# TECHNICAL EVALUATION REPORT

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

ROCHESTER GAS AND ELECTRIC CORPORATION

ROBERT EMMETT GINNA NUCLEAR POWER PLANT UNIT 1

NRC DOCKET NO. 50-244
NRC TAC NO. 41500
NRC CONTRACT NO. NRC-03-79-118

FRC PROJECT C5257
FRC ASSIGNMENT 11
FRC TASK 322

*Prepared by*

Franklin Research Center
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FRC Group Leader: T. C. Stilwell

*Prepared for*

Nuclear Regulatory Commission
Washington, D.C. 20555

Lead NRC Engineer: D. Persinko

May 27, 1982





Franklin Research Center
A Division of The Franklin Institute
The Benjamin Franklin Parkway, Phila., Pa. 19103 (215) 448-1000

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## 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.

# 1. INTRODUCTION

For the Seismic Category I buildings and structures at the R. E. Ginna Plant, 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 Ginna 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 Ginna plant.

# 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 by Lawrence Livermore Laboratory. |

## 5. MARGINS OF SAFETY

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

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

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

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 Ginna plant 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 warn that this may 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 Ginna 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 Ginna 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. GINNA SEISMIC CATEGORY I STRUCTURES

SEP Topic III-1 has for its objective the classification of components, structures, and systems with respect to both quality group and seismic designation. Based upon the review of the Ginna FSAR [5] and Gilbert Associates, Inc. drawings [6] showing the location of Seismic Category I equipment, the present report considers the following to be Seismic Category I structures:

A. Containment

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

B. Internal Structures

- Steam generator/reactor coolant pump compartments (reviewed in Generic Task A-2)
- Biological shield (reviewed in Generic Task A-2)
- Fuel transfer canal

C. External Structures

1. Auxiliary Building

Contains the following Seismic Category I structures:
- Spent fuel storage pit
- New fuel storage area
- 480-V switchgear room
- Portions of the fuel transfer tube

Houses the following Seismic Category I equipment:
- Safety injection pumps and residual heat removal pumps (in pit beneath basement floor)
- Refueling water storage tank
- Boric acid tanks
- Containment spray pumps
- Waste holdup tanks
- 480-V switchgear

2. Control Room Building

   Contains:
   Control room
   Battery room
   Relay room

3. Portions of the intermediate building (which house auxiliary feedwater pumps)

4. Cable tunnel

5. Intake/discharge structure and screen house

6. Diesel generator annex.

Major structures not classified as Seismic Category I are the turbine building and the service building.

## 9. STRUCTURAL DESIGN CRITERIA

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

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

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

| Structure | Design Criteria | Current Criteria |
|---|---|---|
| 3. Control Room Building | AISC-1963<br>ACI 318-63 | AISC-1980<br>ACI 349-80 |
| 4. Portions of the Intermediate Building | AISC-1963<br>ACI 318-63 | AISC-1980<br>ACI 349-80 |
| 5. Cable Tunnel | ACI 318-63 | ACI 349-80 |
| 6. Intake/Discharge Structure and Screen House | AISC-1963<br>ACI 318-63 | AISC-1980<br>ACI 349-80 |
| 7. Diesel Generator Annex | AISC-1963<br>ACI 318-63 | AISC-1980<br>ACI 349-80 |

REFERENCES IDENTIFYING MAJOR CODES USED FOR THE ORIGINAL DESIGN:

1. Final Facility Description and Safety Analysis Report for the Robert Emmett Ginna Nuclear Power Plant No. 1.

2. Rochester Gas and Electric Corporation's response to NRC's Request For Information letter, Topic III-7.B.

# 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"

COMPARISON OF DESIGN BASIS LOADS

| STRUCTURE: |
|---|
| CONTAINMENT STRUCTURE (concrete) |

PLANT: GINNA

| | 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 | ----- | Yes | No | ---- | |
| | L | Yes | Yes | ----- | Yes | No | ---- | |
| Pressure | F | Yes | Yes | ----- | Yes | No | ---- | |
| | H | Yes | Yes | III-5.A | * | * | * | 5. |
| | $P_o$ | Yes | Yes | ----- | Yes | No | ---- | |
| | $P_a$ | Yes | Yes | VI-2.D, III-7.B | * | * | * | |
| | $P_s$ | No | --- | ----- | --- | No | ---- | |
| Thermal | $T_o$ | Yes | Yes | ----- | Yes | No | ---- | 1. |
| | $T_a$ | Yes | Yes | VI-2.D, III-7.B | * | * | * | 1. |
| | $T_s$ | No | --- | ----- | --- | No | ---- | |
| Pipe & Mech. | $R_o$ | Yes | Yes | ----- | Yes | No | ---- | |
| | $R_a$ | Yes | Yes | ----- | Yes | No | ---- | |
| | $R_s$ | No | --- | ----- | --- | No | ---- | |
| Environmental | E' | Yes | Yes | III-6 | * | * | $A_x$ | 6. |
| | E | Yes | Yes | III-6 | * | * | * | |
| | W' | Yes | No | III-2, III-4.A | * | * | $A_x$ | |
| | W | Yes | Yes | III-2, III-4.A | * | * | * | 2. |
| Impulse | $Y_r$ | Yes | Yes | III-5.A | * | * | * | 3. |
| | $Y_j$ | Yes | Yes | III-5.A | * | * | * | 3. |
| | $Y_m$ | No | Yes | III-5.A | * | * | * | 3. |

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. FSAR (Pg. 5.1.2-56) states penetrations were analyzed for these loads.
2. FSAR (Pg. 5.1.2-6) indicates wind loads were considered. They do not appear in Table 5.1.2-41, (75 MPH used).
3. These loads were reviewed in all operating plants.
4. FSAR (Section 5.1.2.8) states that all sources of internal missiles are shielded from containment walls.
5. R G&E engineers report hydrological soil pressure was considered. Water table taken at elev. 250 ft.
6. Equivalent static analysis used for original design and checked by response spectrum analysis using housner spectrum.

STRUCTURE:
SPENT FUEL POOL (concrete)

COMPARISON OF DESIGN BASIS LOADS

PLANT: GINNA

| | 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 | ----- | | | | |
| | L | Yes | Yes | ----- | | | $A_x$ | 4. |
| Pressure | F | No | --- | ----- | ----- | ----- | ----- | |
| | H | Yes | Yes | III-3.A | * | * | * | |
| | $P_a$ | No | --- | III-5.B | * | * | ----- | |
| Thermal | $T_o$ | Negl. 5. | --- | ----- | ----- | ----- | ----- | |
| | $T_a$ | Yes | | III-5.B | * | * | * | |
| Pipe & Mech. | $R_o$ | No | --- | ----- | ----- | ----- | ----- | |
| | $R_a$ | No | --- | ----- | ----- | ----- | ----- | |
| Environmental | E' | Yes | | III-6 | * | * | $A_x$ | |
| | E | Yes | | III-6 | * | * | ----- | |
| | W' | Yes | 2. | III-2, III-4.A | * 1. | * | $A_x$ | |
| | W | No | | III-2, III-4.A | * | * | ----- | |
| Impulse | $Y_r$ | --- | --- | III-5.B | * | * | * | |
| | $Y_j$ | --- | --- | III-5.B | * | * | * | |
| | $Y_m$ | --- | --- | 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. SEP Topic III-2 will determine whether or not pool exposure to possible tornado effects is an allowable spent fuel pool load.
2. Applicable only since roof over spent fuel pool is not believed to be tornado resistant.
3. No information on design loads specific to the spent fuel pool was found. However, loads an load combinations for the auxiliary building (in which the pool is located) were provided by Roc' aster G & E's response to NRC III-7B inquiry. These are assumed to apply to the spent fuel pool lso.
4. Roof loads have increased per SEP Topic II-2.A, and may increase per SEP Topic II-3.B for ,arapet roofs.
5. Fuel pool temperature (high density racks, fully loaded) is limited to 90°F for all reactors.

STRUCTURE:
AUXILIARY BUILDING
(Concrete)

COMPARISON OF DESIGN BASIS LOADS

PLANT: GINNA

| | 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 | ----- | Yes | No | ---- | |
| | L | Yes | Yes | ----- | Yes | No | $A_x$ | 4. |
| Pressure | F | No | --- | ----- | No | ---- | ---- | |
| | H | Yes | Yes | III-3.A | * | * | * | 3. |
| | $P_a$ | No | --- | III-5.B | * | * | * | |
| Thermal | $T_o$ | Yes | No | ----- | --- | Yes | B | Effects are small |
| | $T_a$ | No | --- | III-5.B | * | * | ---- | |
| Pipe & Mech. | $R_o$ | Yes | NO INFORMATION FOUND | | | | | |
| | $R_a$ | No | --- | ----- | --- | No | --- | |
| Environmental | E' | Yes | Yes | III-6 | * | * | $A_x$ | 2. |
| | E | Yes | No | III-6 | * | * | * | 2. |
| | W' | Yes | No | III-2, III-4.A | * | * | $A_x$ | |
| | W | Yes | Yes | III-2, III-4.A | * | * | * | |
| Impulse | $Y_r$ | Yes | No | III-5.B | * | * | $A_x$ | 1. |
| | $Y_j$ | Yes | No | III-5.B | * | * | $A_x$ | 1. |
| | $Y_m$ | Yes | --- | III-5.B | * | * | $A_x$ | 1. |

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. Effects of pipe rupture outside containment is being addressed in another SEP topic.
2. Initial design used static earthquake loading (g-loads).
3. Water table taken at elevation 250 feet.
4. Roof loads have increased per SEP Topic II-2.A, and may increase per SEP Topic II-3.B for parapet roofs.

GENERAL NOTE: There are a number of masonry walls in this structure. This subject is addressed in IE Bulletin 80-11 and other SEP Topics.

COMPARISON OF DESIGN BASIS LOADS

STRUCTURE:
AUXILIARY BUILDING (steel)

PLANT: GINNA

| | 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 | ----- | Yes | No | --- | |
| | L | Yes | Yes | ----- | Yes | No | $A_x$ | 3. |
| Pressure | F | No | --- | ----- | --- | No | --- | |
| | H | No | --- | III-3.A | * | * | --- | |
| | $P_a$ | No | --- | III-5.B | * | * | --- | |
| Thermal | $T_o$ | Yes | No | ----- | --- | Yes | B | |
| | $T_a$ | No | --- | III-5.B | * | * | --- | |
| Pipe & Mech. | $R_o$ | No | --- | | --- | NO INFORMATION FOUND | | |
| | $R_a$ | No | --- | | --- | No | --- | |
| Environmental | E' | Yes | Yes | III-6 | * | * | $A_x$ | 1. |
| | E | Yes | No | III-6 | * | * | * | 1. |
| | W' | Yes | No | III-2, III-4.A | * | * | $A_x$ | |
| | W | Yes | Yes | III-2, III-4.A | * | * | * | |
| Impulse | $Y_r$ | Yes | No | III-5.B | * | * | $A_x$ | 2. |
| | $Y_j$ | Yes | No | III-5.B | * | * | $A_x$ | 2. |
| | $Y_m$ | Yes | --- | III-5.B | * | * | $A_x$ | 2. |

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. Seismic loadings for design were taken as static (g-loads).
2. Pipe break outside containment is being considered as a separate SEP Topic (SEP III-5.B)
3. Roof snow loads have increased per SEP Topic II-2.A.

GENERAL NOTE: The auxiliary building employs a number of masonry walls to be investigated in IE Bulletin 80-11 and other SEP Topics.

STRUCTURE:
CONTROL BUILDING (concrete)

COMPARISON OF DESIGN BASIS LOADS

PLANT: GINNA

| | 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 | ----- | Yes | No | ----- | |
| | L | Yes | Yes | ----- | Yes | No | $A_x$ | 5. |
| Pressure | F | No | --- | ----- | --- | No | ----- | |
| | H | Yes | Yes | III-3.A | * | * | * | 4. |
| | $P_a$ | No | --- | III-5.B | * | * | ----- | |
| Thermal | $T_o$ | Yes | No | | --- | Yes | B | 2. |
| | $T_a$ | No | --- | III-5.B | * | * | ----- | |
| Pipe & Mech. | $R_o$ | No | --- | | --- | No | --- | |
| | $R_a$ | No | --- | | --- | No | --- | |
| Environmental | E' | Yes | Yes | III-6 | * | * | $A_x$ | 1. |
| | E | Yes | No | III-6 | * | * | * | 1. |
| | W' | Yes | No | III-2, III-4.A | * | * | $A_x$ | |
| | W | Yes | No | III-2, III-4.A | * | * | * | |
| Impulse | $Y_r$ | No | --- | III-5.B | * | * | --- | 3. |
| | $Y_j$ | Yes | --- | III-5.B | * | * | * | 3. |
| | $Y_m$ | Yes | --- | III-5.B | * | * | * | 3. |

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. Treated earthquake loadings as static (g-load).
2. Effects small. Building has experienced a broad spectrum of them.
3. Shares wall in common with turbine building, missile and jet reaction barrier is understood to have been installed.
4. Water table taken at elev. 250 FT.
5. Roof loads have increased per SEP Topic II-2.A and may increase per SEP Topic II-3.B for parapet roofs.

STRUCTURE: PORTIONS OF INTERMEDIATE BUILDING (Concrete)

COMPARISON OF DESIGN BASIS LOADS

PLANT: GINNA

| | 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 | | | | | | |
| | L | Yes | | | | | | |
| Pressure | F | No | | | | | | |
| | H | Yes | | III-3.A | * | * | * | |
| | $P_a$ | No | | III-5.B | * | * | --- | |
| Thermal | $T_o$ | Yes | | | | | | |
| | $T_a$ | No | | III-5.B | * | * | --- | |
| Pipe & Mech. | $R_o$ | Yes | | | | | | |
| | $R_a$ | Yes | | | | | | 1. |
| Environmental | E' | Yes | | III-6 | * | * | $A_x$ | |
| | E | Yes | | III-6 | * | * | * | |
| | W' | Yes | | III-2, III-4.A | * | * | $A_x$ | |
| | W | Yes | | III-2, III-4.A | * | * | * | |
| Impulse | $Y_r$ | Yes | | III-5.B | * | * | * | 1. |
| | $Y_j$ | Yes | | III-5.B | * | * | * | 1. |
| | $Y_m$ | Yes | | III-5.B | * | * | * | 2. |

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. Main steam, feedwater piping, and relief valve discharge piping.
2. Intermediate building shares common wall with turbine building. No information was found on design loads for the intermediate building. A reasonable assumption is that it was designed to the same conditions as the rest of the building complex. On this basis, design is comparable to that of the auxiliary building.

