TECHNICAL EVALUATION REPORT

DESIGN CODES, DESIGN CRITERIA, AND LOADING COMBINATIONS (SEP, III-7.B)

CONSUMERS POWER COMPANY

BIG ROCK POINT PLANT

NRC DOCKET NO. 50-155

NRC TAC NO. 41495

NRC CONTRACT NO. NRC-03-79-118

FRC PROJECT C5257

FRC ASSIGNMENT 11

FRC TASK 317

Prepared by

Franklin Research Center
20th and Race Street
Philadelphia, PA 19103

FRC Group Leader: T. Stilwell

Prepared for

Nuclear Regulatory Commission
Washington, D.C. 20555

Lead NRC Engineer: D. Persinko

September 13, 1982

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CONTENTS

Section	Title	Page
1	INTRODUCTION	1
2	BACKGROUND	2
3	REVIEW OBJECTIVES	3
4	SCOPE	4
5	MARGINS OF SAFETY	7
6	CHOICE OF REVIEW APPROACH	9
7	METHOD	11
	7.1 Information Retrieval	11
	7.2 Appraisal of Information Content	11
	7.3 Code Comparison Reviews	12
	7.4 Assessment of the Potential Impact of Code Changes	15
	7.4.1 Classification of Code Changes	16
	7.4.1.1 General and Conditional Classifications of Code Change Impacts	17
	7.4.1.2 Code Impacts on Structural Margins	18
	7.5 Plant-Specific Code Changes	20
8	BIG ROCK POINT SEISMIC CATEGORY I STRUCTURES	21
9	STRUCTURAL DESIGN CRITERIA	22
10	LOADS AND LOAD COMBINATION CRITERIA	24
	10.1 Description of Tables of Loads and Load Combinations	24

CONTENTS (Cont.)

Section	Title	Page
	10.2 Load Definitions	28
	10.3 Design Load Tables, "Comparison of Design Basis Loads"	30
	10.4 Load Combination Tables, "Comparison of Load Combination Criteria"	39
11	REVIEW FINDINGS	49
	11.1 Major Findings of AISC-1953 vs. AISC-1980 Code Comparison.	51
	11.2 Major Findings of ACI 318-56 vs. ACI 349-76 Code Comparison	56
	11.3 Major Findings of ASME B&PV Code Comparison, Section VIII, 1956 vs. Section III, Subsection NE, Class MC, 1980 Code	62
12	SUMMARY	67
13	RECOMMENDATIONS	69
14	REFERENCES	74

APPENDIX A - SCALE A AND SCALE A_x CHANGES DEEMED INAPPROPRIATE TO BIG ROCK POINT PLANT

APPENDIX B - SUMMARIES OF CODE COMPARISON FINDINGS

APPENDIX C - COMPARATIVE EVALUATIONS AND MODEL STUDIES

APPENDIX D - ACI CODE PHILOSOPHIES

APPENDIX I - CODE COMPARISON REVIEW OF TECHNICAL DESIGN BASIS DOCUMENTS DEFINING CURRENT LICENSING CRITERIA FOR SEP TOPIC III-7.B (SEPARATELY BOUND)

CONTENTS (Cont.)

Section	Title
APPENDIX II	- CODE COMPARISON REVIEW OF AISC SPECIFICATION FOR THE DESIGN, FABRICATION, AND ERECTION OF STRUCTURAL STEEL FOR BUILDINGS FOR THE YEARS 1980 VS. 1963 (SEPARATELY BOUND)
APPENDIX III	- NOT APPLICABLE TO OYSTER CREEK PLANT
APPENDIX IV	- CODE COMPARISON REVIEW OF CODE REQUIREMENTS FOR NUCLEAR SAFETY-RELATED CONCRETE STRUCTURES ACI 349-76 VS. BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE ACI 318-63 (SEPARATELY BOUND)
APPENDIX V	- NOT APPLICABLE TO BIG ROCK PLANT
APPENDIX VI	- NOT APPLICABLE TO BIG ROCK PLANT
APPENDIX VII	- CODE COMPARISON REVIEW OF AISC SPECIFICATION FOR THE DESIGN, FABRICATION, AND ERECTION OF STRUCTURAL STEEL FOR BUILDINGS FOR THE YEARS 1963 VS. 1953 (SEPARATELY BOUND)
APPENDIX VIII	- NOT APPLICABLE TO BIG ROCK PLANT
APPENDIX IX	- CODE COMPARISON REVIEW OF CODE REQUIREMENTS FOR ASME B&PV CODE SECTION III, SUBSECTION NE, 1980 VS. ASME B&PV CODE SECTION VIII, 1962 (SEPARATELY BOUND)
APPENDIX X	- NOT APPLICABLE TO BIG ROCK PLANT
APPENDIX XI	- CODE COMPARISON REVIEW OF CODE REQUIREMENTS FOR NUCLEAR SAFETY-RELATED CONCRETE STRUCTURES ACI 318-63 VS. BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE ACI 318-56 (SEPARATELY BOUND)
APPENDIX XII	- CODE COMPARISON REVIEW OF CODE REQUIREMENTS FOR ASME B&PV CODE SECTION VIII, 1962 VS. ASME B&PV CODE SECTION VIII, 1956 (SEPARATELY BOUND)

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.

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Dr. E. W. Wallo, Chairman of the Civil Engineering Department, Villanova University, and Dr. R. Koliner, Professor of Civil Engineering, Villanova University, provided assistance both as contributing authors and in an advisory capacity as consultants under subcontract with the Franklin Research Center.

The report also incorporates the suggestions, guidance, and supportive efforts provided by Mr. D. Persinko, the NRC Lead Engineer for this task.

1. INTRODUCTION

For the Seismic Category I buildings and structures at the Big Rock Point Nuclear Power Station, this report provides a comparison of the structural design codes and loading criteria used in the actual plant design against the corresponding codes and criteria currently used for 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 Big Rock Point 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 involving 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 the 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 this report is to present the results of Task III-7.B as they relate to the Big Rock Point Nuclear Power Station.

4. SCOPE

In general, the scope of work requires comparison of the provisions of the structural codes and standards used for the design of SEP plant Seismic Category I civil engineering structures* against 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.

The review scope consists of the following specific tasks:

1. Identify current design requirements, based on a review of NRC Regulations; 10CFR50.55a, "Codes and Standards"; 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 Seismic 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

*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 the scope of this review).

c. results of any comparative stress analyses performed in order to assess the significance of the code changes on safety margins

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.A	Effects of High Water Level on Structures
III-4	Missile Generation and Protection
III-5	Evaluation of Pipe Breaks
III-6	Seismic Design Considerations
III-7.D	Structural Integrity Tests
VI-2	Mass and Energy Release for Postulated Pipe Break.

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

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 generically 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 the review only to the extent that it affects design criteria and 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 within scope.
Shield walls and subcompartments inside containment	Reviewed in Generic Task A-2.
Masonry walls	Reviewed generically in IE Bulletin 80-11.
Seismic analysis	Being reviewed as an independent SEP Topic.

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.

Because the yield strengths of common structural steels are generally well below their ultimate strengths, the engineer knows that in most (but not 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 systematic evaluation 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 on which key questions (of Section 5) 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. The initial step in this approach is not easy, nor are the necessary evaluations. One is dealing with highly loaded structures in regions where materials behave inelastically. Rulemaking 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 unresolved 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 will be 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 the approach 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 outset, NRC forwarded files relevant to the work. These submittals included pertinent sections of plant FSARs, Standard Review Plan (SRP) 3.8, responses to questions on Topic III-7.B previously requested of licensees by the NRC, and other relevant data and reports.

These submittals were organized into Topic III-7.B files on a plant-by-plant basis. The files also contain subsequently received information, as well as 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 items were not referenced at all or 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 (NRC's Standard Review Plan), the operative document providing guidance to NRC reviewers on licensing matters (see Reference 1).

Next, the Seismic Category I structures at the Big Rock Point Nuclear Power Station were identified (see Section 8). For these, the codes and standards which were used for actual design were likewise identified on a structure-by-structure basis (see Section 9). Each code was then paired with its counterpart which 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 were entered. Typical comments address either the reason the change had been introduced, the intent of the change, 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 overly 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.

Whenever it was found that code requirements had truly been relaxed, this was noted in the reviewer's comments in the code comparison review. 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 as a reviewer comment. 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 flagged for further consideration.

Sometimes the effects of a code change are not apparent. 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 in the review, 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, a typical structural element and a simple loading were selected; the element was then designed to the older code requirements. Next, the load carrying capacity of this structure was reexamined using current code criteria. Finally, the load carrying capacities of the element, as shown by the older criteria and as determined by the current criteria, were compared. Examples of investigations performed to assess code change impacts are found in Appendix C.

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

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 indicates, a limited objective is sought in assessing the effects of code changes on Seismic Category I structures.

The scope of this review is not set at the level of appraisal of individual, as-built structures on plant sites. Consequently, 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 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.

   A 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 structural integrity 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 could potentially impair perceived margins of safety, the following scheme classifying code change impacts was adopted.

7.4.1 Classification of Code Changes

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 are designated as "Scale A," and others as "Scale C." Others 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 thereby downgrade the ranking to, say, a Scale B change for that member. The

*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?

scale ranking is neither 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 into the code and is framed in terms of the ratio of the effective column length to its 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 the 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 Impact 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) and (b) changes that have the potential to enhance perceived margins of safety (Scale C).

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, it may or may not pertain.

*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 which might impair perceived margins of safety, and that assessment of their pertinence 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. Several plants, for example, followed the provisions of ACI-318, 1963 edition, in designing major concrete structures.

Thus, the initial work of 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, while 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, such activities were not attempted. However, occasional reference to such documents was necessary to the review work. Consequently, it was possible to cull from the list some items that were obviously inappropriate to the Big Rock Point 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 may be significant for structures in general but did not appear applicable to any of the Seismic Category I structures at the Big Rock Point plant 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. BIG ROCK POINT SEISMIC CATEGORY I STRUCTURES

The objective of SEP Topic III-1 is the classification of components, structures, and systems with respect to both quality group and seismic designation. Based upon the review of the Big Rock Point FSAR [5] and Bechtel Corporation drawings [6] showing the location of Seismic Category I equipment, the present report considers the following to be Seismic Category I structures:

Spherical Containment Vessel

Internal Structures

- o support for reactor enclosure plenum
- o fuel pit

External Structures

- o water intake structure
- o control room
- o waste storage vaults
- o structures housing liquid radwaste
- o stack
- o diesel generator enclosure/screen well and pump house
- o battery rooms.

According to Reference 7, the stack is a Seismic Category II structure. It may be appropriate, however, to include the stack in this report as a Seismic Category I structure based on its proximity to other Seismic Category I equipment and structures. The turbine building, except for the control room, is considered a Seismic Category II structure. The waste storage vaults and structures housing liquid radwaste are included above and in Section 9 for information only and are not considered further in this report.

9. STRUCTURAL DESIGN CRITERIA

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

Structure	Design Criteria	Current Criteria
1. Spherical Containment Vessel	ASME B&PV Code Sect. VIII, 1956	ASME B&PV Code, Sect. III Div. I Subsect. NE, Class MC Components, 1980
Internal Structures		
2. Support for reactor enclosure plenum	ACI 318-56	ACI 349-76
3. Fuel pit	ACI 318-56	ACI 349-76
External Structures		
4. Water intake structure	AISC 1953; ACI 318-56	AISC 1980; ACI 349-76
5. Control room	AISC 1953; ACI 318-56	AISC 1980; ACI 349-76
6. Waste storage vaults	AISC 1953; ACI 318-56	AISC 1980; ACI 349-76
7. Structures housing liquid radwaste	ACI 318-56	ACI 349-76
8. Stack*	Design criteria not stated	ACI 307-79
9. Diesel generator enclosure/ screen well and pump house	Design criteria not stated	AISC 1980; ACI 349-76
10. Battery rooms	AISC 1953; ACI 318-56	AISC 1980; ACI 349-76

* Although the provisions of ACI-349 currently govern design of all Seismic Category I structures external to containment, nonconflicting provisions of ACI-307 also apply. Comparisons of these design codes with previous versions of ACI chimney codes are not carried out in this report since a complete reanalysis of the stack to current criteria will be carried out elsewhere within the SEP program.

**FSAR references UBC 1958. This, in turn, invokes provisions of the then current editions of the ACI and AISC Codes.

The reference identifying major codes used for original design is "Seismic Design Bases and Criteria for Big Rock Point Nuclear Generating Station" by Engineering Decision Analysis Company, Jan. 1979.

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 this section of this report.

The NRC Regulatory Guides and Standard Review Plans provide guidance as to 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, 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 (as enumerated in NRC's Standard Review Plan), 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.

Current criteria require consideration during plant design of 13 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 safe shutdown earthquake 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.

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.

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 consistent basis for the tables be maintained, they are based upon load combinations 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 that were not included in the original design or that have increased in magnitude and have not been specifically addressed in another SEP topic should be addressed by the Licensee.

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.)

L Live loads or their related internal moments and forces (such as movable equipment loads).

P_a Pressure load generated by accident conditions (such as those generated by the postulated pipe break accident).

P_o or P_v Loads resulting from pressure due to normal operating conditions.

*See, for example, SRP 3.8.2.

P_s All pressure loads which are caused by the actuation of safety relief valve discharge including pool swell and subsequent hydrodynamic loads.

R_a or R_r Pipe reactions under accident conditions (such as those generated by thermal transients associated with an accident).

R_o Pipe reactions during startup, normal operating, or shutdown conditions, based on the critical transient or steady-state condition.

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_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.

Y_r Equivalent static load on the structure generated by the reaction on the broken pipe during the design basis accident.

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:
SPHERICAL CONTAINMENT VESSEL

COMPARISON OF DESIGN BASIS LOADS

PLANT: BIG ROCK POINT

	Current Design Basis Loads	Is Load Applicable To This Structure?	Is Load Included In Plant Design Basis?	SEP Topic Reviewing This Load	Does Load Magnitude Correspond To Present Criteria?	Does Deviation Exist In Load Basis?	Code Impact Scale Ranking	Comments
Gravity	D	Yes	Yes	——	——	No	——	
	L	Yes	Yes	——	——	No	A_x	1.
Pressure	F	No	——	——	——	——	——	
	H	Yes	No	III-5.A	*	*	*	
	P_o	Yes	Yes	——	——	No	——	
	P_a	Yes	Yes	VI-2.D, III-7.B	*	*	*	2.
	P_s	Yes	No	——	——	Yes	A_x	
Thermal	T_o	Yes	Negl	——	——	——	——	
	T_a	Yes	Yes	VI-2.D, III-7.B	*	*	*	3.
	T_S	Yes	No	——	——	Yes	——	
Pipe & Mech.	R_o	Yes	No	——	——	Yes	——	
	R_a	Yes	No	——	——	Yes	A_x	
	R_S	Yes	No	——	——	Yes	A_x	
Environmental	E'	Yes	Yes	III-6	*	*	A_x	4., 5.
	E	Yes	No	III-6	*	*	*	4.
	W'	Yes	No	III-2, III-4.A	*	*	A_x	
	W	Yes	Yes	III-2, III-4.A	*	*	*	6.
Impulse	Y_r	Yes	No	III-5.A	*	*	*	
	Y_j	Yes	No	III-5.A	*	*	*	
	Y_m	Yes	No	III-5.A	*	*	*	

Ref.; SRP(1981) Section 3.8.1 or 3.8.2

Comments

* To be determined per results of SEP topics. Scale ranking shown for SEP topic items are independent judgments, based on information in the FSAR or other original design documents.

1. Snow loads have increased per topic II-2.A.
2. FSAR states that containment is designed for internal pressure resulting from worst accident. Design internal pressure is 27 psig (Ref. FSAR 3.1.2).
3. Design max. temp. is 235°F (Ref: FSAR 3.2.2).
4. According to NRC's letter to C.P. 5-19-81, a .05g (static) seismic lateral load was used. Current requirements call for dynamic analysis for containment structures.
5. Presently a 0.12g SSE is deemed appropriate for this structure.
6. Design wind load used is 100 mph., per reference 9.

COMPARISON OF DESIGN BASIS LOADS

STRUCTURE: SUPPORT FOR REACTOR ENCLOSURE PLENUM

PLANT: BIG ROCK POINT

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

Ref.; SRP(1981) Section 3.8.4

Comments

* To be determined per results of SEP topics. Scale ranking shown for SEP topic items are independent judgments, based on information in the FSAR or other original design documents.

1. FSAR information insufficient to evaluate these items.
2. According to NRC's letter to C.P., 5-19-81, a .05g (static) seismic lateral load was used. Current requirements call for dynamic analysis.
3. Presently a 0.12g SSE is deemed appropriate for this structure.

COMPARISON OF DESIGN BASIS LOADS

STRUCTURE:

FUEL POOL

PLANT: BIG ROCK POINT PLANT 1

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

Ref.; SRP(1981) Section 3.8.4

Comments

* To be determined per results of SEP topics. Scale ranking shown for SEP topic items are independent judgments, based on information in the FSAR or other original design documents.

1. According to NRC's letter to CP, 5-19-81, a static analysis was used with .05g as seismic lateral load. Current requirements call for dynamic analysis.

2. Presently a 0.12g SSE is deemed appropriate for this structure.

STRUCTURE:
INTAKE STRUCTURE

COMPARISON OF DESIGN BASIS LOADS

PLANT: BIG ROCK POINT

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

Ref.; SRP(1981) Section 3.8.4

Comments

* To be determined per results of SEP topics. Scale ranking shown for SEP topic items are independent judgments, based on information in the FSAR or other original design documents.

1. Static analysis was used with .025g as lateral seismic load. Current requirements call for dynamic analysis (See Section 3.3 Reference 9).

2. Presently a 0.12g SSE is deemed appropriate for this structure.

COMPARISON OF DESIGN BASIS LOADS

STRUCTURE: CONTROL ROOM (SERVICE BUILDING)

PLANT: BIG ROCK POINT

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

Ref.; SRP(1981) Section 3.8.4

Comments

* To be determined per results of SEP topics. Scale ranking shown for SEP topic items are independent judgments, based on information in the FSAR or other original design documents.

1. Not a major structural concern, but might affect control room habitability.
2. Static analysis was used, with .025g as seismic laterial load. Current requirements call for dynamic analysis.
3. Roof loads have increased per SEP Topic II-2A and may increase per SEP Topic II-3B for parapet roofs.
4. For the turbine building, D'Apalonia Consulting Engineering, Inc. reported this loading is insignificant.

COMPARISON OF DESIGN BASIS LOADS

STRUCTURE: STACK (Concrete)

PLANT: BIG ROCK POINT

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

Ref.; SRP(1981) Section 3.8.4

Comments

* To be determined per results of SEP topics. Scale ranking shown for SEP topic items are independent judgments, based on information in the FSAR or other original design documents.

1. Static analysis was used, with .025g as seismic lateral load. Current requirements call for dynamic analysis.

COMPARISON OF DESIGN BASIS LOADS 1.

STRUCTURE:
DIESEL GENERATOR ENCLOSURE/
SCREEN WELL AND PUMP HOUSE

PLANT: BIG ROCK POINT

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

Ref.; SRP(1981) Section 3.8.4

Comments

* To be determined per results of SEP topics. Scale ranking shown for SEP topic items are independent judgments, based on information in the FSAR or other original design documents.

1. Information on original design basis is not stated in material provided for FRC review.

COMPARISON OF DESIGN BASIS LOADS

STRUCTURE:
BATTERY ROOM (TURBINE BUILDING)

PLANT: BIG ROCK POINT

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

Ref.; SRP(1981) Section 3.8.4

Comments

* To be determined per results of SEP topics. Scale ranking shown for SEP topic items are independent judgments, based on information in the FSAR or other original design documents.

1. Static analysis was used, with .025g as seismic lateral load. Current requirements call for dynamic analysis.

2. Report reference 9 states that snow, wind, & seismic loads were considered. No values are given.

10.4 LOAD COMBINATION TABLES

"COMPARISON OF LOADING COMBINATION CRITERIA"

COMPARISON OF LOADING COMBINATION CRITERIA

PLANT: BIG ROCK POINT

STRUCTURE SPHERICAL CONTAINMENT VESSEL

	Combined Loading Cases	Gravity Dead, Live	Thermal	Pressure	Mechanical	Natural Phenomena	Impulsive Loading	Scale Ranking	
Service Level A	1	D + L	T_o	P_o	R_o				2.
	2	D + L	T_s	P_s	R_s				
	3	D + L	T_a	P_a	R_a				3.
	4	D + L	$T_a + T_s$	$P_a + P_s$	$R_a + R_s$				
Service Level B	1	D + L	T_a (3)	P_a	R_a	E W			4.
	2	D + L	T_o	P_o	R_o	E			5., 6.
	3	D + L	T_s	P_s	R_s	E			
	4	D + L	$T_a + T_s$	$P_a + P_s$	$R_a + R_s$				
Service Level C	1	D + L	T_a	P_a	R_a	~~E'~~ W_T		A_x	7.
	2	D + L	T_o	P_o	R_o	E'			
	3	D + L	$T_a + T_s$	$P_a + P_s$	$R_a + R_s$	E'			
Service Level D	1	D + L	T_a	P_a	R_a	E'	$Y_r+Y_j+Y_m$		
	2	D + L	$T_a + T_s$	$P_a + P_s$	$R_a + R_s$	E'	$Y_r+Y_j+Y_m$	A_x	7.
Post - LOCA Flooding	1	D + L		F_L		E			

Ref.: SRP Section 3.8.2 Steel Containment

Notes

1. Encircled loads are those actually considered in the design per FSAR. When load factors different from those currently required were used, the factor used is also encircled.
2. 0.5 psi external pressure plus dead load considered, Reference 9.
3. Maximum temperature gradient plus 27 psi internal pressure considered, Reference 9.
4. Snow load plus 60 mph wind considered, Reference 9.
5. Dead load plus snow plus 0.05g seismic considered, Reference 9.
6. Credited here as an OBE Combination, because the 0.05g static load used is approximately half the value currently deemed appropriate for the SSE (i.e. 0.12g).
7. For purposes of the SEP Review, demonstration that structural integrity is maintained for load cases indicated above (per current criteria) may be considered as providing reasonable assurance that this structure meets the intent of current design criteria.

