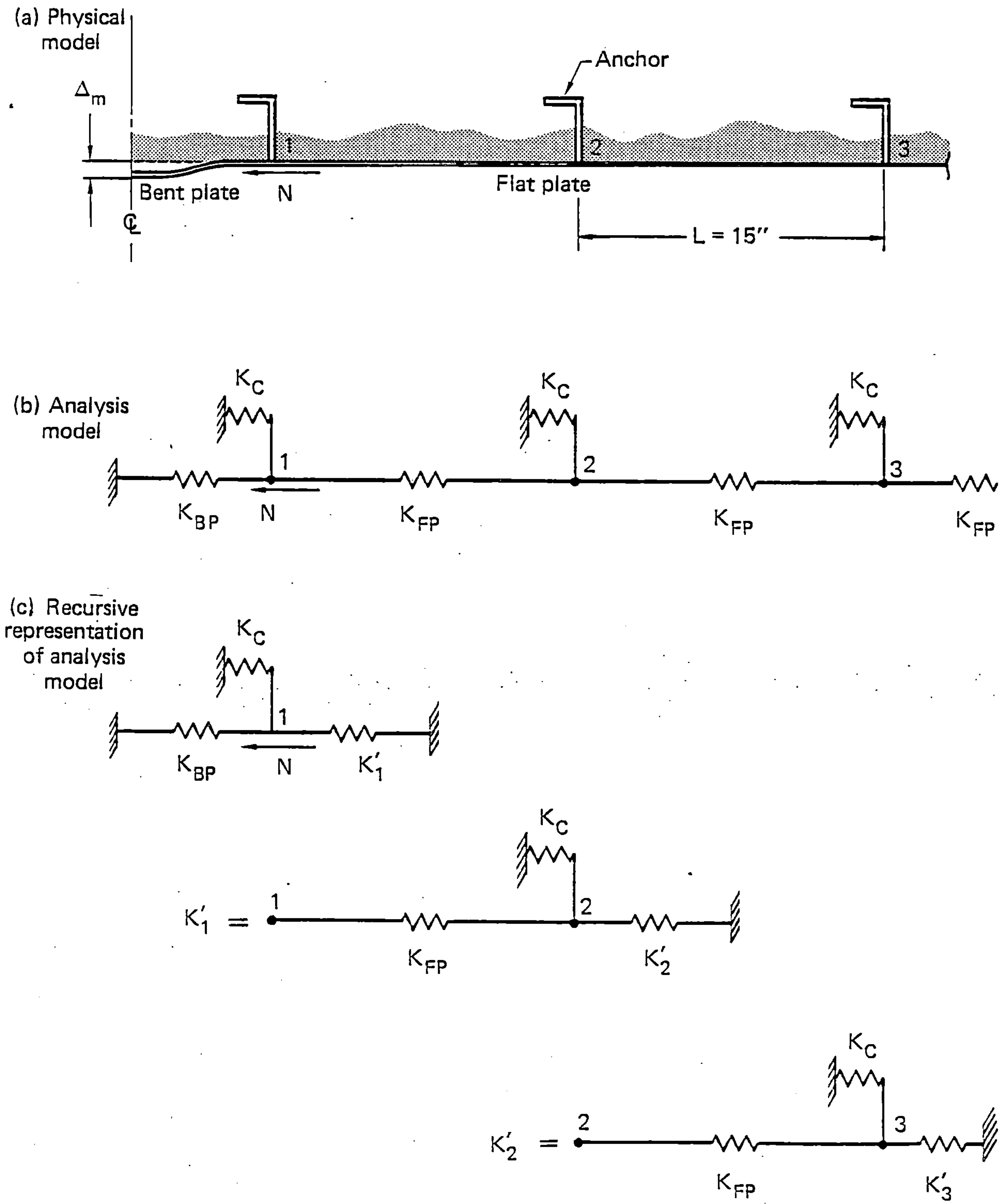


Fig. A.2. Analysis model of the liner system at the cylinder base.

A.2 ANALYSIS MODEL

As shown in Fig. A.2(b), the analysis model is composed of three types of springs. K_c represents the shear resistance of the liner anchor in the concrete, K_{BP} represents the in-plane stiffness of the bent plate, and K_{FP} represents the in-plane stiffness of a flat plate panel. The assumed initial inward curvature of the bent plate results in an in-plane unbalanced force, N, at anchor point No. 1: this results from the differential strains between the liner and concrete and to the pressure acting on the bent plate. This force generates tangential movements of all the anchors toward the bent plate panel. The anchor movement will maximize at anchor point No. 1 and diminish rapidly as the distance from anchor point No. 1 increases.