STRUCTURE: PORTIONS OF INTERMEDIATE BUILDING (steel)

COMPARISON OF DESIGN BASIS LOADS

PLANT: GINNA

| | 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 | | | | | | |
| | L | Yes | | | | | | |
| Pressure | F | No | | | | | | |
| | H | No | | III-3.A | * | * | --- | |
| | $P_a$ | No | | III-5.B | * | * | --- | |
| Thermal | $T_o$ | Yes | | | | | | |
| | $T_a$ | No | | III-5.B | * | * | --- | |
| Pipe & Mech. | $R_o$ | Yes | | | | | | |
| | $R_a$ | Yes | | | | | | 1. |
| Environmental | E' | Yes | | III-6 | * | * | $A_x$ | |
| | E | Yes | | III-6 | * | * | * | |
| | W' | Yes | | III-2, III-4.A | * | * | $A_x$ | |
| | W | Yes | | III-2, III-4.A | * | * | * | |
| Impulse | $Y_r$ | Yes | | III-5.B | * | * | * | 1. |
| | $Y_j$ | Yes | | III-5.B | * | * | * | 1. |
| | $Y_m$ | Yes | | III-5.B | * | * | * | 2. |

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. Main steam and feedwater piping pass through intermediate building. Relief valve discharge piping also.
2. Intermediate building shares common wall with turbine building. No information concerning loads specific to the intermediate building. A reasonable assumption is that design was to same criteria as other structures of the building complex. On this basis, design is comparable to the auxiliary building.

STRUCTURE:

CABLE TUNNEL

COMPARISON OF DESIGN BASIS LOADS

PLANT: GINNA

| | 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 | | | | | | |
| | L | Yes | | | | | | |
| Pressure | F | No | | | | | | |
| | H | Yes | | III-3.A | * | * | * | - |
| | $P_a$ | No | | III-5.B | * | * | — | |
| Thermal | $T_o$ | Yes | | | | | | |
| | $T_a$ | No | | III-5.B | * | * | — | |
| Pipe & Mech. | $R_o$ | No | | | | | | |
| | $R_a$ | No | | | | | | |
| Environmental | E' | Yes | | III-6 | * | * | $A_x$ | 1. |
| | E | Yes | | III-6 | * | * | — | 1. |
| | W' | No | | III-2, III-4.A | * | * | $A_x$ | |
| | 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. No information on the design of the cable tunnel was found.

COMPARISON OF DESIGN BASIS LOADS

STRUCTURE: INTAKE/DISCHARGE STRUCTURE & SCREEN HOUSE (concrete)

PLANT: GINNA

| | 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 | ----- | Yes | No | --- | |
| | L | Yes | Yes | ----- | Yes | No | $A_x$ | 3. |
| Pressure | F | No | --- | | --- | No | --- | |
| | H | Yes | Yes | III-3.A | * | * | * | |
| | $P_a$ | No | --- | III-5.B | * | * | --- | - |
| Thermal | $T_o$ | Yes | No | | --- | Yes | B | 2. |
| | $T_a$ | No | --- | III-5.B | * | * | --- | |
| Pipe & Mech. | $R_o$ | Yes | No | | --- | Yes | B | 2. |
| | $R_a$ | No | --- | | --- | 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 | $Y_r$ | Yes | --- | III-5.B | * | * | $A_x$ | |
| | $Y_j$ | Yes | --- | III-5.B | * | * | $A_x$ | |
| | $Y_m$ | Yes | --- | III-5.B | * | * | $A_x$ | |

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. Earthquake loadings taken as static (g-load) in original design.
2. Small effects.
3. Roof loads have increased per SEP Topic II-2A and may increase per SEP Topic II-3.B for parapet roofs.

STRUCTURE: DIESEL GENERATOR BUILDING (concrete)

COMPARISON OF DESIGN BASIS LOADS

PLANT: GINNA

| | 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 | --- | Yes | No | --- | |
| | L | Yes | Yes | --- | Yes | No | $A_x$ | 5. |
| Pressure | F | No | --- | --- | --- | No | --- | |
| | H | Yes | Yes | III-3.A | * | * | * | |
| | $P_a$ | No | --- | III-5.B | * | * | --- | |
| Thermal | $T_o$ | Yes | No | --- | --- | Yes | B | 3. |
| | $T_a$ | No | --- | III-5.B | * | * | --- | |
| Pipe & Mech. | $R_o$ | No | --- | --- | --- | No | --- | |
| | $R_a$ | No | --- | --- | --- | No | --- | |
| Environmental | E' | Yes | Yes | III-6 | * | * | $A_x$ | 1. |
| | E | Yes | Yes 4. | III-6 | * | * | * | 1. |
| | W' | Yes | No | III-2, III-4.A | * | * | $A_x$ | |
| | W | Yes | No | III-2, III-4.A | * | * | * | |
| Impulse | $Y_r$ | No | --- | III-5.B | * | * | --- | |
| | $Y_j$ | NO INFO | --- | III-5.B | * | * | * | 2. |
| | $Y_m$ | Yes | No | III-5.B | * | * | * | 2. |

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. Earthquake loads treated as static (g-load).
2. D/G bldg. shares common wall with turbine bldg.
3. Effect is small.
4. Considered for wall panels only.
5. Roof loads have increased per SEP Topic II-2.A and may increase per SEP Topic II-3.B for parapet roofs.

## 10.4 LOAD COMBINATION TABLES

"COMPARISON OF LOADING COMBINATION CRITERIA"

STRUCTURE

CONCRETE CONTAINMENT

COMPARISON OF LOADING COMBINATION CRITERIA

PLANT: GINNA

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

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

Notes

1. Encircled loads are those considered in the design. When load factors different from those currently required were used, the factor used is also encircled.
2. Loads deemed inapplicable or negligible struck from loading combinations.
3. FSAR (Pg. 5.1.2-6) indicates that wind load was considered; but the loading combinations actually calculated do not include it (See Table 5.1.2-4I) thus, it may be that stresses from load combination 11 are less than those from case 10 and 12 everywhere in the structure; but explicit documentation of this was not found. It is understood that 75 MPH wind was used.
4. $R_r = R_{rr} + R_{rj} + R_{rm}$. For this containment, according to FSAR (Pg. 5.1.2-83a), $R_{rm}$ may be taken as zero in this expression.
5. For purposes of the SEP Review, demonstration that structural integrity is maintained for load case 7, 8 & 14 (per current criteria) may be considered as providing reasonable assurance that this structure meets the intent of current design criteria.

| STRUCTURE |
|---|
| CONTAINMENT LINER |

COMPARISON OF LOADING COMBINATION CRITERIA

PLANT: GINNA

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

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

NOTES

1. Encircled loads are those actually considered in the design. When load factors different from those currently required were used, the factor used is also encircled.
2. Loads deemed inapplicable or negligible when struck from loading combinations.
3. The liner may have been considered non-load-bearing in the case of some of the mechanical loads.
4. The liner should be shown leak-free under tornado load and its missiles and under credible events generating $R_a + R_r$ loading concurrent with loading combination 14.
5. For purposes of the SEP Review, demonstration that structural integrity is maintained for load cases 7, 8 & 14 (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: GINNA

STRUCTURE:
SPENT FUEL POOL (concrete)

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

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✓ consequently no load factors were used / ~~ultimate strength~~
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. No information on design loads specific to the spent fuel pool was found. However, loads and load combinations for the auxiliary building (in which the pool is located) were provided by Rochester G & E's response to NRC III-7B Inquiry. These are assumed to apply to the spent fuel pool also.
6. Method of seismic analysis does not correspond to current criteria.
7. For purposes of the SEP Review, demonstration that structural integrity is maintained for load cases 10 and 13 (per current criteria) may be considered as providing reasonable assurance that this structure meets the intent of current design criteria.

COMPARISON OF LOADING COMBINATION CRITERIA

CONCRETE STRUCTURES

PLANT: GINNA

STRUCTURE:
AUXILIARY BUILDING (concrete)

| Combined Loading Cases | Gravity Dead, Live | Thermal | Pressure | Mechanical | Natural Phenomena | Impulsive Loading | Scale Ranking |
|---|---|---|---|---|---|---|---|
| 1 | 1.4D + 1.7L 2.,5. | | | | | | |
| 2 | 1.4D + 1.7L | | | | 1.9E | | |
| 3 | 1.4D + 1.7L | | | | 1.7W | | |
| 4 | .75 (1.4D + 1.7L) | .75 x 1.7 $T_o$ | | .75 x 1.7 $R_o$ | | | |
| 5 | .75 (1.4D + 1.7L) | .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 5. | $T_o$ | | $R_o$ | E' 6. | | |
| 10 | D + L | $T_o$ | | $R_o$ | $W_t$ | | $A_x$ |
| 11 | D + L | $\cancel{T_a}$ | 1.5 $\cancel{P_a}$ | $\cancel{R_a}$ | | | |
| 12 | D + L | $\cancel{T_a}$ | 1.25 $\cancel{P_a}$ | $\cancel{R_a}$ | 1.25E | $Y_r + Y_j + Y_m$ | |
| 13 | D + L | $\cancel{T_a}$ | $\cancel{P_a}$ | $\cancel{R_a}$ | 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. H included in D+L.
6. Earthquake loading taken as static g-load.
7. Snow load coefficients in accordance with ANSI A58.1 may be used, or provisions of UBC Section 2311 (j) invoked.
8. For purposes of the SEP Review, demonstration that structural integrity is maintained for load cases 10 and 13 (per current criteria) may be considered as providing reasonable assurance that this structure meets the intent of current design criteria.

COMPARISON OF LOADING COMBINATION CRITERIA

STEEL STRUCTURES (Elastic Analysis)

PLANT: GINNA

STRUCTURE:
AUXILIARY BUILDING (steel)

| Combined Loading Cases | Gravity Dead, Live | Thermal | Pressure | Mechanical | Natural Phenomena | Impulsive Loading | Scale |
|---|---|---|---|---|---|---|---|
| 1 | D + L | | | | | | |
| 2 | D + L | | | | E | | |
| 3 | D + L | | | | W | | |
| 4 | D + L | $T_o$ | | $\cancel{R_o}$ | | | |
| 5 | D + L | $T_o$ | | $\cancel{R_o}$ | E | | |
| 6 | D + L | $T_o$ | | $\cancel{R_o}$ | W | | |
| 7 | D + L | $T_o$ | | $\cancel{R_o}$ | E' 3. | | |
| 8 | D + L | $T_o$ | | $\cancel{R_o}$ | $W_t$ | | $A_x$ |
| 9 | D + L | $\cancel{T_a}$ | $\cancel{P_a}$ | $\cancel{R_a}$ | | | |
| 10 | D + L | $\cancel{T_a}$ | $\cancel{P_a}$ | $\cancel{R_a}$ | E | $Y_j + Y_r + Y_m$ | |
| 11 | D + L | $\cancel{T_a}$ | $\cancel{P_a}$ | $\cancel{R_a}$ | E' | $Y_j + Y_r + Y_m$ | $A_x$ |

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

Notes

1. Encircled loads are those actually considered in the design. When load factors are different from those currently required were used, the factor used is also encircled.
2. Loads deemed inapplicable or negligible struck from loading combinations.
3. Earthquake loading taken as static g-load.
4. Snow load coefficients in accordance with ANSI A58.1 may be used, or provisions of UBC Section 2311(j) invoked.
5. For purposes of the SEP Review, demonstration that structural integrity is maintained for load cases 8 and 11 (per current criteria) may be considered as providing reasonable assurance that this structure meets the intent of current design criteria.

COMPARISON OF LOADING COMBINATION CRITERIA

CONCRETE STRUCTURES

PLANT: GINNA

STRUCTURE:

CONTROL 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 | | | | 1.9E | | |
| 3 | 1.4D + 1.7L | | | | 1.7W | | |
| 4 | .75 (1.4D + 1.7L) | .75 x 1.7 $T_o$ | | .75 x 1.7 $\cancel{R_o}$ | | | |
| 5 | .75 (1.4D + 1.7L) | .75 x 1.7 $T_o$ | | .75 x 1.7 $\cancel{R_o}$ | .75 x 1.9E | | |
| 6 | .75 (1.4D + 1.7L) | .75 x 1.7 $T_o$ | | .75 x 1.7 $\cancel{R_o}$ | .75 x 1.7W | | |
| 7 | 1.2D | | | | 1.9E | | |
| 8 | 1.2D | | | | 1.7W | | |
| 9 | D + L | $T_o$ | | $\cancel{R_o}$ | E' 5. | | |
| 10 | D + L | $T_o$ | | $\cancel{R_o}$ | $W_t$ | | $A_x$ |
| 11 | D + L | $\cancel{T_a}$ | 1.5 $\cancel{P_a}$ | $\cancel{R_a}$ | | | |
| 12 | D + L | $\cancel{T_a}$ | 1.25 $\cancel{P_a}$ | $\cancel{R_a}$ | 1.25E | $Y_r + Y_j + Y_m$ | 6. |
| 13 | D + L | $\cancel{T_a}$ | $\cancel{P_a}$ | $\cancel{R_a}$ | E' | $Y_r + Y_j + Y_m$ | $A_x$ 6. |

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. Concrete walls were originally designed by applying an earthquake loading of 0.2g (SSE) to midspan of wall panel.
6. Missile barrier has been installed.
7. Snow load coefficients in accordance with ANSI A58.1 may be used, or provisions of UBC Section 2311(j) invoked.
8. 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: GINNA

STRUCTURE: PORTIONS OF THE INTERMEDIATE BUILDING (Concrete)

| Combined Loading Cases | Gravity Dead, Live | Thermal | Pressure | Mechanical | Natural Phenomena | Impulsive Loading | Scale Ranking |
|---|---|---|---|---|---|---|---|
| 1 | 1.4D + 1.7L | | | | | | |
| 2 | 1.4D + 1.7L | | | | 1.9E | | |
| 3 | 1.4D + 1.7L | | | | 1.7W | | |
| 4 | .75 (1.4D + 1.7L) | .75 x 1.7 $T_o$ | | .75 x 1.7 $R_o$ | | | |
| 5 | .75 (1.4D + 1.7L) | .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✓ consequently no load factors were used / ~~ultimate strength~~ }
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. No information found on building design basis. A reasonable assumption is that design was to same basis as rest of building complex. Consequently, intermediate building is taken as the auxiliary building.
6. 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

STEEL STRUCTURES (Elastic Analysis)

PLANT: GINNA

STRUCTURE: PORTIONS OF THE INTERMEDIATE BUILDING (steel)

| Combined Loading Cases | Gravity Dead, Live | Thermal | Pressure | Mechanical | Natural Phenomena | Impulsive Loading | Scale |
|---|---|---|---|---|---|---|---|
| 1 | D + L | | | | | | |
| 2 | D + L | | | | E | | |
| 3 | D + L | | | | W | | |
| 4 | D + L | $T_o$ | | $R_o$ | | | |
| 5 | D + L | $T_o$ | | $R_o$ | E | | |
| 6 | D + L | $T_o$ | | $R_o$ | W | | |
| 7 | D + L | $T_o$ | | $R_o$ | E' | | |
| 8 | D + L | $T_o$ | | $R_o$ | $W_t$ | | $A_x$ |
| 9 | D + L | $T_a$ | $P_a$ | $R_a$ | | | |
| 10 | D + L | $T_a$ | $P_a$ | $R_a$ | E | $Y_j + Y_r + Y_m$ | |
| 11 | D + L | $T_a$ | $P_a$ | $R_a$ | E' | $Y_j + Y_r + Y_m$ | $A_x$ |

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

Notes

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

2. Loads deemed inapplicable or negligible struck from loading combinations.

3. Information not found. A reasonable assumption is that the design was to the same basis as other structures in the building complex. Consequently, intermediate building is taken as the auxiliary building.