COMPARISON OF STRESS LIMITS

FOR

STEEL CONTAINMENT STRUCTURES

PLANT: BIG ROCK POINT

SERVICE LEVEL	CURRENT CRITERIA (REF.: TABLE NE - 3221-1, ASME SECTION III, 1980)		DESIGN CRITERIA (REF.: ASME B&PV CODE SECTION VIII, 1956)	
	CRITERIA	VALUE, psi	CRITERIA	VALUE, psi
A	P_m 1.0 S_{mc} P_L 1.5 S_{mc} $P_L + P_b$ 1.5 S_{mc} $P_L + P_b + Q$ 3.0 S_{mi} (See note 6)	16,500 24,750 24,750 57,870		
B	P_m 1.0 S_{mc} P_L 1.5 S_{mc} $P_L + P_b$ 1.5 S_{mc} $P_L + P_b + Q$ 3.0 S_{mi} (See note 6)	16,500 24,750 24,750 57,870	$(P_L$ or $P_m) + P_b + Q$ 1.5S (Ref: FSAR Sect. 3.2.2.)	22500
C	P_m 1.2 S_{mc} or 1.0 S_y P_L 1.8 S_{mc} or 1.5 S_y $P_L + P_b$ 1.8 S_{mc} or 1.5 S_y (See notes 3, 4 & 6)	32,000 48,000 48,000		
D	P_m 1.0 S_f P_L 1.5 S_f $P_L + P_b$ 1.5 S_f (See notes 2, 5 & 6)	33,660 50,490 50,490		
POST-FLOODING CONDITION	P_m 1.2 S_{mc} or 1.0 S_y P_L 1.8 S_{mc} or 1.5 S_y $P_L + P_b$ 1.8 S_{mc} or 1.5 S_y $P_L + P_b + Q$ 3.0 S_{mi} (See notes 4 & 6)	32,000 48,000 48,000 57,870		

(SEE NOTE 7) SHELL MATERIAL SPEC. NO. SA-201 GRADE: B TO SA 300 (8)	
YIELD STRESS (S_y) = 32,000 psi ULT. STRENGTH (S_u) = 60,000 psi	
CURRENT PRIMARY STRESS INTENSITY LIMIT	S_{mc} = 16,500 psi S_{mi} = 19,250 psi @ 235 ^oF (See note 1)
DESIGN PRIMARY MEMBRANE STRESS LIMIT	S = 15,000 psi @ 235 ^oF REF: ASME B&PV CODE, VIII, 1962

8. FSAR SECT. 3.2.3 STATES THAT "THE SHELL IS CONSTRUCTED OF SA-201 GRADE B FIREBOX STEEL PRODUCED TO SA-300 SPECIFICATIONS"

NOTES:
1. NOTE THAT CURRENT PRIMARY STRESS INTENSITY LIMITS PRESUME (AMONG OTHER CODE QUALITY CONTROLS) MODERN COMPUTERIZED METHODS OF ANALYSIS. CONSEQUENTLY, CAUTION SHOULD BE OBSERVED IN MAKING DIRECT COMPARISONS WITH DESIGN STRESS LIMITS APPROPRIATE FOR LESS MODERN ANALYTICAL PROCEDURES.
2. THE COMPARABLE CURRENT CRITERIA ASSUMING ELASTIC METHODS WERE USED FOR THE ORIGINAL DESIGN ANALYSIS.
3. VALUES SHOWN PERTAIN TO INTEGRAL AND CONTINUOUS STRUCTURES ONLY.
4. THE LARGER OF THE TWO LIMITS IS APPLICABLE.
5. S_f IS 85% OF THE GENERAL PRIMARY MEMBRANE ALLOWABLE PERMITTED IN APPENDIX F OF SECTION III, ASME CODE.
6. IN ALL INSTANCES FATIGUE AND BUCKLING CRITERIA MUST ALSO BE SATISFIED.
7. IN ACCORDANCE WITH ASME III, DIV 1, SUBSECT. NE, SUBPARA. NE 2121, THIS MATERIAL IS NOT LISTED AMONG THOSE CURRENTLY PERMITTED. REF.: APPENDICES TABLE I-10.1 "CURRENT" STRESS VALUES LISTED ARE DERIVED USING S_{mc} = 1.1 x 1/4 x Su, and S_m @ 235^{o}F FROM TABLE N-421 ASME B&PV CODE SECT III, CLASS A, (1965).

COMPARISON OF LOADING COMBINATION CRITERIA

CONCRETE STRUCTURES

PLANT: BIG ROCK POINT

STRUCTURE: SUPPORT FOR REACTOR ENCLOSURE PLENUM

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

Ref.: SRP (1981) SEC 3.8.3 Concrete and Steel Internal Structures of Containment

Notes

1. Ultimate strength method required by ACI-349 (1977).
2. Method used in design { working stress ✓ ~~ultimate strength~~ } Consequently no load factors were used.
3. Loads deemed inapplicable or negligible struck from loading combinations.
4. 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.
5. "Equipment" loads considered for internal concrete structures. (See Table 4-1, Ref. 9).
6. Credited here as an OBE Combination, because the 0.05g static load used is approximately half the value currently deemed appropriate for the SSE (i.e., 0.12g).
7. For purposes of the SEP Review, demonstration that structural integrity is maintained for load case 9 (per current citeria) 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: BIG ROCK POINT

STRUCTURE:
FUEL POOL

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		6.
3	.75 (1.4D + 1.7L)	.75 x 1.7 T_o		$\cancel{.75 \times 1.7 R_o}$			
4	.75 (1.4D + 1.7L)	.75 x 1.7 T_o		$\cancel{.75 \times 1.7 R_o}$	.75 x 1.9E		
5	$\boxed{D}$ + $\boxed{L}$	T_o		$\cancel{R_o}$	$\boxed{E'}$		A_x
6	D + L	T_o		$\cancel{R_o}$	$\cancel{W_t}$		
7	D + L	T_a	$\cancel{1.5 P_a}$	$\cancel{R_a}$			
8	D + L	T_a	$\cancel{1.25 P_a}$	$\cancel{R_a}$	1.25E	$\cancel{Y_r} + Y_j + Y_m$	
9	D + L	T_a	$\cancel{P_a}$	$\cancel{R_a}$	E'	$\cancel{Y_r} + Y_j + Y_m$	

Ref.: SRP (1981) SEC 3.8.3 Concrete and Steel Internal Structures of Containment

Notes

1. Ultimate strength method required by ACI-349 (1977).
2. Method used in design { working stress ✓ ~~ultimate strength~~ } Consequently no load factors were used.
3. Loads deemed inapplicable or negligible struck from loading combinations.
4. 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.
5. Load combinations shown by dashed-line boxes were considered in the review of the integrity of the plant structures under 0.12g earthquake by D'Appolonia Consulting Engineering, Inc.
6. Credited here as an OBE Combination, because the 0.05g static load used is approximately half the value currently deemed appropriate for the SSE (i.e. 0.12g).
7. For purposes of the SEP Review, demonstration that structural integrity is maintained for load case 5 (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: BIG ROCK POINT

STRUCTURE:
INTAKE 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)	$\cancel{.75 \times 1.7\ T_o}$		$.75 \times 1.7\ R_o$			
5	.75 (1.4D + 1.7L)	$\cancel{.75 \times 1.7\ T_o}$		$.75 \times 1.7\ R_o$	.75 x 1.9E		
6	.75 (1.4D + 1.7L)	$\cancel{.75 \times 1.7\ T_o}$		$.75 \times 1.7\ R_o$	.75 x 1.7W		
7	1.2D				1.9E		
8	1.2D				1.7W		
9	$\boxed{D}$ + L	$\cancel{T_o}$		R_o	$\boxed{E'}$		A_x
10	D + L	$\cancel{T_o}$		R_o	W_t		A_x
11	D + L	$\cancel{T_a}$	$\cancel{1.5\ P_a}$	$\cancel{R_a}$			
12	D + L	$\cancel{T_a}$	$\cancel{1.25\ P_a}$	$\cancel{R_a}$	1.25E	$\cancel{Y_r} + \cancel{Y_j} + \cancel{Y_m}$	
13	D + L	$\cancel{T_a}$	$\cancel{P_a}$	$\cancel{R_a}$	E'	$\cancel{Y_r} + \cancel{Y_j} + \cancel{Y_m}$	

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

Notes

1. Ultimate strength method required by ACI-349 (1977).
2. Methods used in design { working stress ✓ / ~~ultimate strength~~ } Consequently no load factors were used.
3. Loads deemed inapplicable or negligible struck from loading combinations.
4. 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.
5. Load combinations shown by dashed-line boxes were considered in the review of the integrity of the plant structures under 0.12g earthquake by D'Applonia Consulting Engineering, Inc.
6. Load combinations applicalbe to the steel portions of these structures (See NRC Standard Review Plan 3.8.4, Structural Steel) are essentially the same as shown above for the concrete portions.
7. For purposes of the SEP Review, demonstration that structural integrity is maintained for load cases 9 and 10 (per current criteria) may be considered as providing reasonable assurance that this structure meets theintent of current design criteria.

COMPARISON OF LOADING COMBINATION CRITERIA

CONCRETE STRUCTURES

PLANT: BIG ROCK POINT

STRUCTURE: CONTROL ROOM (IN SERVICE BUILDING)

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 [5]				1.9E			
3	1.4D + 1.7L				1.7W			6.
4	.75 (1.4D + 1.7L)	.75 x 1.7 T_o		~~.75 x 1.7 R_o~~				
5	.75 (1.4D + 1.7L)	.75 x 1.7 T_o		~~.75 x 1.7 R_o~~	.75 x 1.9E			
6	.75 (1.4D + 1.7L)	.75 x 1.7 T_o		~~.75 x 1.7 R_o~~	.75 x 1.7W			
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	1.5 P_a	~~R_a~~				
12	D + L	T_a	1.25 P_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

1. Ultimate strength method required by ACI-349 (1977).
2. Methods used in design { working stress ✓ ~~ultimate strength~~ } Consequently no load factors were used.
3. Loads deemed inapplicable or negligible struck from loading combinations.
4. 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.
5. Snow and "equipment" loads were considered for turbine building (See Sect. 3.3, Ref. 9).
6. Wind loads considered, according to Sect. 3.3, Ref. 9 but load combinations not stated in FSAR.
7. Load combinations shown by dashed-line boxes were considered in the review of the integrity of the plant structures under 0.12g earthquake by D'Appolonia Consulting Engineering, Inc.
8. Load combinations applicable to the steel portions of these structures, (See NRC Standard Review Plan 3.8.4., Structural Steel) are essentially the same as shown above for the concrete portions.
9. Credited here as an OBE Combination, because the 0.05g static load used is approximately half the value currently deemed appropriate for the SSE (i.e. 0.12g).
10. For purposes of the SEP Review, demonstration that structural integrity is maintained for load cases 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: BIG ROCK POINT

STRUCTURE: STACK

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		$\cancel{.75 \times 1.7 R_o}$				
5	.75 (1.4D + 1.7L)	.75 x 1.7 T_o		$\cancel{.75 \times 1.7 R_o}$	.75 x 1.9E			
6	.75 (1.4D + 1.7L)	.75 x 1.7 T_o		$\cancel{.75 \times 1.7 R_o}$	.75 x 1.7W			
7	1.2D				1.9E			
8	1.2D				1.7W			
9	$\boxed{D}$ + L	$\boxed{T_o}$		$\cancel{R_a}$	$\boxed{E'}$		A_x	5.
10	D + L	T_o		$\cancel{R_a}$	W_t		A_x	5.
11	D + L	$\cancel{T_a}$	$\cancel{1.5\ P_a}$	$\cancel{R_a}$				
12	D + L	$\cancel{T_a}$	$\cancel{1.25\ P_a}$	$\cancel{R_a}$	1.25E	$\cancel{Y_r} + \cancel{Y_j} + \cancel{Y_m}$		
13	D + L	$\cancel{T_a}$	$\cancel{P_a}$	$\cancel{R_a}$	E'	$\cancel{Y_r} + \cancel{Y_j} + \cancel{Y_m}$		

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

Notes

1. Ultimate strength method required by ACI-349 (1977).
2. Methods used in design { working stress ✓ ~~ultimate strength~~ } Consequently no load factors were used.
3. Loads deemed inapplicable or negligible struck from loading combinations.
4. 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.
5. The principal loads on the stack are D, E, E', W, & W_t. Reanalysis of all ventilation stacks for these loadings is being carried out within the SEP Program. Therefore, no action need be taken by licensee in response to this item.

COMPARISON OF LOADING COMBINATION CRITERIA

CONCRETE STRUCTURES

PLANT: BIG ROCK POINT

STRUCTURE: DIESEL GENERATOR ENCLOSURE/SCREEN WELL AND PUMP HOUSE

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)	$\cancel{.75 \times 1.7\ T_o}$		$\cancel{.75 \times 1.7\ R_o}$			
5	.75 (1.4D + 1.7L)	$\cancel{.75 \times 1.7\ T_o}$		$\cancel{.75 \times 1.7\ R_o}$	.75 x 1.9E		
6	.75 (1.4D + 1.7L)	$\cancel{.75 \times 1.7\ T_o}$		$\cancel{.75 \times 1.7\ R_o}$	.75 x 1.7W		
7	1.2D				1.9E		
8	1.2D				1.7W		
9	$\boxed{D}$ + L	$\cancel{T_o}$		$\cancel{R_o}$	$\boxed{E''}$		A_x
10	D + L	$\cancel{T_o}$		$\cancel{R_o}$	W_t		A_x
11	D + L	$\cancel{T_a}$	$\cancel{1.5\ P_a}$	$\cancel{R_a}$			
12	D + L	$\cancel{T_a}$	$\cancel{1.25\ P_a}$	$\cancel{R_a}$	1.25E	$\cancel{Y_r} + \cancel{Y_j} + \cancel{Y_m}$	
13	D + L	$\cancel{T_a}$	$\cancel{P_a}$	$\cancel{R_a}$	E'	$\cancel{Y_r} + \cancel{Y_j} + \cancel{Y_m}$	

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

Notes

1. Ultimate strength method required by ACI-349 (1977).
2. Methods used in design { working stress ✓ ~~ultimate strength~~ Consequently no load factors were used.
3. Loads deemed inapplicable or negligible struck from loading combinations.
4. 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.
5. Load combinations shown by dashed-line boxes were considered in the review of the integrity of the plant structures under 0.12g earthquake by D'Appolonia Consulting Engineering, Inc.
6. For purposes of the SEP Review, demonstration that structural integrity is maintained for load cases 9 & 10 (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: BIG ROCK POINT

STRUCTURE: BATTERY ROOM (IN TURBINE BUILDING)

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 5,6				1.9E			9.
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 + 1.7L)	~~.75 x 1.7 T_o~~		~~.75 x 1.7 R_o~~	.75 x 1.9E			
6	.75 (1.4D + 1.7L)	~~.75 x 1.7 T_o~~		~~.75 x 1.7 R_o~~	.75 x 1.7W			
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~~	1.5 P_a	~~R_a~~				
12	D + L	~~T_a~~	1.25 P_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

1. Ultimate strength method required by ACI-349 (1977).
2. Methods used in design { working stress ✓ ~~ultimate strength~~ Consequently no load factors were used.
3. Loads deemed inapplicable or negligible struck from loading combinations.
4. 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.
5. Snow and "equipment" loads were considered for turbine building. (See Sect. 3.3, Ref. 9).
6. Wind loads considered according to Sect. 3.3, Ref. 9 but load combinations not stated in FSAR.
7. Load combinations shown by dashed-line boxes were considered in the review of the integrity of the plant structures under 0.12g earthquake by D'Appolonia Consulting Engineering, Inc.
8. Load combinations applicable to the steel portions of these structures, (See NRC Standard Review Plan 3.8.4, Structural Steel) are essentially the same as shown above for the concrete portions.
9. Credited here as an OBE Combination, because the 0.05g static load used is approximately half the value currently deemed appropriate for the SEE (i.e. 0.12g).
10. For purposes of the SEP Review, demonstration that structural integrity is maintained for load cases 10 & 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 Big Rock Point Nuclear Power Station were:

1. Uniform Building Code 1958 (invokes AISC, "Specification for Design, Fabrication, and Erection of Structural Steel for Buildings," 1953)
2. ACI 318-56, "Building Code Requirements for Reinforced Concrete," 1956
3. ASME Boiler and Pressure Vessel Code, Section VIII, 1956.

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 original 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 Big Rock Point structures. When it could be determined 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. The appendix 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-1953 VS. AISC-1980 CODE COMPARISON

MAJOR FINDINGS OF AISC 1953 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	AISC 1953	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.
1.5.1.4.1	1.5.1.4.1	15(a)(3)	Rolled sections, plate girders and built up members.	New requirements added in the 1963 Code limiting the allowable stresses for tension due to bending.
1.6	1.6	12(a)	Members subject to axial and bending stresses	New requirement for combined stresses added in the 1963 Code
1.8.3	1.8.3	16	Axially loaded compression members where sideway is not prevented	New requirements for slenderness ratio added in the 1963 Code

MAJOR FINDINGS OF AISC 1953 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	AISC 1953	Structural Elements Potentially Affected	Comments
1.9.1.2 and Appendix C	1.9.1	18(b)	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 1963 and the 1980 Code, Appendix C.
1.10.4	1.10.4	26(d)	Partial length cover plates in plate girders and rolled beams	New requirements added in the 1963 Code
1.10.7	1.10.7	--	Plate girder web	New requirements for combined shear and tension stress added to the 1963 Code
1.10.10.2	1.10.10.2	26	Stiffeners for web plate girders	Change in the requirements of the 1953 Code
1.11.1	1.11.1	13(a)	Composite construction	Limitation on effective width of concrete flange is introduced in the 1953 Code

MAJOR FINDINGS OF AISC 1953 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	AISC 1953	Structural Elements Potentially Affected	Comments
1.11.4	1.11.4	13	Shear connectors in composite beams	New requirements added in the 1963 Code and the 1980 Code
1.11.5	--	--	Composite beams or girders with formed steel deck	New requirements 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
1.14.6.1	1.14.7	15(f)	Effective throat thickness for partial penetration weld	
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

MAJOR FINDINGS OF AISC 1953 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	AISC 1953	Structural Elements Potentially Affected	Comments	Scale
1.18.3	1.18.3	28(b)	Built up members under tension	New requirement added in the 1963 Code	
2.9	2.8	--	Lateral bracing of members to resist lateral and torsional displacement	0.0 < M/Mp < 1.0 0.0 > M/Mp > -1.0 See case study 7 for details.	A C

11.2 MAJOR FINDINGS OF ACI 318-56 VS. ACI 349-76 CODE COMPARISON

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MAJOR FINDINGS OF ACI 318-56 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	ACI 318-63	ACI 318-56	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 318-56 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	ACI 318-56	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.
Chap. 12	Chap. 18	--	All	New chapter; old code did not have ultimate strength criteria for bond. This chapter presents some changes in bond stresses allowed and a change in philosophy. Allowable bond values are higher on small bars, but lower on large bars because of this shift in philosophy introduced by ultimate strength logic here.

MAJOR FINDINGS OF ACI 318-56 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	ACI 318-56	Structural Elements Potentially Affected	Comments
Chapter 12 (cont.)				Splice lengths in column steel are the same as the 56 code and permissible bond stress for compression bars was set to match when reduced to working stress.
--	1301(c)	Table 305(a)	All	Allowable bond stresses are presented in the new code as a function of concrete strength and bar diameter. Values in the new code are higher for small diameter bars and lower for large diameter bars as compared to the old code. See case study (14).
Chap. 17	Chapter 25	--	Composite construction	New chapter; ACI 318-56 did not contain specific sections on composite concrete flexural members and composite construction.

MAJOR FINDINGS OF ACI 318-56 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	ACI 318-56	Structural Elements Potentially Affected	Comments
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	For structures subject to effects of pipe break, especially jet impingement, thermal stresses may be significant. Scale A for areas of jet impingement or where the conditions could develop causing concrete temperature to exceed limitations of A.4.2. For structures not subject to effects of pipe break accident, thermal stresses are unlikely to be significant (Scale B).
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.**

**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.

MAJOR FINDINGS OF ACI 318-56 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	ACI 318-56	Structural Elements Potentially Affected	Comments
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.

11.3 MAJOR FINDINGS OF ASME B&PV CODE COMPARISON, SECTION VIII, 1956 VS. SECTION III, SUBSECTION NE, 1980

ASME B&PV CODE COMPARISON
SECTION VIII, 1956 VS. SECTION III, SUBSECTION NE, 1980

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

Scale A

Referenced Subsection: Section III 1980	Section VIII 1962	Section VIII 1956	Structural Elements Potentially Affected	Comments
NE-3112.4	---	UG-5(b)	Plates, if understrength	The 1956 Code permits conditional use of understrength plate, if: 1. The local allowable stress is correspondingly reduced; and 2. The UTS range is maintained. This practice has been terminated and current codes are blind to such situations in older structures. Scale A - if additional loads, not originally designed for, are required by current criteria. No scale ranking applicable otherwise.
NE-3112.4	UG-23	UG-23	Vessels of materials no longer listed as Code acceptable	Section III, 1980 Code references some materials which are identical to those referenced in Section VIII, 1956 Code. However, several materials which were referenced in Section VIII, are no longer listed in Section III, 1980. Justification of such use would be necessary to show equivalence to current requirements.

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

Scale **A** (Cont.)

Referenced Subsection				
Section III 1980	Section VIII 1962	Section VIII 1956	Structural Elements Potentially Affected	Comments
NE-3131	---	Various Paragraphs	Containment shells designed by formula	Section VIII, 1956 Code calls for the design of vessels by formula, while Section III, 1980 Code requires that the rules of Subsection NE-3200 (Design by Analysis) be satisfied. In the absence of substantial thermal or mechnical loads other than pressure, the rules of "Design by Formula" may still be used. The scale rating for containment shells where substantial thermal or mechnical loads other than pressure are absent is Scale B; otherwise it is Scale A.
NE-3133.5(a)	UG-29	UG-29	Stiffening rings for cylindrical shells subject to buckling loads.	The requirements of the 1980 Code defining the minimum moment of inertia for stiffening rings as compared to the requirements of the 1956 Code may result in a lower margin of safety.

	Scale
$I_s' > 1.28\ I_s$	C
$I_s' > 1.22\ I_s$	B
$I_s' < 1.22\ I_s$	A

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ASME B&PV CODE COMPARISON

SECTION VIII, 1956 VS. SECTION III, SUBSECTION NE, 1980

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

Scale A (Cont.)

Referenced Subsection				
Section III 1980	Section VIII 1962	Section VIII 1956	Structural Elements Potentially Affected	Comments
NE-3133.5(b)	---	---	Shell and stiffening rings of different materials.	This new insert in Section III of the 1980 Code requires using the material chart which gives the larger value of the factor A. This may result in a larger stiffening ring section needed to meet the requirements of the Code. Scale A for ring-stiffened shells where (1) the ring and the shell are of different materials and, in addition, (2) the "factor A" (as computed by the procedures of NE-3133.5) for the two materials differs by more than 6%; otherwise Scale B.
NE-3221.5	---	---	Containment components subject to cyclic loadings.	Requirements for fatigue analysis of vessels or parts which experience cyclic loadings are provided in Section III, Subsection NE, of the 1980 Code. No specific guidance was provided by Section VIII, 1956.

ASME B&PV CODE COMPARISON
SECTION VIII, 1956 VS. SECTION III, SUBSECTION NE, 1980

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

Scale A (Cont.)

Referenced Subsection				
Section III 1980	Section VIII 1962	Section VIII 1956	Structural Elements Potentially Affected	Comments
NE-3325 Figs. (c) and (m)	UG34(d) Figs. (d) and (p)	UG-34(d) Figs. (b) and (a)	Unstayed flat heads and covers of the designs in the referenced figures	Present Code requires thicker plates.
NE-3327	UG-35	Footnote to UG-35	Quick-acting closures	Subsection NE, 1980 has expanded requirements for safety devices including: o positive interlocks on remotely operated doors o warning devices on manually operated doors o visibility of pressure indicators from operating floor.
NE-3334.1 NE-3334.2	UG-40(b) UG-40(c)	UG-40	Reinforcement for vessel openings	New requirements in the 1980 Code impose additional restrictions on metal that may be counted as reinforcement.
NE-3365	---	--	Bellows and bellows expansion joints	The 1980 Code imposes new design requirements.

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 Seismic Category I structures at the Big Rock Point Nuclear Power Station.

The first and second columns of the table show the number of changes in requirements found for the design codes used for Seismic Category I structures external to containment, classified by scale ranking. The first column applies to the concrete portion of these structures and the internal structures; the second column applies to the portions of the external structures which are of steel frame construction. The third column applies only to the primary containment.

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 Big Rock Point plant, to be an effort of tractable size.