The spring stiffness properties are described below:

(a) K_{FP}, The in-plane stiffness of a flat plate is

$$K_{FP} = AE_s/L \tag{A-1}$$

A = section area of the 1-inch wide liner plate strip (0.25 in. x 1 in.)

E_s = Young's modulus of liner (30,000 ksi)

L = hoop direction spacing of anchors (15 in. typical)

Hence,

$$K_{FP} = 500 \text{ kip/in./in.} \tag{A-2}$$

(b) K_C, The tangential shear resistance capacity of the L3x2x1/4 angle embedded in concrete was established from tests.[9] The idealized K_C, corresponding to a concrete having $E_c = 5.4 \times 10^3$ ksi, is reproduced in Fig. A.3.

(c) K_{BP}, The in-plane stiffness of the bent plate was also adopted from Ref. 9. Figure A.4 illustrates the idealized K_{BP} corresponding to liner material having a minimum yield stress of 32 ksi.

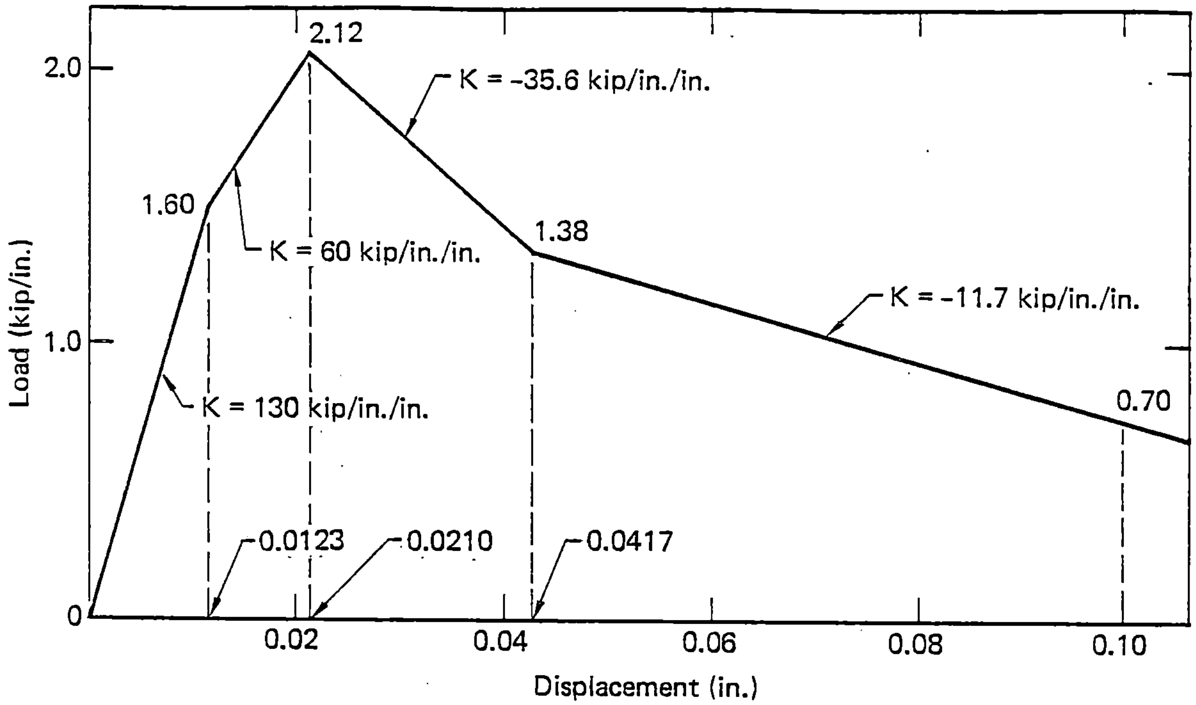


Fig. A.3. Load vs. displacement curve, K_{BP}, for the bent plate.