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

COMPARISON OF LOADING COMBINATION CRITERIA

CONCRETE STRUCTURES

PLANT: GINNA

STRUCTURE:
CABLE TUNNEL

| Combined Loading Cases | Gravity Dead, Live | Thermal | Pressure | Mechanical | Natural Phenomena | Impulsive Loading | Scale Ranking |
|---|---|---|---|---|---|---|---|
| 1 | 1.4D + 1.7L | | | | | | |
| 2 | 1.4D + 1.7L | | | | 1.9E | | |
| 3 | 1.4D + 1.7L | | | | 1.7W | | |
| 4 | .75 (1.4D + 1.7L) | .75 x 1.7 $T_o$ | | .75 x 1.7 $R_o$ | | | |
| 5 | .75 (1.4D + 1.7L) | .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 | $T_o$ | | $R_o$ | $W_t$ | | |
| 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. No information on design of the cable tunnel was found.
6. For purposes of the SEP Review, demonstration that structural integrity is maintained for load case 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: GINNA

STRUCTURE: INTAKE/DISCHARGE STRUCTURE & SCREEN HOUSE (concrete)

| Combined Loading Cases | Gravity Dead, Live | Thermal | Pressure | Mechanical | Natural Phenomena | Impulsive Loading | Scale Ranking |
|---|---|---|---|---|---|---|---|
| 1 | 1.4D + 1.7L 2. | | | | | | |
| 2 | 1.4D + 1.7L 2. | | | | 1.9E | | |
| 3 | 1.4D + 1.7L 2. | | | | 1.7W | | |
| 4 | .75 (1.4D + 1.7L) | $.75 \times 1.7\ T_o$ | | $.75 \times 1.7\ R_o$ | | | |
| 5 | .75 (1.4D + 1.7L) | $.75 \times 1.7\ T_o$ | | $.75 \times 1.7\ R_o$ | .75 x 1.9E | | |
| 6 | .75 (1.4D + 1.7L) | $.75 \times 1.7\ T_o$ | | $.75 \times 1.7\ R_o$ | .75 x 1.7W | | |
| 7 | 1.2D | | | | 1.9E | | |
| 8 | 1.2D | | | | 1.7W | | |
| 9 | D + L | $T_o$ | | $R_o$ | E' | | |
| 10 | D + L | $T_o$ | | $R_o$ | $W_t$ | | $A_x$ |
| 11 | D + L | $\cancel{T_a}$ | $1.5\ \cancel{P_a}$ | $\cancel{R_a}$ | | | |
| 12 | D + L | $\cancel{T_a}$ | $1.25\ \cancel{P_a}$ | $\cancel{R_a}$ | 1.25E | $Y_r + Y_j + Y_m$ | |
| 13 | D + L | $\cancel{T_a}$ | $\cancel{P_a}$ | $\cancel{R_a}$ | 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 ✓ consequently no load factors were used. ~~ultimate strength~~
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 load coefficients in accordance with ANSI A58.1 may be used, or provisions of UBC Section 2311(j) invoked.
6. For purposes of the SEP Review, demonstration that structural integrity is maintained for load case 10 & 13 (per current criteria) may be considered as providing reasonable assurance that this structure meets the intent of current design criteria.

COMPARISON OF LOADING COMBINATION CRITERIA

CONCRETE STRUCTURES

PLANT: GINNA

STRUCTURE:
DIESEL GENERATOR ANNEX
(concrete)

| 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 2. | | | | 1.9E 5. | | |
| 3 | 1.4D + 1.7L | | | | 1.7W | | |
| 4 | .75 (1.4D + 1.7L) | .75 x 1.7 $T_o$ | | .75 x 1.7 $\cancel{R_o}$ | | | |
| 5 | .75 (1.4D + 1.7L) | .75 x 1.7 $T_o$ | | .75 x 1.7 $\cancel{R_o}$ | .75 x 1.9E | | |
| 6 | .75 (1.4D + 1.7L) | .75 x 1.7 $T_o$ | | .75 x 1.7 $\cancel{R_o}$ | .75 x 1.7W | | |
| 7 | 1.2D | | | | 1.9E | | |
| 8 | 1.2D | | | | 1.7W | | |
| 9 | D + L | $T_o$ | | $\cancel{R_o}$ | E' 5. | | |
| 10 | D + L | $T_o$ | | $\cancel{R_o}$ | $W_t$ | | $A_x$ |
| 11 | D + L | $\cancel{T_a}$ | 1.5 $\cancel{P_a}$ | $\cancel{R_a}$ | | | |
| 12 | D + L | $\cancel{T_a}$ | 1.25 $\cancel{P_a}$ | $\cancel{R_a}$ | 1.25E | $Y_r + Y_j + Y_m$ | |
| 13 | D + L | $\cancel{T_a}$ | $\cancel{P_a}$ | $\cancel{R_a}$ | 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. Earthquake loadings taken as static g-loads.
6. Snow load coefficients in accordance with ANSI A58.1 may be used, or provisions of UBC Section 2311(j) invoked.
7. For purposes of the SEP Review, demonstration that structural integrity is maintained for load cases 10 and 13 (per current criteria) may be considered as providing reasonable assurance that this structure meets the intent of current design criteria.

## 11. REVIEW FINDINGS

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

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

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

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

These tables provide:

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

Some listed provisions may apply only to elements that do not exist in the Ginna 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. This provides:

1. full texts of each revised provision in both the former and current versions
2. comments or conclusions, or both, relevant to the code change
3. the scale ranking of the change.

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

MAJOR FINDINGS OF AISC 1963 VS. AISC 1980 CODE COMPARISON

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

Scale A

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

MAJOR FINDINGS OF AISC 1963 VS. AISC 1980 CODE COMPARISON

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

Scale A (Cont.)

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

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

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

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

Scale A

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

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

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

Scale A (Cont.)

| Referenced Subsection ACI 349-76 | ACI 318-63 | Structural Elements Potentially Affected | Comments |
|---|---|---|---|
| 11.16 | -- | All structural walls - those which are primary load carrying, e.g., shear walls and those which serve to provide protection from impacts of missile-type objects. | Guidelines for these kinds of wall loads were not provided by older codes; therefore, structural integrity may be seriously endangered if the design fails to fulfill these requirements. |
| Appendix A | -- | All elements subject to time-dependent and position-dependent temperature variations and restrained so that thermal strains will result in thermal stresses. | For structures subject to effects of pipe break, especially jet impingement, thermal stresses may be significant. Scale A for areas of jet impingement or where the conditions could develop causing concrete temperature to exceed limitation of A.4.2.<br><br>For structures not subject to effects of pipe break accident, thermal stresses are unlikely to be significant (Scale B). |

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

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

Scale A (Cont.)

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

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

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

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

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

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

Scale A

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

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

Scale A (Cont.)

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

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

Scale A (Cont.)

| Referenced Subsection Sec. III 1980 | ACI 318-63 | Structural Elements Potentially Affected | Comments |
|---|---|---|---|
| CC-3440(b),(c) | --- | All concrete elements which could possibly be exposed to short term high thermal loading | New limitations are imposed on short-term thermal loading. No comparable provisions existed in the ACI 318-63. |
| CC-3532.1.2 | --- | Where biaxial tension exists | ACI 318-63 did not consider the problem of development length in biaxial tension fields. |

## 12. SUMMARY

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

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

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

SUMMARY
NUMBER OF CODE CHANGE IMPACTS
FOR GINNA CATEGORY I STRUCTURES

| SCALE RANKING | | ACI 318-63 VS. ACI 349-76 | AISC 1963 VS. AISC 1980 | ACI 301-63 VS. ACI 301-72 (1975 Rev.) | ACI 318-63 VS. ASME B&PV SEC.III Div. 2, 1980 |
|---|---|---|---|---|---|
| Total Changes Found | | 82 | 33 | 37 | 40 |
| Do Not Require Further Investigation | A or $A_x$ Not Applicable to Ginna | 1 + 4* | 11 | 0 | 3* |
| | B | 63 | 10 | 21 | 27 |
| | C | 7 | 4 | 16 | 4 |
| To Be Further Investigated | A | 7 | 8 | 0 | 6 |
| | $A_x$ | 0 | 0 | 0 | 0 |

SCALE RATINGS:

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

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

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

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

# 13. RECOMMENDATIONS

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

It is recommended that the Rochester Gas and Electric Corporation be requested to take three actions:

1. Review individually all Seismic Category I structures at the Ginna 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 Ginna plant.

LIST OF STRUCTURAL ELEMENTS TO BE EXAMINED

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

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

LIST OF STRUCTURAL ELEMENTS TO BE EXAMINED (Cont.)

| Structural Elements to be Examined | Code Change Affecting These Elements | | |
|---|---|---|---|
| | New Code | Old Code | Scale |
| Members Designed to Operate in an Inelastic Regime | AISC 1980 | AISC 1963 | |
| Spacing of lateral bracing | 2.9 | 2.8 | A |
| Short Brackets and Corbels having a shear span-to-depth ratio of unity or less | ACI 349-76<br>11.13 | ACI 318-63<br>-- | A |
| Shear Walls used as a primary load-carrying member | ACI 349-76<br>11.16 | ACI 318-63<br>-- | A |
| Precast Concrete Structural Elements, where shear is not a measure of diagonal tension | ACI 349-76<br>11.15 | ACI 318-63<br>-- | A |
| Concrete Regions Subject to High Temperatures | ACI 349-76 | ACI 318-63 | |
| Time-dependent and position-dependent temperature variations | Appendix A | -- | A |
| Columns with Spliced Reinforcement | ACI 349-76 | ACI 318-63 | |
| subject to stress reversals; $f_y$ in compression to 1/2 $f_y$ in tension | 7.10.3 | 805 | A |
| Steel Embedments used to transmit load to concrete | ACI 349-76<br>Appendix B | ACI 318-63<br>-- | A |
| Elements Subject to Impulsive and Impactive Loads whose failure must be precluded | ACI 349-76<br>Appendix C | ACI 318-63<br>-- | A |
| Containment and Other Elements, transmitting In-plane shear | B&PV Code Section III, Div. 2, 1980<br>CC-3421.5 | ACI 318-63<br>-- | A |

LIST OF STRUCTURAL ELEMENTS TO BE EXAMINED (Cont.)

| Structural Elements to be Examined | Code Change Affecting These Elements New Code | Old Code | Scale |
|---|---|---|---|
| Region of shell carrying concentrated forces normal to the shell surface (see case study 13 for details) | B&PV Code, Section III, Div. 2, 1980 CC-3421.6 | ACI 318-63 1707 | A |
| Region of shell under torsion | B&PV Code Section III, Div. 2, 1980 CC-3421.7 | ACI 318-63 921 | A |
| Elements Subject to Short-term High Temperature Loading | B&PV Code, Section III, Div. 2, 1980 CC-3440(b),(c) | ACI 318-63 -- | A |
| Elements Subject to Biaxial Tension | B&PV Code, Section III, Div. 2, 1980 CC-3532.1.2 | ACI 318-63 -- | A |
| Brackets and Corbels | B&PV Code, Section III, Div. 2, 1980 CC-3421.8 | ACI 318-63 -- | A |
| Roofs | -- | -- | A(1) |

Extreme environmental snow loads are provided by SEP Topic II-2.A Regulatory Guide 1.102 (Position 3) provides guidance to preclude adverse consequences from ponding on 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, NUREG-0800 (Formerly NUREG-75/087), Rev. 1, NRC, July 1981

2. Specification for Design, Fabrication, and Erection of Structural Steel for Buildings
   American Institute of Steel Construction (AISC), 1963

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

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

5. Rochester Gas and Electric Corp.
   Final Facility Description and Safety Analysis
   Report for Robert Emmett Ginna Nuclear Power Plant Unit 1

6. Gilbert Associates, Inc.
   Drawings Nos. 04-4750-D-024-002 through 04-4750-D-024-020

7. 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
   TER-C5257-327

APPENDIX A

SCALE A AND SCALE $A_x$ CHANGES
DEEMED INAPPROPRIATE TO GINNA PLANT

Franklin Research Center
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The Benjamin Franklin Parkway, Phila., Pa. 19103 (215) 448-1000

# APPENDIX A-1

## AISC 1963 VS. AISC 1980 CODE COMPARISON

(SCALE A AND SCALE $A_x$ CHANGES DEEMED INAPPROPRIATE TO MILLSTONE UNIT 1
OR CODE CHANGES RELATED TO LOADS OR LOAD COMBINATIONS
AND THEREFORE TREATED ELSEWHERE)

# AISC 1963 VS. AISC 1980 CODE COMPARISON

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

AISC 1963 VS. AISC 1980 CODE COMPARISON

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

AISC 1963 VS. AISC 1980 CODE COMPARISON

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

APPENDIX A-2

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

(SCALE A AND SCALE $A_x$ CHANGES DEEMED INAPPROPRIATE TO GINNA
OR CODE CHANGES RELATED TO LOADS OR LOAD COMBINATIONS
AND THEREFORE TREATED ELSEWHERE)

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

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

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

APPENDIX A-3

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

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

(SCALE A AND SCALE $A_x$ CHANGES DEEMED INAPPROPRIATE TO GINNA OR CODE CHANGES RELATED TO LOAD COMBINATIONS AND THEREFORE TREATED ELSEWHERE)

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

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

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

APPENDIX B

SUMMARIES OF CODE COMPARISON FINDINGS

APPENDIX B

SUMMARIES OF CODE COMPARISON FINDINGS

APPENDIX B-1

AISC 1963 VS. AISC 1980

SUMMARY OF CODE COMPARISON

AISC 1963 VS. AISC 1980
SUMMARY OF CODE COMPARISON

Scale A

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

AISC 1963 VS. AISC 1980
SUMMARY OF CODE COMPARISON

Scale A

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

AISC 1963 VS. AISC 1980
SUMMARY OF CODE COMPARISON

Scale A (Cont.)

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

AISC 1963 VS. AISC 1980
SUMMARY OF CODE COMPARISON

Scale B

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

AISC 1963 VS. AISC 1980
SUMMARY OF CODE COMPARISON

Scale B (Cont.)

