TECHNICAL EVALUATION REPORT

SUPPLEMENTARY REPORT

REVIEW OF LICENSEE RESPONSE TO

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

ROCHESTER GAS AND ELECTRIC CORPORATION

R. E. GINNA NUCLEAR POWER PLANT UNIT 1

NRC DOCKET NO. 50-244

NRC TAC NO. 48881

NRC CONTRACT NO. NRC-03-81-130

FRC PROJECT C5506

FRC ASSIGNMENT 18

FRC TASK 423

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July 29, 1983

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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.

Summary

Information concerning the Ginna Nuclear Power Plant Unit 1 supplied to the NRC by Rochester Gas and Electric Corporation (RG&E) dealing with Topic III-7.B of NRC's Systematic Evaluation Program was reviewed. Topic III-7.B assesses the impact of perceived margins of safety of Seismic Category I structures that may result from changes in design codes and from differences between loads and loading combinations used for design and those currently specified.

The review was conducted by the Franklin Research Center with the objective of assisting the NRC in the evaluation of RG&E's compliance status with respect to implementation of the Systematic Evaluation Program by appraising the technical content of the information submitted.

The review found that RG&E has made a substantial engineering effort toward resolution of Topic III-7.B concerns. Although open items were found to remain, these primarily relate to assessment of effects of design code changes when appraised for loadings associated with extreme environmental and faulted service conditions. RG&E plans to address these concerns in due course as part of the Structural Reanalysis Program.

1. INTRODUCTION

Current design criteria for nuclear power plant structures contain requirements that were not in effect when older plants were designed and licensed. Consequently, one aspect (designated Topic III-7.B) of the implementation of NRC's Systematic Evaluation Program requires licensees to review changes that have occurred in structural design criteria since their plant was built and also to review the loads and load combinations used for design of plant structures by comparing them with the loads and load combinations now specified for current construction. The licensee's objective is to assess the impact that these changes may have on margins of safety of Seismic Category I structures as they were originally perceived and as they would be perceived under current criteria. Upon completion of this work, licensees report their findings to the NRC.

To assist in this review, the NRC provided licensees with plant-specific Technical Evaluation Reports (TERs) concerning these issues (e.g., Reference 1). The TERs listed design code changes and, on a building-by-building basis, the load and loading combination changes to be addressed in the licensee review. The items listed were ones judged to have the greatest potential to degrade the originally perceived margins of safety.

In May 1983, under contract NRC-03-81-130, the NRC retained the Franklin Research Center (FRC) to assist in its review of licensee findings. This report describes the review for the R. E. Ginna Nuclear Power Plant Unit 1 and summarizes Rochester Gas and Electric Corporation's (RG&E) compliance status with respect to the implementation of SEP Topic III-7.B.

2. DESIGN CODE CHANGES DESIGNATED SCALE A

Current structural design codes contain provisions that differ from, or did not appear in, the codes to which older plants were designed and constructed. Changes that were judged to have the potential to significantly affect perceived margins of safety have been designated as Scale A. These changes are discussed item-by-item in this section of the report.

2.1 SHEAR CONNECTORS FOR COMPOSITE BEAMS

Four major modifications to the 1963 AISC Code [2] related to the type, distribution, and spacing of shear connectors for composite beams occur in the 1980 Code [3]. These modifications are:

a. Permission to use lightweight structural concrete (concrete made with C330 aggregates) in composite designs

b. Allowance of design for composite action in the negative moment region of continuous beams and provision of design guidance for including the longitudinal reinforcing steel in the negative moment resisting section

c. Design requirements for the minimum number of shear connectors in regions of concentrated load

d. Maximum and minimum spacing requirements in terms of stud diameters.

The first two modifications will not affect old designs because they were not allowed by the previous code. The new provisions concerning the number of studs in the region near concentrated loads and the new limits concerning spacing of studs may adversely affect the margin of safety in older designs when checked against the new code provisions. These new requirements are of special concern in the case of composite beams subject to large concentrated loads, such as those associated with extreme environmental or critical accident conditions.

2.2 COMPOSITE BEAMS OR GIRDERS WITH FORMED STEEL DECK

The 1980 AISC Code [3] contains a new section covering stay-in-place formed steel deck when used in a composite design. These provisions for formed steel decking, depending on the rib geometry and the direction of the ribs relative to the beam, may affect the load capacity of the shear studs and the effective flange width of the assumed concrete compression flange. They provide for reduction factors, to be applied to the shear stud allowable capacity, which account for the structural irregularity introduced into the composite slab.

Composite beams with formed steel decks that were designed to the previous code could have less conservative margins of safety when compared to present requirements, especially in cases where extreme loadings are to be considered.

2.3 FLANGE STRESS IN HYBRID GIRDERS

The AISC Code section covering reduction of bending stress in the compression flange was modified in the 1980 Code.

The original flange stress reduction formula in the old code was needed to account for stress transfer which may occur in ordinary beam webs if the compression region should deflect laterally, thereby changing the bending capacity of the cross section. In hybrid girders, the amount of the loss of bending resistance resulting from this phenomenon will vary depending on the relative properties of the web and flange steel. A reduced bending stress formula reflecting this interaction was introduced. In order to keep the formulation relatively simple, the reduced bending stress was made applicable to both flanges of the hybrid member.

Beams or girders fabricated from plate where the flange and web steels are different could have lower margins of safety under the new code than were thought to exist under older code requirements where the ratio of web yield stress to flange yield stress is less than 0.45 and the ratio of the web area to flange area is low.

2.4 STRESSES IN UNSTIFFENED COMPRESSION ELEMENTS

New requirements provide stress reduction factors for unstiffened elements subject to compression with one edge free parallel to the compressive stress.

Previous code provisions allowed the designer to neglect a portion of the area of such elements. The new code requirements provide equations for various elements based on the critical buckling stress for plates. The new analytical approach is more conservative for the steams of tees and less conservative for all other cases.

Where structural tees are used as main members and the tee stem is in compression, the margin of safety for older designs checked under the new code could be significantly less than was thought under prior code requirements. Since buckling is a non-ductile type failure, these new requirements are of special concern in the case of tee shapes subjected to the extreme environmental or critical accident conditions.

2.5 MAXIMUM LOAD IN RIVETED OR BOLTED TENSILE MEMBERS

The 1980 AISC Code [3] introduces codes changes which affect the maximum load permitted in tensile members.

Two interacting code changes are involved in establishing this limit, and the mutual effects of both must be considered in assessing the impact of the new code upon the perception of margins of safety in tension members. The two provisions involved concern:

1. the tensile area permitted to be used in establishing load carrying capacities
2. the allowable stresses to be used in conjunction with these areas.

Both effects are taken into account in ranking this change. The potential magnitude of the mutual effects of the two changes is discussed below.

The 1980 AISC Specification definition of "Effective Net Area" introduces a reduction coefficient which is to be applied to the traditional definition of net area. This essentially changes the design capacity of a tension member when compared to older versions of these specifications. First consider only the effect of the critical area used for the design of a tension member as defined in the new code compared to the critical area used for the design of the same member as defined in the old code. Clearly, if all other factors are equal, the new code is more conservative. However, all other factors are not the same. The changes in allowable tensile stress definition (on the gross area and on the effective net area) which were introduced simultaneously with the new definition of effective net area modify the above conclusion. In addition, the traditional upper limit on the critical net area of 85% of the gross area (a requirement of the old code) is no longer a requirement of the new code. Both of these changes interact with the new effective net area requirement.

A valid assessment of the effect of these changes is best accomplished by a comparison of the allowable load each code permits in tension members. If one considers the allowable load on the effective net area, the value based on the new code is a function of three variables: the new reduction coefficient, the net area,* and the ultimate tensile strength of the steel. The allowable load based on the old code is a function of only two variables: the net area and the yield strength of the steel. First, form the load ratio of the allowable load defined by the new code criteria to the allowable load defined by the old code criteria. Next, consider the ranges of all of the parameters mentioned above, this ratio will have defined upper and lower limits which are a function of the ratio of the net areas, the new code net area reduction factor, and the ratio of the steel ultimate strength to the yield strength.

For all the steels allowed under the new code, this load ratio ranges from 1.5 to 0.69. For all the steels allowed under the old code, this load ratio ranges from 1.6 to 0.88. It is apparent that, for those steels with load ratios less than 1.0, the new code is less conservative than the old. The margin of safety of some older designs therefore could be significantly lower when checked against the new code requirements.

2.6 SHEAR LOAD IN COPED BEAMS

The 1980 AISC Code [3] introduces additional control over the shear load permitted at beam end connections where the top flange has been coped.

*In making this comparison, one must be careful to note that the net area is not always the same under the old and new codes.

Web shear control in older codes did not distinguish between coped and uncoped beams or between shear allowed at connections and over the free span (except for requiring reinforcement of thin webs at connections). The shear load allowed was given by:

allowable shear load = 0.4 (yield strength) (gross web section).

The 1980 Code retains this limit, but introduces an additional requirement to protect against a failure mode associated with coped beams. For coped beams (and similar situations), a portion of the web may sever, failing along the perimeter of the connection holes. In particular, coped beam web connections where the fastener holes lie close to the butt end of the beam may be prone to such failures.

This web "tear out" failure is actually a combination of shear failure through the line of fasteners together with tensile failure across the shortest path to the beam end. The failure surface turns a corner with shear failure along a line trending upward through the holes, combined with tensile failure across a more-or-less horizontal line running out to the beam end.

The newly introduced shear limit is given as a function of the minimum net failure surface and the steel ultimate strength. Thus, the new requirements may or may not control a coped beam's allowable capacity in shear. Whether or not it does depends on both the connection geometry and the type of steel used.

When this requirement is controlling, coped beams designed by previous rules may be found, if checked against the new criteria, to have significantly smaller margins of safety than previously thought.

2.7 COLUMN WEB STIFFENERS AT FRAME JOINTS

The more recent editions of the AISC code mandate which columns must be stiffened at locations where beams of girders are rigidly attached to the column flange and also establish requirements for the geometry of such web stiffeners. These requirements are introduced to preclude local crippling at such frame joints.

No such guidance was provided by AISC-63 [2]. Older codes (such as AISC-63) left such matters to the designer's discretion. Consequently, there is no assurance that all such columns are adequately stiffened for current accident and faulted loadings.

2.8 LATERAL SUPPORT SPACING IN FRAMES (PLASTIC DESIGN METHOD)

The 1980 AISC Code contains changed spacing requirements for lateral supports in portions of members in frames where failure mechanisms are expected to form at ultimate load.