SUMMARY

NUMBER OF CODE CHANGE IMPACTS FOR BIG ROCK POINT CATEGORY I STRUCTURES

SCALE RANKING		ACI 318-56 VS. ACI 349-76	AISC 1953 VS. AISC 1980	ASME B & PV SEC. VIII, 1956 VS. SEC. III SUBSECT. NE, CLASS MC, 1980
TOTAL CHANGES FOUND		113	50	30
Do Not Require Further Investigation	A or A_x not Applicable to Big Rock Point	3 + 4*	13	4 + 3*
	B	84	13	10
	C	12	8	3
To Be Further Investigated	A	10	16	10
	A_x	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 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.

---

*These changes are related to loads and load combinations. Loading criteria are addressed in Section 10. Consequently, to avoid duplication, such items are not counted in the above tabulation of code changes to be addressed under Section 11.

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 Big Rock Point 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 Big Rock Point plant.

LIST OF STRUCTURAL ELEMENTS TO BE EXAMINED

Structural Elements to be Examined	Code Change Affecting These Elements New Code	Old Code	Scale
Composite Construction	AISC 1980	AISC 1953	
1. Shear connectors in composite beams	1.11.4	13	A
2. Composite beams or girders with formed steel deck	1.11.5	--	A
3. Width of concrete flange - limitations	1.11.1	13(a)	A
Compression Elements	AISC 1980	AISC 1953	
1. With width-to-thickness ratio higher than specified in 1.9.1.2	1.9.1.2 and Appendix C	18(b)	A
2. Members where sideway is not prevented	1.8.3	16	A
Tension Members	AISC 1980	AISC 1953	
1. When load is transmitted by bolts or rivets	1.14.2.2	--	A
2. Built up members	1.18.3	28(b)	A
Connections	AISC 1980	AISC 1953	
1. Beam ends with top flange coped, if subject to shear	1.5.1.2.2	--	A
2. Connections carrying moment or restrained member connection	1.15.5.2 1.15.5.3 1.15.5.4	--	A

*Double dash (--) indicates that older code had no provisions.

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 1953	
Spacing of lateral bracing	2.9	--	A
Rolled Sections and Built up Members	AISC 1980 1.5.1.4.1	AISC 1953 15(a)(3)	A
Partial length cover plates	1.10.4	26(d)	A
Members Subject to Axial and Bending Stresses	AISC 1980 1.6	AISC 1953 12(a)	A
Web Plate Girders	AISC 1980	AISC 1953	
1. Subject to shear and tension stresses	1.10.7	--	A
2. Stiffeners	1.10.10.2	26	A
Partial Penetration Weld Effective throat thickness	1.14.6.1	15(f)	A
Short Brackets and Corbels having a shear span-to-depth ratio of unity or less	ACI 349-76 11.13	ACI 318-56 --	A
Shear Walls used as a primary load-carrying member	ACI 349-76 11.16	ACI 318-56 --	A
Precast Concrete Structural Elements, where shear is not a measure of diagonal tension	ACI 349-76 11.15	ACI 318-56 --	A
Concrete Regions Subject to High Temperatures	ACI 349-76	ACI 318-56	
Time-dependent and position-dependent temperature variations	Appendix A	--	A

LIST OF STRUCTURAL ELEMENTS TO BE EXAMINED (Cont.)

Structural Elements to be Examined	Code Change Affecting These Elements: New Code	Old Code	Scale
All Structural Elements	ACI 349-76	ACI 318-56	
1. Ultimate bond strength	Chapter 12	--	A
2. Allowable bond stress	--	Table 305(a)	A
Columns with Spliced Reinforcement	ACI 349-76	ACI 318-56	
subject to stress reversals; f_y in compression to 1/2 f_y in tension	7.10.3	--	A
Steel Embedments used to transmit load to concrete	ACI 349-76 Appendix B	ACI 318-56 --	A
Element Subject to Impulsive and Impactive Loads whose failure must be precluded	ACI 349-76 Appendix C	ACI 318-56 --	A
Composite Construction	ACI 349-76 Chapter 17	ACI 318-56 --	A
Containment Vessels			
1. Plates, if understrength	ASME Sec. III, 1980 NE-3112.4	ASME Sec. VIII, 1956 UG-5(b)	A
2. Containment vessels of materials no longer listed as code acceptable	ASME Sec. III, 1980 NE-3112.4	ASME Sec. VIII, 1956 UG-23	A
3. Containment vessels designed by formula and subject to substantial thermal or mechanical loads	ASME Sec. III, 1980 NE-3131	ASME Sec. VIII, 1956 Various paragraphs	A
4. Stiffening rings for cylindrical shells subject to buckling loads	ASME Sec. III, 1980 NE-3133.5(a)	ASME Sec. VIII, 1956 UG-29	A

LIST OF STRUCTURAL ELEMENTS TO BE EXAMINED (Cont.)

Structural Elements to be Examined	Code Change Affecting These Elements: New Code	Old Code	Scale
5. Stiffening rings of material different than shell material	ASME Sec. III, 1980 NE-3133.5(b)	ASME Sec. VIII, 1956 --	A
6. Vessels with Quick Actuating Closures	ASME Sec. III, 1980 NE-3327.1	ASME Sec. VIII, 1956 Footnote to UG-35	A
Shell Openings and Attachments			
1. Unstayed flat heads and covers	ASME Sec. III, 1980 NE-3325 Figs. (c) and (m)	ASME Sec. VIII, 1956 UG-34(d) Figs. (b) and (a)	A
2. Openings and reinforcements; subject to cyclic loads	ASME Sec. III, 1980 NE-3331(b)	ASME Sec. VIII, 1956 --	A
3. Reinforcement for openings	ASME Sec. III, 1980 NE-3334.1, NE-3334.2	ASME Sec. VIII, 1956 UG-40	A
4. Bellows and bellows expansion joints	ASME Sec. III, 1980 NE-3365	ASME Sec. VIII, 1956 --	A
Roofs	---	--	A[1]

Extreme environmental snow loads are provided by SEP Topic II-2.A. NRC Regulatory Guide 1.102 (Position 3) provides guidance to preclude adverse consequences from ponding or parapet roofs. Failure of roofs not designed for such circumstances could generate impulsive loadings and water damage, possibly extending to Seismic Category I components of all floor levels.

1. Not shown in tabular summary of code change impacts.

14. REFERENCES

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

2. American Institute of Steel Construction, Inc.
Specification for Design, Fabrication, and Erection of Structural Steel for Buildings
New York, NY, 1953

3. American Concrete Institute
Building Code Requirements for Reinforced Concrete
Detroit, MI, 1956
ACI 318-63

4. American Society of Mechanical Engineers
Boiler and Pressure Vessel Code, Section VIII
New York, NY, 1956

5. Consumers Power Company
Final Hazards Summary Report for Big Rock Point Plant
14-Nov-61

6. Bechtel Corporation
Final Hazards Summary Report for Big Rock Point Plant
Vol. II, Part 1 - Engineering Drawings
14-Nov-61

7. NRC Letter to Consumer Power Company
Subject: Codes Standards for Seismic Category I Structures
19-May-81

8. Appendix I to Technical Evaluation Report, "Design Codes, Design Criteria, and Loading Combinations"
Contains List of Basic Documents Defining Current Licensing Criteria for SEP Topic III-7.B
Franklin Research Center, 1981
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9. Engineering Decision Analysis Company, Inc. (EDAC)
Seismic Design Bases and Criteria for Big Rock Point Plant
January 1979

APPENDIX A

SCALE A AND SCALES A_x CHANGES
DEEMED INAPPROPRIATE TO BIG ROCK POINT

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APPENDIX A-1

AISC 1953 VS. AISC 1980 CODE COMPARISON

(SCALE A AND SCALE A_x CHANGES DEEMED INAPPROPRIATE TO BIG ROCK POINT
OR CODE CHANGES RELATED TO LOADS OR LOAD COMBINATIONS
AND THEREFORE TREATED ELSEWHERE)

AISC 1953 VS. AISC 1980 CODE COMPARISON

Scale A

Referenced Subsection				
AISC 1980	AISC 1963	AISC 1953	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 Big Rock Cat. I structures is A-7. Thus, $F_y < 0.83\ F_u$ Therefore, Scale C for Big Rock.
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 Big Rock 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

Limitations for 1.5.1.1:

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

AISC 1953 VS. AISC 1980 CODE COMPARISON

Scale A (Cont.)

Referenced Subsection				
AISC 1980	AISC 1963	AISC 1953	Structural Elements Potentially Affected	Comments
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 Big Rock Cat. I structures; therefore; not applicable
1.5.2.2	1.7	11(b)	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	1.7	11	Members and connections subject to 20,000 cycles or more	Cat. I structures are not subject to such cyclic loading; therefore, not applicable
1.9.2.1 and Appendix C	1.9.2	18(c)	Stiffened Compression members	All structural steel is A-7, Fy < 40 ksi; therefore, Scale C
1.9.2.3 and Appendix C	--	--	Circular tubular elements subject to axial compression	New requirements added in the 1980 Code

AISC 1953 VS. AISC 1980 CODE COMPARISON

Scale A (Cont.)

Referenced Subsection				
AISC 1980	AISC 1963	AISC 1953	Structural Elements Potentially Affected	Comments
1.10.6	1.10.6	26	Hybrid girder - reduction in flange stress	All structural steel is A-7. No hybrid girders found in Big Rock, 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)	
2.4 1st Para.	2.3 1st Para.	--	Slenderness ratio for columns. Must satisfy: $\frac{l}{r} \leq \frac{2\pi^2 E}{F_y}$	

	Scale	
$F_y \leq 40$ ksi	C	Scale C for Big Rock. See case study 4 for details.
$40 \leq F_y < 44$ ksi	B	
$F_y \geq 44$ ksi	A	

AISC 1953 VS. AISC 1980 CODE COMPARISON

Scale A (Cont.)

Referenced Subsection AISC 1980	AISC 1963	AISC 1953	Structural Elements Potentially Affected	Comments
2.7	2.6	--	Flanges of rolled W, M, or S shapes and similar built-up single-web shapes subject to compression $F_y \leq 36$ ksi — Scale C $36 < F_y < 38$ ksi — Scale B $F_y \geq 38$ ksi — Scale A	Scale C for Big Rock. See case study 6 for details.
Appendix D	--	--	Web tapered members	New requirements added in the 1980 Code Web tapered member are not found to be used in Big Rock Cat. I structures, therefore, not applicable

APPENDIX A-2

ACI 318-56 VS. ACI 349-76 CODE COMPARISON

(SCALE A AND SCALE A_x CHANGES DEEMED INAPPROPRIATE TO BIG ROCK PLANT OR CODE CHANGES RELATED TO LOADS OR LOAD COMBINATIONS AND THEREFORE TREATED ELESEWHERE)

ACI 318-56 VS. ACI 349-76 SUMMARY OF CODE COMPARISON

Scale A

Referenced Subsection ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
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.*
Chap. 18	Chap. 26	--	Prestressed concrete.	New chapter; ACI 318-56 did not contain specific sections or criteria for prestressed concrete. No prestressed elements outside primary containment; therefore, not applicable.
18.1.4 and 18.4.2	--	--	Prestressed concrete elements.	New load combinations here refer to Chapter 9 load combinations.* No prestressed elements outside containment; therefore, not applicable.
Chap. 19	Chap. 19	--	Shell structures with thickness equal to or greater than 12 inches.	No concrete shell structure; therefore, not applicable.

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

ACI 318-56 VS. ACI 349-76 SUMMARY OF CODE COMPARISON

Scale A (Cont.)

Referenced Subsection				
ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
3.5	405 (e),(f)	--	Prestressed elements.	New insert lists ASTM specifications for prestressing wire and strands. 318-56 did not have sections dealing with prestressed concrete. Controls other than ACI Codes or recommended practice would apply to this type of construction prior to 1963. No prestressed elements outside containment; therefore, not applicable.
Chap. 9 9.1, 9.2, & 9.3 most specifically	Chap. 15	A604	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.*

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

ACI 318-56 VS. ACI 349-76 SUMMARY OF CODE COMPARISON

Scale A (Cont.)

Referenced Subsection ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
Chap. 19 (Cont.)	--	--	Shell Structures	This chapter is completely new; therefore, shell structures designed by the general criteria of older codes may not satisfy all aspects of this chapter. In addition, this chapter refers to Chapter 9 provisions.

APPENDIX A-3

ASME B&PV CODE COMPARISON

SECTION VIII, 1956, VS. SECTION III, SUBSECTION NE, 1980

(SCALE A OR SCALE A_x CHANGES DEEMED NOT APPLICABLE TO BIG ROCK PLANT

OR CODE CHANGES RELATED TO LOAD COMBINATIONS

AND THEREFORE TREATED ELSEWHERE)

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ASME B&PV CODE COMPARISON
SECTION VIII, 1956 VS. SECTION III, SUBSECTION NE, 1980

(Scale A Changes Deemed Not Applicable or Changes Related to Loadings)

Scale A

Referenced Subsection				
Section III 1980	Section VIII 1962	Section VIII 1956	Structural Elements Potentially Affected	Comments
---	UG-25(d)	UG-25(d) UW-15(b)	Vessels containing telltale holes	The 1956 Code required telltale holes at reinforcing plates and saddles at nozzles to be left open. The 1962 Code permitted plugging. The removal of these provisions from Section III, 1962 Code, bans the use of telltale holes. Moreover, the more recent version of Section VIII specifically excludes using telltale holes for lethal substances.
NE-3324.3	UG-27(c)	UG-27(c)	Vessel components where welding efficiency of circumferential joints is less than half the longitudinal joint efficiency	The 1956 Code did not require computation of axial stress in cylindrical shells. The wide disparity in welding efficiencies is deemed improbable.
NE-3325	UG-34(c)3	UG-34(c)3	Heads, covers, or blind flanges on non-circular shape	The 1956 Code did not distinguish between circular and non-circular plates when specifying plate thickness. These thicknesses are now regarded as inadequate for most non-circular plates.

ASME B&PV CODE COMPARISON

SECTION VIII, 1956 VS. SECTION III, SUBSECTION NE, 1980

(Scale A Changes Deemed Not Applicable or Changes Related to Loadings)

Scale A

Referenced Subsection				
Section III 1980	Section VIII 1962	Section VIII 1956	Structural Elements Potentially Affected	Comments
NE-2124(b)	UG-16(c)	UG-16(c)	Pressure retaining plates less than 0.167 in thick	The 1956 Code granted a blanket 0.010 in mill undertolerance on all plate. The present Code allows 6% or 0.010 in undertolerance (whichever is least).
NE-3111	UG-22	UG-22	Loading as applied to load-carrying components*	Section III, 1980 Code, specifies additional loads to be considered in designing the vessel. These include: o dynamic head of liquids o snow loads and vibration loads o reaction to steam and water jet impingement
NE-3112.2	---	UG-20	Design temperature as applied to the vessel and its components*	The effects of internal heat generation due to radiation (in addition to all external sources) must be included in establishing design temperature.
NE-3112.3	---	---	Design mechanical loads as applied to the vessel and its components*	Currently, the design load combination includes mechanical loads. In 1956, the Code considered pressure at temperature only.

*Treatment of loads and load combinations is addressed in Section 10.

APPENDIX B

SUMMARIES OF CODE COMPARISON FINDINGS

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APPENDIX B-1

AISC 1953 VS. AISC 1980

SUMMARY OF CODE COMPARISON

(SYNTHESIS OF AISC 1953 VS. AISC 1963 VS. AISC 1980 CODE COMPARISONS)

AISC 1953 VS. AISC 1980
SUMMARY OF CODE COMPARISON

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Scale A

Referenced Subsection				Comments	
AISC 1980	AISC 1963	AISC 1953	Structural Elements Potentially Affected	Limitations	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	1.5.1.4.1	15(a)(3)	Rolled sections, plate girders and built up members.	New requirements added in the 1963 Code limiting the allowable stresses for tension due to bending.	
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	

AISC 1953 VS. AISC 1980
SUMMARY OF CODE COMPARISON

Scale A (Cont.)

Referenced Subsection				
AISC 1980	AISC 1963	AISC 1953	Structural Elements Potentially Affected	Comments
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	11(b)	Rivets, bolts, and threaded parts subject to 20,000 cycles or more	Change in the requirements
1.6	1.6	12(a)	Members subject to axial and bending stresses	New requirement for combined stresses added in the 1963 Code
1.7 and Appendix B	1.7	11	Members and connections subject to 20,000 cycles or more	Change in the requirements
1.8.3	1.8.3	16	Axially loaded compression members where sideway is not prevented	New requirements for slenderness ratio added in the 1963 Code

AISC 1953 VS. AISC 1980
SUMMARY OF CODE COMPARISON

Scale A (Cont.)

Referenced Subsection				
AISC 1980	AISC 1963	AISC 1953	Structural Elements Potentially Affected	Comments
1.9.1.2 and Appendix C	1.9.1	18(b)	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 1963 and the 1980 Code, Appendix C.
1.9.2.1 and Appendix C	1.9.2	18(c)	Stiffened compression members	New requirements added in the 1963 Code and the 1980 Code
1.9.2.3 and Appendix C	--	--	Circular tubular elements subject to axial compression	New requirements added in the 1980 Code
1.10.4	1.10.4	26(d)	Partial length cover plates in plate girders and rolled beams	New requirements added in the 1963 Code
1.10.6	1.10.6	26	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.

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AISC 1953 VS. AISC 1980
SUMMARY OF CODE COMPARISON

Scale A (Cont.)

Referenced Subsection				
AISC 1980	AISC 1963	AISC 1953	Structural Elements Potentially Affected	Comments
1.10.7	1.10.7	--	Plate girder web	New requirements for combined shear and tension stress added to the 1963 Code
1.10.10.2	1.10.10.2	26	Stiffeners for web plate girders	Change in the requirements of the 1953 Code
1.11.1	1.11.1	13(a)	Composite construction	Limitation on effective width of concrete flange is introduced in the 1953 Code
1.11.4	1.11.4	13	Shear connectors in composite beams	New requirements added in the 1963 Code and the 1980 Code
1.11.5	--	--	Composite beams or girders with formed steel deck	New requirements added in the 1980 Code
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)	

AISC 1953 VS. AISC 1980
SUMMARY OF CODE COMPARISON

Scale A (Cont.)

Referenced Subsection				
AISC 1980	AISC 1963	AISC 1953	Structural Elements Potentially Affected	Comments
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
1.14.6.1	1.14.7	15(f)	Effective throat thickness for partial penetration weld	
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
1.15.7	1.15.7	21(g)	Connections of tension and compression members in trusses	
1.18.3	1.18.3	28(b)	Built-up members under tension	New requirement added in the 1963 Code

AISC 1953 VS. AISC 1980
SUMMARY OF CODE COMPARISON

Scale A (Cont.)

Referenced Subsection					
AISC 1980	AISC 1963	AISC 1953	Structural Elements Potentially Affected	Comments	Scale
2.4 1st Para.	2.3 1st Para.	--	Columns, Slenderness ratio for columns. Must satisfy: $\frac{l}{r} \leq \frac{2\pi^2 E}{F_y}$	See case study 4 for details. $F_y \leq 40$ ksi $40 < F_y < 44$ ksi $F_y \geq 44$ ksi	Scale C B 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. $F_y \leq 36$ ksi $36 < F_y < 38$ ksi $F_y \geq 38$ ksi	Scale C B 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 1953 VS. AISC 1980
SUMMARY OF CODE COMPARISON

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Scale B

Referenced Subsection				
AISC 1980	AISC 1963	AISC 1953	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.
1.10.5	1.10.5	26(e)	Intermediate stiffeners for plate girders and rolled beams	Change of in the requirements of the 1953 Code
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

AISC 1953 VS. AISC 1980
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Subsection				
AISC 1980	AISC 1963	AISC 1953	Structural Elements Potentially Affected	Comments
1.14.2	1.14.3	19(g)	Member with through hole	The 1963 Code specifies slightly more stringent requirements
1.14.6.1.3	--	--	Flare type groove welds when flush to the surface of the solid section of the bar	
1.15.5.5	--	--	Connections having high shear in the column web	New insert in the 1980 Code
1.15.11	1.15.11	--	Friction type joints	
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	
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 1953 VS. AISC 1980
SUMMARY OF CODE COMPARISON

Scale C

Referenced Subsection				
AISC 1980	AISC 1963	AISC 1953	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.
1.5.1.3.1	1.5.1.3.1	15(a)(2)	Axially loaded members under compression	New requirements added the 1963 Code - See Case Study 15 for details
1.5.1.5.3	1.5.2.2	--	Bolts and rivets - bearing stress on projected area - in bearing type connections $F_p = 1.5\ F_u$ (1980 Code) $F_p = 1.35\ F_y$ (1963 Code)	New provisions added in the 1963 Code.
1.10.2	1.10.2	26(b)	Web girders and rolled beams	The requirements of the 1963 Code are more liberal

AISC 1953 VS. AISC 1980
SUMMARY OF CODE COMPARISON

Scale C (Cont.)

Referenced Subsection				
AISC 1980	AISC 1963	AISC 1953	Structural Elements Potentially Affected	Comments
1.10.5.3	1.10.5.3	--	Stiffeners in girders - added spacing between stiffeners at end panels, at panels containing large holes, and at panels adjacent to panels containing large holes	New design concept 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
1.14.5	1.14.6	19(g)	Pin Connected Members	
1.15.1	1.15.1	21(a)	Connections	More stringent requirements were specified in the 1953 Code.

APPENDIX B-2

ACI 318-56 VS. ACI 349-76

SUMMARY OF CODE COMPARISON

(SYNTHESIS OF ACI 318-56 VS. ACI 318-63 VS. ACI 349-76 CODE COMPARISONS)

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ACI 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale A

Referenced Subsection				
ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
3.5	405 (e),(f)	--	Prestressed elements	New insert lists ASTM specifications for prestressing wire and strands. 318-56 did not have sections dealing with prestressed concrete. Controls other than ACI Codes or recommended practice would apply to this type of construction prior to 1963.
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.
Chap. 9 9.1, 9.2, & 9.3 most specifically	Chap. 15	A604	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.*

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

Scale A (Cont.)

Referenced Subsection ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
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 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.

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

Scale A (Cont.)

Referenced Subsection ACI 349-76	ACI 318-63	ACI 318-56	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.
Chapter 12	Chapter 18	--	All	New chapter; old code did not have ultimate strength criteria for bond. This chapter presents some changes in bond stresses allowed and a change in philosophy. Allowable bond values are higher on small bars, but lower on large bars because of this shift in philosophy introduced by ultimate strength logic here. Splice lengths in column steel are the same as the 56 code and permissible bond stress for compression bars was set to match when reduced to working stress.

ACI 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale A (Cont.)

Referenced Subsection				
ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
--	1301(c)	Table 305(a)	All	Allowable bond stresses are presented in the new code as a function of concrete strength and bar diameter. Values in the new code are higher for small diameter bars and lower for large diameter bars as compared to the old code. See case study (14).
Chap. 17	Chapter 25	--	Composite construction	New chapter; ACI 318-56 did not contain specific sections on composite concrete flexural members and composite consruction.
Chap. 18	Chapter 26	--	Prestressed concrete	New chapter; ACI 318-56 did not contain specific sections or criteria for prestressed concrete.
18.1.4 and 18.4.2	--	--	Prestressed concrete elements	New load combinations here refer to Chapter 9 load combinations.*
Chap. 19	Chap. 19	--	Shell structures with thickness equal to or greater than 12 inches	This chapter is completely new; therefore, shell structures designed by the general

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

ACI 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale A (Cont.)

Referenced Subsection ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
Chap. 19 (Cont.)				criteria of older codes may not satisfy all aspects of this chapter. 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 did not give specific guidelines on short-term temperature limits for concrete. The possible effects of strength loss in concrete at high temperatures should be assessed. Scale A for any accident temperature or other thermal condition exceeding limits of paragraph A.4.2.
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.**

**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 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale A (Cont.)