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

## AISC 1963 VS. AISC 1980
## SUMMARY OF CODE COMPARISON

Scale C

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

APPENDIX B-2

ACI 318-63 VS. ACI 349-76

SUMMARY OF CODE COMPARISON

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

Scale A

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

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

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

Scale A (Cont.)

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

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

Scale A (Cont.)

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

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

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

Scale B

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

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

Scale B (Cont.)

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

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

Scale B (Cont.)

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

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

Scale B (Cont.)

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

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

Scale B (Cont.)

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

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

Scale B (Cont.)

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

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

Scale B (Cont.)

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

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

Scale B (Cont.)

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

Scale B (Cont.)

| Referenced Section ACI 349-76 | ACI 318-63 | Structural Elements Potentially Affected | Comments |
|---|---|---|---|
| 11.13.1<br>11.13.2 | -- | Columns | No problem anticipated since previous code required design consideration by some analysis. |
| Chapter 12 | -- | Reinforcement | Development length concept replaces bond stress concept in ACI 318-63.<br>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<br>through<br>12.1.6.3 | 918(C) | Reinforcement | Modified with minimum added to ACI 318-63, 918(C). |
| 12.2.2<br>12.2.3 | -- | Reinforcement | New insert in ACI 349-76. |
| 12.4 | -- | Reinforcement of special members | New insert.<br>Gives emphasis to special member consideration. |
| 12.8.1<br>12.8.2 | -- | Standard hooks | Based on ACI 318-63 bond stress allowables in general; therefore, no major change. |

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

Scale B (Cont.)

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

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

Scale B (Cont.)

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

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

Scale B (Cont.)

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

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

Scale C

| Referenced Section ACI 349-76 | ACI 318-63 | Structural Elements Potentially Affected | Comments |
|---|---|---|---|
| 7.13.4 | -- | Reinforcement in flexural slabs | |
| 10.8.1<br>10.8.2<br>10.8.3 | 912 | Compression members, limiting dimensions | Minimum size limitations are deleted in newer Code, giving the designer more freedom in cross-sectional dimensioning. |
| 10.14 | 2306 | Bearing - sections controlled by design bearing stresses | ACI 318-63 is more conservative, allowing a stress of $1.9(0.25\ f'_c) = 0.475\ f'_c < 0.6\ f'_c$ |
| 11.2.5 | 1706 | Reinforcement concrete members without prestressing | Allowance of spirals as shear reinforcement is new. Requirement of two lines of web reinforcement, 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. |
| 17.5.4<br>17.5.5 | -- | Permissible horizontal shear stress for any surface, ties provided or not provided | Nominal increase in allowable shear stress under new code. |

APPENDIX B-3

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

SUMMARY OF CODE COMPARISON

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

Scale B

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

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

Scale B (Cont.)

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

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

Scale B (Cont.)

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

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

Scale B (Cont).

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

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

Scale C

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

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

Scale C (Cont.)

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

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

Scale C (Cont.)

| Referenced Section ACI 301-72 | ACI 301-63 | Structural Elements Potentially Affected | Comments |
|---|---|---|---|
| 5.2.2.1<br>5.2.2.2 | -- | Better strength, less chance of cracked reinforcing bars | ACI 301-72 (Rev. 1975) has less stringent bending requirement for reinforcing bars than does ACI 318-63. |
| 5.5.4<br>5.5.5 | 505b | Better strength from reinforcement | ACI 301-63 provides for more overlap in welded wire fabric. |
| 12.2.3 | 1201d | Better strength from better cure of concrete | ACI 301-63 provides for final curing for 7 days with air temperature above 50°F. ACI 301-72 (Rev. 1975) provides for curing for 7 days and compressive strength of test cylinders to be 70 percent of specified strength. This could allow termination of cure too soon. |
| 14.4.1 | 1404 | Better strength resulting from better uniformity | ACI 301-63 provides for a maximum slump of 2 in. ACI 301-72 (Rev. 1975) gives a tolerance on the maximum slump which could lead to nonuniformity in the concrete in place. |
| 15.2.1.1 | 1502-c1b | Higher strength from higher yield prestressing bars | ACI 301-63 requires higher yield stress than does ACI 301-72 (Rev. 1975) |
| 15.2.1.2 | 1502-c2 | Higher strength from better prestressing steel | ACI 301-63 requires that stress curves from the production lot of steel be furnished. ACI 301-72 (Rev. 1975) requires that a typical stress-strain curve be submitted. The use of the typical curve may miss lower strength material. |

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

Scale C (Cont.)

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

APPENDIX B-4

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

SUMMARY OF CODE COMPARISON

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

Scale A

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

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

Scale A (Cont.)

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

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

Scale A (Cont.)

| Referenced Subsection Sec. III 1980 | Referenced Subsection ACI 318-63 | Structural Elements Potentially Affected | Comments |
|---|---|---|---|
| CC-3421.8 | --- | Bracket and corbels | New provisions. No comparable section in ACI 318-63; therefore, any existing corbels or brackets may not meet these criteria and failure of such elements could be non-ductile type failure.<br><br>Structural integrity may be seriously endangered if the design fails to fulfill these requirements. |
| CC-3440(b),(c) | --- | All concrete elements which could possibly be exposed to short-term high thermal loading | New limitations are imposed on short term thermal loading. No comparable provisions existed in the ACI 318-63. |
| CC-3532.1.2 | --- | Where biaxial tension exists | ACI 318-63 did not consider the problem of development length in biaxial tension fields. |
| CC-3900 All sections in this chapter | --- | Concrete containment* | New design criteria. ACI 318-63 did not contain design criteria for loading such as impulse or missile impact. Therefore, no comparison is possible for this section. |

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

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

Scale B

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

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

Scale B (Cont.)

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

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

Scale B (Cont.)

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

Scale B (Cont.)

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

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

Scale B (Cont.)

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

Scale B (Cont).

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

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

Scale B (Cont).

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

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

Scale B (Cont.)

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

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

Scale B (Cont.)

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

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

Scale C

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

APPENDIX C

# COMPARATIVE EVALUATIONS AND MODEL STUDIES

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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 | 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

$$P = 0.85 \left[A_g \left(0.25 f_c' + f_s p_g\right)\right]$$

where $p_g = \frac{A_{st}}{A_g}$ and allowable $f_s = 0.4f_y \leq$ 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

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

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 $P_{allow} = \frac{P_u}{L.F.}$

(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

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

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

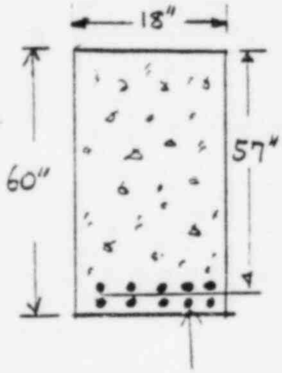
Comparison of these resulting equations to the $P_{allow}$ by working stress design criteria shows that the new code allows from 1.24 to 1.62 times more load on the concrete in a tied column and from 0.97 to 1.18 times more load on the longitudinal steel in a tied column.

Therefore, Scale C

CASE STUDY -3-

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

Sample Section




Allowable Stresses

Concrete : 3000 lb/in² grade
( $f'_c = 3{,}000$, $f_c = 1350$, $n = 9$ )

Reinforcing Steel : Grade 40
( $f_y = 40{,}000$ lb/in², $f_s = 20{,}000$ lb/in² )

$A_s = 10\text{-}\#10$ bars $= 12.66$ in²

I. By Strength Design

$\rho = \frac{12.66}{18 \times 57} = .01234$ (There is a limit of .0278, But a "reasonable" design is half of this.)

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

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

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

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

II. BY <u>Alternate</u> Design

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

$18\, x\, (x/2) = 9\,(12.66)(57 - x)$

solving, $x = kd = 21.27''$

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

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

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

(Governs)

III Comparison:

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

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

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 $C_c$, ... "

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

$E = 29 \times 10^3$ KSI

$F_y$ = yield stress

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

Ref AISC 1963 Code

Subsection 2.3 Columns

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

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

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

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

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

then,

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

2) The 1980 Code is 5% more conservative when

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

then, $F_y = 44$ KSI

Conclusion:

Scale

$F_y \leq 40$ KSI ——— C

$40 < F_y < 44$ ——— B

$F_y > 44$ ——— A

CASE STUDY -5-

Ref AISC 1980 Code

Subsection 1.10.5.3

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

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

Where

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

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

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

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

Ref AISC 1963 Code

Subsection 1.10.5.3

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

$$\frac{11000t}{\sqrt{f_v}}$$ "

REF AISC Sub section 1.10.5.3

EXAMPLE

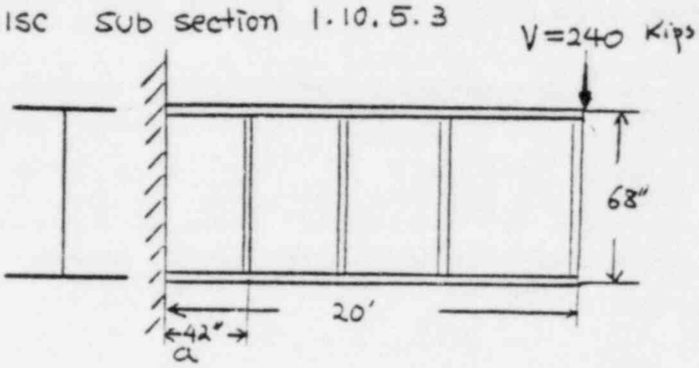
$h = 68''$

$t = .375''$

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

$V = 240$ 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 $\quad \frac{h}{t} = \frac{68}{.375} = 181 \ \& \ \frac{a}{h} = \frac{42}{68} = .618$

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

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

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

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

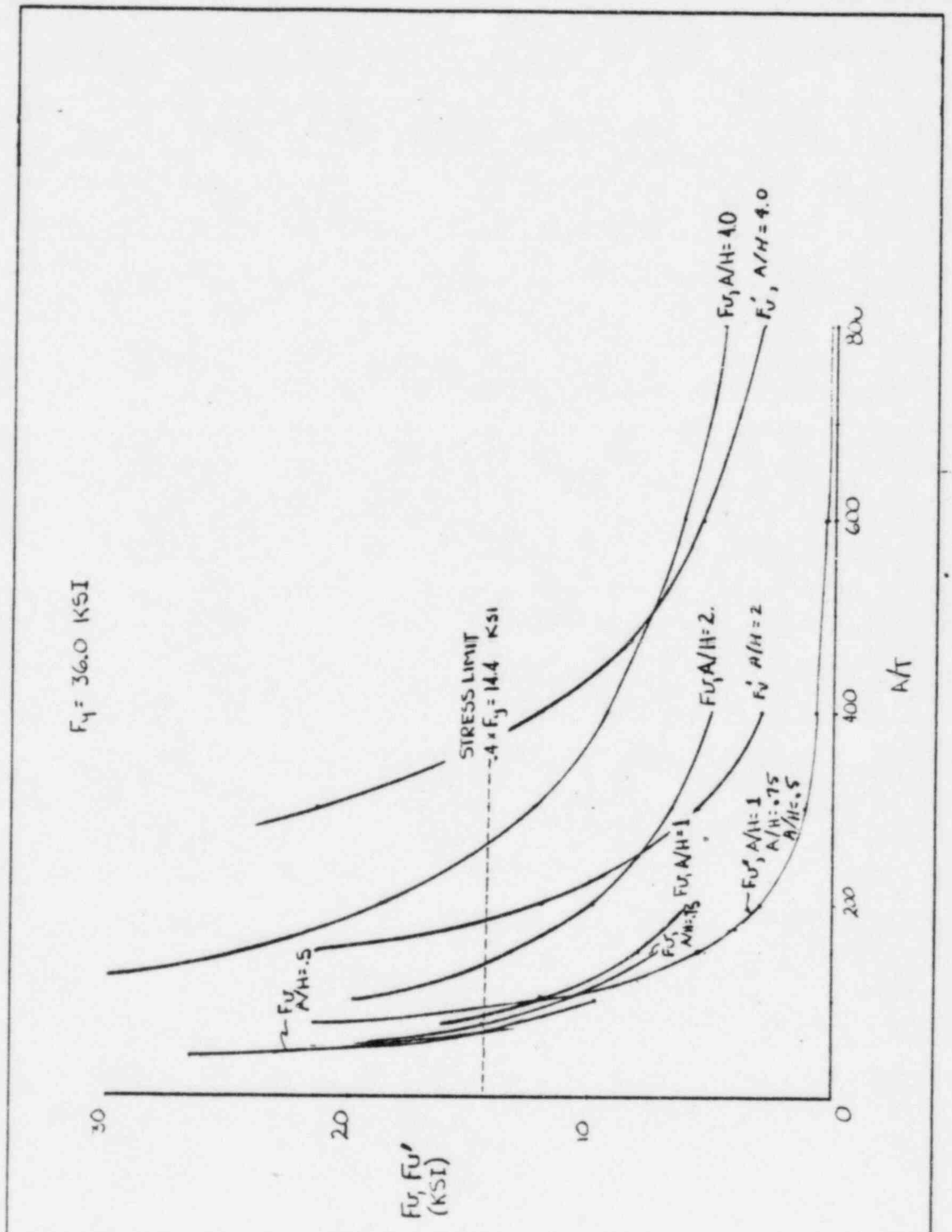
Allowable shear stress ≈ 8.6 Ksi (checks Computed Value)