Members of such frames must not only be capable of developing a plastic hinge, but must also be stable enough to sustain moments larger than those computed on an elastic-perfect-plastic theory (because real steels work-harden at strains expected to occur at hinge locations). Previous lateral bracing requirements were developed for a limited range of steels. Research on high-strength steels has shown that, for certain ranges of slenderness ratio of the compression flange of such frame members, older specification bracing requirements were not sufficiently conservative.

The new specification requirements make the slenderness ratio limits a function of the steel yield strength and the member curvature (as expressed by the ratio of the lesser bending moment at the ends of the unbraced segment to the plastic moment).

The new specifications are more conservative for (1) any segment bent in double curvature regardless of its steel specification and (2) very high-strength steel members. The adequacy of frame members bent in single curvature and constructed of steels whose yield strength exceeds 36 ksi should be examined on a case-by-case basis.

The new requirements may reduce the margins of safety thought to exist in:

1. structures designed under the plastic requirements of older codes
2. elastically designed structures sized to carry a smaller maximum load than is now required by current accident and faulted load combinations. In this case, plastic logic may have to be invoked to justify the adequacy of exisiting structures. Nonconformance with

current bracing requirements may substantially restrict the capability of frame members to carry code-acceptable overloads.

2.9 BRACKETS AND CORBELS

ACI 349-76 [4], Section 11.13 contains design requirements for short brackets and corbels which are considered primary load-carrying members; no comparable requirements are provided in ACI 318-63 [5].

The requirements apply to brackets and corbels having a shear span-to-depth ratio of unity or less. They provide minimum and maximum limits on tension and shear reinforcement, limits on ultimate shear stress in concrete, and constraints on member geometry and location of reinforcement.

Brackets and corbels designed under earlier codes may or may not satisfy the newly imposed limits. If they do not, they may be prone to non-ductile failure (which occurs suddenly and without warning) and may exhibit smaller margins of safety than those currently required.

2.10 SPECIAL PROVISIONS FOR WALLS

2.10.1 Shear Walls

ACI 349-76, Sections 11.15.1 through 11.15.6 specify requirements for reinforcing and permissible shear stresses for in-plane shear loads on walls. The ACI 318-63 Code had no specific requirements for in-plane shear on shear walls.

2.10.2 Punching Shear

ACI 349-76, Section 11.15.7 specifies permissible punching shear stresses for walls. ACI 318-63 had no specific provisions for walls for these stresses. Punching loads are caused by relatively concentrated lateral loads on the walls. These loads may be from pipe supports, equipment supports, duct supports, conduit supports, or any other component producing a lateral load on a wall.

2.11 ELEMENTS LOADED IN SHEAR WITH NO DIAGONAL TENSION (SHEAR FRICTION)

The provisions for shear friction given in ACI 349-76 did not exist in ACI 318-63. These provisions specify reinforcing and stress requirements for situations where it is inappropriate to consider shear as a measure of diagonal tension.

2.12 ELEMENTS SUBJECT TO TEMPERATURE VARIATIONS

The ACI 349-76 [4], Appendix A requirements for consideration of temperature variations in concrete were not contained in ACI 318-63. These new provisions require that the effects of temperature gradients and the difference between mean temperature and base temperature during normal operation or accident conditions be considered. The new provisions also require that thermal stresses be evaluated considering the stiffness and rigidity of members and the degree of restraint of the structure.

2.13 COLUMNS WITH SPLICED REINFORCING

The ACI 349-76, Section 7.10.3 requirements for columns with spliced reinforcing did not exist in the ACI 318-63 Code. The ACI 349-76 Code requires that splices in each face of a column, where the design load stress in the longitudinal bars varies from fy in compression to 1/2 fy in tension, be developed to provide at least twice the calculated tension in that face of the column (splices in combination with unspliced bars can provide this if applicable). This code change requires that a minimum of 1/4 of the yield capacity of the bars in each face of the column be developed by both spliced and unspliced bars in that face of the column.

2.14 EMBEDMENTS

Appendix B of ACI 349-80 provides rules for the design of steel embedments in concrete; the design of embedments is not specifically addressed in ACI 318-63.

Current requirements of Appendix B are based upon ultimate strength design using factored loads. The anchorage design is controlled by the ultimate strength of the embedment steel. Ductile failure (i.e., steel yields before concrete fails) is postulated.

Under the provisions of ACI 318-63, the design of embedments was left to the discretion of the designer. Working stress design methods were widely used.

Consequently, it is likely that original embedment designs do not fully conform to current criteria. Review of such designs to determine the implications with respect to margins of safety is therefore judged a desirable precaution.

2.15 DUCTILE RESPONSE TO IMPULSE LOADS

Appendix C to ACI 349-76 [4] contains design rules for structures which may be subjected to impulse or impact loads; no such provisions occur in ACI 318-63 [5].

The rules of Appendix C are intended to foster ductile response (i.e., steel yields prior to concrete failure) of nuclear structures if and when they experience impulse or impact loads. For structures built to codes not containing such provisions, there is no assurance that sufficient design effort was directed toward proportioning members to provide energy absorbtion capability. Consequently, such structures might be prone to non-ductile, sudden failure should they ever experience postulated accident loadings such as jet impingement, pipe whip, compartment depressurization, or tornado missiles.

2.16 TANGENTIAL SHEAR (CONTAINMENTS)

Paragraph CC-3421.5, Tangential Shear, of Section III, Division 2 of the ASME Boiler and Pressure Vessel Code [6] addresses the capacity of reinforced concrete containments to carry horizontal shear load. It provides code-acceptable levels of horizontal shear stress that the designer may credit to the concrete. No specific guidance in this matter exists in ACI 318-63.

The provisions associate the allowable concrete stress in horizontal shear with the concrete properties, the manner in which lateral loads are imposed on the structure, and the presence of sufficient reinforcement to assure that the assumed shear capacity of concrete can be developed.

Sufficient diagonal reinforcement (or its demonstrated equivalent) is to be supplied to carry, without excessive strain, shear in excess of that permitted in the concrete. A major consideration here is the preservation of the structural integrity of the liner.

In containments constructed to older codes, such matters were left to the discretion of the designer, who may or may not have provided the horizontal shear capacity at controlled strains that the code currently requires.

2.17 AREAS OF CONTAINMENT SHELL SUBJECT TO PERIPHERAL SHEAR

Concrete containment design is currently governed by the ASME Boiler and Pressure Vessel Code. Section III, Division 2, 1980 [6]. The provisions for peripheral (punching) shear appear in code Section CC-3421.6. These provisions are similar to the ACI 318-63 Code [5] provisions for slabs and footings, except that the allowable punching shear stress in CC-3421.6 includes the effect of shell membrane stresses. For membrane tension, the allowable concrete punching shear stress in the ASME Code is less than that allowed by ACI 318-63.

2.18 AREAS OF CONTAINMENT SHELL SUBJECT TO TORSION

Concrete containment design is currently governed by the ASME Boiler and Pressure Vessel Code, Section III, Division 2, 1980. Section CC-3421.7 of the code contains provisions for the allowable torsional shear stress in the concrete. Such provisions were not contained in the ACI 318-63 Code. The present allowable torsional shear stress includes the effects of the membrane stresses in the containment shell and is based on a criterion that limits the principal membrane tension stress in the concrete.

2.19 THERMAL LOADS

ACI 349-76 Appendix A and ASME B&PV Code, Section III, Div. 2, CC-3440 contains requirements for consideration of temperature variations in concrete that are not contained in ACI 318-63.

The new provisions require consideration of the effects of thermal gradients and of the effects depending on the mean temperature distribution and the base temperature distribution during normal operation or accident conditions. The new provisions also require that thermal stresses be evaluated considering the stiffness and rigidity of members and the degree of restraint of the structure.

An assessment is to be made of the analytical methods used to determine thermal stresses as compared to current code-acceptable practices, e.g., those discussed in ACI 349.1R-80 and the commentary to ACI 349R-80.

If the methods used for design produce stress results which are significantly different from those current procedures generate, perceived margins of safety could be affected.

2.20 AREAS OF CONTAINMENT SHELL SUBJECT TO BIAXIAL TENSION

Increased tensile development lengths are required by Section CC-3532.1.2 of Reference 6 for reinforcing steel bars terminated in areas of reinforced concrete containment structures which may experience biaxial tension. For biaxial tension loading, bar development lengths, including both straight embedment lengths and equivalent straight length for standard hooks, are required to be increased by 25% over the standard development lengths required for uniaxial loading. Nominal temperature reinforcement is excluded from these special provisions. ACI 318-63 had no requirements related to this increase in development length.

2.21 BRACKETS AND CORBELS (ON THE CONTAINMENT SHELL)

The ACI 318-63 Code did not specify requirements for brackets and corbels. Provisions for these components are included in the ASME Boiler and Pressure Vessel Code, Section III, Division 2, Section CC-3421.8. These provisions apply to brackets and corbels having a shear-span-to-depth ratio of unity or less. The provisions specify minimum and maximum limits for tension and shear reinforcing, limits on shear stresses, and constraints on the member geometry and placement of reinforcing within the member.

3. . REVIEW METHOD AND TABULAR PRESENTATIONS

The information relating to SEP Topic III-7.B which was supplied to the NRC by Rochester Gas and Electric Corporation and made available for this review is contained in the following documents:

1. J. E. Maier, Rochester Gas and Electric Corporation
   Letter to D. M. Crutchfield, Chief, Operating Reactor Branch No. 5, USNRC
   Subject: Structural Reanalysis Program, SEP Topics II-2.A, III-2, III-4.A, and III-7.B, R. E. Ginna Nuclear Power Plant, Docket No. 50-244
   April 22, 1983

2. J. E. Maier, Rochester Gas and Electric Corporation
   Letter to D. M. Crutchfield, Chief, Operating Reactor Branch No. 5, USNRC
   Subject: Structural Reanalysis Program, SEP Topics II-2.A, III-2, III-4.A, and III-7.B, R. E. Ginna Nuclear Power Plant, Docket No. 50-244
   May 19, 1983

3. J. E. Maier, Rochester Gas and Electric Corporation
   Letter to D. M. Crutchfield, Chief, Operating Reactor Branch No. 5, USNRC
   Subject: SEP Topic III-7.B, Design Codes, Design Criteria, and Load Combinations, R. E. Ginna Nuclear Power Plant, Docket No. 50-244
   May 27, 1983

4. Gilbert Commonwealth calculations for the Ginna Nuclear Power Plant Unit 1 on the following subjects:

   a. Integrity of structural walls against punching shear (5.6, Attachment 3 of Reference 3). Specific example: Main steam penetration under postulated LOCA.

   b. Integrity of elements loaded in shear with no diagonal tension (5.3, Attachment 3 of Reference 3). Specific example: Shear capacity of beam pockets supporting the intermediate building floor.

   c. Development length of lapped splices in columns (5.1, Attachment 3 of Reference 3). Specific example: Column group which includes control room column.

   d. Coped beams (4.2.6 of Reference 2). Specific example: Integrity of roof beams (if coped) under extreme environmental load.

e. Steel embedments (4.2.9 of Reference 2). Specific example: Frame columns under low roof of the auxiliary building.