Referenced Subsection ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
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 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B

Referenced Subsection				
ACI 349-76	ACI 318-63	ACI 318-56	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.
Chap. 3	Chap. 4	Chap. 2	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.
--	1208	--	Elements where lightweight concrete was used.	Probably does not apply to nuclear structures.

ACI 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Subsection				
ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
3.2	402	205	Cement	This serves to clarify intent of previous code.
3.3	403	206	Aggregate	Eliminated reference to lightweight aggregate.
3.3.1	403	206	Any structural concrete covered by ACI 349-76 and expected to provide for radiation shielding in addition to structural capacity	Controls of ASTM C567, "Standard Specifications for Aggregates for Radiation Shielding Concrete," closely parallel those for ASTM C33, "Standard Specification for Concrete Aggregates."
3.3.3	403	206	Aggregate	To ensure adequate control.
3.4.2	404	207	Water for concrete	Improve quality control measures.
3.5	405	208	Metal reinforcement	Removed all reference to steel with f_y > 60,000 psi.
3.5.1	405(a)	--	Reinforcing bar welds	Older code did not reference A.W.S. literature, but specific jobs that allowed welding of reinforcing bars normally listed requirements in the job specifications.

ACI 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Subsection ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
3.6	406, 407, & 408	--	Concrete admixtures	Added requirements to improve quality control.
3.6.3 & 3.6.4	407 & 408	--	Concrete where admixtures were used	Extensive use of these admixtures before 1963 was not common.
4.1 & 4.2	501 & 502	302 & 303	Concrete proportioning	Proportioning logic improved to account for statistical variation and statistical quality control.
4.2.5 & 4.2.7	501(c) & 501(d)	--	Concrete exposed to freezing or chemically aggressive environments	Past practice used other sources to guide designs in chemically aggressive environments.
4.3	504	304	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.

Scale B (Cont.)

Referenced Subsection				
ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
4.3.3	504(c)	304(c)	Concrete quality control	Changed to separate quality control on strength for working stress and ultimate strength. Control for working stress in new code made somewhat more conservative.
--	505	--	Lightweight concrete	New section added for lightweight aggregate concrete diagonal tension control. Old code did not specify this parameter.
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 of 150°F, or 200°F in localized areas not insulated from the concrete	Previous codes did not address the problem of long periods of exposure to high temperature and did not provide for reduction in design allowables to account for strength reduction at high (>150°F) temperatures.

ACI 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Subsection				
ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
7.5.5.1	805(d)	1103 (c) (3)	Welded splices	Welded splice requirement is more conservative as the 56 Code only required splices in compression to develop 100% of yield. Design allowables were reasonably below yield. This is not considered critical.
7.5, 7.6, & 7.8	805	506, 1002(d), 1103(c)	Members with spliced reinforcing steel	Sections on splicing and tie requirements amplified to better control strength at splice locations and provide ductility.
7.8.1 & 7.8.2	805(f)	--	Elements which used welded wire fabric as main reinforcement	This type of reinforcement not generally used in large structures and main structural elements; therefore, not considered a problem.
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.

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Scale B (Cont.)

Referenced Subsection ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
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 thicknesses in nuclear-related structures.
8.6	--	--	Continuous nonprestressed flexural members.	Allowance for redistribution of negative moments has been redefined as a function of the steel percentage.
9.2	1504(b)	--	All	Concept of a capacity reduction factor ϕ applied to the ultimate strength equations is new. This in a way replaces the old code use of different load factors for different structural elements.
9.3.1 & 9.3.2	1506	A604	All	Load factors have changed - also the use of different load factors for different structural elements was dropped. These changes have been offset by the introduction of the capacity reduction factor; therefore, overall effect not critical.

ACI 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Subsection ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
9.4	1505	A603(c)	Reinforcing steel - design strength limitation	See comments in Chapter 3 summary.
9.5.1.1	--	--	Reinforced concrete members subject to bending - deflection limits	Allows for more stringent controls on deflection in special cases.
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	New section on control of deflections needed because of use of new high strength steels and concrete. Will, generally, not be a problem in structures carrying heavy loads as minimum. thickness would not control.
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 strength requires more attention to deflection controls.

ACI 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Subsection				
ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
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 prestressed	Limits on axial design load for these members given in terms of design equations. See case study 2
10.3.6	Chapter 19	A600	Columns	The introduction of the capacity reduction factor ϕ viewed alone would significantly effect the ultimate design code results; however, the introduction of lower load factors at the same time minimizes the effect. Sample calculations show reasonable parity between safety margins with the older code being generally more conservative.

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ACI 318-56 VS. ACI 349-76

SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Subsection				
ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
10.6.1 through 10.6.4	1508	A604(a)	Beams and one-way slabs	Changes in distribution of reinforcement for crack control.
10.6.5	--	--	Beams	New insert
10.7	910	--	Deep beams	Older code did not address "deep beams" as a specific case.
10.11	916	1107	Long columns	For long columns, h/t limit removed and a new strength reduction logic, which includes factors such as resistance to lateral displacement of the ends and mode of curvature in the formulation, replaces load reduction based on h/t. The old code designs were generally conservative and long slender columns were not allowed.
10.11.1 through 10.11.7 & 10.12	915 & 916	1107	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.

ACI 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Subsection: ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
--	1102(c)	--	Flexural elements which contain compression steel	New requirements defined for computing the compression steel contribution to the transformed area. This was to account for stress increase which results from creep. Will not be significant where design dead load is not a large part of the design load.
10.15.1 through 10.15.6	1404 through 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.
--	1407	1109	Columns	Both codes use interaction logic; however, new code working stress interaction diagram is derived from the ultimate strength diagram. The definition of the tension controlled region changes since balanced eccentricity is the new limit as opposed to the old "Kern" definition.

ACI 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Subsection				
ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
-- (Cont.)				Comparison is complex but in general it is probable that the old code is more conservative.
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.
11.3	Chapter 17	--	All	This chapter is completely new; previous codes did not contain ultimate strength design criteria for shear and diagonal tension.
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

ACI 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Subsection				
ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
11.7 through 11.8.6 (Cont.)				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.
11.9 through 11.9.6	--	--	Deep beams	Special provisions for shear stresses in deep beams are 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.

ACI 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Subsection ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
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 shearhead reinforcement is used. Logic consistent with ACI 318-63 for these conditions and change is not considered major.
--	1207	808-809	Slabs and footings	Shear stress logic for working stress design in ACI 318-63 was developed by applying a factor of 2 to the ultimate strength logic. In slabs and footings, the critical section for shear was defined at a distance d/2 (not d) from the face of the support or column. Allowable stresses in the new code are larger; however, overall differences are not great in the final design.
--	2101(e)(2)	--	Slabs	New section added to give a specific method of defining the effect of a slab opening on the critical section around a column.

Scale B (Cont.)

Referenced Subsection				
ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
--	1604	--	Members with nonsymmetrical cross sections	Old code did not address this problem. Old designs generally done by very conservative assumptions.
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.

ACI 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Subsection				
ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
11.12	--	--	Openings in slabs and footings	Modification for inclusion of shearhead design. See above conclusion.
11.13.1 & 11.13.2	--	--	Columns	No problem anticipated since previous code required design consideration by some analysis.
Chap. 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 12.1.6.3	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.

ACI 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Franklin Research Center
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Scale B (Cont.)

Referenced Subsection ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
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.
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.
12.11	918	--	Beams	Tensile steel cut off conditions are new. Older design practice did not terminate bars in high tension zones and generally bent up bars where not needed.
12.13.1.4	--	--	Wire fabric	New insert. Use of this material

ACI 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Subsection ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
12.13.1.4 (Cont.)				for stirrups not likely in heavy members of a nuclear plant.
13.2.4	2102(g)	--	Slabs	New section added to ensure moment transfer between supports and the slab.
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.
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.

Scale B (Cont.)

Referenced Subsection				
ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
15.5	2305(d)	1205(e)	Footings	Removal of the 85% shear used to compute tensile reinforcement bond in two-way reinforced footings; now 100% shear is required.
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 makes design computations easier.
18.4.1	--	--	Concrete immediately after prestress transfer	Change allows more tension, thus is less conservative but not considered a problem.

ACI 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

Referenced Subsection ACI 349-76	ACI 318-63	ACI 318-56	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 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale C

Referenced Subsection				
ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
7.13.4	--	--	Reinforcement in flexural slabs	
Chapter 7	2408, 2409 and 2410	--	Precast elements	New sections identify special conditions allowed by new code as exceptions to the general code provisions. Old code required precase elements to meet all Code provisions.
10.3.6	1403(a)	1104(a)	Tied columns	New code allows more load to be carried on tied columns, i.e., 85% as compared to 80% factor in old code. Also new code allows a higher % of steel to be used in tied columns. This is less conservative than the old code.
10.8.1 10.8.2 10.8.3	912	1101	Compression members, limiting dimensions	Minimum size limitations are deleted in newer code giving the designer more freedom in cross-sectional dimensioning.

ACI 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale C

Referenced Subsection ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
--	1502(d)	--	Continuous beams	New Code allows for moment redistribution where sufficient ductility exists. Old designs produce steel % on the order of 0.4 p_b; therefore, ductility was there.
10.14	2306	1206	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	805 & 806	Reinforcement concrete members without prestressing	Allowance of spirals as shear reinforcement is new. Requirement of 2 lines of web reinforcment, where shear stress exceeds $6\phi\sqrt{f'_c}$, 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.

ACI 318-56 VS. ACI 349-76
SUMMARY OF CODE COMPARISON

Scale C (Cont.)

Referenced Subsection				
ACI 349-76	ACI 318-63	ACI 318-56	Structural Elements Potentially Affected	Comments
15.6	2306(b)	1206(b)	Columns	New code requires only transfer of actual stress carried by the column longitudinal bars. Old code required transfer of full working value. Older code more conservative.
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

SECTION VIII, 1956 VS. SECTION III, SUBSECTION NE, 1980

SUMMARY OF CODE COMPARISONS

Franklin Research Center
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ASME B&PV CODE COMPARISON
SECTION VIII, 1956 VS. SECTION III, SUBSECTION NE, 1980

Scale A

Referenced Subsection				
Section III 1980	Section VIII 1962	Section VIII 1956	Structural Elements Potentially Affected	Comments
NE-2124(b)	UG-16(c)	UG-16(c)	Pressure retaining plates less than 0.167 in thick	The 1956 Code granted a blanket 0.010 in mill undertolerance on all plate. The present code allows 6% or 0.010 inch undertolerance (whichever is least).
NE-3111	UG-22	UG-22	Load-carrying components	Section III, 1980 Code, specifies additional loads to be considered in designing the vessel. These include: o dynamic head of liquids o snow loads and vibration loads o reaction to steam and water jet impingement
NE-3112.2	---	UG-20	Vessel and components	The effects of internal heat generation due to radiation (in addition to all external sources) must be included in establishing design temperature.
NE-3112.3	---	---	Vessel and components	Currently, the design load combination includes mechanical loads. In 1956, the code considered pressure at temperature only.

ASME B&PV CODE COMPARISON
SECTION VIII, 1956 VS. SECTION III, SUBSECTION NE, 1980

Scale A (Cont.)

Referenced Subsection				
Section III 1980	Section VIII 1962	Section VIII 1956	Structural Elements Potentially Affected	Comments
NE-3112.4	---	UG-5(b)	Plates, if understrength	The 1956 Code permits conditional use of understrength plate, if: 1. The local allowable stress is correspondingly reduced; and 2. The UTS range is maintained. This practice has been terminated and current codes are blind to such situations in older structures. Scale A - if additional loads, not originally designed for, are required by current criteria. No scale ranking applicable otherwise.
NE-3112.4	UG-23	UG-23	Vessels of materials no longer listed as Code acceptable	Section III, 1980 Code, references some materials which are identical to those referenced in Section VIII, 1956 Code. However, several materials which were referenced in Section VIII, are no longer listed in Section III, 1980. Justification of such use would be necessary to show equivalence to current requirements.

ASME B&PV CODE COMPARISON
SECTION VIII, 1956 VS. SECTION III, SUBSECTION NE, 1980

Scale A (Cont.)

Referenced Subsection				
Section III 1980	Section VIII 1962	Section VIII 1956	Structural Elements Potentially Affected	Comments
NE-3131	---	Various Paragraphs	Containment shells designed by formula	Section VIII, 1962 Code, calls for the design of vessels by formula, while Section III, 1980 Code requires that the rules of Subsection NE-3200 (Design by Analysis) be satisfied. In the absence of substantial thermal or mechnical loads other than pressure, the rules of "Design by Formula" may still be used. The scale rating for containment shells where substantial thermal or mechnical loads other than pressure are absent is Scale B; otherwise it is Scale A.
---	UG-25(d)	UG-25(d) UW-15(b)	Vessels containing telltale holes	The 1956 Code required telltale holes at reinforcing plates and saddles at nozzles to be left open. The 1962 Code permitted plugging. The removal of these provisions from Section III, 1962 Code, bans the use of telltale holes. Moreover, the more recent version of Section VIII specifically excludes using telltale holes for lethal substances.

Scale A (Cont.)

Referenced Subsection				
Section III 1980	Section VIII 1962	Section VIII 1956	Structural Elements Potentially Affected	Comments
NE-3133.5(a)	UG-29	UG-29	Stiffening rings for cylindrical shells subject to buckling loads.	The requirements of the 1980 Code defining the minimum moment of inertia for stiffening rings as compared to the requirements of the 1956 Code may result in a lower margin of safety. Scale $I_s' > 1.28\ I_s$ C $I_s' > 1.22\ I_s$ B $I_s' < 1.22\ I_s$ A
NE-3133.5(b)	---	---	Shell and stiffening rings of different materials.	This new insert in Section III of the 1980 Code requires using the material chart which gives the larger value of the factor A. This may result in a larger stiffening ring section needed to meet the requirements of the Code. Scale A for ring-stiffened shells where (1) the ring and the shell are of different materials and, in addition, (2) the "factor A" (as computed by the procedures of NE-3133.5) for the two materials differs by more than 6%; otherwise Scale B.

ASME B&PV CODE COMPARISON
SECTION VIII, 1956 VS. SECTION III, SUBSECTION NE, 1980

Scale A (Cont.)

Referenced Subsection				
Section III 1980	Section VIII 1962	Section VIII 1956	Structural Elements Potentially Affected	Comments
NE-3221.5	---	---	Containment components subject to cyclic loadings.	Requirements for fatigue analysis of vessels or parts which experience cyclic loadings are provided in Section III, Subsection NE, of the 1980 Code. No specific guidance was provided by Section VIII, 1956.
NE-3324.3	UG-27(c)	UG-27(c)	Vessel components where welding efficiency of circumferential joints is less than half the longitudinal joint efficiency	The 1956 Code did not require computation of axial stress in cylindrical shells. The wide disparity in welding efficiencies is deemed improbable.
NE-3325	UG-34(c)3	UG-34(c)3	Heads, covers, or blind flanges on non-circular shape	The 1956 Code did not distinguish between circular and non-circular plates when specifying plate thickness. These thicknesses are now regarded as inadequate for most non-circular plates.
NE-3325 Figs. (c) and (m)	UG34(d) Figs. (d) and (p)	UG-34(d) Figs. (b) and (a)	Unstayed flat heads and covers of the designs in the referenced figures	Present code requires thicker plates.

ASME B&PV CODE COMPARISON
SECTION VIII, 1956 VS. SECTION III, SUBSECTION NE, 1980

Scale A (Cont.)

Referenced Subsection				
Section III 1980	Section VIII 1962	Section VIII 1956	Structural Elements Potentially Affected	Comments
NE-3327	UG-35	Footnote to UG-35	Quick-acting closures	Subsection NE, 1980 has expanded requirements for safety devices including: o positive interlocks on remotely operated doors o warning devices on manually operated doors o visibility of pressure indicators from operating floor.
NE-3334.1 NE-3334.2	UG-40(b) UG-40(c)	UG-40	Reinforcement for vessel openings	New requirements in the 1980 Code impose additional restrictions on metal that may be counted as reinforcement.
NE-3365	---	--	Bellows and bellows expansion joints	The 1980 Code imposes new design requirements.

ASME B&PV CODE COMPARISON

SECTION VIII, 1956 VS. SECTION III, SUBSECTION NE, 1980

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

Scale B

Referenced Subsection				
Section III 1980	Section VIII 1962	Section VIII 1956	Structural Elements Potentially Affected	Comments
NE-3133.1 & NE-3133.6	UG-28	UG-28 & UG-29	Components under external pressure and axial compression	The curves associated with the buckling of short cylinders appear to have been replotted to slightly different values.
NE-3324.8(c)	---	---	Torisphereical heads made of materials having minimum tensile strength exceeding 80 ksi	The allowable stress is restricted to values less than 22 ksi at room temperature by the the 1980 Code. Allowable stresses for some plate materials specified in the 1956 Code are slightly higher.
NE-3325 No figure	UG-34(d) Fig. (s)	UG-34(d) Fig. (m)	Unstayed flat heads and covers secured by spinning	Not a code-recommended practice for Section III vessels.
NE-3328	---	---	Combinations units	This new insert gives the design requirements for pressure vessels consisting of more than one independent pressure

Scale B (Cont.)

Referenced Subsection				
Section III 1980	Section VIII 1962	Section VIII 1956	Structural Elements Potentially Affected	Comments
NE-3328 (cont.)				chamber. These requirements are standard practice for designing such vessels.
NE-3335	UG-40	UG-45	Reinforcement in nozzles and vessel walls	These new provisions of Section III, 1980 Code, detail specific requirements which are usually considered in good design practice.
NE-3336	UG-41(a)	---	Reinforcement for openings where welding is counted as reinforcement	The 1962 Code has provision that weld strength be taken as that of the weaker of the metal joined.
NE-3700	---	---	Electrical and mechanical penetration assemblies	Provisions usually adopted in standard engineering design of such assemblies.
NE-4120	UG-11(c)	---	Welded pressure parts other than the vessel shell	Documentation of code acceptability of welding practices as presently required by 1980 Code was not code enforced in 1956 and may not be available.

ASME B&PV CODE COMPARISON
SECTION VIII, 1956 VS. SECTION III, SUBSECTION NE, 1980

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

Scale B (Cont.)

Referenced Subsection				
Section III 1980	Section VIII 1962	Section VIII 1956	Structural Elements Potentially Affected	Comments
NE-4232.1	UG-36(d)5	---	Reducers	Restriction on alignment of joints introduced. Local bending moments could be induced if offset joints were controlled.
NA-3767.4(a)2	UG-85	---	Heat-treated components	Requirements for written documentation of heat-treatment process were not provided by the 1956 Code.
NE-6000	UG-101	---	Vessel shell and other pressure retaining parts	The code has expanded the methods for, and exerts greater control over, the acceptable methods of proof testing.

ASME B&PV CODE COMPARISON
SECTION VIII, 1956 VS. SECTION III, SUBSECTION NE, 1980

Scale C

Referenced Subsection				
Section III 1980	Section VIII 1962	Section VIII 1956	Structural Elements Potentially Affected	Comments
NE-3332.2	UG-37(b)	UG-37(b)	Area of reinforcement - vessels under internal pressure	The 1980 Code includes a correction factor, F, in the equation for required area for reinforcement. This area is the same or less than the uncorrected equation required.
NE-3325 Figs. (a), (b), & (f)	UG-34(d) Figs. (a), (c), & (g)	UG-34(d) Figs. (d), (c), & (f)	Well-proportioned flat heads or covers of circular shape of the configurations shown in the referenced figures.	Thinner heads for these designs are now code acceptable.
NE-3362(b)	UG-43	UG-43	Studded connections	These paragraphs (although addressing different issues) provide rules for minimum depth of studs in general. The minimum engagement length can be less under the 1980 Code.

APPENDIX C

COMPARATIVE EVALUATIONS AND MODEL STUDIES

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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-

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

FY,PSI	FU,PSI	H,IN	C1	C2	ALLOWABLE LOAD,LB 1963 CODE	ALLOWABLE LOAD,LB 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

AXIALLY LOADED COLUMNS

Maximum allowable axial load on tied columns by working stress design criteria is defined by

$$P = 0.85\ [A_g\ (0.25\ f'_c + f_s\ p_g)]$$

where $p_g = \dfrac{A_{st}}{A_g}$ and allowable $f_s = 0.4 f_y \leq$ 30,000 psi

that is, max $f_y \leq$ 75,000 psi

therefore, the maximum load could be expressed as:

$$P_{allow} = (0.21\ A_g\ f'_c\ + 0.34\ f_y\ A_{st})$$

Maximum allowable axial load on tied columns by strength design criteria is defined by

$$P_{allow} = \phi P_o = \phi 0.8\ [0.85\ f'_c\ (A_g - A_{st}) + A_{st}\ f_y]$$

for a tied column in axial compression $\phi = 0.7$ and $P_u = 1.4\ D + 1.7\ L$

Reducing these equations to be comparable to working stress limits and considering all extremes of steel % and D. to L. load ratios, we get

if $A_{st} = 0.01\ A_g \quad P_u = \phi P_o = \phi\ (0.673\ f'_c\ A_g + 0.8\ A_{st}\ f_y)$

if $A_{st} = 0.08\ A_g \quad P_u = \phi P_o = \phi\ (0.626\ f'_c\ A_g + 0.8\ A_{st}\ f_y)$

and to bracket extremes, consider the following three cases.

(a) $D = 0$

(b) $L = D$ and

(c) $L = 0$ with $P_{allow} = \dfrac{P_u}{L.F.}$

(a) for L.F. = 1.7

$$P_{allow} = 0.28\ f_c'\ A_g + 0.33\ f_y\ A_{st} \quad \text{or}$$

$$P_{allow} = 0.26\ f_c'\ A_g + 0.33\ f_y\ A_{st}$$

(b) for L.F. = 1.55

$$P_{allow} = 0.30\ f_c'\ A_g + 0.36\ f_y\ A_{st} \quad \text{or}$$

$$P_{allow} = 0.28\ f_c'\ A_g + 0.36\ f_y\ A_{st}$$

(c) for L.F. = 1.4

$$P_{allow} = 0.34\ f_c'\ A_g + 0.40\ f_y\ A_{st} \quad \text{or}$$

$$P_{allow} = 0.31\ f_c'\ A_g + 0.40\ f_y\ A_{st}$$

Comparison of these resulting equations to the P_{allow} by working stress design criteria shows that the new code allows from 1.24 to 1.62 times more load on the concrete in a tied column and from 0.97 to 1.18 times more load on the longitudinal steel in a tied column.

Therefore, Scale C

CASE STUDY 3

FLEXURAL MEMBERS

Sections with Tension Reinforcing Only:

For purposes of code comparison, with emphasis on comparing safety margins of designs conforming to older codes and practices with corresponding margins provided by current criteria, the following case studies were prepared.

For designs prepared by working stress criteria, a comparison with strength design was made by reducing the strength equation to an allowable moment by the following definition.

$$M_{allow} = \frac{\phi M_u}{L.F.}$$

To bracket extremes of load ratios, the following three cases were considered in each working stress comparison.