however, lower than $f_v$ of 9.06 Ksi

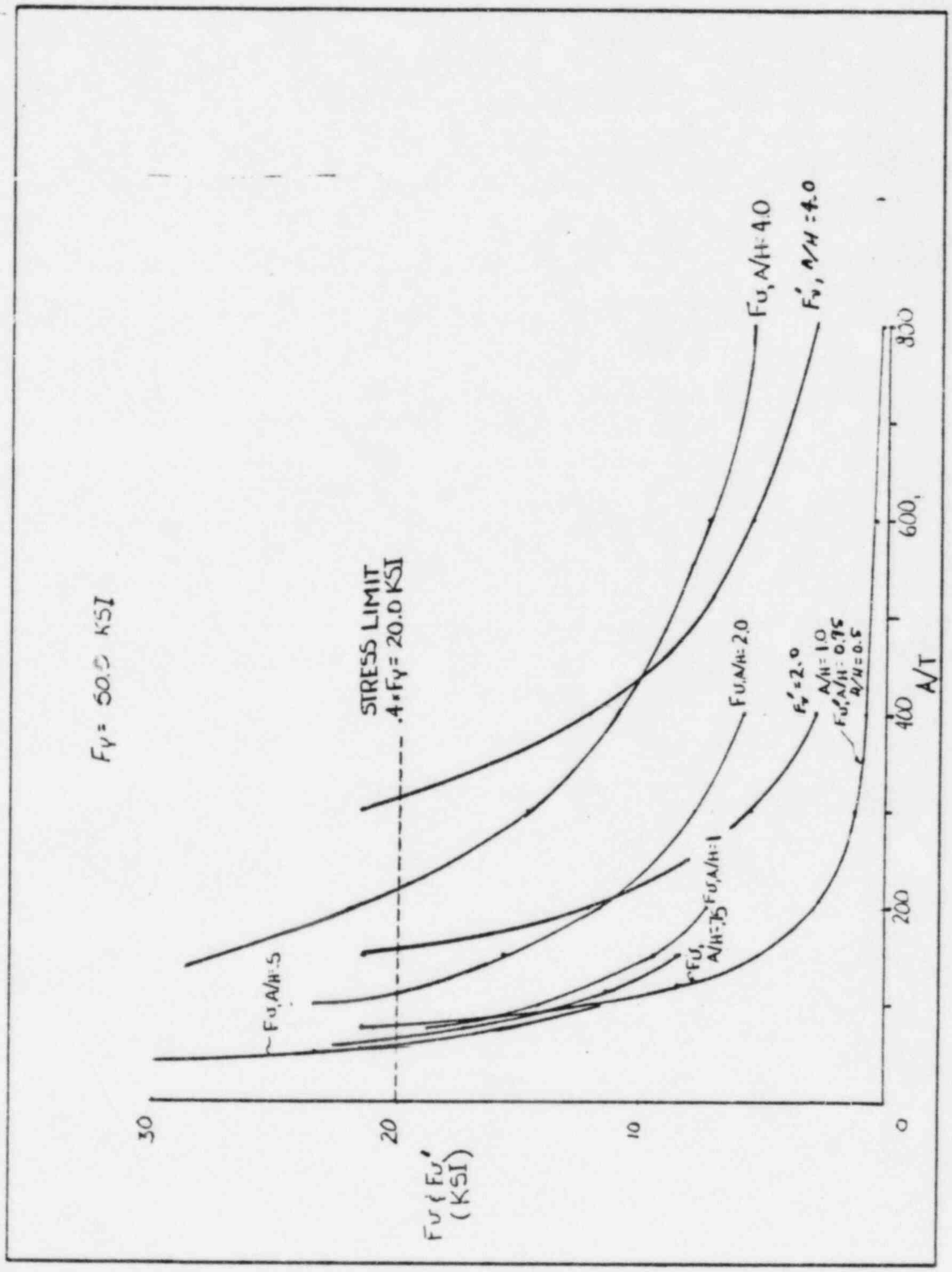
∴ Scale B for this example

Remarks

The following two figures show Fv vs. A/T for various values of A/H and Fy. By knowing the shear stress Fv or Fv' the A/T value can be abtained and compared with the design A/T. Thus comparison should be examined on a case by case basis.



$F_y$ = 36.0 KSI
STRESS LIMIT
.4 × $F_y$ = 14.4 KSI
Fu, A/H = 4.0
Fu', A/H = 4.0
Fu, A/H = 2.
Fu' A/H = 2
Fu, A/H = 1
Fu' A/H = 1
A/H = .75
A/H = .5
Fu' A/H = .5
Fu A/H = .5
Fu, Fu' (KSI)
A/T
0
10
20
30
200
400
600
800


Fy = 50.0 KSI
STRESS LIMIT
.4 × Fy = 20.0 KSI
Fu, A/H = 4.0
Fv', A/H = 4.0
Fu, A/H = 2.0
Fv' = 2.0
A/H = 1.0
Fu, A/H = 0.75
A/H = 0.5
Fu, A/H = 1
Fu, A/H = .75
Fu, A/H = .5
30
20
10
0
200
400
600
800
A/T
Fu & Fu' (KSI)

CASE STUDY -6-

Ref AISC 1980 Code
Section 2.7

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

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

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

Example

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

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

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

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

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

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

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

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

Ref AISC 1963 Code
Section 2.6

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

$b_f/2t_f \leq 8.5$ Rolled Shapes

$b_f/t_f \leq 32$ Box Sections

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

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

Remarks

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

$F_y \leq 36$ KSI Ⓒ

$36 < F_y < 38$ KSI Ⓑ

$F_y \geq 38$ KSI Ⓐ

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

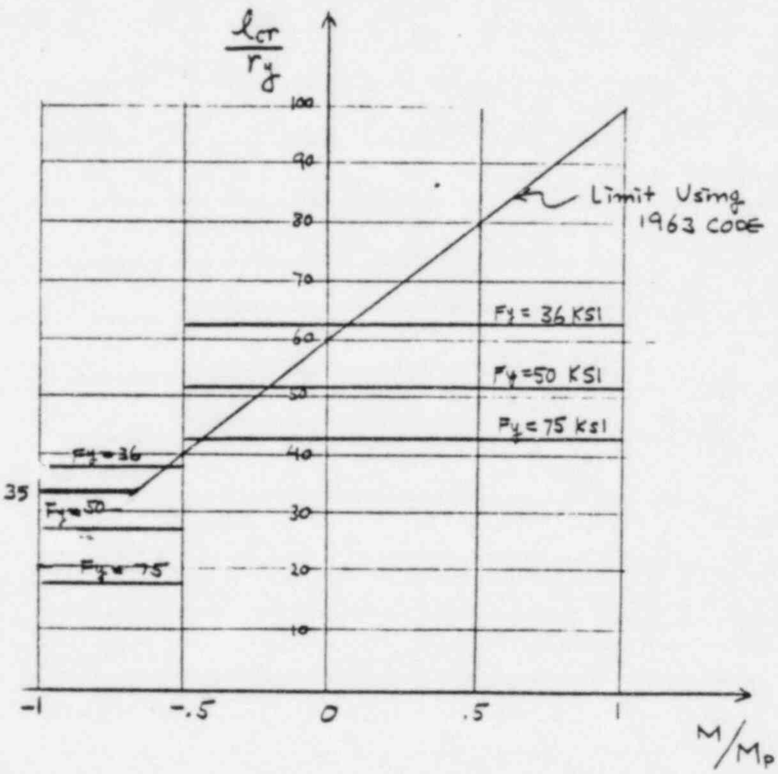
The figure which follows ($l_{cr}/r_y$ vs. $M/M_p$) indicates that for A-36 steel ($F_y$=36 ksi)

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
Fy = 36 KSI
Fy = 50 KSI
Fy = 75 KSI
Fy = 36
Fy = 50
Fy = 75
-1
-.5
0
.5
1
M/Mp

# CASE STUDY -8-

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

| AISC 1963 | AISC 1980 |
|---|---|
| 1. Slenderness ratio for columns in 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 f.. $F_y = 36$ 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)

$$M \leq M_p \quad \text{for } P/P_y \leq 0.15$$

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

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 ($0.6 \le C_m \le 1.0$) and double Curvature ($0.4 \le C_m \le 0.6$) cases.

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

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

Assume braced in weak direction

$\therefore M_m = M_p$

**1963 Code**

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

$M \le M_p$

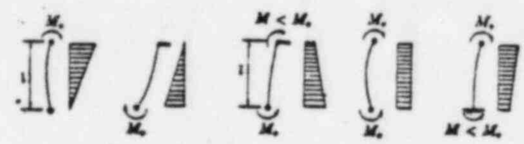
Formula (23) $\frac{M}{M_p} \le 1.0 - H(P/Py) - J(P/Py)^2$

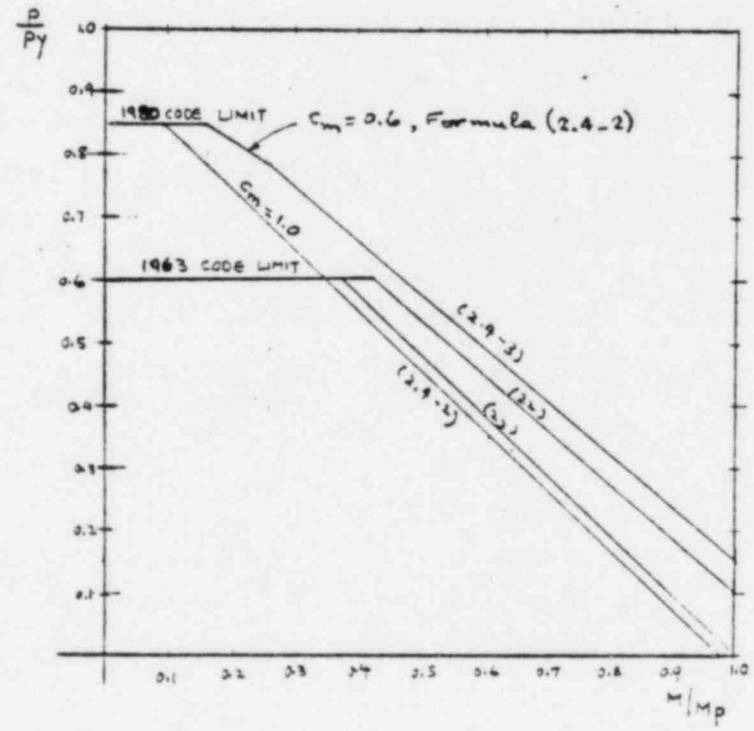
**1980 Code**

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

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

TYPICAL EXAMPLES



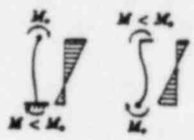



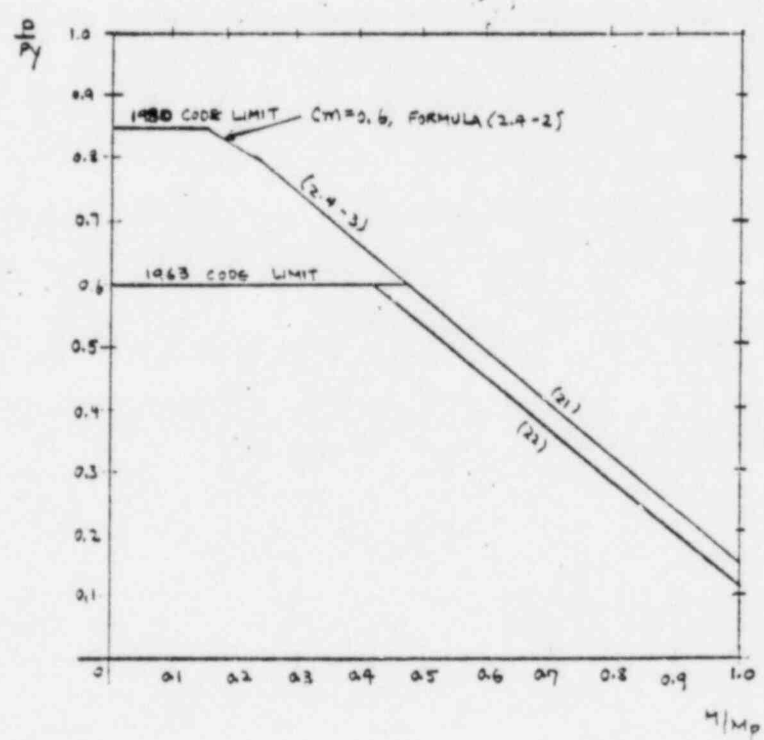
$F_y = 36$ ksi $\quad \frac{kl}{r} = 30 \quad$ 14 w 150

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

| 1963 Code | 1980 Code |
|---|---|
| Formula (21) $M = M_p$ when $P/Py \leq 0.15$ <br> $\frac{M}{M_p} \leq 1.18 - 1.18(P/Py) \leq 1.0$ | (2.4-2) $\frac{P}{P_{cr}} + \frac{C_m M}{(1 - \frac{P}{P_e})M_p} \leq 1.0$ <br> $0.4 \leq C_m \leq 0.6$ |
| Formula (22) $\frac{M}{M_p} \leq B - G(P/Py) \leq 1.0$ <br> $M \leq M_p$ | (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 $\frac{kl}{r} = 70$ 14 → 150 SINGLE CURVATURE

Assume braced in weak direction

∴ $M_m = M_p$

1963 Code

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

$M \leq M_p$

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

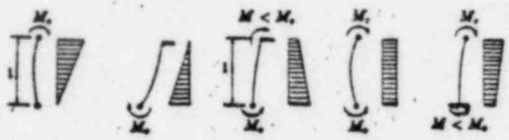
1980 Code

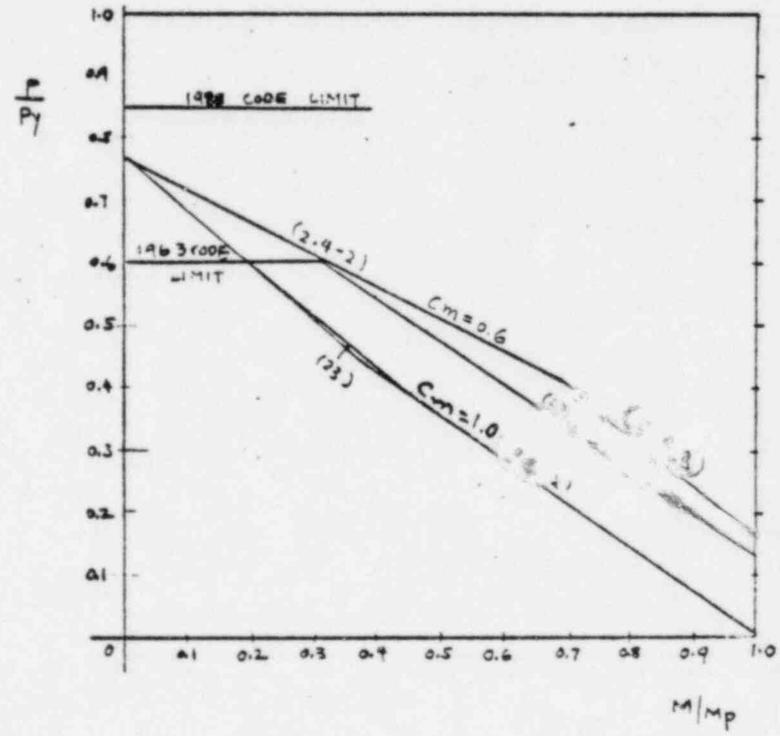
(2.4-2) $\frac{P}{P_{cr}} + \frac{C_m M}{(1 - \frac{P}{P_e})M_p} \leq 1.0$

$0.6 \leq C_m \leq 1.0$

(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 $\quad \frac{kl}{r} = 70 \quad$ 14 wf 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) $\frac{M}{M_p} \leq B - G(P/Py) \leq 1.0$