Before undertaking licensee report reviews, FRC prepared tabular forms to be used as a working tool during the review process and also to document the review work and its findings when the review was completed.

These tables are intended to:

1. establish a systematic and comprehensive review procedure
2. standardize, as much as possible, the review process for all licensees
3. present a relatively compact overview of each licensee's SEP Topic III-7.B compliance status.

Two such forms were prepared, one related to design code changes and the other to the differences between loads and load combinations used for design and loads and load combinations current today.*

The form sheets provide space to summarize key information reported in licensee responses. Certain items (such as descriptions of Scale A code changes, conclusions, and comments) frequently are not adaptable to abbreviated summary. For such items, the form sheets refer the reader either to sections of this TER where the matter is developed more fully or to an extended note list compiled on separate sheets. The note list, although detached from the main table in order to allow a fuller discussion, accompanies each table and should be regarded as an integral part of it.

The form sheet consists of four major columnar sections which:

1. identify each Scale A item
2. state the action that the licensee took or the logic that the licensee presented to resolve the item
3. provide an assessment of engineering conclusions that may be reasonably drawn from the evidence provided

*The tables for load and load combinations do not appear in this report because RG&E plans to address these matters fully and in due course as part of their "Structural Reanalysis Program." However, for each Seismic Category I structure, RG&E listed currently appropriate load combinations; this is discussed later.

4. summarize the licensee's compliance status with respect to the item.

Items listed on the tables are designed code changes (or itemized load combinations) designated Scale A. This list is drawn directly from TER-C5257-322, the earlier report on this topic [1].

Licensees may choose to address potential concerns stemming from Scale A items in two ways:

1. generically, i.e., on an overall basis which resolves the concern for all plant structures collectively, or

2. on a structure-by-structure basis.

The form sheets are compiled in a manner matching the licensee's approach, with one form sheet containing generically treated matters and with structure-specific form sheets for each structure-specific matter.

Form sheets summarizing the review findings concerning the licensee's compliance status with respect to the implementation of SEP Topic III-7.B aspects related to design code changes follow in Section 4. A discussion of the review findings concerning the licensee's compliance status with respect to load and load combination changes is presented in Section 5.

4. TABULAR SUMMARY OF REVIEW FINDINGS OF LICENSEE COMPLIANCE STATUS CONCERNING IMPLEMENTATION OF SEP TOPIC III-7.B IMPACT OF DESIGN CODE CHANGES

Form sheets summarizing the review findings concerning technical aspects with respect to the implementation of SEP Topic III-7.B as related to design code changes follow.

SUMMARY OF LICENSEE COMPLIANCE STATUS --
IMPACT OF DESIGN CODE CHANGES

PLANT: Ginna
STRUCTURE: All steel structures

CODE CHANGE CITED AS SCALE A- IN TER-5257-322			LICENSEE'S ACTION TO RESOLVE POTENTIAL CONCERN			EVALUATION OF LICENSEE'S ACTION			LICENSEE STATUS	
REFERENCED CODES AND PARAGRAPH		DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE		APPROACH	IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
CURRENT	DESIGN		DOCUMENT	PAGE NUMBER						
AISC 1980	AISC 1963									
1.11.4	1.11.4	Shear connectors in composite beams (2.1)	Ref. 2	p. 4-2 App. A	Calculations and construction drawings were reviewed for the use of shear connectors for composite beams.	Yes	Yes	C-1	Resolved	None
1.11.5	--	Composite beams or girders with formed steel deck (2.2)	Ref. 2	p. 4-2 App. A	Calculations and construction drawings were reviewed for composite beams with steel decking. Selected beams were analyzed for loads shown on the drawings.	Yes	See Notes 1 & 2	C-2	OK for loads shown on drawings.	Further investigation required for C and D service conditions.
1.10.6	1.10.6	Hybrid girders (2.3)	Ref. 2	p. 4-3 App. A	Construction drawings and specifications were reviewed for the existence of hybrid girders.	Yes	Yes	C-3	Not applicable	None
1.9.1.2 and App. C	1.9.1	Compression elements having width/thickness ratio greater than specified in 1980 Code (2.4)	Ref. 2	p. 4-3 App. A	The plant structural model was reviewed to determine where tee sections were used in compression. These were evaluated under normal operating load combinations.	Yes	See Notes 1 & 2	C-4	OK for normal operating load combinations.	Further investion required for C and D service conditions.
1.14.2.2	--	Tension members, when load is transmitted by bolts or rivets (2.5)	Ref. 2	p. 4-4 App. A	Using the formulas and allowables for each code, the structural capacity of a generic design example was computed and compared.	Yes	Yes	C-5	Resolved	None

Notes: 1. The Licensee has not yet considered this code change in conjunction with current accident and faulted service loading conditions.
2. Paragraph 4.1 in Appendix A of Reference 2 states, "The effects of seismic loads are not a part of the code comparison of this report."

SUMMARY OF LICENSEE COMPLIANCE STATUS --
IMPACT OF DESIGN CODE CHANGES

PLANT: Ginna
STRUCTURE: All steel structures
Sheet 2 of 11

CODE CHANGE CITED AS SCALE A IN TER-5257-322			LICENSEE'S ACTION TO RESOLVE POTENTIAL CONCERN			EVALUATION OF LICENSEE'S ACTION			LICENSEE STATUS	
REFERENCED CODES AND PARAGRAPH		DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE		APPROACH	IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
CURRENT	DESIGN		DOCUMENT	PAGE NUMBER						
AISC 1980	AISC 1963									
1.5.1.2.2	--	Beam end connection with top flange coped, if subject to shear (2.6)	Ref. 2	p. 4-5 App. A	Steel fabrication drawings were reviewed for major members with bolted connections and coped top flanges. Lightly loaded girts, platforms, stair stringers, etc. were not included. The block shear capacity of each beam was compared with: 1. loads shown on the construction drawings 2. the shear capacity of the bolts, or 3. the maximum allowable load for the beam span.	Yes	See Notes 1 & 2 (Sheet 1)	C-6	OK for loads shown on the construction drawings.	Further investigation required for C and D service conditions.
1.15.5.2 through 1.15.5.4	--	Column web stiffeners for connections carrying moment or restrained member connection (2.7)	Ref. 2	p. 4-6 App. A	Construction and fabrication drawings were reviewed for use of moment connections. Only screenhouse roof beams were so designed. These were checked against the AISC 1980 Code using the original applied loads.	Yes	See Notes 1 & 2 (Sheet 1)	C-7	OK for original applied loads.	Further investigation required for C and D service conditions.

SUMMARY OF LICENSEE COMPLIANCE STATUS --
IMPACT OF DESIGN CODE CHANGES

PLANT: Ginna
STRUCTURE: All steel structures
Sheet 3 of 11

CODE CHANGE CITED AS SCALE A IN TER-5257-322			LICENSEE'S ACTION TO RESOLVE POTENTIAL CONCERN			EVALUATION OF LICENSEE'S ACTION			LICENSEE STATUS	
REFERENCED CODES AND PARAGRAPH		DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE			IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
CURRENT	DESIGN		DOCUMENT	PAGE NUMBER	APPROACH					
AISC 1980	AISC 1963									
2.9	2.8	Spacing of lateral supports of members designed using plastic design methods (2.8)	Ref. 2	p. 4-6 App. A	Available calculations and the Ginna FSAR were reviewed for evidence of plastic design methods.	Yes	See Notes 1 & 2 (Sheet 1)	C-8	OK for all loadings when reactions remain elastic at beam supports.	No action required unless plastic logic is subsequently used to justify the integrity of the existing structures under Scale A loading combination. If so, Licensee-stated conclusions must be reexamined.

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PLANT: Ginna
STRUCTURE: All concrete structures
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CODE CHANGE CITED AS SCALE A IN TER-5257-322			LICENSEE'S ACTION TO RESOLVE POTENTIAL CONCERN			EVALUATION OF LICENSEE'S ACTION			LICENSEE STATUS	
REFERENCED CODES AND PARAGRAPH		DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE			IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
CURRENT	DESIGN		DOCUMENT	PAGE NUMBER	APPROACH					
ACI 349-76	ACI 318-63									
11.13	--	Short brackets and corbels (not on the containment shell) (2.9)	Ref. 3	p. 17 Sect. 5.2 Att. 3	Concrete outline drawings and available original calculations were reviewed to determine where brackets and corbels were used. Twelve corbels were found. Significantly loaded corbels having similar geometry were grouped. A corbel from each group (judged to have the worst load) was evaluated for compliance with ACI 349-76 configuration requirements. If all requirements were met, the capacity of the corbel was calculated in accordance with ACI 349-76. If a corbel did not conform to configuration requirements, the concrete shear stresses was computed, taking no credit for reinforcing.	Yes	Yes	C-9	Resolved	None

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CODE CHANGE CITED AS SCALE A IN TER-5257-322			LICENSEE'S ACTION TO RESOLVE POTENTIAL CONCERN			EVALUATION OF LICENSEE'S ACTION			LICENSEE STATUS	
REFERENCED CODES AND PARAGRAPH		DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE		APPROACH	IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
CURRENT	DESIGN		DOCUMENT	PAGE NUMBER						
ACI 349-76	ACI 318-63									
11.16.1 through 11.16.6	--	Shear walls used as primary load-carrying members (2.10.1)	Ref. 3	p. 20 Sect. 5.4.1 Att. 3	A total of 187 shear walls was identified. The walls in each building were taken as a group, and further classified as either interior or exterior. One wall representative of each classification was evaluated. For the controlling load combination, in-plane vertical, in-plane horizontal, and lateral loads on the wall were evaluated to code provisions. (Shear walls in the screenhouse were evaluated by comparison with auxiliary building walls.)	Yes	Yes	C-10	Resolved except for diesel generator building.	RG&E has committed to make modifications to the diesel generator building.
11.16.7	--	Punching shear stress for walls (2.10.2)	Ref. 3	p. 22 Sect. 5.4.2 Att. 3	Load sheets from the Ginna Seismic Upgrade Program were reviewed to determine punching loads from pipe and equipment supports. For pipe supports, the most severe loads found were applied to the thinnest wall, using a 6-in square area of application. The capacity of the wall calculated in accordance with the ACI 349-76 provisions was determined. Equipment punching loads were individually treated.	Yes	Yes	C-11	Resolved	None

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CODE CHANGE CITED AS SCALE A IN TER-5257-322			LICENSEE'S ACTION TO RESOLVE POTENTIAL CONCERN			EVALUATION OF LICENSEE'S ACTION			LICENSEE STATUS	
REFERENCED CODES AND PARAGRAPH: CURRENT	REFERENCED CODES AND PARAGRAPH: DESIGN	DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE: DOCUMENT	REFERENCE: PAGE NUMBER	APPROACH	IS METHOD VALID AND APPROPRI-ATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CON-CLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
ACI 349-76	ACI 318-63									
11.15	--	Structural elements loaded in shear where it is inappropriate to consider shear as a measure of diagonal tension (shear friction) (2.11)	Ref. 3	p. 18 Sect. 5.3 Att. 3	Review of concrete outline drawings and available calculations revealed 203 shear-friction conditions from a variety of beam and slab supports and other situations. Similar configurations were grouped together in 15 categories. Taking credit only for reinforcement meeting ACI 349-76 provisions, the shear capacity of one member (the most heavily loaded) of each group was determined. This capacity was checked against a code-required factor of safety or (failing this) against actual failure.	Yes	Yes, but see Note C-11 and status comment	C-12	OK for loads stated in example given in Reference 5.b.	Further investigation may be required for C and D service conditions.