(a) when L = 0 L.F. = 1.4

(b) when L = D L.F. = 1.55

(c) when D = 0 L.F. = 1.7

For designs prepared by yield-strength criteria, a comparison with strength design was made directly with a load factor equal to 1.0. The yield-strength definition used here was not a code endorsed practice; but was the method widely adopted by architect engineers, at the time, to design for the extreme loadings postulated for accident and faulted conditions. It possesses the practical advantage of permitting an extended use of linearly elastic computer codes to provide design guidance for extreme loading cases and is documented in Ref. 1*

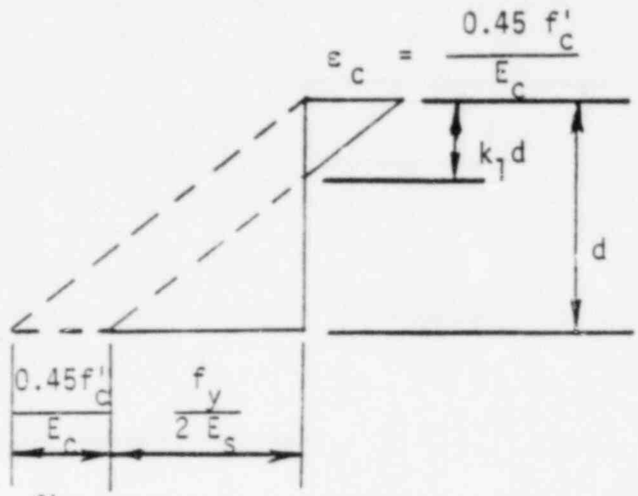
Since older codes did not contain any strict limitation on the percent of reinforcement, the comparisons presented here used the defined balanced steel percentage and additionally steel percentages 60 percent lower and 50 percent higher than balanced in order to show the effect of this parameter on the comparisons.

*Ref. 1
A Study of the Design and Construction Practices of Prestressed Concrete and Reinforced Concrete Containment Vessels by C. P. Tan prepared by FIRL for the U. S. Atomic Energy Commission, Aug. 1969 under contract to the ORNL (TID 25176).

For Working Stress Design

The definition of balanced design is that both concrete and steel reach their theoretical working stress allowable limit simultaneously.

The strain diagram and neutral axis location for this condition are:



$$\frac{k_1 d}{d} = \frac{0.45\, \frac{f'_c}{E_c}}{\frac{0.45\, f'_c}{E_c} + \frac{f_y}{2\, E_s}} \qquad k_1 = \frac{1}{1 + \left(\frac{1}{0.9}\right)\left(\frac{f_y}{f'_c}\right)\left(\frac{E_c}{E_s}\right)}$$

let $r = \frac{f_y}{f'_c}$ and $n = \frac{E_s}{E_c}$

then for elastic balanced design:

$$k_1 = \frac{1}{1 + 1.11\left(\frac{r}{n}\right)}$$

and from equilibrium:

$$\frac{F_s}{F_c} = \frac{A_s\, f_y}{(0.45\, f'_c)\; b\; k\; d} = \left(\frac{A_s}{bd}\right)\left(\frac{f_y}{f'_c}\right)\frac{1}{0.45k} = 1$$

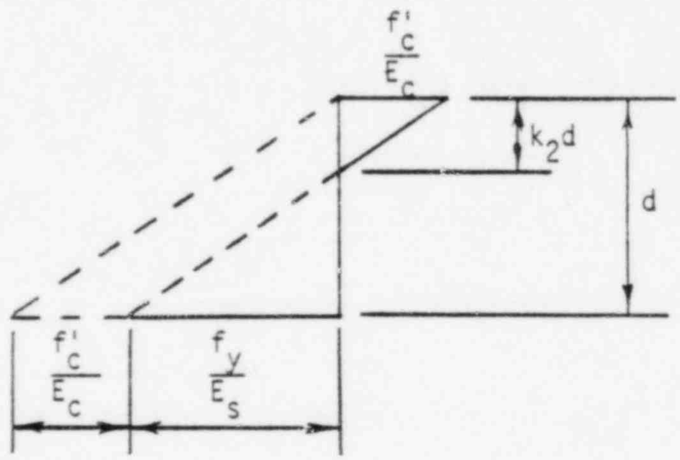
$$\rho_1 = \frac{A_s}{bd} = 0.45 \frac{k_1}{r}$$

$$M_t = f_s A_s jd \quad \text{or } M_c = 1/2\ f_c bd^2 j\ k$$

For Yield-Limit Design

The Yield-Limit concept assumes that the system behaves in a linear fashion up to the yield of the steel or to the ultimate strength of the concrete. For the balanced condition again $f_s = f_y$ and $f_c = f'_c$ simultaneously.

The strain diagram and neutral axis location for this condition are:



$$\frac{k_2 d}{d} = \frac{\dfrac{f'_c}{E_c}}{\dfrac{f'_c}{E_c} + \dfrac{f_y}{E_s}} \qquad k_2 = \frac{1}{1 + \dfrac{f_y}{f'_c}\dfrac{E_c}{E_s}}$$

then for balanced conditions

$$k_2 = \frac{1}{1 + \left(\dfrac{r}{n}\right)}$$

and from equilibrium

$$\frac{F_s}{F_c} = \frac{A_s f_y}{1/2(f'_c)\ bkd} = 2\ \rho_2 \frac{r}{k_2} = 1$$

$$\rho_2 = \frac{k_2}{2r}$$

$$M_t = f_y A_s jd \quad \text{or } M_c = 1/2\ f'_c\ bd^2 jk$$

For Strength Design

Ultimate strength capacity is defined as:

$$M_u = A_s f_y d \left[1 - 0.59\ \rho\ \frac{f_y}{f'_c}\right]$$

Example 1.

for Yield-Limit design at balanced design

$$M_t = f_y A_s jd \qquad M_c = 1/2\ f'_c\ bd^2 jk = 1/2\left(\frac{f'_c}{f_y}\right)\frac{jk}{p}\ (A_s f_y d)$$

$$k_2 = \frac{1}{1 + \frac{f_y}{f'_c}\ \frac{E_c}{E_s}} \qquad \rho_2 = 1/2\ k_2\ \frac{f'_c}{f_y} \qquad n = \frac{E_s}{E_c} = \frac{508}{\sqrt{f'_c}}$$

for $f'_c = 4{,}000$ psi $\quad f_y = 40{,}000$ psi $\quad n = 8$

$$k_2 = \frac{1}{1 + 10\ (1/8)} = 0.444 \qquad \rho_2 = 1/2\ (0.444)\ 4/40 = 0.022$$

$$j = 0.852$$

$$M_t = 0.852\ f_y A_s d$$

$$M_u = A_s f_y d\ [1 - 0.59(0.022)10] = 0.869\ A_s f_y d$$

$$\frac{M_u}{M_t} = \frac{0.869}{0.852} = 1.02$$

Also:

if $\rho < \rho_2$ (say 60% ρ_2)

$$\rho = 0.6\ (0.022) = 0.0132$$

$\alpha = 2\rho n = 2\ (0.0132)(8) = 0.211$

$$k = \left(0.211 + \frac{(0.211)^2}{4}\right)^{1/2} - \frac{0.211}{2} = 0.366$$

$j = 0.878$

$M_t = 0.878\ f_y A_s d$

$M_u = A_s f_y d\ [1-0.59\ (0.0132)10] = 0.922\ A_s f_y d$

$$\frac{M_u}{M_t} = 1.05$$

and; similarly,

if $\rho > \rho_2$ $\qquad \rho = 1.5\ \rho_2 = 1.5\ (0.022) = 0.033$

One finds M_c controls, and:

$$\frac{M_u}{M_c} = 1.26$$

For working stress design at balanced design

$k_1 = \frac{1}{1 + 1.11\ (10/8)} = 0.419$ $\qquad \rho_1 = 0.45\ \frac{0.419}{(10)} = 0.0188$

$j = 0.86$

$M_t = 0.86\ f_s A_s d = 0.86\ \frac{f_y}{2}\ A_s d = 0.43\ A_s f_y d$

$M_u = A_s f_y d\ [1-0.59(0.0188)10] = 0.889\ A_s f_y d$

$$\frac{M_u}{M_t} = 2.07$$

$$\therefore\ \frac{M_{allow}}{M_t} = \frac{\frac{0.9}{L.F.}\ M_u}{M_t} = \begin{cases} 1.33 \text{ if } L = 0 \\ 1.20 \text{ if } L = D \\ 1.09 \text{ if } D = 0 \end{cases}$$

Also:

if $\rho < \rho_1$ (say 60% ρ_2)

$\rho = 0.6\ (0.0188) = 0.0.0113$ $\alpha = 2\ pn = 0.180$

$k = (0.18 + \frac{(0.18)^2}{4})^{1/2} - \frac{0.18}{2} = 0.344$

$j = 0.885$

$M_t = 0.885 A_s f_s d = 0.885\ A_s \frac{f_y}{2} d = 0.443\ A_s f_y d$

$M_u = A_s f_y d\ [1-0.59(0.0113)10] = 0.933\ A_s f_y d$

$$\frac{M_u}{M_t} = 2.11$$

$$\therefore \quad \frac{M_{allow}}{M_t} = \begin{cases} 1.36 \text{ if } L = 0 \\ 1.22 \text{ if } L = D \\ 1.12 \text{ if } D = 0 \end{cases}$$

and:

if $\rho > \rho_1$ (say 1.5 ρ_1)

One finds concrete controls, and:

$$\frac{M_u}{M_c} = 2.43$$

$$\therefore \quad \frac{M_{allow}}{M_c} = \begin{cases} 1.56 \text{ if } L = 0 \\ 1.41 \text{ if } L = D \\ 1.29 \text{ if } D = 0 \end{cases}$$

In summary,

for yield limit design comparisons:

$$\frac{M_u}{M_t} = 1.02 \text{ to } 1.26$$

for working stress design comparisons:

$$\frac{M_{allow}}{M_t} = 1.09 \text{ to } 1.56$$

$\therefore$ Strength design allows beams to operate at a higher stress level. For these beams the older code is more conservative

Scale C

Example 2.

For Yield-Limit design at balanced design

for $f'_c = 3000$ psi $\quad f_y = 36{,}000$ psi

$$k_2 = \frac{1}{1 + (12)(1/9)} = 0.429$$

$$\rho_2 = 1/2\ (0.429)\ 1/12 = 0.0179$$

$j = 0.857$

$M_t = 0.857\ A_s f_y d$

$M_u = A_s f_y d[1-0.59(0.0179)12] = 0.873\ A_s f_y d$

$$\frac{M_u}{M_t} = 1.02$$

Also:

if $\rho < \rho_2$ (say 60%)

$$\frac{M_u}{M_t} = 1.05$$

And:

if $\rho > \rho_2$ (say $\rho = 1.5\ \rho_2 = 0.0268$)

$$\frac{M_u}{M_c} = 1.26$$

For Working Stre-s Design at balanced design

$f'_c = 3$ ksi $\qquad f_y = 36$ ksi $\qquad n = 9 \qquad \frac{f_y}{f'_c} = 12$

$k_1 = 0.403 \qquad \rho_1 = 0.0151$

$$\frac{M_u}{M_t} = 2.06$$

$$\therefore \frac{M_{allow}}{M_t} = \begin{cases} 1.32 \text{ if } L = 0 \\ 1.20 \text{ if } L = D \\ 1.09 \text{ if } D = 0 \end{cases}$$

Also:

if $\rho < \rho_1$ (say 60%)

$$\frac{M_u}{M_t} = 2.$$

$$\therefore \frac{M_{allow}}{M_t} = \begin{cases} 1.35 \text{ if } L = 0 \\ 1.22 \text{ if } L = D \\ 1.11 \text{ if } D = 0 \end{cases}$$

And:

if $\rho > \rho_1$ (say $1.5\ \rho_1$)

$$\frac{M_u}{M_c} = 2.58$$

$$\therefore \frac{M_{allow}}{M_c} = \begin{cases} 1.66 & \text{if } L = 0 \\ 1.50 & \text{if } L = D \\ 1.36 & \text{if } D = 0 \end{cases}$$

In summary,

for yield limit design comparisons:

$$\frac{M_u}{M_t} = 1.02 \text{ to } 1.26$$

for working stress design comparisons:

$$\frac{M_{allow}}{M_t} = 1.09 \text{ to } 1.66$$

$\therefore$ Strength design allows beams to operate at a higher stress level. For these beams the older code is more conservative.

Scale C

In general, for designs controlled by flexure, beams designed by strength design methods will have higher stresses at service load levels than beams designed for the same service loads by working stress design methods.

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 $\frac{\ell}{r} = 120$ 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

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

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

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

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

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

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

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

REF AISC Sub section 1.10.5.3

EXAMPLE

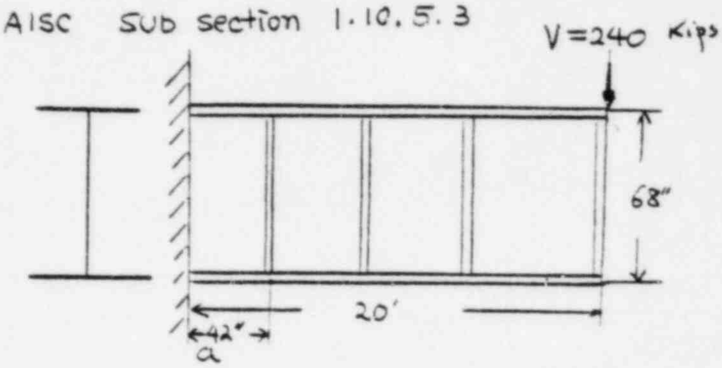
$h = 68''$

$t = .375''$

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

$V = 240$ Kips

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



from 1.10.5.3 1963 Code

$$a \text{ or } h \not> \frac{11000t}{\sqrt{f_v}} = \frac{11000 \times 3/8}{\sqrt{9.06 \times 1000}} = 43 \text{ in}$$

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

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

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

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

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

Allowable shear stress ≈ 8.6 Ksi (checks Computed Value)

however, lower than f_v of 9.06 Ksi

∴ Scale B for this example

Remarks

The following two figures show F_v vs. A/T for various values of A/H and F_y. By knowing the shear stress F_v or F_v' 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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Project			Page
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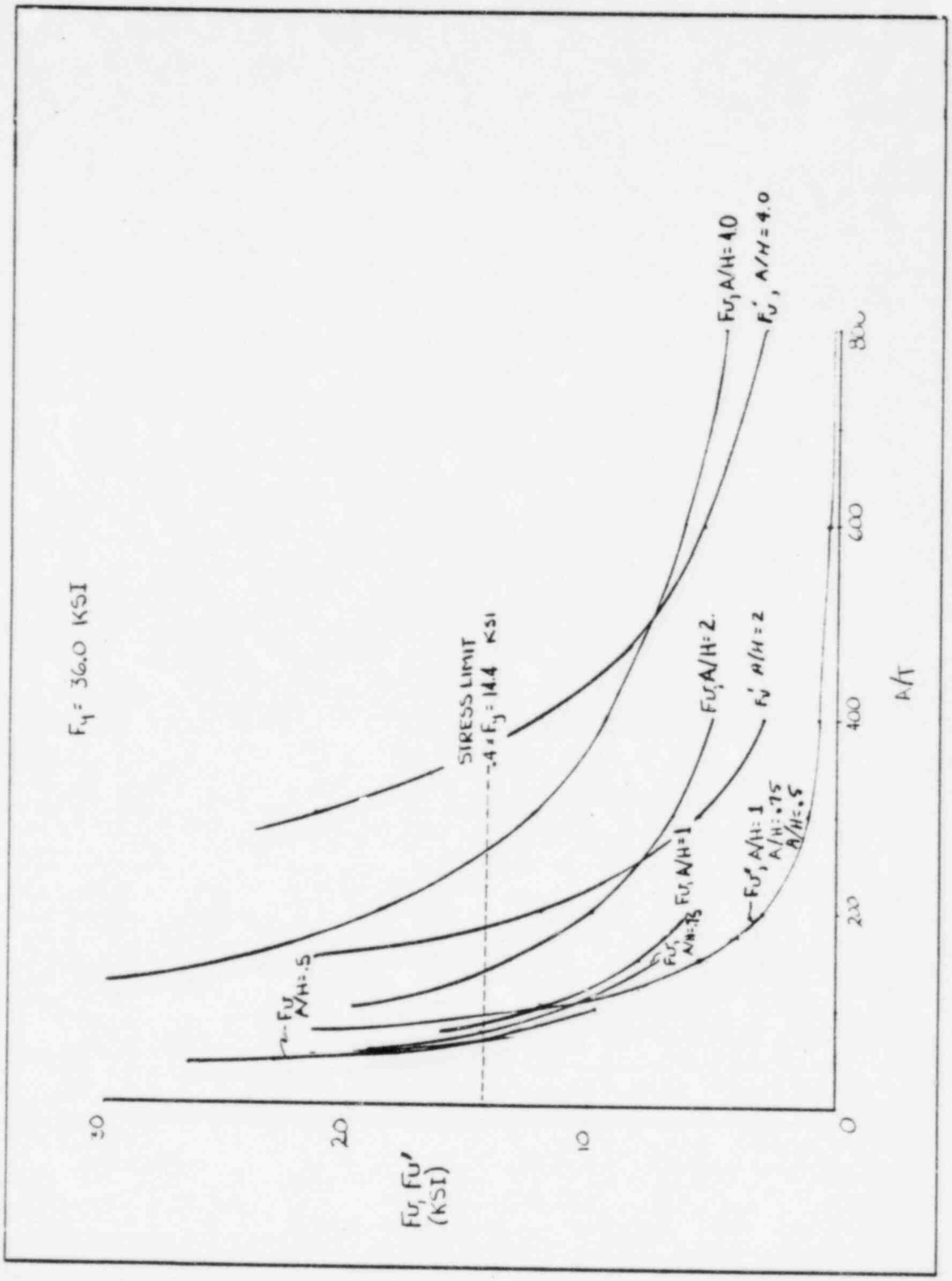

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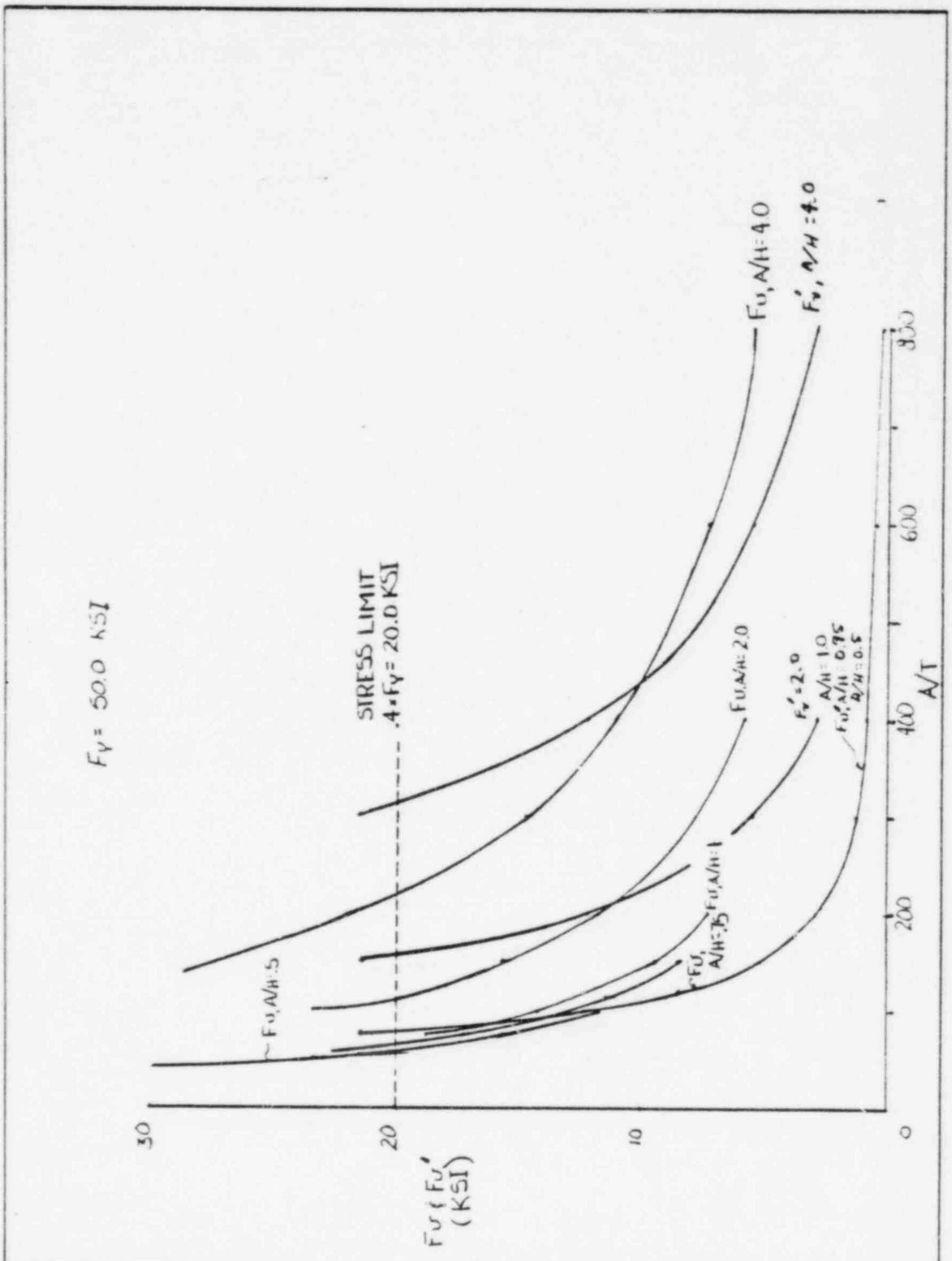


FORM 207-5M-4-80-CP

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 ...."

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

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"

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

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

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

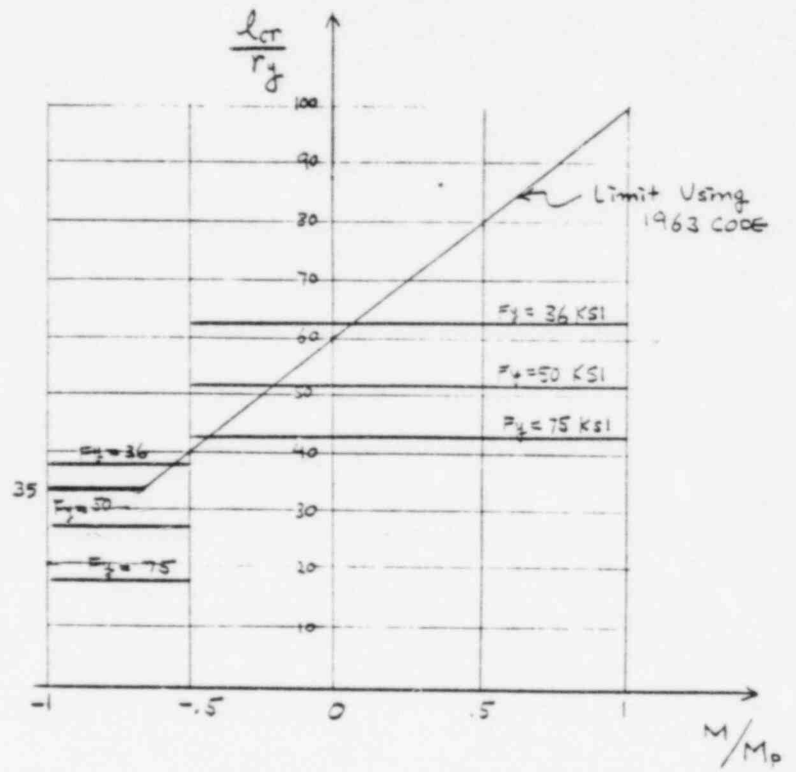
The figure which follows (l_{cr}/r_y vs. M/M_p) indicates that for A-36 steel ($F_y = 36$ ksi)

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.