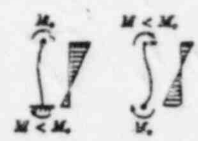
$$M \leq M_p$$

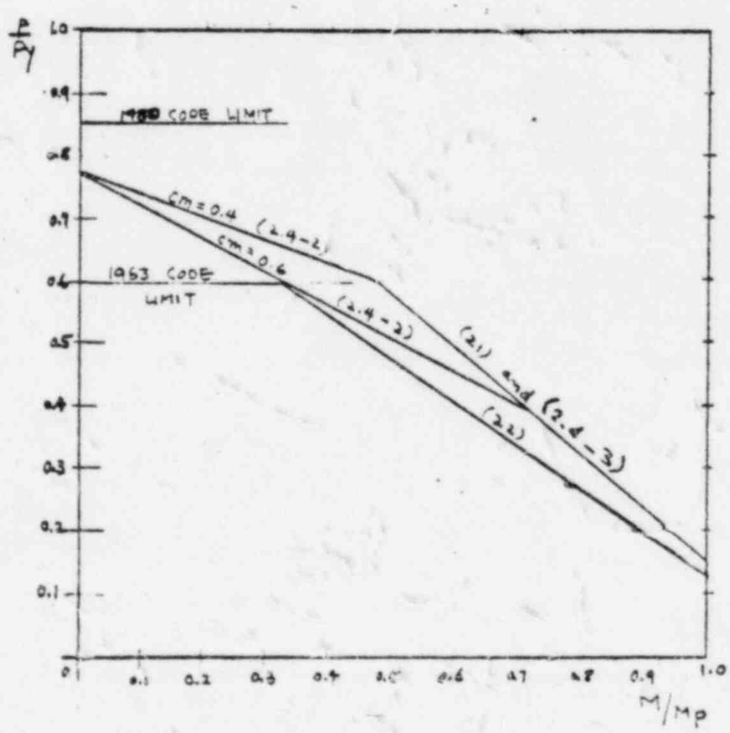
**1980 Code**

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

TYPICAL EXAMPLES



$F_y$ = 36 ksi  $\frac{kl}{r}$ = 100 14 WF 150  SINGLE CURVATURE

Assume braced in weak direction

$\therefore M_m = M_p$

<u>1963 Code</u>

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

$M \le M_p$

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

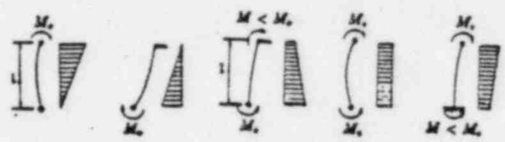
<u>1980 Code</u>

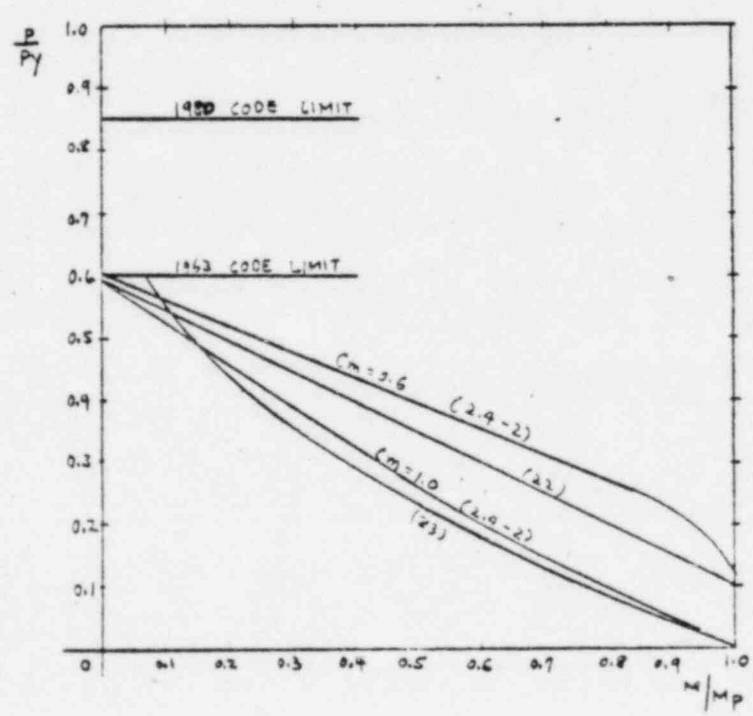
(2.4-2) $\frac{P}{P_{cr}} + \frac{C_m M}{(1 - \frac{P}{P_e})M_p} \le 1.0$

$0.6 \le C_m \le 1.0$

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

<u>TYPICAL EXAMPLES</u>



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

Assume braced in weak direction
∴ $M_m = M_p$

1963 Code

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

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

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

$$M \le M_p$$

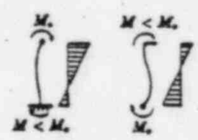
1980 Code

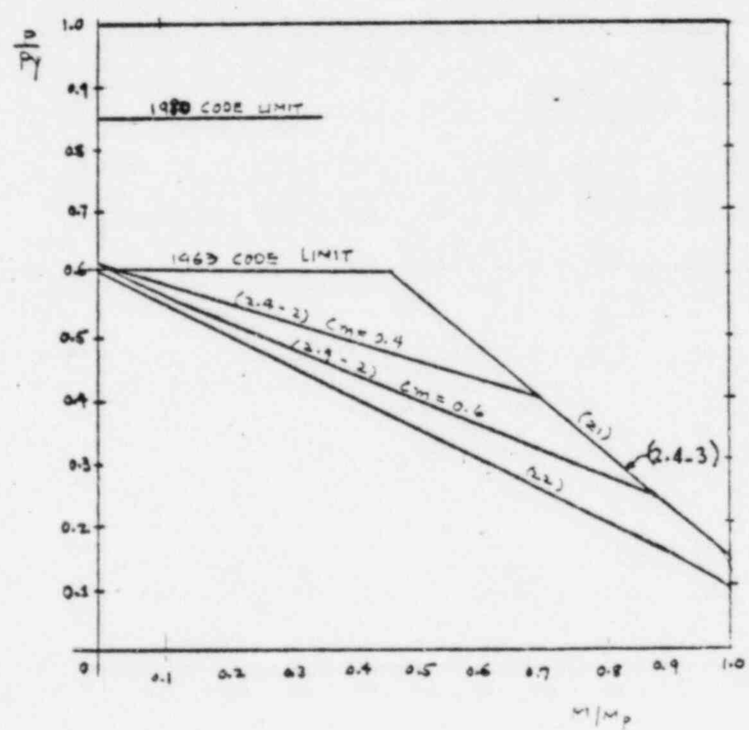
(2.4-2) $\frac{P}{P_{cr}} + \frac{C_m M}{(1 - \frac{P}{P_e})M_p} \le 1.0$

$$0.4 \le C_m \le 0.6$$

(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 $\quad \frac{kl}{r} = 30 \quad$ 12 WF 45 $\quad$ SIDESWAY ALLOWED

Assume braced in weak direction

$\therefore M_{ox} = M_p$

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

$$M \le M_p$$

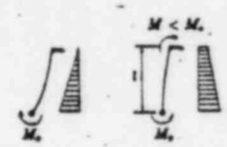
Formula (23) $\frac{M}{M_p} \le 1.0 - H(P/Py) - J(P/Py)^2$

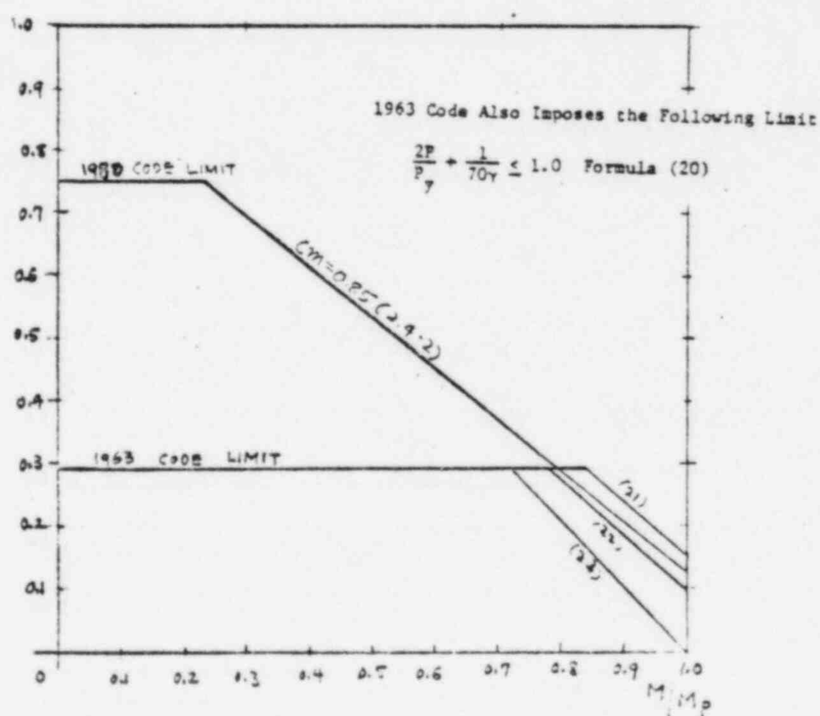
**1980 Code**

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

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

TYPICAL EXAMPLES



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

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

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

$$M \le M_p$$

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

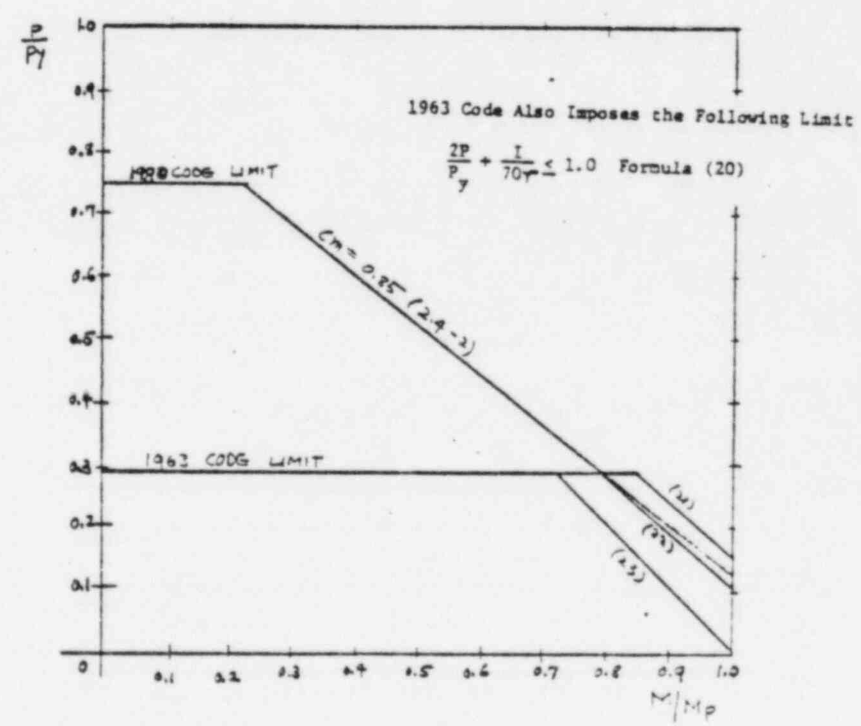
1980 Code

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

$C_m = 0.85$

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

TYPICAL EXAMPLES

CASE STUDY -9-

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

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

But for $0.45 < \alpha \leq 0.75$, Formula (1.10-6) or Formula (1.10-5) could be conservative as compared to each other depending on h/t ratio for given $F_b$. 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 $F_b$ 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 $F_b$ in ksi

**New Formula**

Formula (1.10-6) 1980 code

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

| $\alpha$ | Scale |
|---|---|
| $\leq 0.45$ and low $A_w/A_f$ ratio | A |
| 0.45 to 0.75 | B |
| $\geq 0.75$ | C |

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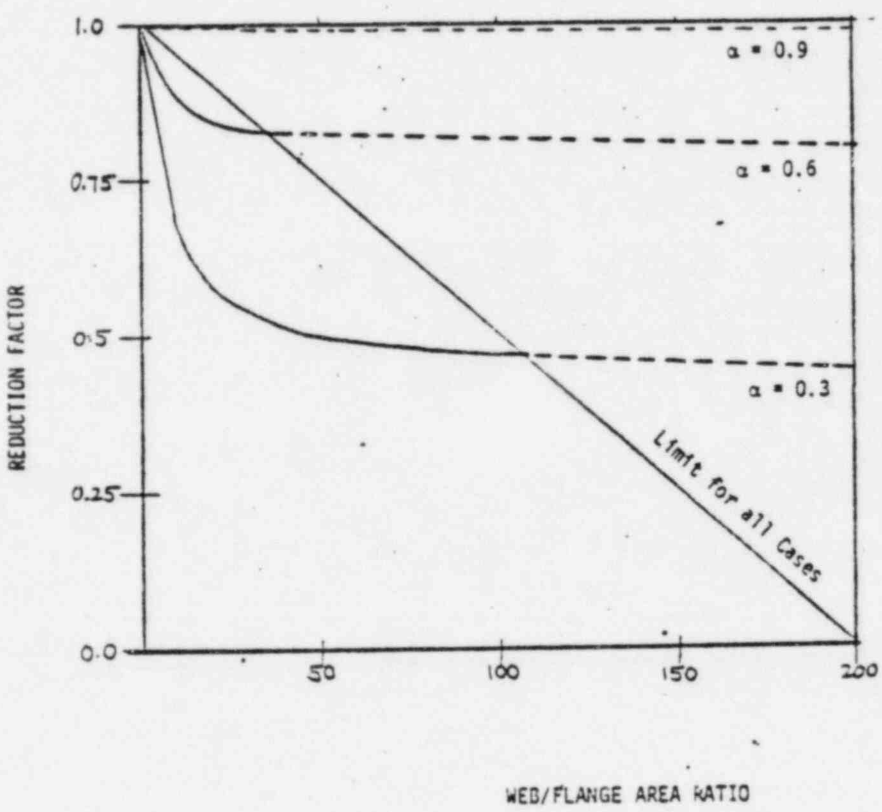
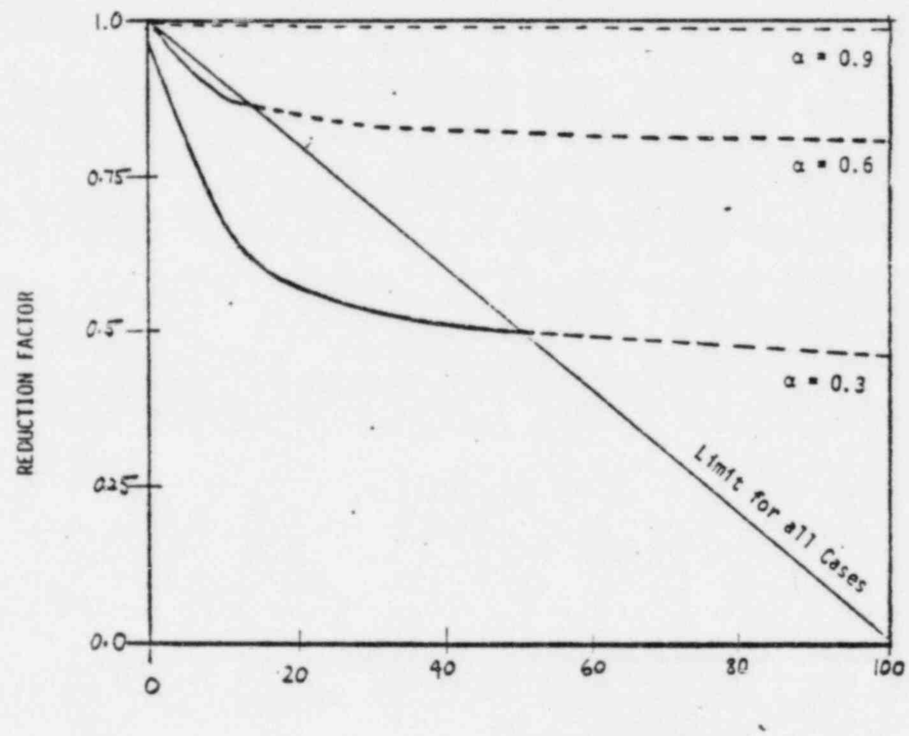
AISC 1.10.6 1963/1980 CODE COMPARISON