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CODE CHANGE CITED AS SCALE A IN TER-5257-322			LICENSEE'S ACTION TO RESOLVE POTENTIAL CONCERN			EVALUATION OF LICENSEE'S ACTION			LICENSEE STATUS	
REFERENCED CODES AND PARAGRAPH: CURRENT	DESIGN	DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE: DOCUMENT	PAGE NUMBER	APPROACH	IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
ACI 349-76	ACI 318-63									
Appendix A	--	Concrete regions subject to high-temperature time-dependent and position-dependent temperature variations (2.12)	Ref. 3	p. 23 Sect. 5.5 Att. 3	In buildings where a possible thermal differential condition of consequence could occur, drawings and calculations were reviewed to determine thermal conditions. Six situations were found. Of these, the cable tunnel condition was judged to be the worst case and evaluated. Using the most severe loading combination, the moments in the cable tunnel were determined and compared to the corresponding moment capacities.	Yes	Yes	C-13	Resolved	None
7.10.3	805	Column with spliced reinforcement subject to stress reversal (2.13)	Ref. 3	p. 15 Sect. 5.1 Att. 3	Drawings and calculations were reviewed to determine columns with spliced reinforcing; 57 were found. All use lap splices at the bottom of the column. These were grouped, according to their reinforcing details and sizes, into nine categories. One heavily loaded column from each group was chosen for evaluation to to ACI 349-76 provisions. The splice	Yes	Yes, for the loads considered in the computations	C-14	OK for loads considered in the report, but the report does not clearly state that all extreme load cases have been considered for all column groups.	Further investigation may be required for C and D service conditions.

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REFERENCED CODES AND PARAGRAPH		DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE		APPROACH	IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
CURRENT	DESIGN		DOCUMENT	PAGE NUMBER						
ACI 349-76	ACI 318-63									
7.10.3 (Cont.)					capacity was calculated. If the splices did not have the minimum required splice length to fully develop the bar, splice capacities were reduced in proportion to their length.					
Appendix B	--	Steel embedment used to transmit load to concrete (2.14)	Ref. 2	p. 4-7 Sect. 4.2.9	From a total population of 194 columns, 46 (having concrete anchorage) were selected as a statistical sample for evaluation. These column anchorages were checked for ductile failure and other requirements of the ACI 349-80 Code. If code requirements were met, the anchorage was deemed acceptable. If not, the ultimate capacity of the anchorage was compared to the normal design load combinations.	Yes	See Notes 1 & 2 (Sheet 1)	C-15	OK for normal design loads using current load combinations.	Further investigation may be required for C and D service conditions.
Appendix C	--	Elements subject to impulsive and impactive loads, whose failure must be precluded (2.15)	--	--	This item will be addressed in the Structural Upgrade Program.					

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REFERENCED CODES AND PARAGRAPH		DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE							
CURRENT	DESIGN		DOCUMENT	PAGE NUMBER	APPROACH	IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
ASME B&PV Code Section III Div 2, 1980	ACI 318-63									
CC-3421.5	--	Containment transmitting inplane shear (2.16)	Ref. 3	p. 2 Sect. 1.2 Att. 3	Resolved in SER transmitting TER-C5257-322 to RG&E (Section 1.2, Attachment 3 of Reference 3).					
CC-3421.6	1707	Region of the containment shell subject to peripheral shear (2.17)	Ref. 3	p. 24 Sect. 5.6 Att. 3	A total of 126 penetrations was identified, and grouped by sleeve diameter into ten categories. Some groups were adequate by inspection; for others a "worst case" penetration from each group was chosen and the shell capacity of these penetrations was evaluated. Actual factors of safety were calculated and compared to the factor of safety required by the code.	Yes	Yes	C-16	Resolved	No further action required.
CC-3421.7	921	Region of containment shell subject to torsion (2.18)	Ref. 3	p. 26 Sect. 5.7 Att. 3	Structural drawings were reviewed to identify penetrations which rely upon concrete capacity to resist torsion. Only the main steam and feedwater penetrations were found to be so designed.	Yes	Yes	C-17	Resolved	No further action required.

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REFERENCED CODES AND PARAGRAPH		DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE			IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
CURRENT	DESIGN		DOCUMENT	PAGE NUMBER	APPROACH					
ASME B&PV Code Section III Div 2, 1980	ACI 318-63									
CC-3440 (b), (c)	--	Elements subject to transient thermal loading (2.18)	--	--	--	--	--	--	Not addressed, but is considered in NUREG/CR-2580.	Per resolution of NUREG/CR-2580 findings.
CC-3532.1.2	--	Areas of containment shell subject to biaxial tension (2.19)	Ref. 3	p. 27 Sect. 5.9 Att. 3	Containment concrete drawings were examined to identify the areas where main reinforcing bars are terminated. Nine areas were found where the main reinforcing bars in the wall and dome are terminated and seven additional areas where supplementary bars are terminated. Thirteen of the 16 areas were individually evaluated. The tensile development lengths required for the controlling load combination were compared to the development lengths provided.	Yes	Yes	C-18	OK, for load combination considered.	Licensee should provide assurance that all containment service loads were considered in this evaluation.

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REFERENCED CODES AND PARAGRAPH		DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE		APPROACH	IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
CURRENT	DESIGN		DOCUMENT	PAGE NUMBER						
ASME B&PV Code Section III Div 2, 1980	ACI 318-63									
CC-3421.8	--	Brackets and corbels in containment shell (2.21)	Ref. 3	p. 27 Sect. 5.8 Att. 3	Drawings and calculations for the containment shell were reviewed to determine where corbels were used.	--	--	--	No brackets or corbels were found in the containment shell.	None

NOTES:

In the following notes, the Licensee's conclusion is presented first, followed by the reviewer's comments, if any, in brackets.

C-1. The review showed that the steel beam section was adequate to carry the applied loads and that composite action was not relied upon.

C-2. The analysis showed that composite design was not required for these beams and the Licensee surmised that the existing shear connectors were provided to preclude lateral torsional buckling in the top flange.

C-3. The review showed no use of hybrid girders in plant structures.

C-4. The review showed that, under normal load combinations, none of the tee section failed the code check for members in compression.

C-5. The results of the generic review showed that, for the structural materials used in the Ginna plant, the AISC 1963 Code provides a more conservative design.

[For this design code change, the conservatism or nonconservatism of the 1963 AISC Code is material dependent. For the Ginna plant, where all structural members are of A-36 steel, the conclusion that the 1963 AISC Code is more conservative is correct. However, this is not necessarily true of plants which also use other construction materials, particularly the higher strength steels.]

C-6. In all cases, it was found that the beam capacity was controlled by one of the three other loading limits cited and not by the block shear capacity.

[Positive evidence that coping will not reduce safety margins is provided for those beams which pass comparison tests 2 and 3. For such beams, the critical section controlling beam capacity is not through the coping but elsewhere.

Determination of coping acceptability by test 1 shows that safety margins (although smaller than formerly perceived to be) are still code acceptable for the loads that the Licensee considered, i.e., normal operating conditions.

In any case, since to date only normal operating loads have been considered in the Licensee's review of this item, any structural concern about the acceptability of coped members at the Ginna plant is relegated to the review of member acceptability under the portion of the III-7.B topic devoted to loads and load combinations.]

C-7. It was determined that no column web stiffeners are required to safely carry the original applied loads.

C-8. No evidence was found of plastic design methods being used.

C-9. Evaluation of the twelve corbels showed:

a. Six of seven corbels supporting primary structural elements met code requirments. One did not conform with the minimum reinforcing requirements but its stresses were too small to be of concern.

b. The five corbels which support secondary elements did not comply with the code requirements for reinforcing. However, all five have insignificant stresses.

C-10. The evaluation showed:

a. The shear walls in the auxiliary building, intermediate building, control building, containment interior structures, and screenhouse met the code requirements.

b. The shear walls in the diesel generator building did not meet current code criteria because of the new code provision for in-plane shear.

C-11. The evaluation found that, in all cases, the walls met the code required factor of safety for punching shear.

C-12. The results showed:

a. Six groups representing 26 conditions had safety factors that were equal to or greater than the code-required factor of safety, considering only code-satisfying reinforcing.

b. Five groups representing 108 conditions met the code-required factor of safety, considering code-satisfying reinforcing plus taking credit for any additional well-anchored reinforcing installed.

c. Two groups representing three conditions met the code-required factor of safety for shear stresses in unreinforced concrete.

d. One group representing six conditions (beam pockets for beam supporting the intermediate building floor) had an actual factor of safety less than the code requires, but greater than unity against ultimate failure.

[Computations for the beam pockets for beams supporting the floor at elevation 271 ft in the intermediate building were examined during of the review. This was one of several sample calculations arbitrarily chosen by the reviewers and provided by the Licensee to serve as examples typical of computations made by the Licensee in support of its conclusions. It was noted that the loading combination used in this computation was the most severe of the operating loads (which included the operational basis earthquake). However, a more severe loading would appear to occur under accident conditions (for example, a load combination including the safe shutdown earthquake).

If the same procedure used for the check computations made by the Licensee were applied to the latter loading, it appears that the beam pockets would exhibit a factor of safety less than 1. However, the check computation is conservative. It relies on the shear capacity of the concrete alone and takes no credit for additional shear resistance provided by existing bearing plate anchors and other reinforcement that may also be present.]

e. One group representing two conditions met the code required factor of safety assuming an in-situ concrete strength (f'c) of 3300 psi, as opposed to the 28-day strength of 3000 psi. This in-situ strength is judged to be reasonable.