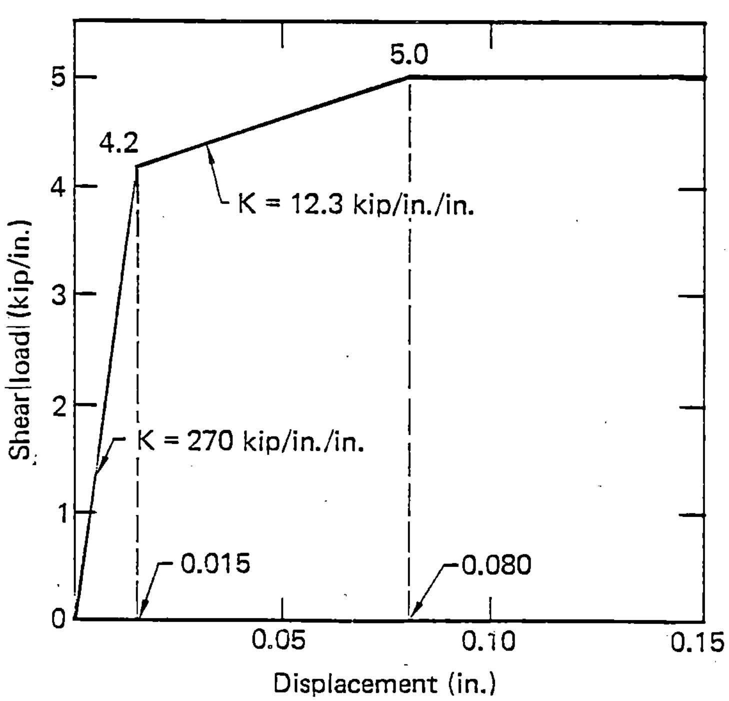


Fig. A.4. Load-displacement curve, K_C, for the liner anchor.

A.3 LINER STRAINS AND UNBALANCED FORCE

The force, N, to be applied at anchor point No. 1 is composed of two parts. The first is N_n. This is due to the differential strain between the liner and concrete that arises from the applicable mechanical and thermal loads. The second part is N_n', which is due to pressure directly acting on the bent plate. The methods used to compute N_n and N_n' are discussed below.

(a) N_n' Maximum concrete stresses or strains due to dead load and seismic load on the containment wall occurs near the cyliner-to-base junction. First, therefore, compute the meridional and hoop concrete strains near the base junction for the following loads: dead load, seismic load, effective prestress load, and pressure load. Strain due to inital concrete shrinkage was assumed to be -100 μ ($\mu = 10^{-6}$ in./in.).

For the mechanical loads, the concrete strains also represented the differential strains between liner and concrete that were imposed upon the liner by way of the anchors. For thermal loads, the differential strains imposed on the liner were conservatively calculated as follows:

Extreme Environment: $$e_h = e_z = 6.5\,\mu\left[T_i - (T_i + T_o)/2\right]$$
$$= 6.5\,\mu\,(T_i - T_o)/2 \qquad \text{(A-3)}$$

Abnormal/Extreme: $$e_h = e_z = 6.5\,\mu\left[T_a - (T_i + T_o)/2\right] \qquad \text{(A-4)}$$

e_h = differential liner-concrete strain in hoop direction.
e_z = differential liner-concrete strain in meridional direction.
T_i = ambient temperature inside containment.
T_o = temperature outside containment.
T_a = peak temperature on liner surface for abnormal condition.

The above expressions are conservative. They are based on the following simplifications:

- Thermal expansion coefficients for both liner and concrete were 6.5×10^{-6} in./in./^oF.
- The concrete did not crack and the concrete wall was restrained from rotation.

- Under abnormal conditions, the liner was assumed to be instantaneously heated to T_a, while the temperature gradient in the concrete wall still remained $(T_i - T_o)$.

The membrane force N_h can then be determined:

$$N_h = \frac{tE_s(e_h + \nu e_z)}{1-\nu^2} \qquad \text{(A-5)}$$

where t is the thickness, E_s is Young's modulus, and ν is Poisson's ratio of the liner plate.

(b) N_h' The membrane reaction-force at both edges of the bent plate, when subjected to a normal pressure acting directly on the plate, may be approximately computed as follows.[9]

$$N_h' = PL^2/2\pi^2\Delta_m \qquad \text{(A-6)}$$

in which

P = pressure
L = plate span (15 in.)
Δ_m = initial inward deflection at center of the bent plate (1/8 in.)