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

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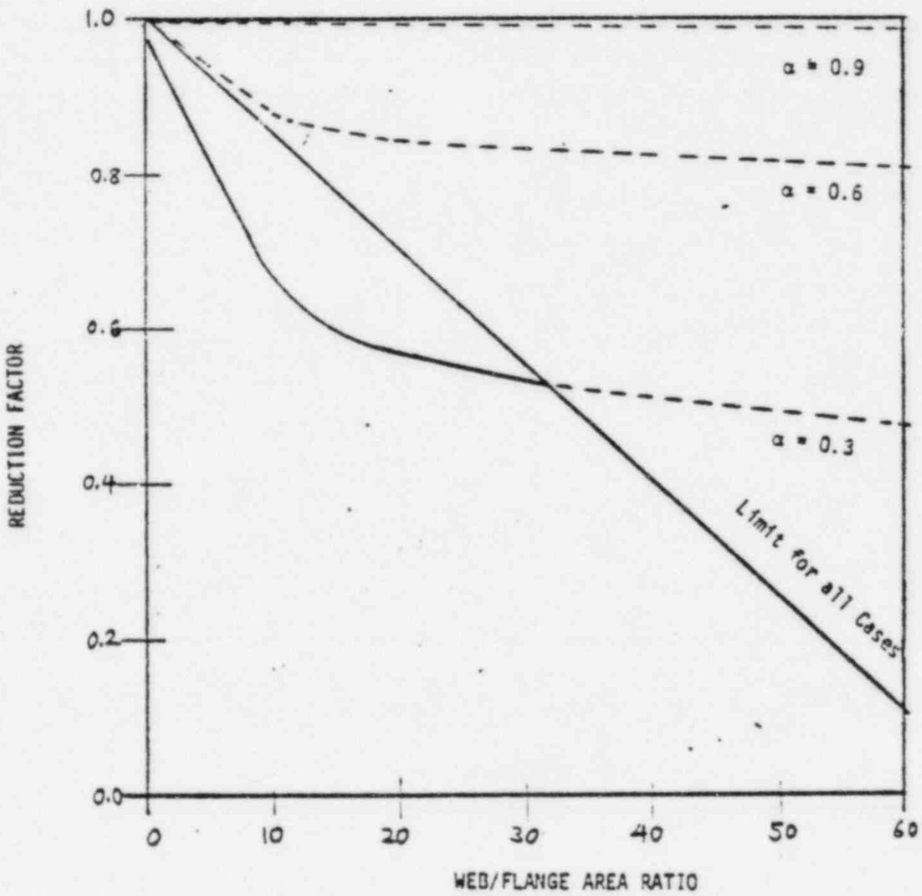
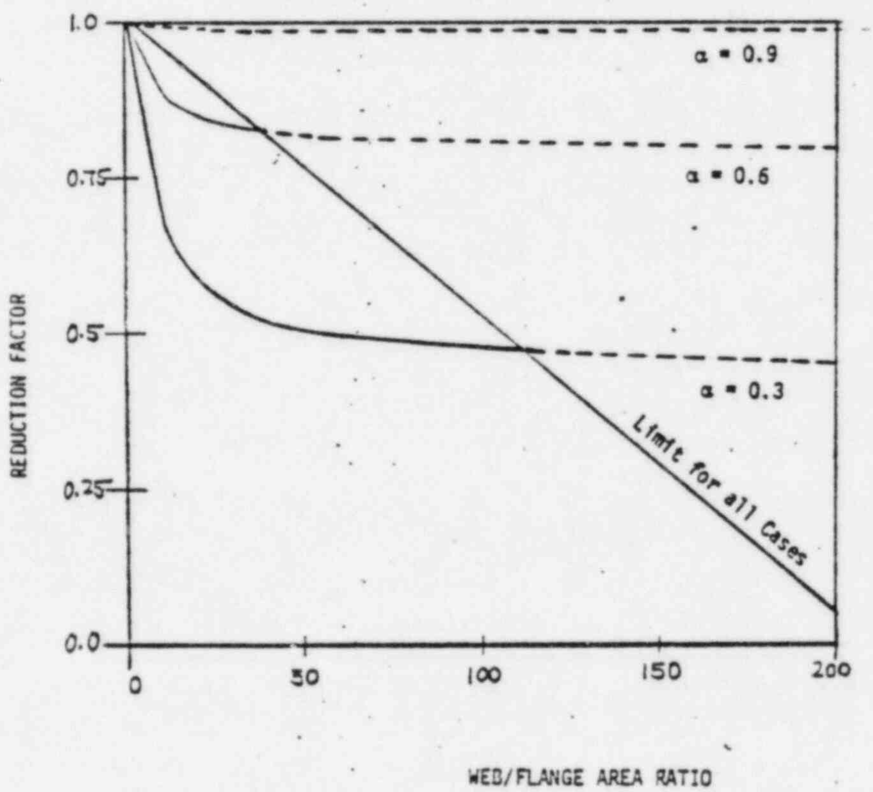
AISC 1.10.6 1963/1980 CODE COMPARISON



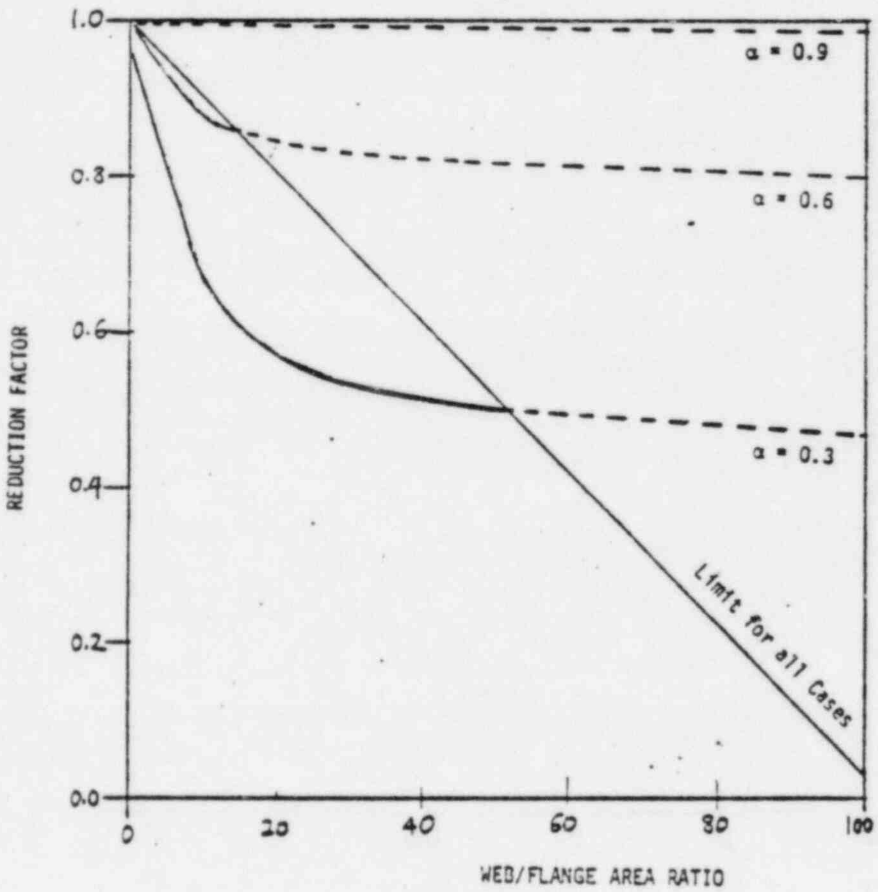

BENDING STRESS = 25KSI ALPHA=0.3, 0.6, 0.9, H/T RATIO = 182

AISC 1.10.6 1963/1980 CODE COMPARISON



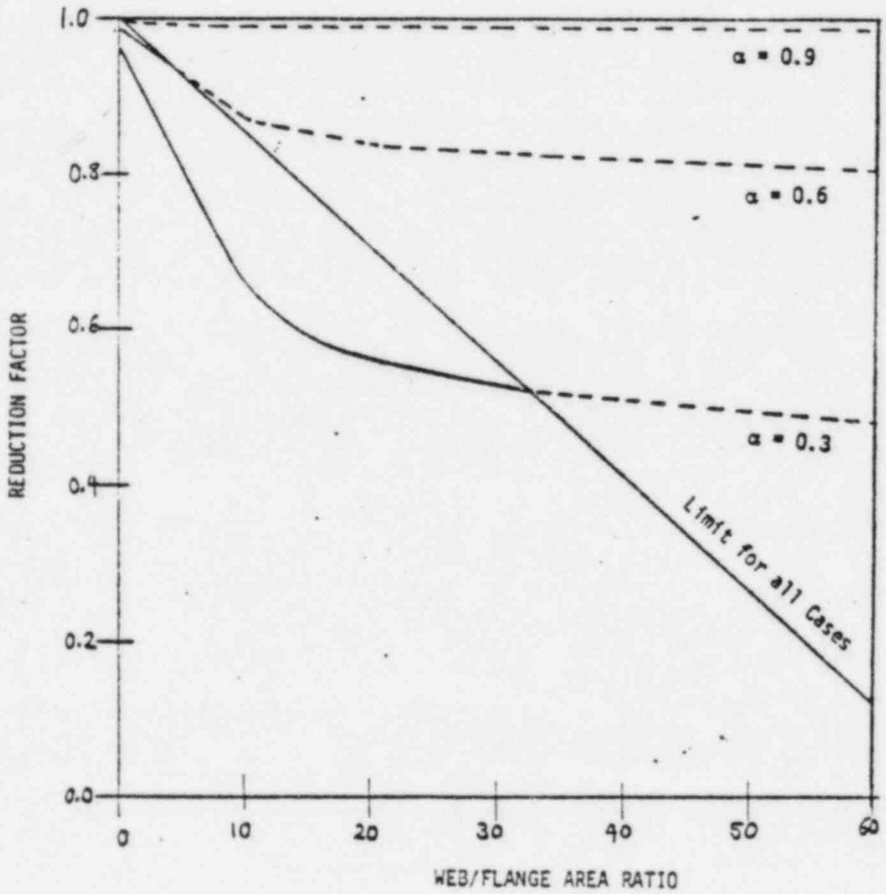

BENDING STRESS = 50KSI ALPHA=0.3, 0.6, 0.9, H/T RATIO = 117

AISC 1.10.6 1963/1980 CODE COMPARISON




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

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

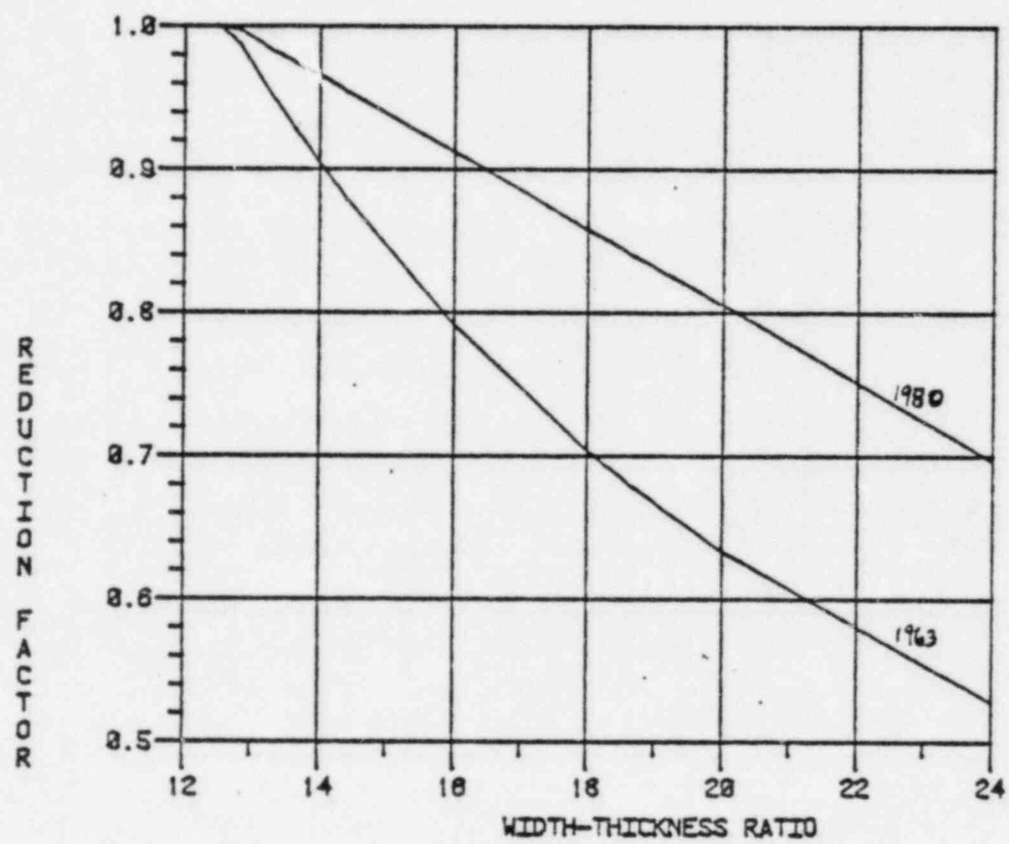
In AISC, 1980, according to the specifications for the above cases, when compression members exceed the allowable width-thickness ratio, the allowable stresses are reduced by a factor based on formulas given in appendix C which depends on yield stress ($F_y$) and the width-thickness ratio.

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.