   [The reviewers concur that it is reasonable to expect a long-term strength increase of at least this much.]

f. The results for the screenhouse show the safety factors are greater than those required by the code.

C-13. The factor of safety found for the cable tunnel was greater than the code requires. Based on this "worst case," the remaining five elements were judged to meet the current code requirements.

C-14. The evaluation found all concrete columns examined met the code required factor of safety.

C-15. Results of the evaluation:

a. Of the 46 column anchorages evaluated, 22 did not meet the ACI 349-80 Code.

b. Of the 22 that did not meet the code, 5 anchorages were unacceptable for the applied loads.

Using statistical projection, at a 95% confidence level, no more than 27% of the population of 194 column anchorages would have unacceptable margins of safety for normal load combinations.

C-16. The results of the evaluation:

a. For penetration groups with 6-in, 12-1/2-in, and 14-1/4-in diameter sleeves, the code-specified punching shear capacity of the concrete exceeded the ultimate axial load of the pipe penetration.

b. For penetration groups with 24-in and 54-in diameter sleeves, the shell capacity was judged to be adequate, since no significant punching shear loads were identified.

c. At equipment and personnel locks, significant punching shear loads occur under containment internal pressure only. Under the abnormal loading condition (1.5 Pa), adequacy against punching failure local to the penetration was demonstrated by calculations.

d. For the groups with 10-in and 24-1/4-in diameter sleeves, the shell capacity was shown adequate.

e. For the 29-in and 45-1/4-in diameter sleeve groups (feedwater and mainstream penetrations), the shell was found not to meet the current code-required factor of safety using pipe rupture loads from the original plant design calculations. However, the actual factor of safety is greater than 1.0.

C-17. A torsional shear stress check was not required.

C-18. In all of the 13 areas evaluated, the provided tensile development lengths exceeded ASME Code requirements.

5. REVIEW FINDINGS - LOADS AND LOAD COMBINATIONS

This section presents, on a structure-by-structure basis, the review findings concerning the Licensee's compliance status with respect to the loads and load combination aspects of SEP Topic III-7.B.

5.1 CONCRETE CONTAINMENT SHELLS

The reviewers concur with the RG&E conclusion (see Page 10, Attachment 3, Reference 9) that the following set of loads is, as reduced by building-specific considerations, a proper loading combination under current criteria.

1. D + L + F + Pv + To + Ro
2. D + L + F + Pv + To + Eo + Ro
3. D + L + F + Pv + To + W + Ro
4. D + 1.3L + F + Pv + To + 1.5Eo + Ro
5. D + 1.3L + F + Pv + To + 1.5W + Ro
6. D + L + F + Pv + To + Ess + Ro
7. D + L + F + Pv + To + Wt + Ro
8. D + L + F + 1.5Pa + Ta + Ra
9. D + L + F + Pa + Ta + 1.25Ra
10. D + L + F + 1.25Pa + Ta + 1.25Eo + Ra
11. D + L + F + 1.25Pa + Ta + 1.25W + Ra

*12. D + L + F + To + Eo

*13. D + L + F + To + W

14. D + L + F + Pa + Ta + Ess + Ra + Yr + Yj

TER-C5257-322 had cited load combinations 7, 8, and 14 as Scale A_x.

RG&E has demonstrated in Section 1, Attachment 2 of Reference 3 that load combinations 7 and 8 may be removed from Scale A_x classification. This conclusion is based on the results of SEP Topics II-2 and III-6.

Based on the conclusions drawn in NUREG/CR-1821 (substantiating the seismic adequacy of the containment to withstand SSE considered as acting alone) and the findings of NUREG/CR-2580 (where seismic stresses were considered in combination with other loadings), the Scale A_x rating may also be removed from load combination 14.

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*The Licensee's response references a single load combination (designated as License No. 12) representing the combined load combinations 12 and 13.

5.2 CONTAINMENT LINER

Based on the information provided by RG&E (Section 2, Attachment 2, Reference 3), the following set of loads appears to be a proper loading combination under current criteria when reduced by plant-specific considerations.

1. D + L + F + To
2. D + L + F + To + Eo
3. D + L + F + To + W
4. D + L + F + To + Eo
5. D + L + F + To + W
6. D + L + F + To + Ess
7. D + L + F + To + Wt
8. D + L + F + Pa + Ta + Ra
9. D + L + F + Pa + Ta + Ra
10. D + L + F + Pa + Ta + Eo + Ra
11. D + L + F + Pa + Ta + W + Ra

*12. D + L + F + Ha + To + Eo

*13. D + L + F + Ha + To + W

14. D + L + F + Pa + Ta + Ess + Ra

Load combinations 7, 8, and 14 are cited in TER-C5257-322 as Scale A_x.

Although the concrete shell and liner form an intergral structure and are currently designed to the same code provisions, the liner was given individual attention in the Topic III-7.B study because of the special considerations associated with it. Primary among these considerations is maintenance of liner integrity. Loading cases 7, 8, and 14 are retained as Scale A_x pending:

1. Resolution under Topic III-6 of effects associated with pipe reactions occurring under accident or faulted service conditions.
2. Resolution of concerns for dome liner integrity raised in NUREG/CR-2580.

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*See footnote on page 33.

5.3 SPENT FUEL POOL (CONCRETE)

The reviewers concur with the RG&E conclusion that the following set of loads is, as reduced by building-specific considerations, a proper load combination under current criteria.

1. 1.4D + 1.7L
2. 1.4D + 1.7L + 1.9Eo
3. 1.4D + 1.7L + 1.7W
4. 0.75 (1.4D + 1.7L)
5. 0.75 (1.4D + 1.7L + 1.9Eo)
6. 0.75 (1.4D + 1.7L + 1.7W)
7. 1.2D + 1.9Eo
8. 1.2D + 1.7W
9. D + L + Ess
10. D + L + Wt
11. D + L
12. D + L + 1.25Eo
13. D + L + Ess

TER-C5257-322 had cited load combinations 10 and 13 as Scale A_x.

RG&E has demonstrated in Section 3, Attachment 2 of Reference 3 that load combinations 10 and 13 may be removed from the list.

This was based on the Licensee's response concerning loading case 10 which states:

> "It was shown in the SER's for SEP Topics III-2 and III-4.A, that the Spent Fuel Pool would not be affected by wind and tornado (including missile) loadings."

Concerning loading case 13, the Licensee stated:

> "The spent fuel pool was shown to be adequate to withstand SSE loads, per NUREG/CR-1821. Temperature variations as the result of failures in the Spent Fuel Pool Cooling system were considered, and found acceptable, in the NRC's SER for SEP Topic IX-1, 'Fuel Storage', dated January 27, 1982."

The original analysis of the spent fuel pool treated earthquake loadings using static equivalent forces; current practice requires dynamic analysis. However, because the pool is a massive structure and because of its location, it is expected to respond to earthquake loads without appreciable amplification or structural deformations. Consequently, it seems reasonable to expect that static and dynamic treatment should not produce widely divergent results.

On this basis, for Topic II-7.B objectives, the review finds that pool adequacy has been demonstrated.

5.4 AUXILIARY BUILDING (CONCRETE)

The reviewers concur with the RG&E conclusion that the following set of loads is, as reduced by building-specific considerations, a proper loading combination under current criteria.

1. 1.4D + 1.7L
2. 1.4D + 1.7L + 1.9Eo
3. 1.4D + 1.7L + 1.7W
4. 0.75 (1.4D + 1.7L + 1.7Ro)
5. 0.75 (1.4D + 1.7L + 1.7Ro + 1.9Eo)
6. 0.75 (1.4D + 1.7L + 1.7Ro + 1.7W)
7. 1.2D + 1.9Eo
8. 1.2D + 1.7W
9. D + L + Ro + Ess
10. D + L + Ro + Wt
11. D + L + Ra
12. D + L + Ra + 1.25Eo
13. D + L + Ra + Ess

TER-C5257-322 had cited load combinations 10 and 13 as Scale A_x.

The A_x classification for both of these loading combinations is retained pending:

1. resolution of issues related to masonry walls, and
2. establishment of embedment strength needed to ensure that all columns can withstand loadings found during SEP seismic review and also from tornado loadings at wind magnitudes satisfying SEP objectives.

RG&E states that loading combination 13 reduces to loading combination 9, a case treated in the original analysis of the auxiliary building. Except for regions local to pipe penetrations or pipe supports (or the like), this equivalency does exist. However, it should be made clear that absence of a Scale A citation of a previously analyzed load combination does not necessarily reflect tacit agreement that existing analytical results are in full accord with current criteria. It merely indicates that some other loading combination was deemed likely to be more significant.

5.5 AUXILIARY BUILDING (STEEL)

Based on the information provided by RG&E (Section 5, Attachment 2 of Reference 3), the following set of loads appears to be a proper loading combination under the current criteria.

1. D + L
2. D + L + E
3. D + L + W
4. D + L + Ro
5. D + L + Ro + E
6. D + L + Ro + W
7. D + L + Ro + E'
8. D + L + Ro + Wt
9. D + L + Ra
10. D + L + Ra + E
11. D + L + Ra + E'

TER-C5257-322 had cited load combinations 10 and 13 as Scale A_x.

The A_x classification for both of these loading combinations is retained pending results from the RG&E Structural Reanalysis Program.

5.6 CONTROL BUILDING

The reviewers concur with the RG&E conclusion that the following set of loads is, as reduced by building-specific considerations, a proper loading combination under current criteria.

1. 1.4D + 1.7L
2. 1.4D + 1.7L + 1.9Eo
3. 1.4D + 1.7L + 1.7W
4. 0.75 (1.4D + 1.7L)
5. 0.75 (1.4D + 1.7L + 1.9Eo)
6. 0.75 (1.4D + 1.7L + 1.7W)
7. 1.2D + 1.9Eo
8. 1.2D + 1.7W
9. D + L + Ess
10. D + L + Wt
11. D + L
12. D + L + 1.25Eo
13. D + L + Ess

TER-C5257-322 had cited load combinations 10 and 13 as Scale A_x.

The east wall of the control building incorporates masonry block construction. Although this wall is reinforced and has received analytical attention, criteria acceptable to the NRC are not available as a basis for establishing its acceptability. Consequently, the Scale A_x rating has been retained for both loading cases 10 and 13.

5.7 INTERMEDIATE BUILDING (CONCRETE)

The reviewers concur with the RG&E conclusion that the following set of loads is, as reduced by building-specific considerations, a proper loading combination under current criteria.