$\frac{l_{cr}}{r_y}$
100
90
80
70
60
50
40
30
20
10
35
Limit Using 1963 CODE
F_y = 36 KSI
F_y = 50 KSI
F_y = 75 KSI
F_y = 36
F_y = 50
F_y = 75
-1
-.5
0
.5
1
M/M_p

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 continuous frames where sideway is not prevented, is limited by Formula (20) $$\frac{2P}{P_y} + \frac{l}{70r} \leq 1.0$$ This limits slenderness Ratio $\frac{l}{r} \leq 70$ and axial load not to exceed $0.5\,P_y$ for $\frac{l}{r} = 0$. Also limited by Formula (26) given below.	1. Slenderness ratio for Columns in continuous 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 $\frac{l}{r}$ 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, $C_c = 126.1$

3b. 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)

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

(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 ($0.6 \leq C_m \leq 1.0$) and double Curvature ($0.4 \leq C_m \leq 0.6$) 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, for all graphs

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 $\frac{Kl}{r} = 30$ the Formula (2.4-2) line for $C_m = 1.0$ is below the formula (23) line, but for $\frac{Kl}{r} = 70$, they overlap and for $\frac{Kl}{r} = 100$, the Formula (2.4-2) for $C_m = 1.0$ is above the formula (23) line. Thus for $\frac{Kl}{r} = 30$ 1980 code being more conservative, while for $\frac{Kl}{r} = 100$, 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 $\frac{kl}{r} = 30$ 14 W 150 SINGLE CURVATURE

Assume: braced in weak direction

$\therefore M_m = M_p$

1963 Code

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

$M \leq M_p$

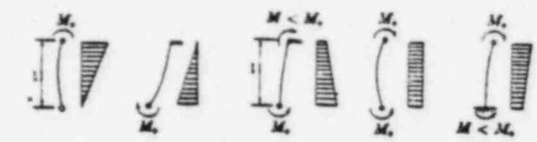
Formula (23) $\frac{M}{M_p} \leq 1.0 - H(P/Py) - J(P/Py)^2$

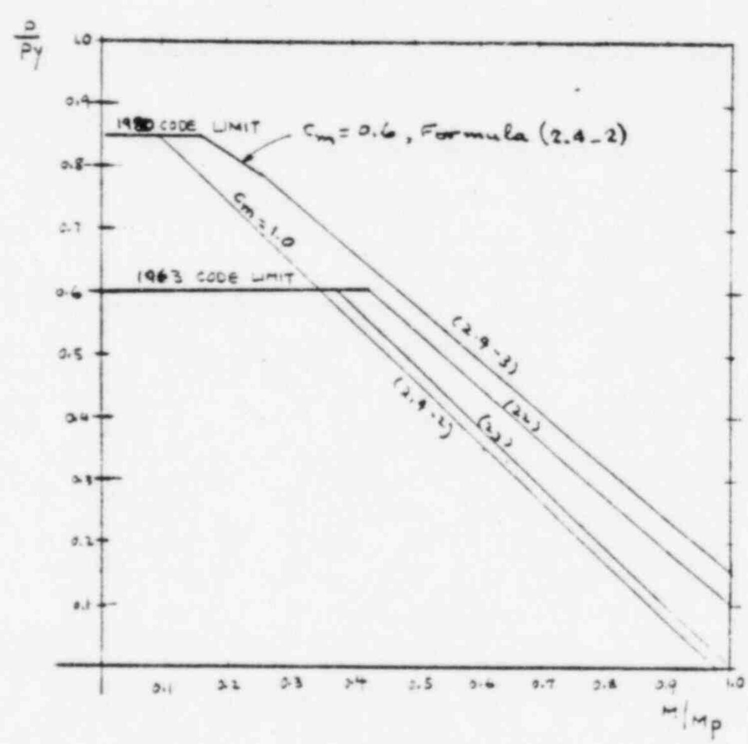
1980 Code

(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





$F_y = 36$ ksi $\quad \frac{kl}{r} = 30 \quad$ 14 ₩ 150

DOUBLE CURVATURE
Assume braced in weak direction
$\therefore M_m = M_p$

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

$$M \leq M_p$$

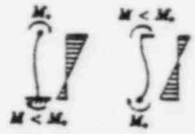
1980 Code

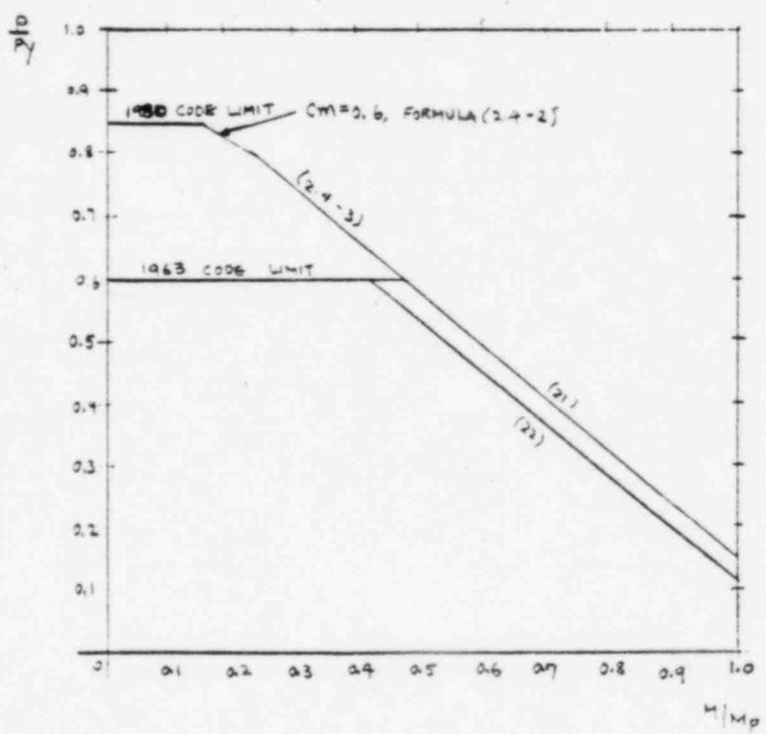
$$(2.4\text{-}2) \quad \frac{P}{P_{cr}} + \frac{C_m M}{(1 - \frac{P}{P_e})M_p} \leq 1.0$$

$$0.4 \leq C_m \leq 0.6$$

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

TYPICAL EXAMPLES





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

Assume braced in weak direction

$\therefore M_m = M_p$

<u>1963 Code</u>

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

$M \leq M_p$

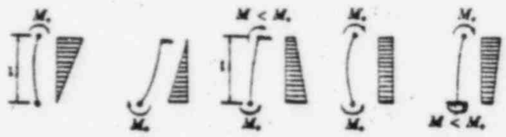
Formula (23) $\frac{M}{M_p} \leq 1.0 - H(P/Py) - J(P/Py)^2$

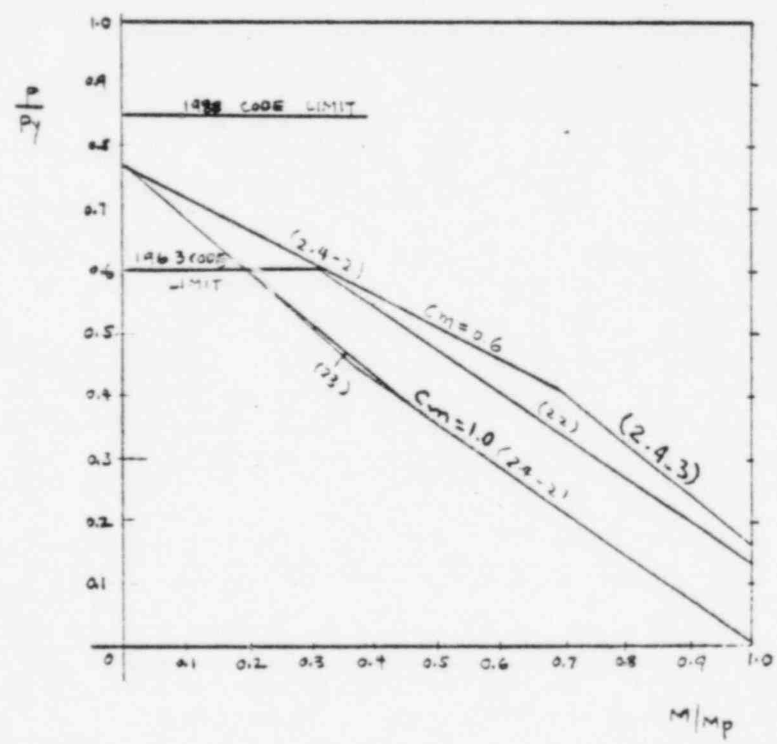
<u>1980 Code</u>

(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>





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

Assume braced in weak direction

$\therefore M_m = M_p$

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

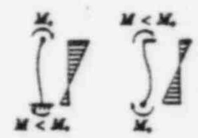
$$M \leq M_p$$

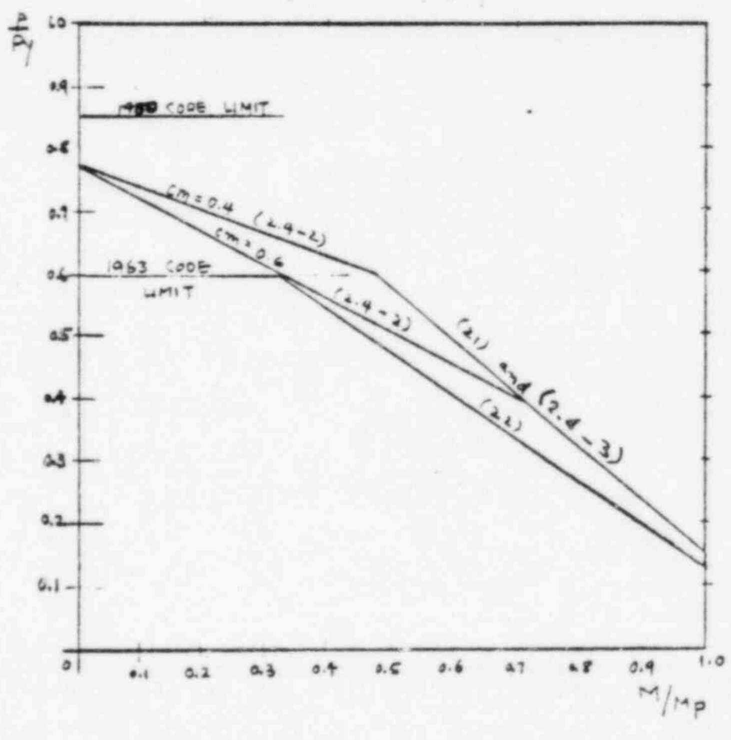
1980 Code

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

(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 $\quad \frac{kl}{r} = 100$ 14 W 150

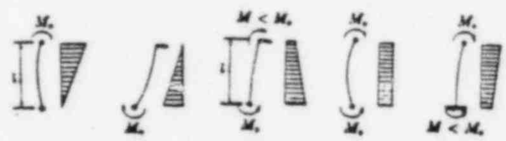
SINGLE CURVATURE

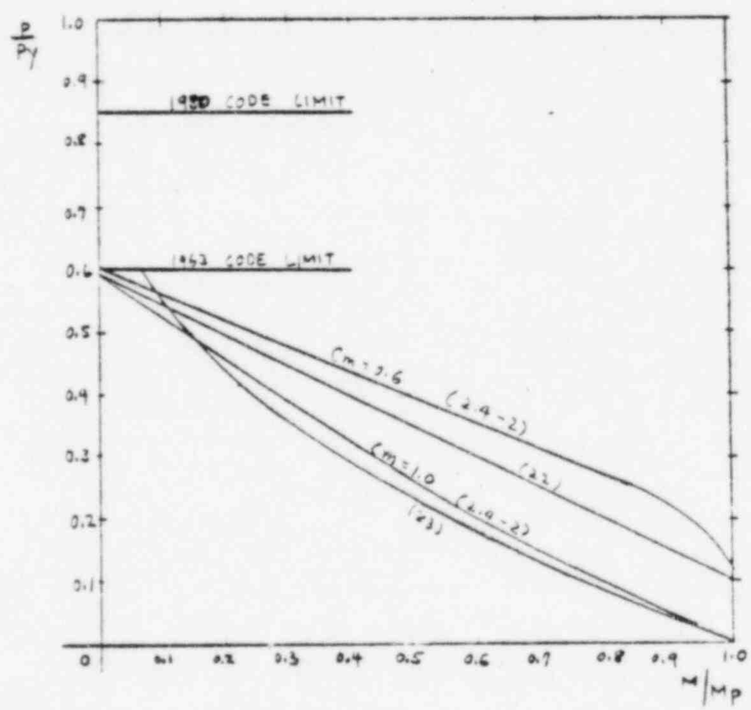
Assume braced in weak direction

$\therefore M_m = M_p$

1963 Code	1980 Code
Formula (22) $\frac{M}{M_p} \leq B - G(P/Py) \leq 1.0$, $M \leq M_p$	(2.4-2) $\frac{P}{P_{cr}} + \frac{C_m M}{(1 - \frac{P}{P_e})M_p} \leq 1.0$ $\quad 0.6 \leq C_m \leq 1.0$
Formula (23) $\frac{M}{M_p} \leq 1.0 - H(P/Py) - J(P/Py)^2$	(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 $\frac{kl}{r} = 100$ 14WF 150

DOUBLE CURVATURE
Assume braced in weak direction
$\therefore M_m = M_p$

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

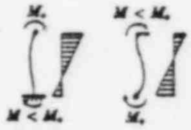
$M \le M_p$

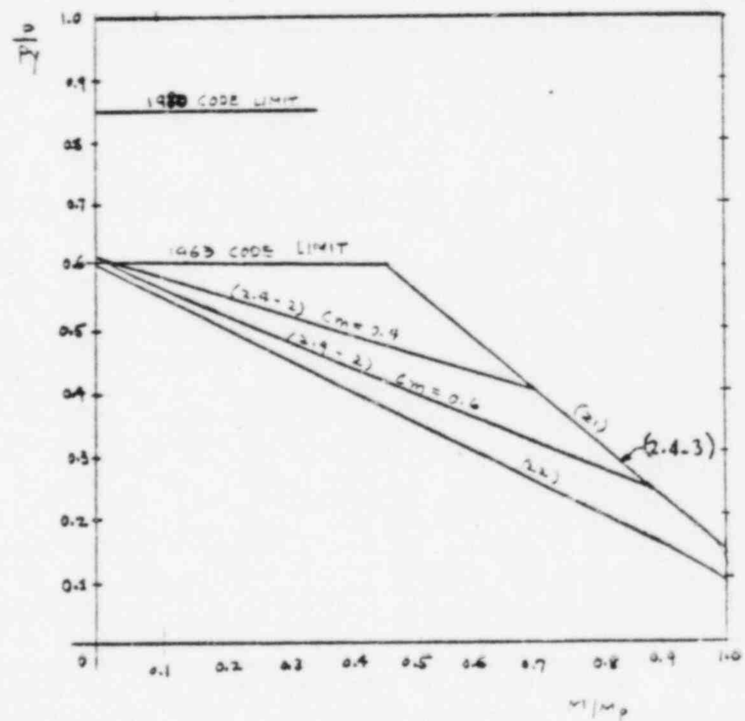
1980 Code

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

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

TYPICAL EXAMPLES





F_y = 36 ksi $\frac{kl}{r}$ = 30 12 WF45

SIDESWAY ALLOWED
Assume braced in weak direction
$\therefore M_m = M_p$

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

$M \le M_p$

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

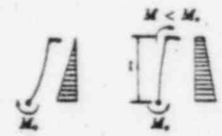
<u>1980 Code</u>

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

$C_m = 0.85$

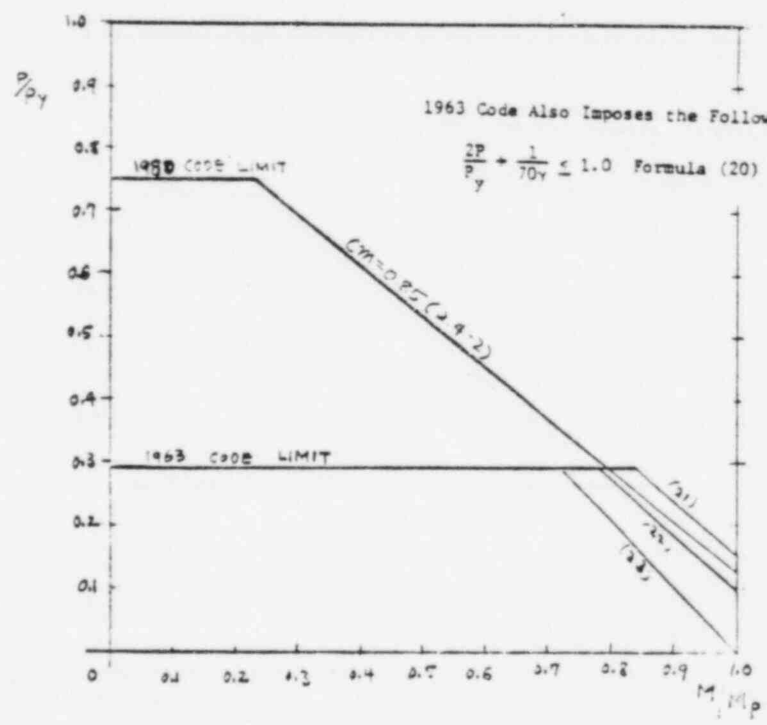
(2.4-3) $\frac{P}{P_y} + \frac{M}{1.18M_p} \le 1.0$, $M \le M_p$

<u>TYPICAL EXAMPLES</u>



1963 Code Also Imposes the Following Limit

$\frac{2P}{P_y} + \frac{1}{70r} \le 1.0$ Formula (20)



Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project C5257				Page C.8-12	
	By RA	Date SEPT'81	Ch'k'd EMd	Date 10/81	Rev.	Date

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

SIDESWAY ALLOWED
Assume braced in weak direction
$\therefore M_m = M_p$

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

$$M \le M_p$$

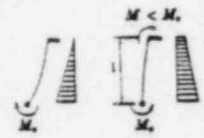
Formula (23) $\frac{M}{M_p} \le 1.0 - H(P/Py) - J(P/Py)^2$

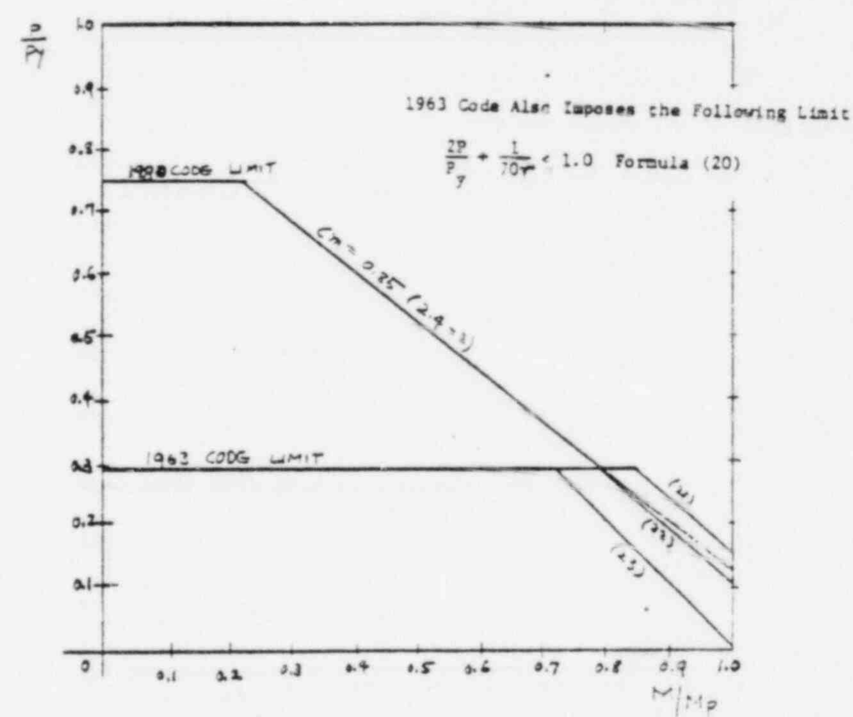
<u>1980 Code</u>

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

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

<u>TYPICAL EXAMPLES</u>





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.

OLD Formulas

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

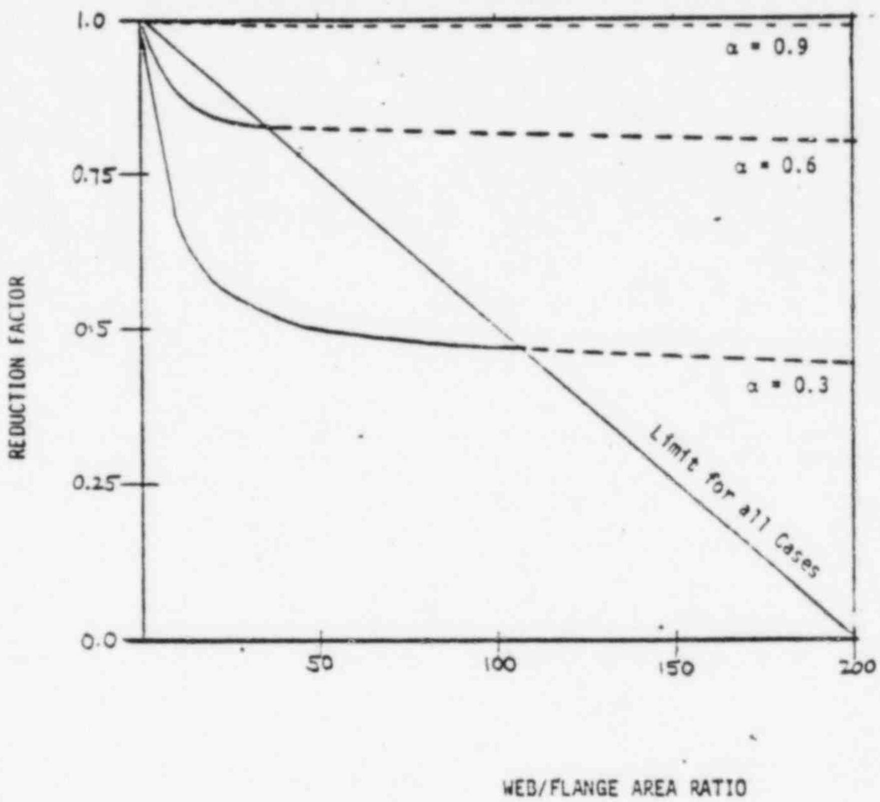
New Formula

Formula (1.10-6) 1980 code

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

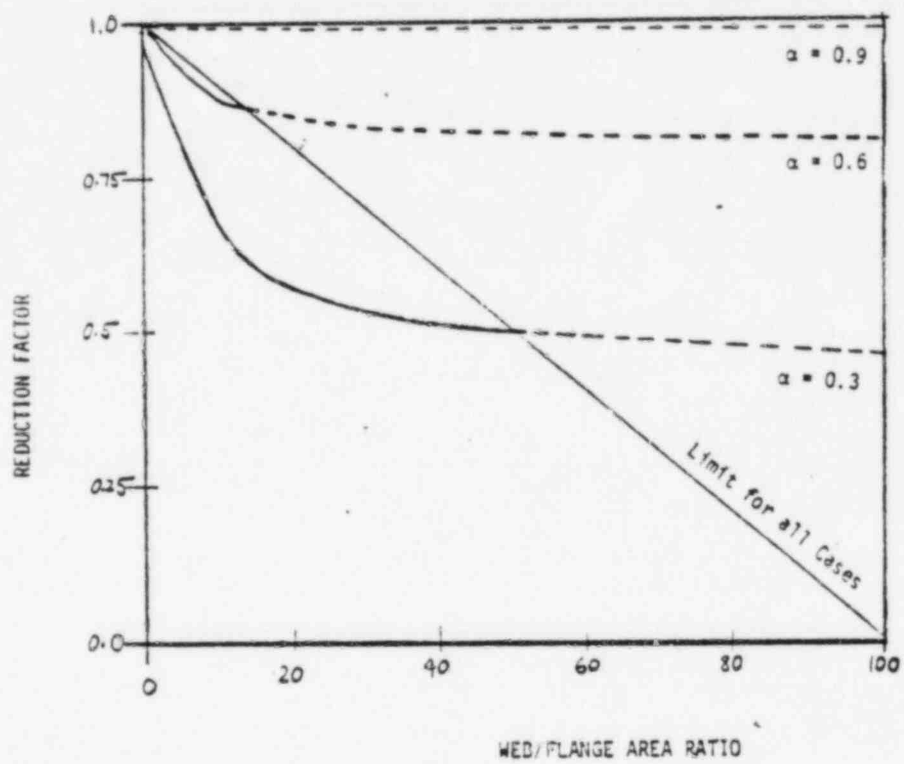
α	Scale
≤ 0.45 and low A_w/A_f ratio	A
0.45 to 0.75	B
≥ 0.75	C