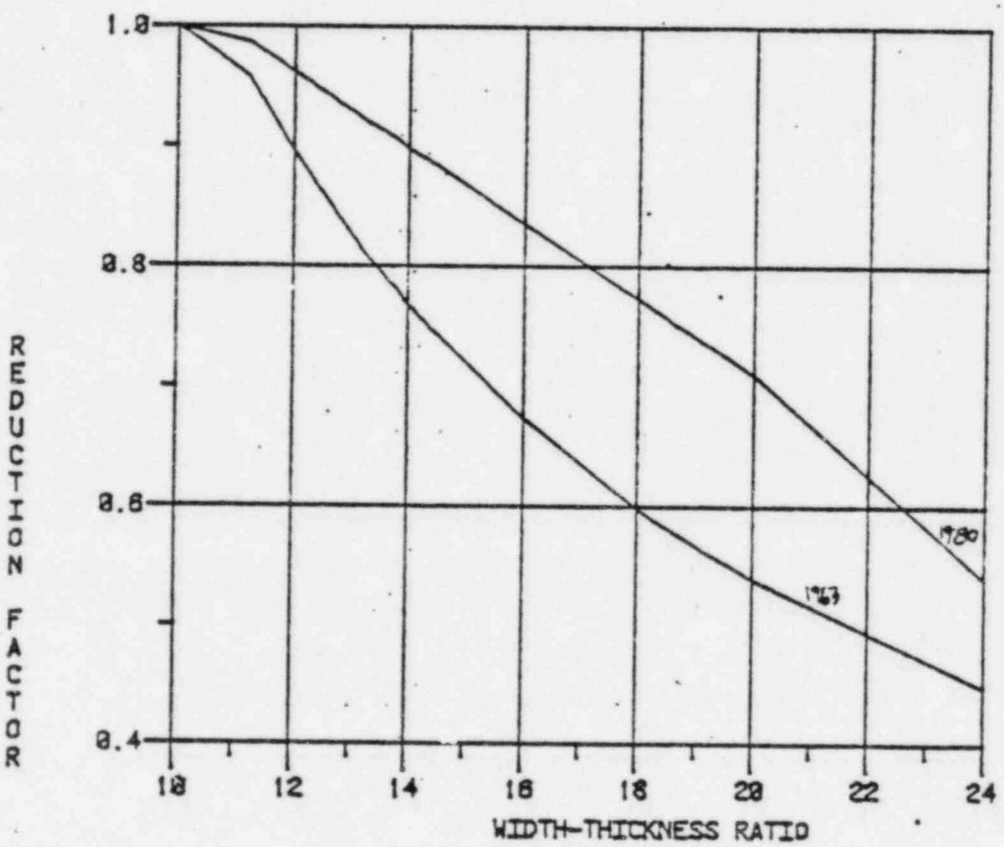
For the case study, two values of Fy 36 ksi and 50 ksi are chosen. For the two values for typical angle section and T sections given in AISC Manual graphs have been plotted for Reduction Factor VS Width – thickness ratio.
Reduction Factor for AISC, 1980 Code is based on formulas given in appendix C and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

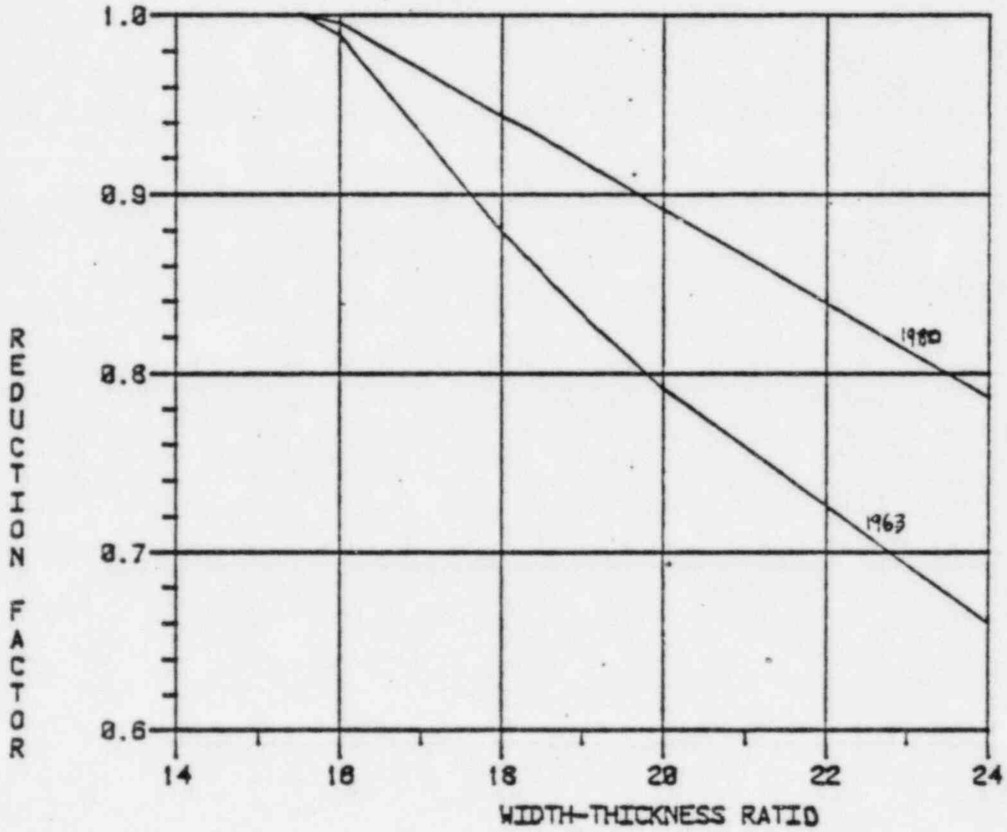
FY=36KSI ANGLES SEPARATED



FY=50KSI ANGLES SEPARATED



FY=36KSI ANGLES IN CONTACT



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| --- | --- | --- | --- | --- |
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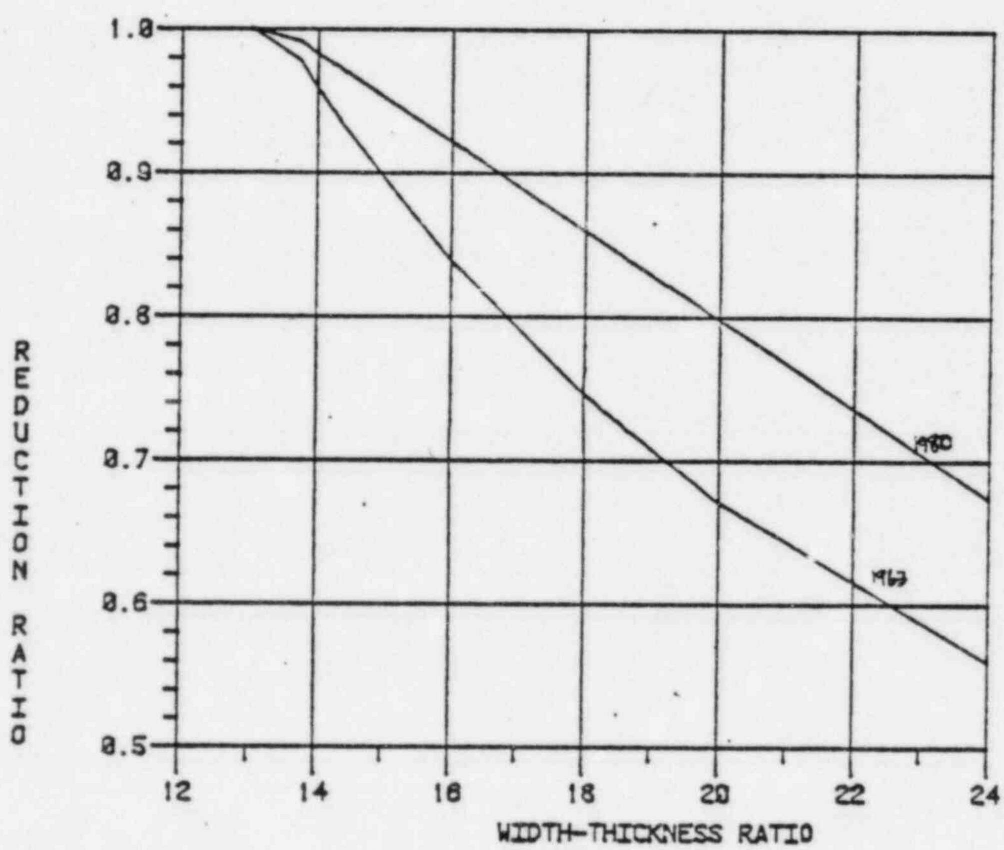
FY=50KSI ANGLES IN CONTACT



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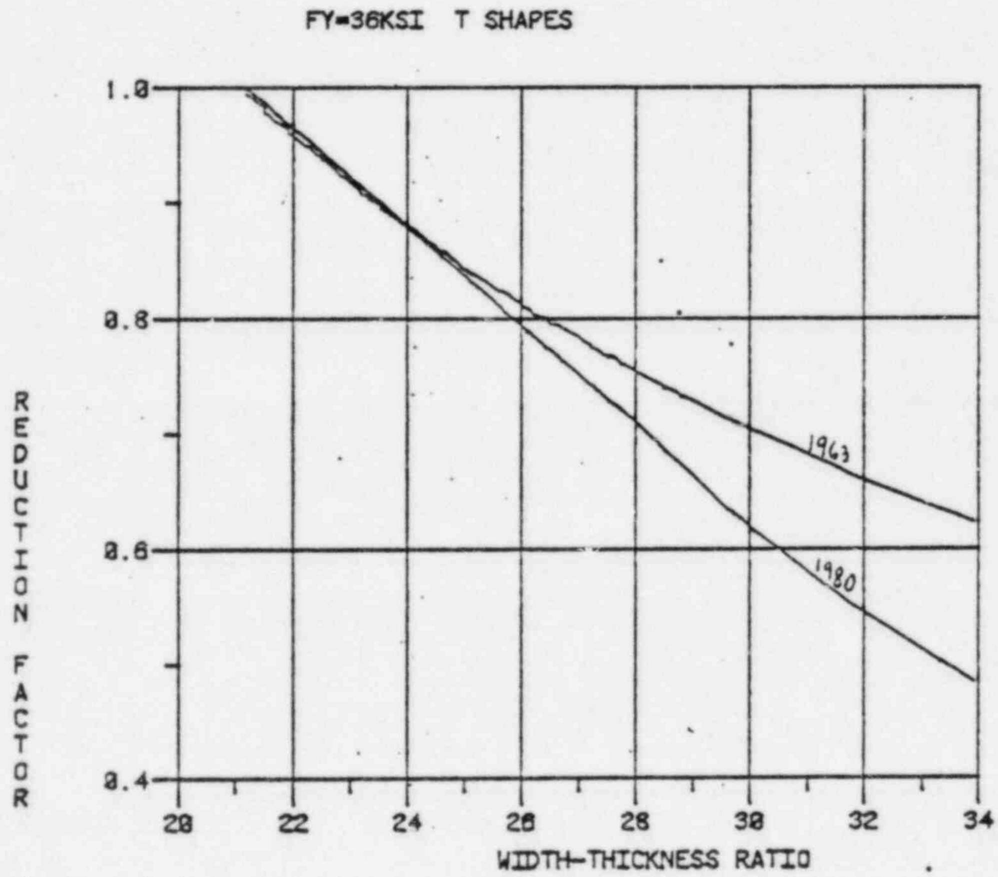
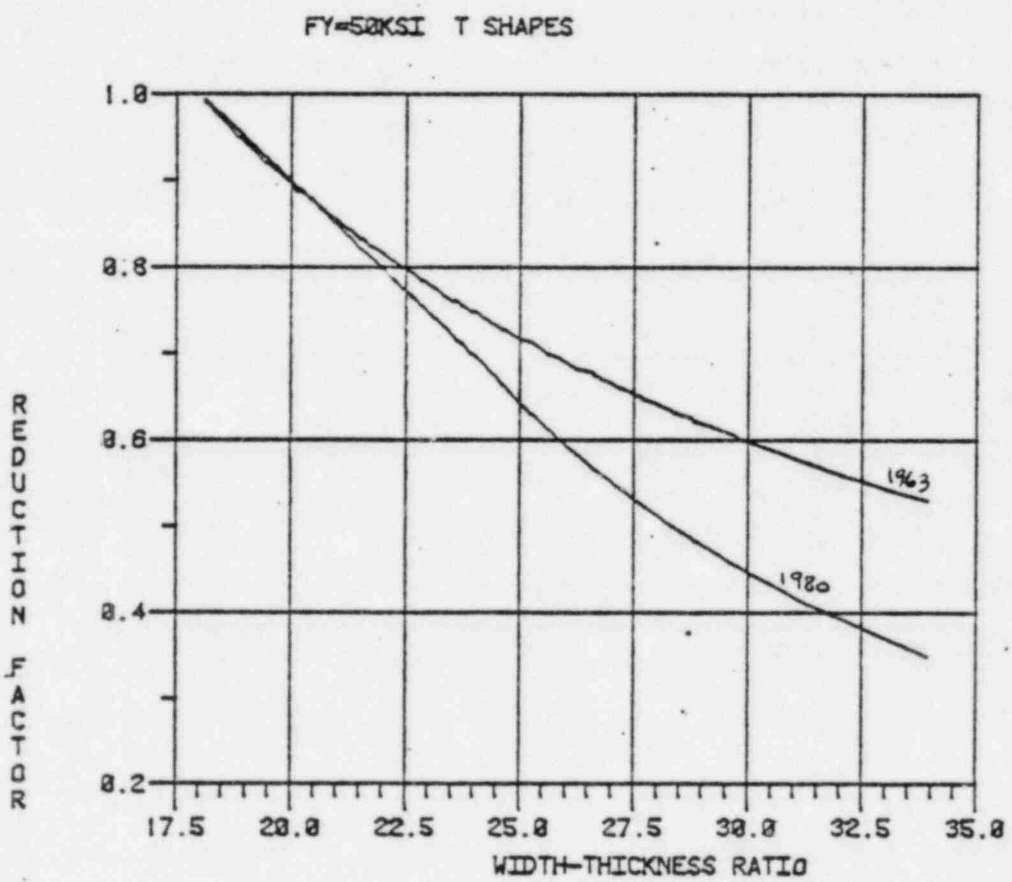
| | By RA | Date SEPT '81 | Ch'k'd EMal | Date 10/81 | Rev. | Date |




FY=50KSI T SHAPES
1.0
0.8
0.6
0.4
0.2
REDUCTION FACTOR
17.5
20.0
22.5
25.0
27.5
30.0
32.5
35.0
WIDTH-THICKNESS RATIO
1963
1980

## CASE STUDY -11-

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

According to AISC 1980, Formula (1.11-5)

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

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

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

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

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

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

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

## CASE STUDY -12-

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

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

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

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

Comparing the above two cases the following is concluded;

When: **Scale**

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

Scale

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

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

## CASE STUDY -13-

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

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

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

$$v_c = 4\sqrt{f_c'}$$

From the above two cases the following is concluded;

When :

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

Scale

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

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

APPENDIX D

ACI CODE PHILOSOPHIES

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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 ($\phi$) 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 ($\phi$). The present ACI code has assigned values to the above factors such that the ratio $U/\phi$ ranges from about 1.5 to 2.4 for reinforced concrete structural elements.

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