1. 1.4D + 1.7L
2. 1.4D + 1.7L + 1.9Eo
3. 1.4D + 1.7L + 1.7W
4. 0.75 (1.4D + 1.7L + 1.7Ro)
5. 0.75 (1.4D + 1.7L + 1.7Ro 1.9Eo)
6. 0.75 (1.4D + 1.7L + 1.7Ro + 1.7W)
7. 1.2D + 1.9Eo
8. 1.2D + 1.7W
9. D + L + Ro + Ess
10. D + L + Ro + Wt
11. D + L + Ra

*12. D + L + Ra + 1.25Eo

*13. D + L + Ra + Ess

TER-C5257-322 had cited load combinations 10 and 13 as Scale A_x.

The A_x scale ratings for load combinations 10 and 13 are retained pending resolution of issues relating to:

1. the wind load magnitude in compliance with SEP objectives, and
2. the structural integrity of the intermediate building's masonry block walls.

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*Current requirements that the effects of an instantaneous guillotine pipe break be considered in these load combinations have been waived. The Licensee stated:

> "As noted in SEP Topic III-5.B, an inservice inspection program has been instituted by RG&E, and accepted by the NRC, which would prevent full diameter breaks in the steam and feedwater piping systems. Thus, only crack breaks in the main piping, or full-diameter breaks in the small branch lines, need to be postulated. The modifications implemented by RG&E as a result of the review of postulated piping failures in the intermediate building (e.g., jet shields and missile barriers) consider the effects of the resultant piping dymanic loads."

5.8 INTERMEDIATE BUILDING (STEEL)

Based on the information provided by RG&E (Section 8, Attachment 2, Reference 3) the following set of loads appears to be a proper loading combination under current criteria.

1. D + L
2. D + L + E
3. D + L + W
4. D + L + Ro
5. D + L + Ro + E
6. D + L + Ro + W
7. D + L + Ro + E'
8. D + L + Ro + Wt
9. D + L + Ra
*10. D + L + Ra + E
*11. D + L + Ra + E'

Load combinations 8 and 11 are cited in TER-C5257-322 as Scale A_x.

A Scale A_x rating is retained on load combination 8 pending determination of the wind speed magnitude deemed necessary to comply with SEP objectives.

A Scale A_x rating is also retained on load combination 13 based on the following consideration. NUREG/CR-1821 found the intermediate building column system, as presently constructed, to be "marginally acceptable" under SSE. Modifications to the intermediate building are currently anticipated in order to provide structural integrity under tornado. Assurance should be provided that such modifications also enhance the structure's earthquake resistance or at least do not detract from it due to an altered dynamic response.

5.9 CABLE TUNNEL

The reviewers concur with the RG&E conclusion that the following set of loads is, as reduced by building-specific considerations, a proper loading combination under current criteria.

*See footnote for corresponding items for intermediate building concrete structures (Section 5.7).

1. 1.4D + 1.7L
2. 1.4D + 1.7L + 1.9Eo
3. 1.4D + 1.7L + 1.7W
4. 0.75 (1.4D + 1.7L)
5. 0.75 (1.4D + 1.7L + 1.9Eo)
6. 0.75 (1.4D + 1.7L + 1.7W)
7. 1.2D + 1.9Eo
8. 1.2D + 1.7W
9. D + L + Ess
10. D + L + Wt
11. D + L + Ta + 1.5Pa
12. D + L + Ta + 1.25Pa + 1.25Eo
13. D + L + Ta + Pa + Ess

TER-C5257-322 had cited load combination 13 as Scale A_x.

Based on conclusions reached in NUREG/CR-1821, the Scale A_x rating for loading combination 13 may be removed, and the structural integrity of the cable tunnel may be considered demonstrated.

5.10 SCREENHOUSE

The reviewers concur with the RG&E conclusion that the following set of loads is, as reduced by building-specific considerations, a proper loading combination under current criteria.

1. 1.4D + 1.7L
2. 1.4D + 1.7L + 1.9Eo
3. 1.4D + 1.7L + 1.7W
4. 0.75 (1.4D + 1.7L)
5. 0.75 (1.4D + 1.7L + 1.9Eo)
6. 0.75 (1.4D + 1.7L + 1.7W)
7. 1.2D + 1.9Eo
8. 1.2D + 1.7W
9. D + L + Ess
10. D + L + Wt
11. D + L

*12. D + L + 1.25Eo

*13. D + L + Ess

TER-C5257-322 had cited load combinations 10 and 13 as Scale A_x.

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*Alternative methods of achieving safe shutdown are proposed under SEP Topic III-5.B in the event of postulated pipe breaks in the screenhouse.

A Scale A_x ranking is retained for load combination 10 pending resolution of wind speed magnitudes deemed satisfactory to assure compliance with SEP objectives under tornado loadings.

The Licensee observes the equivalence, when reduced by building-specific considerations, of load combination 13 (ranked Scale A_x) and load combination 9 (for which an original analysis was made). The original analysis was based on representation of earthquake loading by an equivalent static g load; current criteria presume dynamic methods of analysis.

The Scale A_x ranking is retained pending demonstration that the original analytical methods are adequately conservative.

5.11 DIESEL GENERATOR BUILDING (CONCRETE)

The reviewers concur with the RG&E conclusion that the following set of loads is, as reduced by building-specific considerations, a proper loading combination under current criteria.

1. 1.4D + 1.7L
2. 1.4D + 1.7L + 1.9Eo
3. 1.4D + 1.7L + 1.7W
4. 0.75 (1.4D + 1.7L)
5. 0.75 (1.4D + 1.7L + 1.9Eo)
6. 0.75 (1.4D + 1.7L + 1.7W)
7. 1.2D + 1.9Eo
8. 1.2D + 1.7W
9. D + L + Ess
10. D + L + Wt
11. D + L
12. D + L + 1.25Eo
13. D + L + Ess

TER-C5257-322 had cited load combinations 10 and 13 as Scale A_x.

The Scale A_x rating is retained for load combination 10 pending resolution of tornado wind speed magnitudes deemed necessary to comply with SEP objectives.

The Scale A_x rating may be removed from load combination 13 based on conclusions stated in NUREG/CR-1821.

6. SUMMARY OF REVIEW FINDINGS

Number of Scale A and Scale A_x Rankings for Unresolved Items for Ginna Seismic Category I Structures

	Scale A Code Changes		
Issues	AISC 1963 vs. AISC 1980	ACI 318-63 vs. ACI 349-76	ACI 318-63 vs. ASME B&PV Sect. III Div. 2 1980
Raised by TER-C5257-322	8	8[a]	6
Resolved	3	3	4
Remaining	5	5	2
Planned Resolution per Structural Reanalysis Program[b]	5	5	2

Issues	Scale A_x Load Combinations
Raised by TER-C5257-322	23
Resolved	7
Remaining	16
Planned Resolution per Structural Reanalysis Program[b]	16 (All structural elements except masonry walls)
Open Issues	6 (Masonry walls only)

a. Appears in TER-C5257-322 as seven items. The Licensee provided rational treatment of code shear provisions (Section 11.16) as two separate items.

b. Presumes that RG&E concurs with general recommendations (see Section 7 of this report) and that SEP structural acceptance criteria satisfactory to NRC are adopted in the Structural Reanalysis Program.

7. CONCLUSIONS AND RECOMMENDATIONS

The review disclosed that Rochester Gas and Electric Corporation has undertaken a substantial engineering effort responsive to the objectives of Topic III-7.B and that RG&E has supported its findings concerning Ginna Unit 1 with a considerable body of analytical evidence developed during the course of its review of this topic.

A number of items were found to be unresolved and these are cited in sections of this report dealing with the review findings.

The remaining items primarily relate to the assessment of effects that currently defined loads and loading combinations for extreme environmental and faulted service conditions may have on perceived margins of safety in building structures that are determined to be essential to safe shutdown, especially when these are taken in conjunction with Scale A design code changes.

RG&E plans to address these items in due course under their structural reanalysis program. All plant modifications that may be found necessary to comply with the objectives of the Systematic Evaluation Program are to be constructed to current design codes and to currently specified loads and loading combinations. Thus, for all modified plant structures, Topic III-7.B will be fully resolved.

It is anticipated, however, that some structures determined to be essential to safe shutdown will be found acceptable as built. It is likely that determination of acceptability will be based primarily on a demonstration that the general sizing of major structural elements in these buildings is adequate to sustain current loads and load combinations. A number of the design code changes, however, relate to the adequacy of specific structural details. It is therefore recommended that a review of remaining Topic III-7.B items for essential structures which are retained as-built be incorporated as a specific aspect of RG&E's structural reanalysis program.

8. REFERENCES

1. Franklin Research Center, Technical Evaluation Report
   Design Codes, Design Criteria, and Loading Combinations (SEP Topic II.7.B) Rochester Gas and Electric Corporation, Robert Emmett Ginna Nuclear Power Plant Unit 1, TER-C5257-322
   May 28, 1982

2. "Specification for Design, Fabrication, and Erection of Structural Steel for Buildings," Sixth Edition
   American Institute of Steel Construction, Inc.
   New York, NY
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3. "Specification for Design, Fabrication, and Erection of Structural Steel for Buildings," Eighth Edition
   American Institute of Steel Construction, Inc.
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4. "Code Requirements for Nuclear Safety Related Concrete Structures" (ACI 349-76)
   American Concrete Institute, Detroit, MI

5. "Building Code Requirements for Reinforced Concrete" (ACI 318-63)
   American Concrete Institute, Detroit, MI

6. ASME Boiler and Pressure Vessel Code, Section III, Division 2
   "Code for Concrete Reactor Vessels and Containments"
   New York, NY
   1980

7. J. E. Maier, Rochester Gas and Electric Corporation
   Letter to D. M. Crutchfield, Chief, Operating Reactor Branch No. 5, USNRC
   Subject: Structural Reanalysis Program, SEP Topics II-2.A, III-2, III-4.A, and III-7.B, R. E. Ginna Nuclear Power Plant, Docket No. 50-244
   April 22, 1983

8. J. E. Maier, Rochester Gas and Electric Corporation
   Letter to D. M. Crutchfield, Chief, Operating Reactor Branch No. 5, USNRC
   Subject: Structural Reanalysis Program, SEP Topics II-2.A, III-2, III-4.A, and III-7.B, R. E. Ginna Nuclear Power Plant, Docket No. 50-244
   May 19, 1983

9. J. E. Maier, Rochester Gas and Electric Corporation
   Letter to D. M. Crutchfield, Chief, Operating Reactor Branch No. 5, USNRC
   Subject: SEP Topic III-7.B, Design Codes, Design Criteria, and Load Combinations, R. E. Ginna Nuclear Power Plant, Docket No. 50-244
   May 27, 1983

10. T. C. Stilwell (FRC)
Letter to D. Persinko (NRC)
Subject: Topic list for NRC/RG&E/GC/FRC meeting of June 21, 1983
June 15, 1983

11. Hand-carried Gilbert Commonwealth calculations, in response to Reference 10, on the following subjects:

a. Integrity of structural walls against punching shear (5.6, Attachment 3 of Reference 9). Specific example: Main steam penetration under postulated LOCA.

b. Integrity of elements loaded in shear with no diagonal tension (5.3, Attachment 3 of Reference 9). Specific example: Shear capacity of beam pockets supporting the intermediate building floor.

c. Development length of lapped splices in columns (5.1, Attachment 3 of Reference 9). Specific example: Column group which includes control room column.

d. Coped beams (4.2.6 of Reference 8). Specific example: Integrity of roof beams (if coped) under extreme environmental load.

e. Steel embedments (4.2.9 of Reference 8). Specific example: Frame columns under low roof of the auxiliary building.