A.4 ANCHOR MOVEMENT

To derive the anchor movement of the first anchor when the analysis model is subjected to the unbalanced membrane force of $N = N_h + N_h'$, K_{BP} and all K_C were first assumed to be linear. Letting K_1 represent the effect of all flat plates and anchors other than anchor No. 1, the analysis model became that shown at the top of Fig. A.2(c). From this model,

$$\begin{aligned}\delta_1 &= \frac{N}{K_{BP} + K_C + K_1'} \\ &= \frac{N}{K_{BP} + K_C + K_{FP}} \left(\frac{K_{BP} + K_C + K_{FP}}{K_{BP} + K_C + K_1'}\right) \\ &= \frac{N}{K_{BP} + K_C + K_{FP}} (1 + D) \qquad \text{(A-7)}\end{aligned}$$

where

$$D = \frac{K_{FP} - K_1'}{K_{BP} + K_C + K_{FP}} \quad .$$

Because K_1' is related to K_C, K_{FP} and a spring, K_2', as shown in Fig. A.2(c), i.e.,

$$K_1' = \frac{1}{\dfrac{1}{K_{FP}} + \dfrac{1}{K_C + K_2'}} = \frac{K_{FP}(K_C + K_2')}{K_{FP} + K_C + K_2'} \tag{A.8}$$

we have

$$D = \frac{K_{FP}^2}{K_{FP}K_C + (K_{BP} + K_C)(K_{FP} + K_C) + K_2'(K_{FP} + K_C + K_{BP})} \tag{A-9}$$

Similarly, according to Fig. A.2(c), K_2' is related to K_{FP}, K_C and a certain K_3' as in Eq. (A-8), with K_2' and K_3' replacing K_1' and K_2', respectively. It can then be shown that

$$D = \frac{K_{FP}^2}{(K_C + 2K_{FP})(K_C + K_{BP} + K_{FP}) - K_{FP}^2} \times \left[1 + \frac{K_{FP}^2}{K_{FP}K_C + (K_{BP}' + K_C)(K_{FP} + K_C) + K_3'(K_{FP} + K_{BP}' + K_C)}\right] \tag{A-10}$$

where

$$K_{BP}' = \frac{K_{FP}(K_C + K_{BP})}{K_{FP} + K_C + K_{BP}} \quad . \tag{A-11}$$

Based on Eq. (A-10) and Eq. (A-9), a recursive relationship can be established

$$D = a_1(1 + a_2(1 + a_3(1 + \ldots))) = \sum_{i=1} a_1 a_2 \ldots a_i$$

$$a_i = \frac{K_{FP}^2}{(K_C + 2K_{FP})(K_{FP} + K_C + K_i') - K_{FP}^2}$$

$$K_1' = K_{BP} \qquad \text{(A-12)}$$

$$K_i' = \frac{K_{FP}(K_C + K_{i-1}')}{K_{FP} + K_C + K_{i-1}'} \qquad (i = 2, 3, \ldots)$$

Equation (A.7) now becomes

$$\delta_1 = \frac{\tilde{N}}{K_{FP} + K_C + K_{BP}} \qquad \text{(A-13)}$$

where

$$\tilde{N} = N(1 + D) \,. \qquad \text{(A-14)}$$

The problem is thus reduced to analyzing the equivalent 3-spring system (shown in Fig. A.5) when subjected to the equivalent force $\tilde{N}$. The actual nonlinearity in K_C and K_{BP} can now be taken into account, depending on the magnitude of $\tilde{N}$.

It is advisable to first try a linear solution for δ_1. If the resultant value of δ_1 exceeds the elastic limit of K_{BP} or K_C or both, a nonlinear solution becomes necessary. This can be accomplished by trial and error until a force equilibrium is reached in the solution.

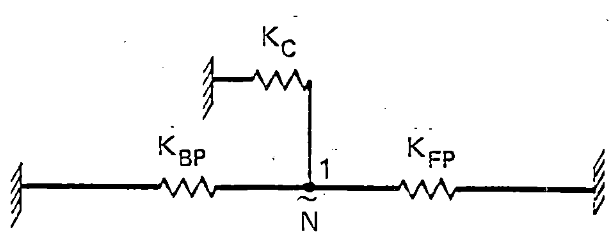


Fig. A.5. The 3-spring equivalent analysis model.

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