AISC 1.10.6 1963/1980 CODE COMPARISON



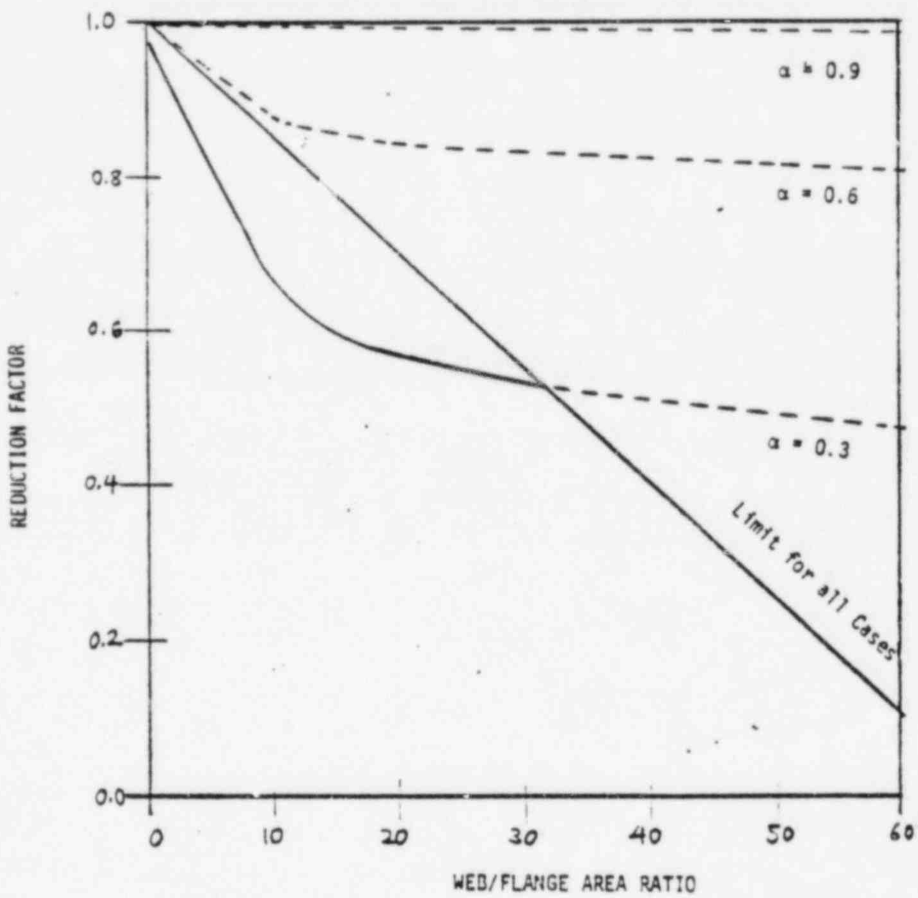
BENDING STRESS = 25KSI ALPHA=0.3, 0.6, 0.9, H/T RATIO = 162

AISC 1.10.6 1963/1980 CODE COMPARISON



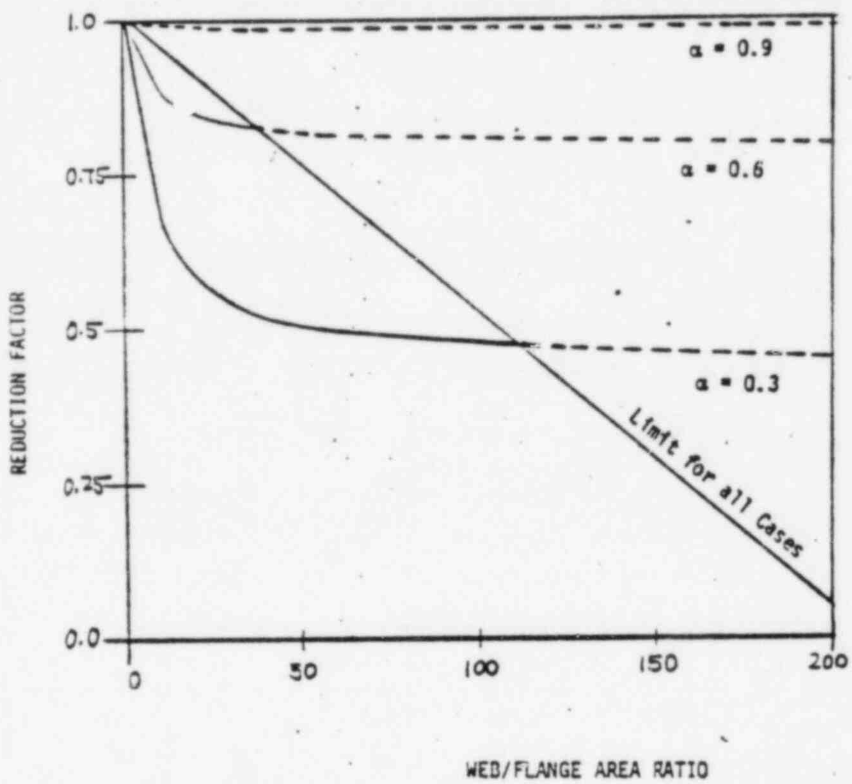
BENDING STRESS = 25KSI   ALPHA=0.3, 0.6, 0.9, H/T RATIO = 172

AISC 1.10.6 1963/1980 CODE COMPARISON



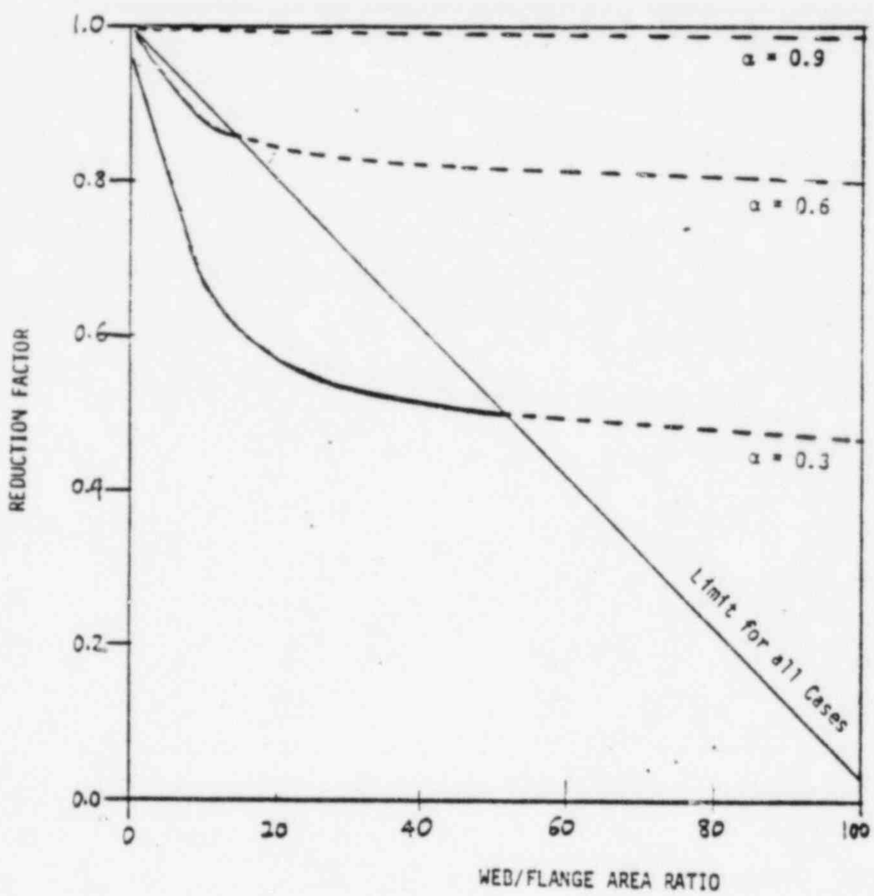
BENDING STRESS = 25KSI   ALPHA=0.3, 0.6, 0.9, H/T RATIO = 182

AISC 1.10.6 1963/1980 CODE COMPARISON



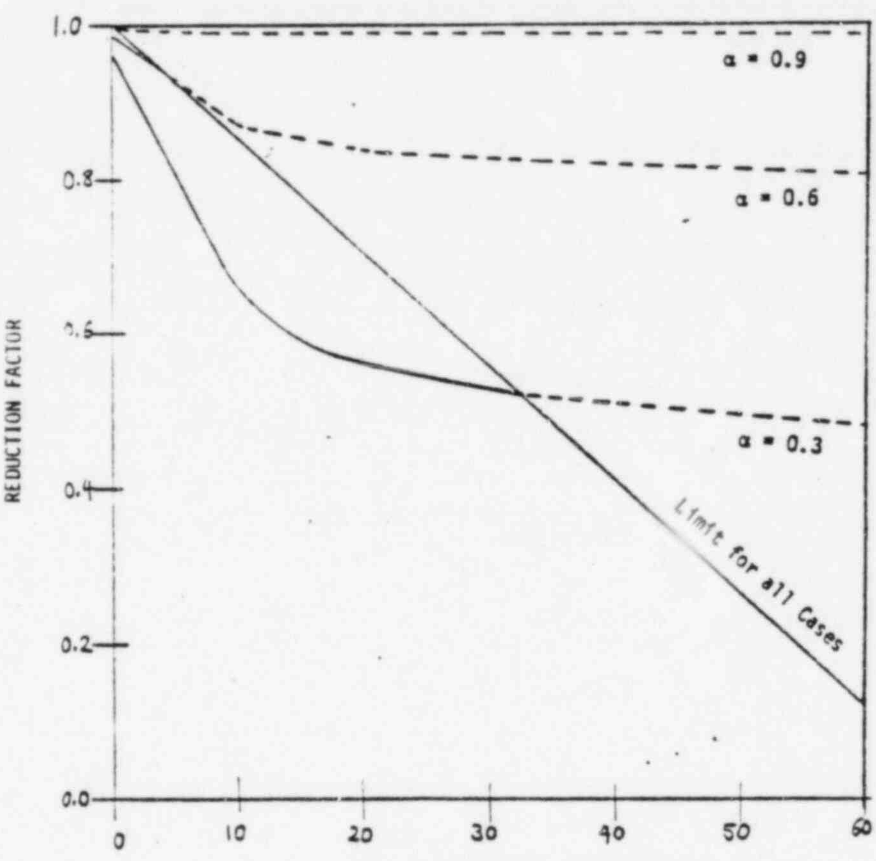
BENDING ST. ‑S = 50KSI   ALPHA=0.3, 0.6, 0.9, H/T RATIO = 117

AISC 1.10.6 1963/1980 CODE COMPARISON



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

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.

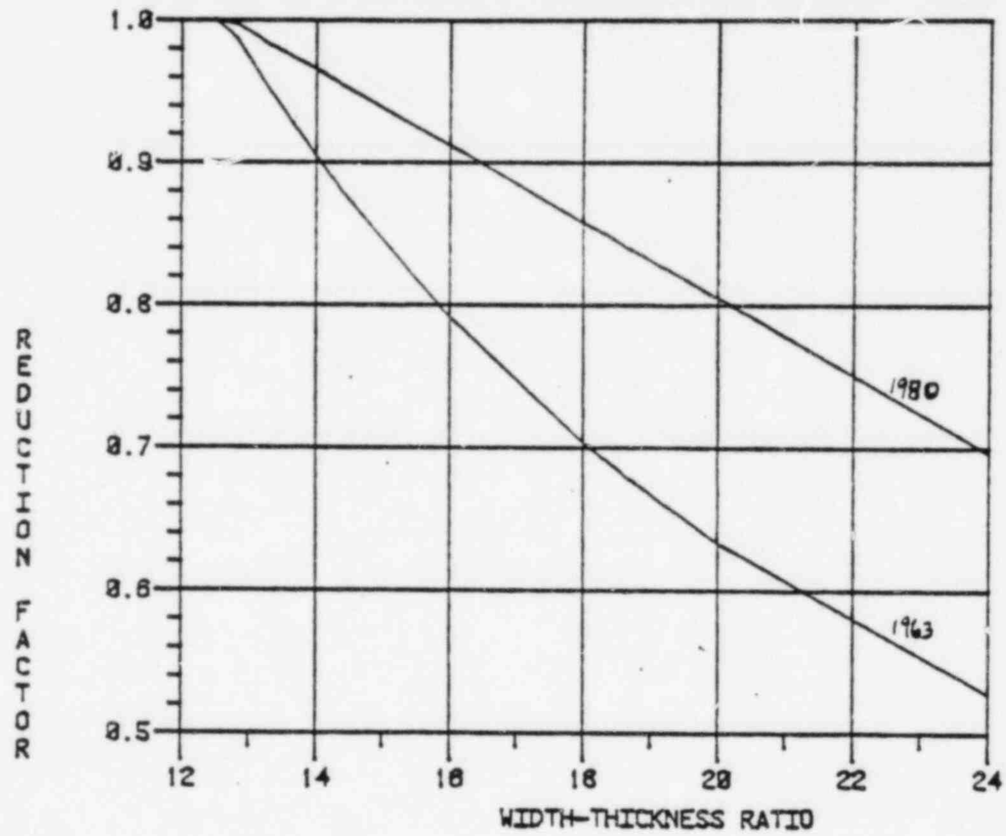
But according to AISC, 1963 Specifications, when compression members exceed the allowable width - thickness ratio, the member is acceptable if it satisfies the allowable stress requirements with a portion of width ie. effective width meets stress requirements.

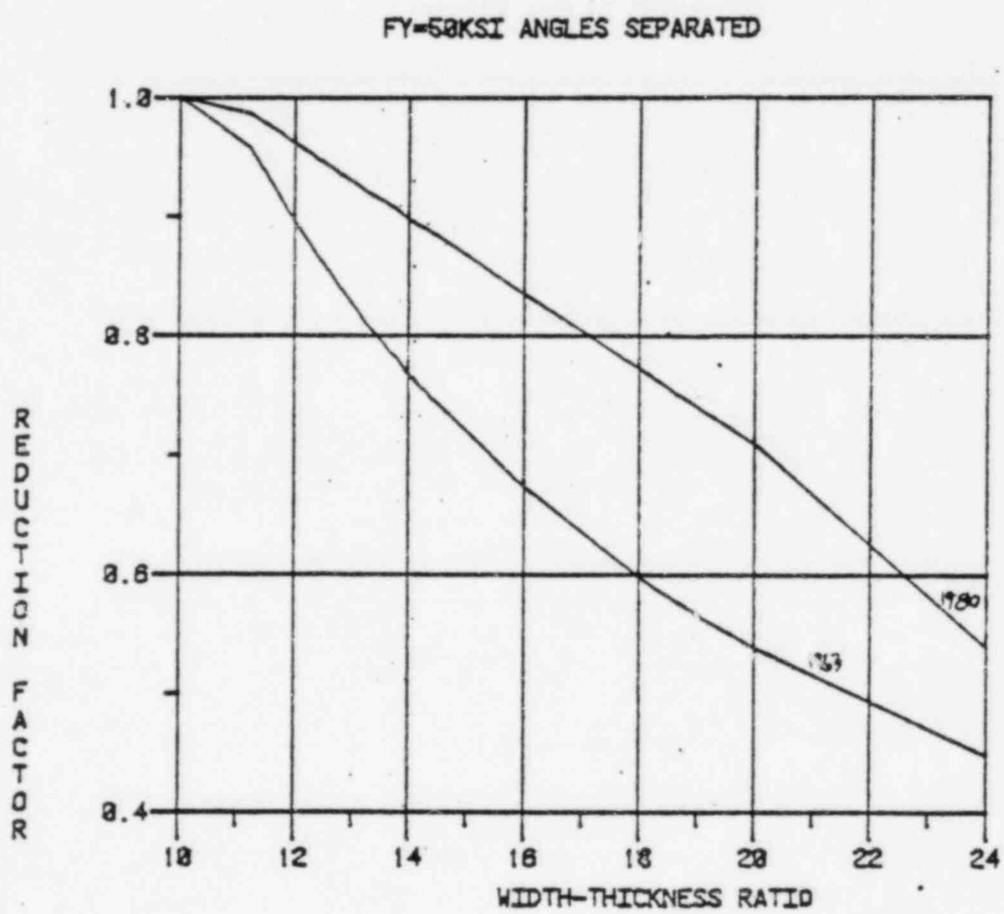
For the case study, two values of Fy 36 ksi and 50 ksi are chosen. For the two values for typical angle section and T sections given in AISC Manual graphs have been plotted for Reduction Factor VS Width - thickness ratio.

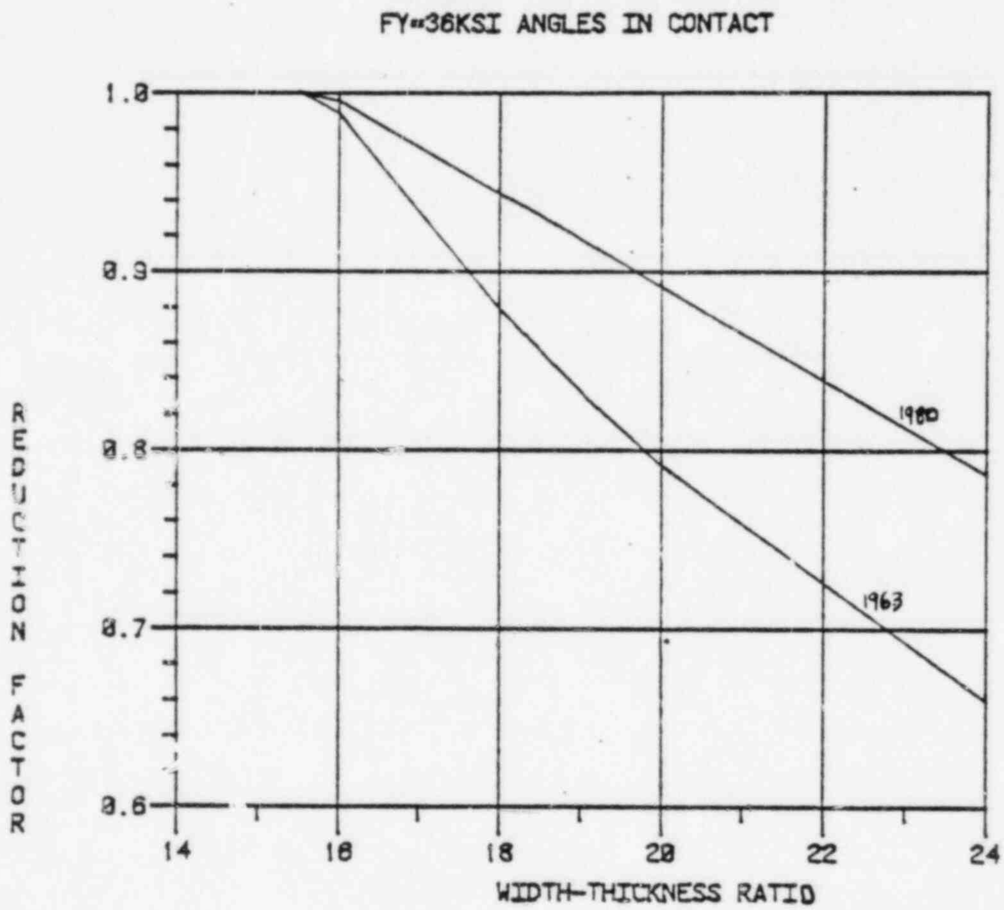
Reduction Factor for AISC, 1980 Code is based on formulas given in appendix C and for AISC, 1963, reduction factor is the ratio of effective width to actual width of the section.

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.