ENCLOSURE 2

TECHNICAL EVALUATION REPORT

SUPPLEMENTARY REPORT

(REVIEW OF LICENSEE RESPONSE TO

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

ROCHESTER GAS AND ELECTRIC CORPORATION

R. E. GINNA NUCLEAR POWER PLANT UNIT 1

NRC DOCKET NO. 50-244

NRC TAC NO. 48881

NRC CONTRACT NO. NRC-03-81-130

FRC PROJECT C5506

FRC ASSIGNMENT 18

FRC TASK 423

Prepared by

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July 29, 1983

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July 29, 1983

Prepared by:	Reviewed by:	Approved by:
M. Darwish	T. C. Stilwell	[illegible]
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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.

Summary

Information concerning the Ginna Nuclear Power Plant Unit 1 supplied to the NRC by Rochester Gas and Electric Corporation (RG&E) dealing with Topic III-7.B of NRC's Systematic Evaluation Program was reviewed. Topic III-7.B assesses the impact of perceived margins of safety of Seismic Category I structures that may result from changes in design codes and from differences between loads and loading combinations used for design and those currently specified.

The review was conducted by the Franklin Research Center with the objective of assisting the NRC in the evaluation of RG&E's compliance status with respect to implementation of the Systematic Evaluation Program by appraising the technical content of the information submitted.

The review found that RG&E has made a substantial engineering effort toward resolution of Topic III-7.B concerns. Although open items were found to remain, these primarily relate to assessment of effects of design code changes when appraised for loadings associated with extreme environmental and faulted service conditions. RG&E plans to address these concerns in due course as part of the Structural Reanalysis Program.

1. INTRODUCTION

Current design criteria for nuclear power plant structures contain requirements that were not in effect when older plants were designed and licensed. Consequently, one aspect (designated Topic III-7.B) of the implementation of NRC's Systematic Evaluation Program requires licensees to review changes that have occurred in structural design criteria since their plant was built and also to review the loads and load combinations used for design of plant structures by comparing them with the loads and load combinations now specified for current construction. The licensee's objective is to assess the impact that these changes may have on margins of safety of Seismic Category I structures as they were originally perceived and as they would be perceived under current criteria. Upon completion of this work, licensees report their findings to the NRC.

To assist in this review, the NRC provided licensees with plant-specific Technical Evaluation Reports (TERs) concerning these issues (e.g., Reference 1). The TERs listed design code changes and, on a building-by-building basis, the load and loading combination changes to be addressed in the licensee review. The items listed were ones judged to have the greatest potential to degrade the originally perceived margins of safety.

In May 1983, under contract NRC-03-81-130, the NRC retained the Franklin Research Center (FRC) to assist in its review of licensee findings. This report describes the review for the R. E. Ginna Nuclear Power Plant Unit 1 and summarizes Rochester Gas and Electric Corporation's (RG&E) compliance status with respect to the implementation of SEP Topic III-7.B.

2. DESIGN CODE CHANGES DESIGNATED SCALE A

Current structural design codes contain provisions that differ from, or did not appear in, the codes to which older plants were designed and constructed. Changes that were judged to have the potential to significantly affect perceived margins of safety have been designated as Scale A. These changes are discussed item-by-item in this section of the report.

2.1 SHEAR CONNECTORS FOR COMPOSITE BEAMS

Four major modifications to the 1963 AISC Code [2] related to the type, distribution, and spacing of shear connectors for composite beams occur in the 1980 Code [3]. These modifications are:

a. Permission to use lightweight structural concrete (concrete made with C330 aggregates) in composite designs

b. Allowance of design for composite action in the negative moment region of continuous beams and provision of design guidance for including the longitudinal reinforcing steel in the negative moment resisting section

c. Design requirements for the minimum number of shear connectors in regions of concentrated load

d. Maximum and minimum spacing requirements in terms of stud diameters.

The first two modifications will not affect old designs because they were not allowed by the previous code. The new provisions concerning the number of studs in the region near concentrated loads and the new limits concerning spacing of studs may adversely affect the margin of safety in older designs when checked against the new code provisions. These new requirements are of special concern in the case of composite beams subject to large concentrated loads, such as those associated with extreme environmental or critical accident conditions.

2.2 COMPOSITE BEAMS OR GIRDERS WITH FORMED STEEL DECK

The 1980 AISC Code [3] contains a new section covering stay-in-place formed steel deck when used in a composite design. These provisions for formed steel decking, depending on the rib geometry and the direction of the ribs relative to the beam, may affect the load capacity of the shear studs and the effective flange width of the assumed concrete compression flange. They provide for reduction factors, to be applied to the shear stud allowable capacity, which account for the structural irregularity introduced into the composite slab.

Composite beams with formed steel decks that were designed to the previous code could have less conservative margins of safety when compared to present requirements, especially in cases where extreme loadings are to be considered.

2.3 FLANGE STRESS IN HYBRID GIRDERS

The AISC Code section covering reduction of bending stress in the compression flange was modified in the 1980 Code.

The original flange stress reduction formula in the old code was needed to account for stress transfer which may occur in ordinary beam webs if the compression region should deflect laterally, thereby changing the bending capacity of the cross section. In hybrid girders, the amount of the loss of bending resistance resulting from this phenomenon will vary depending on the relative properties of the web and flange steel. A reduced bending stress formula reflecting this interaction was introduced. In order to keep the formulation relatively simple, the reduced bending stress was made applicable to both flanges of the hybrid member.

Beams or girders fabricated from plate where the flange and web steels are different could have lower margins of safety under the new code than were thought to exist under older code requirements where the ratio of web yield stress to flange yield stress is less than 0.45 and the ratio of the web area to flange area is low.

2.4 STRESSES IN UNSTIFFENED COMPRESSION ELEMENTS

New requirements provide stress reduction factors for unstiffened elements subject to compression with one edge free parallel to the compressive stress.

Previous code provisions allowed the designer to neglect a portion of the area of such elements. The new code requirements provide equations for various elements based on the critical buckling stress for plates. The new analytical approach is more conservative for the steams of tees and less conservative for all other cases.

Where structural tees are used as main members and the tee stem is in compression, the margin of safety for older designs checked under the new code could be significantly less than was thought under prior code requirements. Since buckling is a non-ductile type failure, these new requirements are of special concern in the case of tee shapes subjected to the extreme environmental or critical accident conditions.

2.5 MAXIMUM LOAD IN RIVETED OR BOLTED TENSILE MEMBERS

The 1980 AISC Code [3] introduces codes changes which affect the maximum load permitted in tensile members.

Two interacting code changes are involved in establishing this limit, and the mutual effects of both must be considered in assessing the impact of the new code upon the perception of margins of safety in tension members. The two provisions involved concern:

1. the tensile area permitted to be used in establishing load carrying capacities

2. the allowable stresses to be used in conjunction with these areas.

Both effects are taken into account in ranking this change. The potential magnitude of the mutual effects of the two changes is discussed below.

The 1980 AISC Specification definition of "Effective Net Area" introduces a reduction coefficient which is to be applied to the traditional definition of net area. This essentially changes the design capacity of a tension member when compared to older versions of these specifications. First consider only the effect of the critical area used for the design of a tension member as defined in the new code compared to the critical area used for the design of the same member as defined in the old code. Clearly, if all other factors are equal, the new code is more conservative. However, all other factors are not the same. The changes in allowable tensile stress definition (on the gross area and on the effective net area) which were introduced simultaneously with the new definition of effective net area modify the above conclusion. In addition, the traditional upper limit on the critical net area of 85% of the gross area (a requirement of the old code) is no longer a requirement of the new code. Both of these changes interact with the new effective net area requirement.

A valid assesament of the effect of these changes is best accomplished by a comparison of the allowable load each code permits in tension members. If one considers the allowable load on the effective net area, the value based on the new code is a function of three variables: the new reduction coefficient, the net area,* and the ultimate tensile strength of the steel. The allowable load based on the old code is a function of only two variables: the net area and the yield strength of the steel. First, form the load ratio of the allowable load defined by the new code criteria to the allowable load defined by the old code criteria. Next, consider the ranges of all of the parameters mentioned above, this ratio will have defined upper and lower limits which are a function of the ratio of the net areas, the new code net area reduction factor, and the ratio of the steel ultimate strength to the yield strength.

For all the steels allowed under the new code, this load ratio ranges from 1.5 to 0.69. For all the steels allowed under the old code, this load ratio ranges from 1.6 to 0.88. It is apparent that, for those steels with load ratios less than 1.0, the new code is less conservative than the old. The margin of safety of some older designs therefore could be significantly lower when checked against the new code requirements.

2.6 SHEAR LOAD IN COPED BEAMS

The 1980 AISC Code [3] introduces additional control over the shear load permitted at beam end connections where the top flange has been coped.

*In making this comparison, one must be careful to note that the net area is not always the same under the old and new codes.

Web shear control in older codes did not distinguish between coped and uncoped beams or between shear allowed at connections and over the free span (except for requiring reinforcement of thin webs at connections). The shear load allowed was given by:

allowable shear load = 0.4 (yield strength) (gross web section).

The 1980 Code retains this limit, but introduces an additional requirement to protect against a failure mode associated with coped beams. For coped beams (and similar situations), a portion of the web may sever, failing along the perimeter of the connection holes. In particular, coped beam web connections where the fastener holes lie close to the butt end of the beam may be prone to such failures.

This web "tear out" failure is actually a combination of shear failure through the line of fasteners together with tensile failure across the shortest path to the beam end. The failure surface turns a corner with shear failure along a line trending upward through the holes, combined with tensile failure across a more-or-less horizontal line running out to the beam end.