FY=36KSI ANGLES SEPARATED




FY=50KSI ANGLES SEPARATED
1.0
0.8
0.6
0.4
REDUCTION FACTOR
10
12
14
16
18
20
22
24
WIDTH-THICKNESS RATIO
1980
1969


FY=36KSI ANGLES IN CONTACT
1.0
0.9
0.8
0.7
0.6
REDUCTION FACTOR
14
16
18
20
22
24
WIDTH-THICKNESS RATIO
1980
1963

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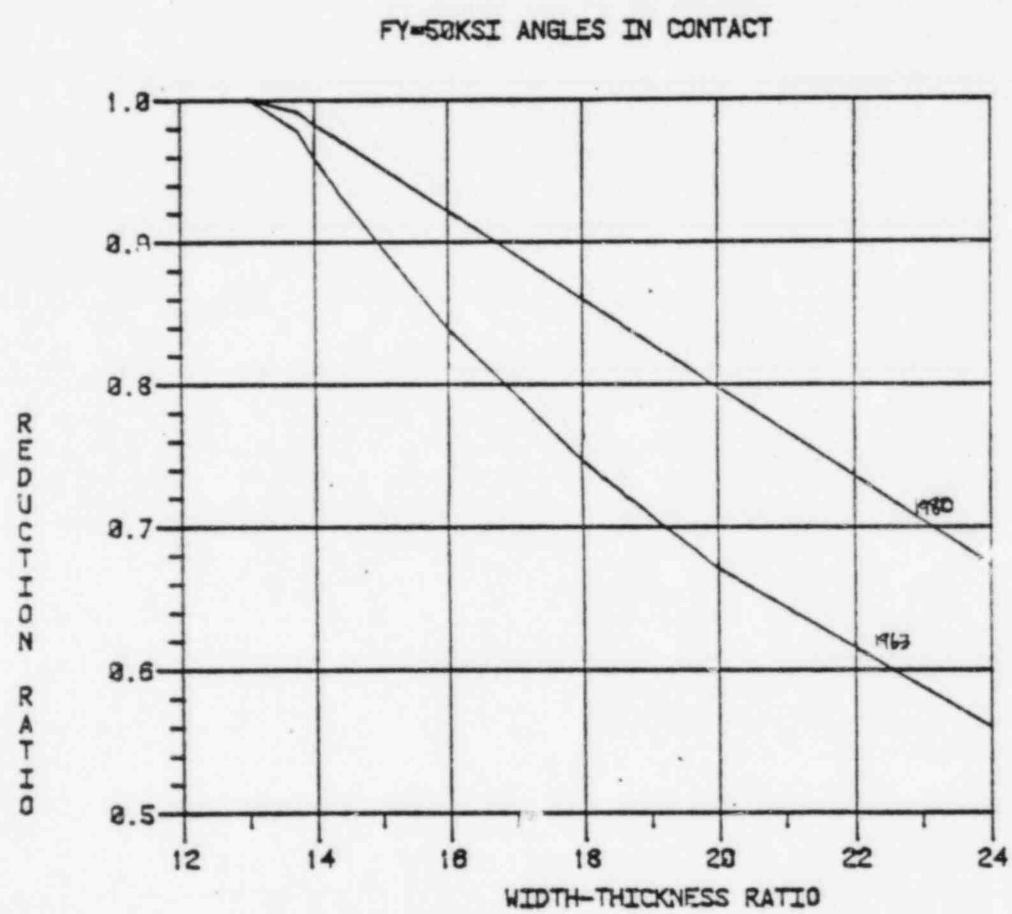
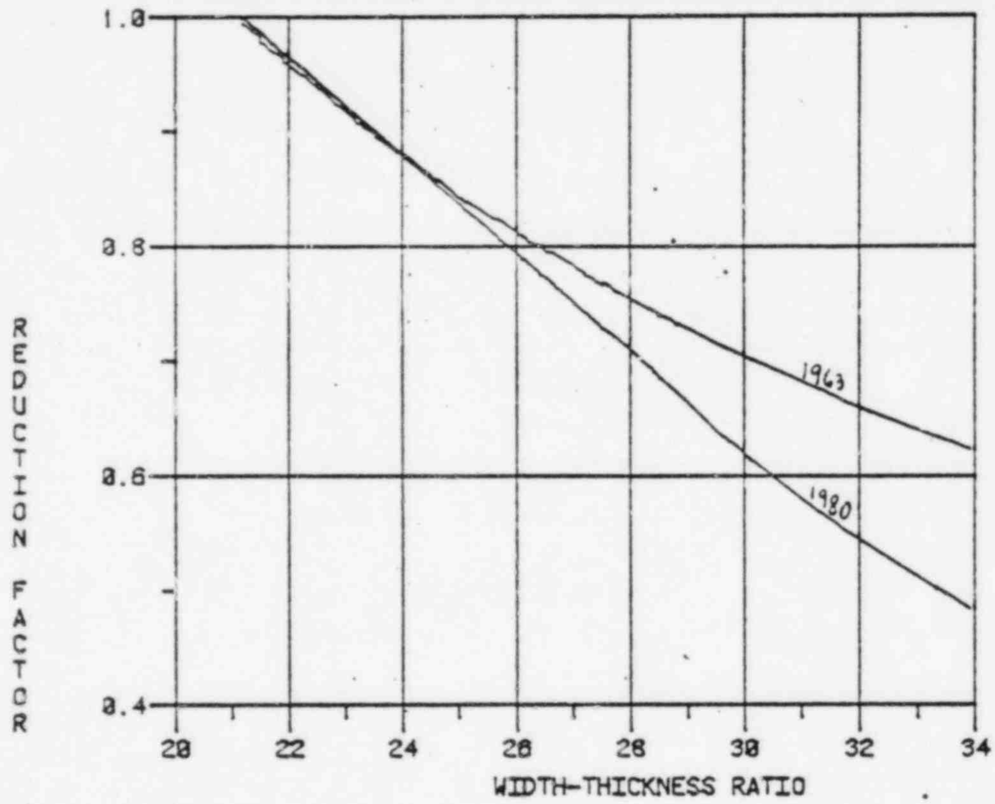
	By RA	Date SEPT'81	Ch'k'd FWD.	Date 10/81	Rev.	Date



FY=36KSI T SHAPES



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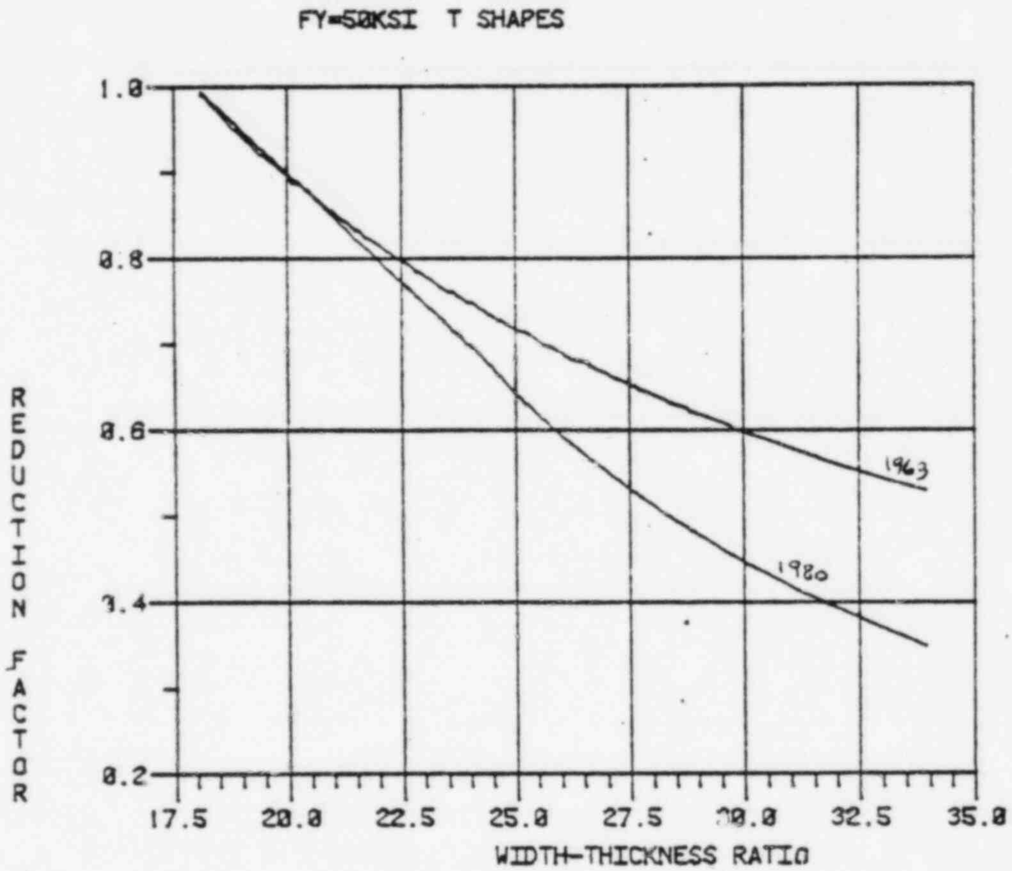

CASE STUDY -11-

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

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

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	By RA	Date OCT '81	Ch'k'd EMD	Date 11/81	Rev.	Date

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 <u>C.</u> 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)

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)

CASE STUDY -14-

Section 1301(c) - Allowable bond stresses - working stress design.

Allowable bond stresses for working stress design in the 318-63 code were newly described as functions of both the square root of concrete compressive strength and reinforcing bar diameter. The 318-56 code defined allowable bond stress as a linear function of concrete compressive strength only.

Plots for three commonly used concrete compressive strengths showing bond stress allowed by each code for deformed bars conforming to ASTM-A-305 plotted against bar diameter show that for small diameter bars the old code is more conservative and for large diameter bars the new code is more conservative. For bars No. 10, 11, 14 and 18 the new code is considerably more conservative.

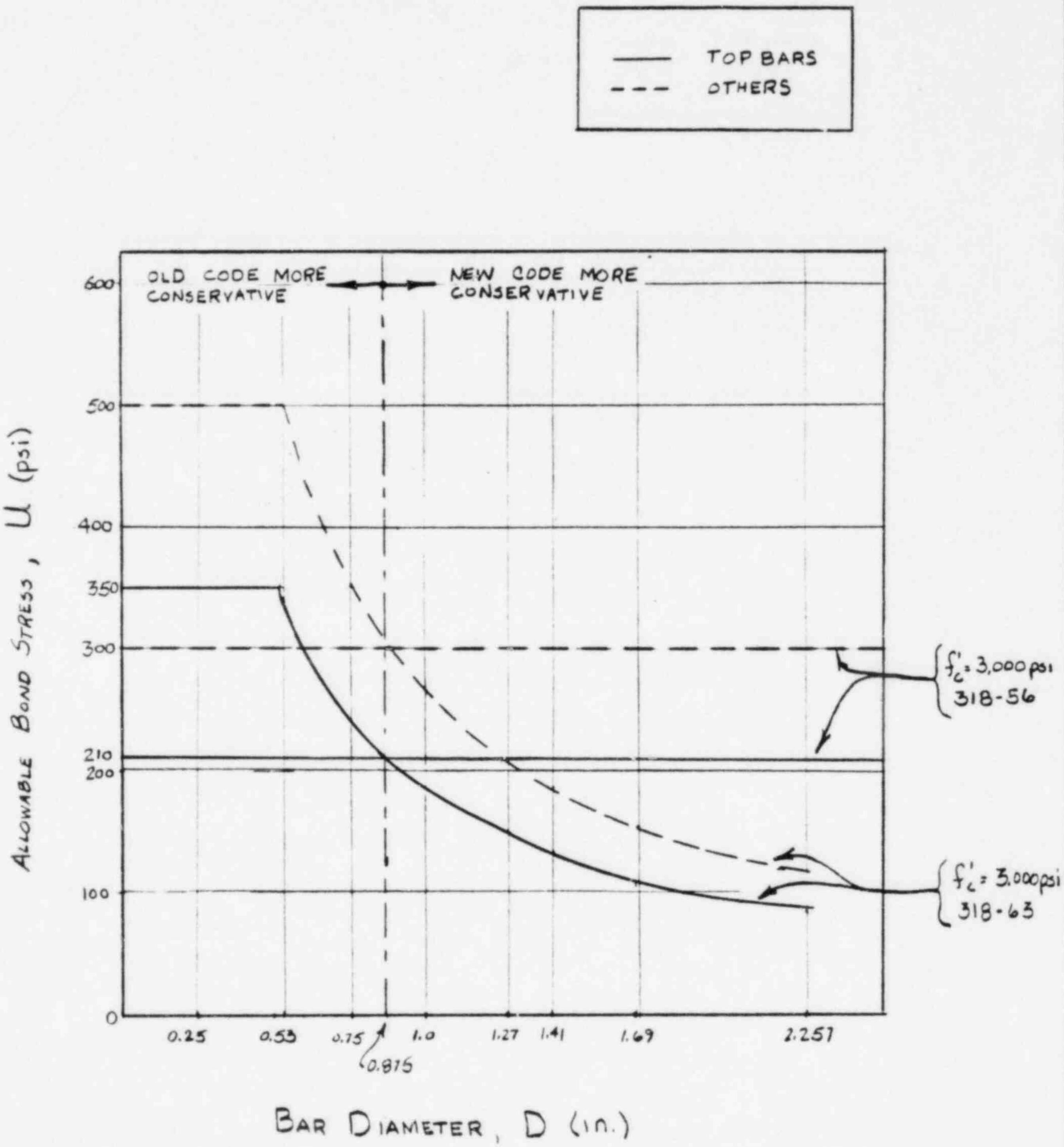
Based on the plots shown, a reasonable interpretation of the code changes as regards scale rating is that for deformed bars conforming to ASTM-A-305:

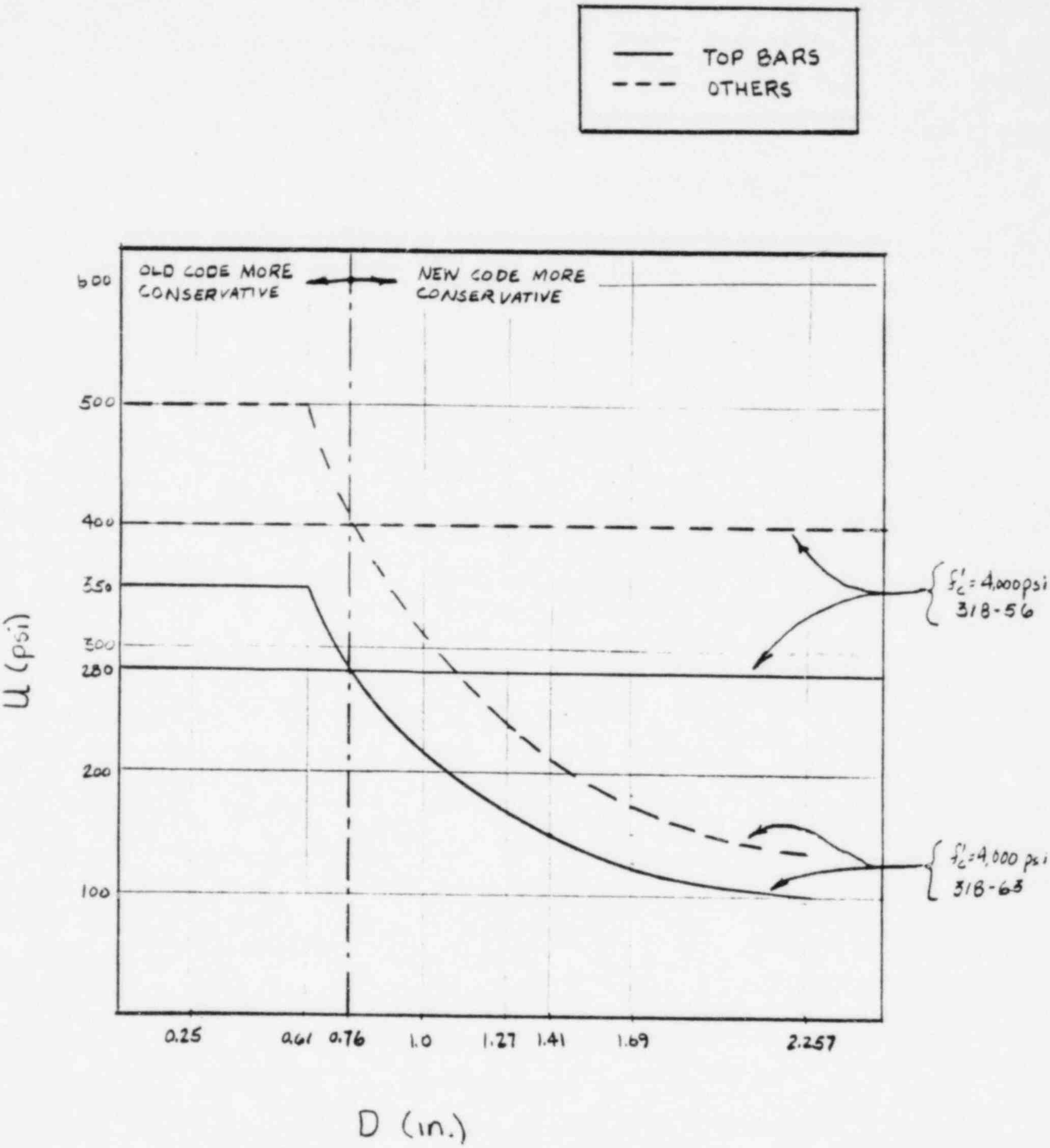
1. For reinforcing bars with diameter less than or equal to 0.875 in. (No. 7 bar) - Scale C

2. For reinforcing bars with diameter greater than 0.875 in. (No. 7 bar) - Scale A

3. For deformed bars conforming to ASTM-A-408 for all diameters - Scale A

Franklin Research Center A Division of The Franklin Institute The Benjamin Franklin Parkway, Phila., Pa. 19103	Project C5257		Page C.14-2
	By MLP Date APRIL '82	Ch'k'd CMW Date 4/82	Rev. Date




TOP BARS
OTHERS
OLD CODE MORE CONSERVATIVE
NEW CODE MORE CONSERVATIVE
600
500
400
350
300
280
200
100
u (psi)
$f'_c = 4{,}000$ psi
318-56
$f'_c = 4{,}000$ psi
318-63
0.25
0.61
0.76
1.0
1.27
1.41
1.69
2.257
D (in.)

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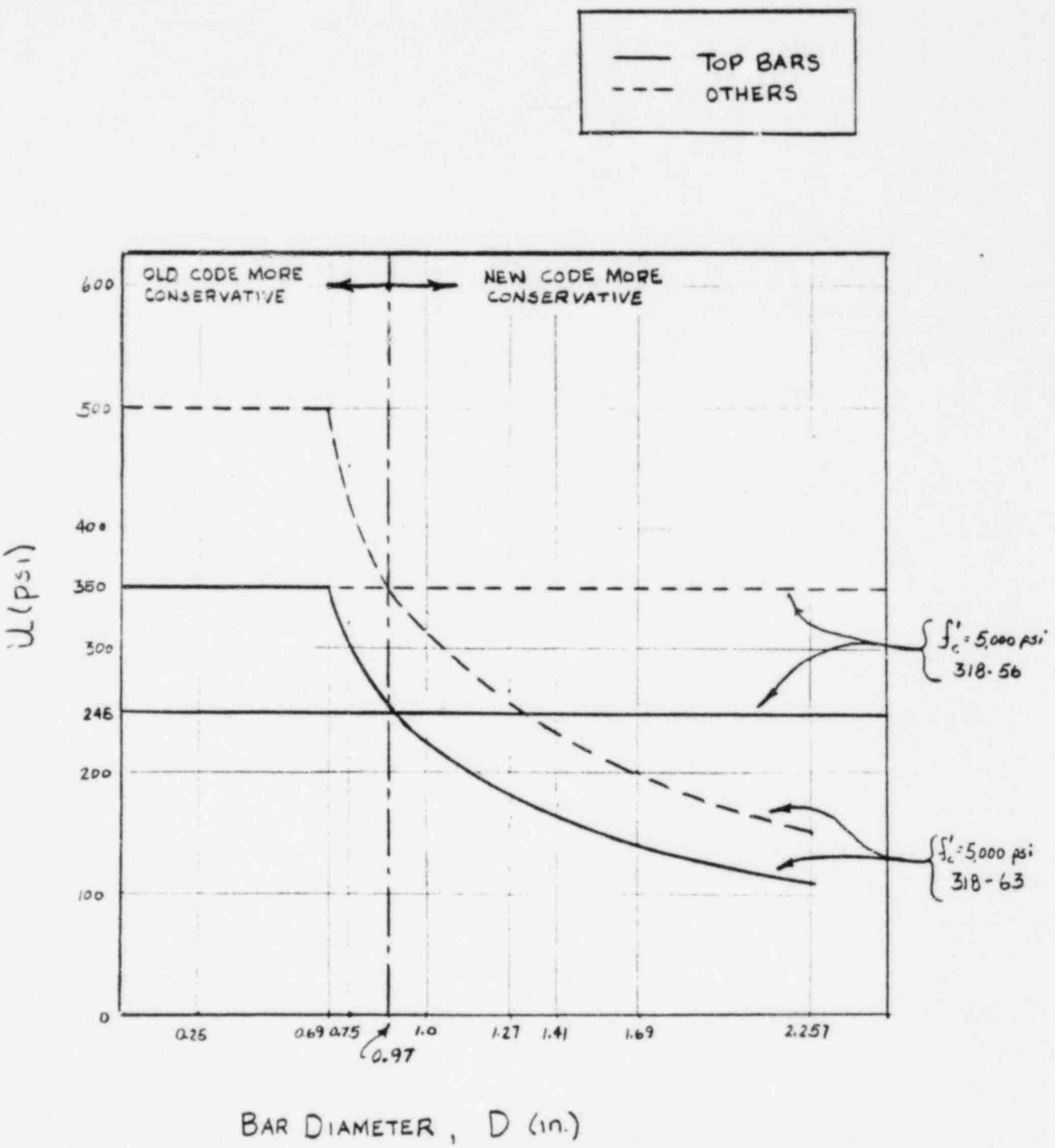
	By MLP Date APRIL '82	Ch'k'd EMW Date 4/82	Rev. Date



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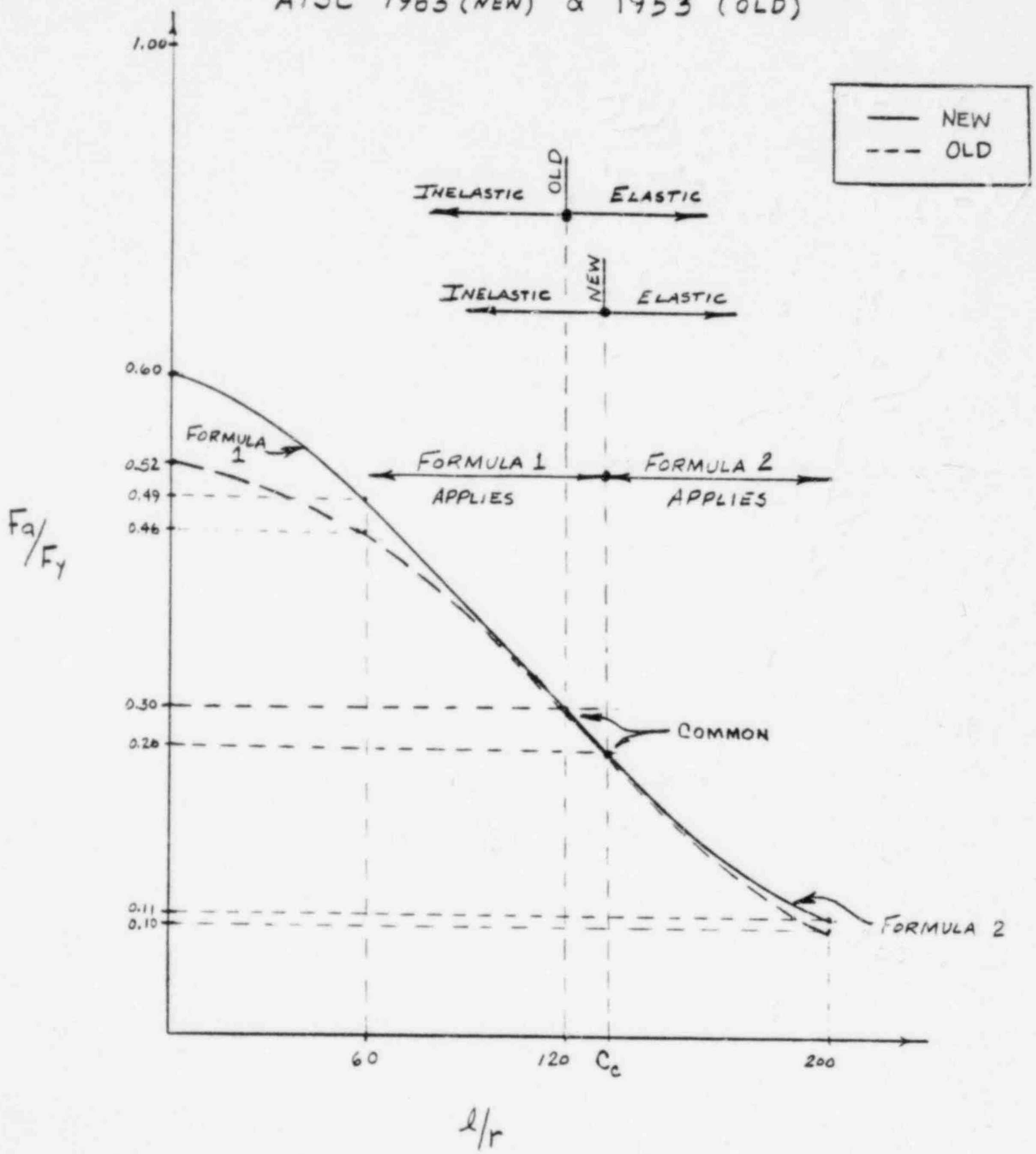
CASE STUDY -15-

(A) Subsection 1.5.1.3.1, 1963 code & 1ST para. section 15(a)(2), 1953 code.

ALLOWABLE COMPRESSION ON MAIN MEMBERS

AISC 1963 (NEW) & 1953 (OLD)



SCALE RATING C

(B) Subsection 1.5.1.3.3, 1963 code & 2ND paragraph section 15(a)(2), 1953 code.

For axially loaded bracing and secondary members when l/r exceeds 120 both the new (Formula 3) and old allowable stress equations are shifted vertically up on the above type plot by the same function

∴ new allowables remain above the old

∴ Scale Rating C

APPENDIX D

ACI CODE PHILOSOPHIES

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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.

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*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 at 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.

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