The newly introduced shear limit is given as a function of the minimum net failure surface and the steel ultimate strength. Thus, the new requirements may or may not control a coped beam's allowable capacity in shear. Whether or not it does depends on both the connection geometry and the type of steel used.

When this requirement is controlling, coped beams designed by previous rules may be found, if checked against the new criteria, to have significantly smaller margins of safety than previously thought.

2.7 COLUMN WEB STIFFENERS AT FRAME JOINTS

The more recent editions of the AISC code mandate which columns must be stiffened at locations where beams of girders are rigidly attached to the column flange and also establish requirements for the geometry of such web stiffeners. These requirements are introduced to preclude local crippling at such frame joints.

No such guidance was provided by AISC-63 [2]. Older codes (such as AISC-63) left such matters to the designer's discretion. Consequently, there is no assurance that all such columns are adequately stiffened for current accident and faulted loadings.

2.8 LATERAL SUPPORT SPACING IN FRAMES (PLASTIC DESIGN METHOD)

The 1980 AISC Code contains changed spacing requirements for lateral supports in portions of members in frames where failure mecnanisms are expected to form at ultimate load.

Members of such frames must not only be capable of developing a plastic hinge, but must also be stable enough to sustain moments larger than those computed on an elastic-perfect-plastic theory (because real steels work-harden at strains expected to occur at hinge locations). Previous lateral bracing requirements were developed for a limited range of steels. Research on high-strength steels has shown that, for certain ranges of slenderness ratio of the compression flange of such frame members, older specification bracing requirements were not sufficiently conservative.

The new specification requirements make the slenderness ratio limits a function of the steel yield strength and the member curvature (as expressed by the ratio of the lesser bending moment at the ends of the unbraced segment to the plastic moment).

The new specifications are more conservative for (1) any segment bent in double curvature regardless of its steel specification and (2) very high-strength steel members. The adequacy of frame members bent in single curvature and constructed of steels whose yield strength exceeds 36 ksi should be examined on a case-by-case basis.

The new requirements may reduce the margins of safety thought to exist in:

1. structures designed under the plastic requirements of older codes
2. elastically designed structures sized to carry a smaller maximum load than is now required by current accident and faulted load combinations. In this case, plastic logic may have to be invoked to justify the adequacy of exisiting structures. Nonconformance with

current bracing requirements may substantially restrict the capability of frame members to carry code-acceptable overloads.

2.9 BRACKETS AND CORBELS

ACI 349-76 [4], Section 11.13 contains design requirements for short brackets and corbels which are considered primary load-carrying members; no comparable requirements are provided in ACI 318-63 [5].

The requirements apply to brackets and corbels having a shear span-to-depth ratio of unity or less. They provide minimum and maximum limits on tension and shear reinforcement, limits on ultimate shear stress in concrete, and constraints on member geometry and location of reinforcement.

Brackets and corbels designed under earlier codes may or may not satisfy the newly imposed limits. If they do not, they may be prone to non-ductile failure (which occurs suddenly and without warning) and may exhibit smaller margins of safety than those currently required.

2.10 SPECIAL PROVISIONS FOR WALLS

2.10.1 Shear Walls

ACI 349-76, Sections 11.15.1 through 11.15.6 specify requirements for reinforcing and permissible shear stresses for in-plane shear loads on walls. The ACI 318-63 Code had no specific requirements for in-plane shear on shear walls.

2.10.2 Punching Shear

ACI 349-76, Section 11.15.7 specifies permissible punching shear stresses for walls. ACI 318-63 had no specific provisions for walls for these stresses. Punching loads are caused by relatively concentrated lateral loads on the walls. These loads may be from pipe supports, equipment supports, duct supports, conduit supports, or any other component producing a lateral load on a wall.

2.11 ELEMENTS LOADED IN SHEAR WITH NO DIAGONAL TENSION (SHEAR FRICTION)

The provisions for shear friction given in ACI 349-76 did not exist in ACI 318-63. These provisions specify reinforcing and stress requirements for situations where it is inappropriate to consider shear as a measure of diagonal tension.

2.12 ELEMENTS SUBJECT TO TEMPERATURE VARIATIONS

The ACI 349-76 [4], Appendix A requirements for consideration of temperature variations in concrete were not contained in ACI 318-63. These new provisions require that the effects of temperature gradients and the difference between mean temperature and base temperature during normal operation or accident conditions be considered. The new provisions also require that thermal stresses be evaluated considering the stiffness and rigidity of members and the degree of restraint of the structure.

2.13 COLUMNS WITH SPLICED REINFORCING

The ACI 349-76, Section 7.10.3 requirements for columns with spliced reinforcing did not exist in the ACI 318-63 Code. The ACI 349-76 Code requires that splices in each face of a column, where the design load stress in the longitudinal bars varies from fy in compression to 1/2 fy in tension, be developed to provide at least twice the calculated tension in that face of the column (splices in combination with unspliced bars can provide this if applicable). This code change requires that a minimum of 1/4 of the yield capacity of the bars in each face of the column be developed by both spliced and unspliced bars in that face of the column.

2.14 EMBEDMENTS

Appendix B of ACI 349-80 provides rules for the design of steel embedments in concrete; the design of embedments is not specifically addressed in ACI 318-63.

Current requirements of Appendix B are based upon ultimate strength design using factored loads. The anchorage design is controlled by the ultimate strength of the embedment steel. Ductile failure (i.e., steel yields before concrete fails) is postulated.

Under the provisions of ACI 318-63, the design of embedments was left to the discretion of the designer. Working stress design methods were widely used.

Consequently, it is likely that original embedment designs do not fully conform to current criteria. Review of such designs to determine the implications with respect to margins of safety is therefore judged a desirable precaution.

2.15 DUCTILE RESPONSE TO IMPULSE LOADS

Appendix C to ACI 349-76 [4] contains design rules for structures which may be subjected to impulse or impact loads; no such provisions occur in ACI 318-63 [5].

The rules of Appendix C are intended to foster ductile response (i.e., steel yields prior to concrete failure) of nuclear structures if and when they experience impulse or impact loads. For structures built to codes not containing such provisions, there is no assurance that sufficient design effort was directed toward proportioning members to provide energy absorbtion capability. Consequently, such structures might be prone to non-ductile, sudden failure should they ever experience postulated accident loadings such as jet impingement, pipe whip, compartment depressurization, or tornado missiles.

2.16 TANGENTIAL SHEAR (CONTAINMENTS)

Paragraph CC-3421.5, Tangential Shear, of Section III, Division 2 of the ASME Boiler and Pressure Vessel Code [6] addresses the capacity of reinforced concrete containments to carry horizontal shear load. It provides code-acceptable levels of horizontal shear stress that the designer may credit to the concrete. No specific guidance in this matter exists in ACI 318-63.

The provisions associate the allowable concrete stress in horizontal shear with the concrete properties, the manner in which lateral loads are imposed on the structure, and the presence of sufficient reinforcement to assure that the assumed shear capacity of concrete can be developed.

Sufficient diagonal reinforcement (or its demonstrated equivalent) is to be supplied to carry, without excessive strain, shear in excess of that permitted in the concrete. A major consideration here is the preservation of the structural integrity of the liner.

In containments constructed to older codes, such matters were left to the discretion of the designer, who may or may not have provided the horizontal shear capacity at controlled strains that the code currently requires.

2.17 AREAS OF CONTAINMENT SHELL SUBJECT TO PERIPHERAL SHEAR

Concrete containment design is currently governed by the ASME Boiler and Pressure Vessel Code, Section III, Division 2, 1980 [6]. The provisions for peripheral (punching) shear appear in code Section CC-3421.6. These provisions are similar to the ACI 318-63 Code [5] provisions for slabs and footings, except that the allowable punching shear stress in CC-3421.6 includes the effect of shell membrane stresses. For membrane tension, the allowable concrete punching shear stress in the ASME Code is less than that allowed by ACI 318-63.

2.18 AREAS OF CONTAINMENT SHELL SUBJECT TO TORSION

Concrete containment design is currently governed by the ASME Boiler and Pressure Vessel Code, Section III, Division 2, 1980. Section CC-3421.7 of the code contains provisions for the allowable torsional shear stress in the concrete. Such provisions were not contained in the ACI 318-63 Code. The present allowable torsional shear stress includes the effects of the membrane stresses in the containment shell and is based on a criterion that limits the principal membrane tension stress in the concrete.

2.19 THERMAL LOADS

ACI 349-76 Appendix A and ASME B&PV Code, Section III, Div. 2, CC-3440 contains requirements for consideration of temperature variations in concrete that are not contained in ACI 318-63.

The new provisions require consideration of the effects of thermal gradients and of the effects depending on the mean temperature distribution and the base temperature distribution during normal operation or accident conditions. The new provisions also require that thermal stresses be evaluated considering the stiffness and rigidity of members and the degree of restraint of the structure.

An assessment is to be made of the analytical methods used to determine thermal stresses as compared to current code-acceptable practices, e.g., those discussed in ACI 349.1R-80 and the commentary to ACI 349R-80.

If the methods used for design produce stress results which are significantly different from those current procedures generate, perceived margins of safety could be affected.

2.20 AREAS OF CONTAINMENT SHELL SUBJECT TO BIAXIAL TENSION

Increased tensile development lengths are required by Section CC-3532.1.2 of Reference 6 for reinforcing steel bars terminated in areas of reinforced concrete containment structures which may experience biaxial tension. For biaxial tension loading, bar development lengths, including both straight embedment lengths and equivalent straight length for standard hooks, are required to be increased by 25% over the standard development lengths required for uniaxial loading. Nominal temperature reinforcement is excluded from these special provisions. ACI 318-63 had no requirements related to this increase in development length.

2.21 BRACKETS AND CORBELS (ON THE CONTAINMENT SHELL)

The ACI 318-63 Code did not specify requirements for brackets and corbels. Provisions for these components are included in the ASME Boiler and Pressure Vessel Code, Section III, Division 2, Section CC-3421.8. These provisions apply to brackets and corbels having a shear-span-to-depth ratio of unity or less. The provisions specify minimum and maximum limits for tension and shear reinforcing, limits on shear stresses, and constraints on the member geometry and placement of reinforcing within the member.

3. REVIEW METHOD AND TABULAR PRESENTATIONS

The information relating to SEP Topic III-7.B which was supplied to the NRC by Rochester Gas and Electric Corporation and made available for this review is contained in the following documents:

1. J. E. Maier, Rochester Gas and Electric Corporation
   Letter to D. M. Crutchfield, Chief, Operating Reactor Branch No. 5, USNRC
   Subject: Structural Reanalysis Program, SEP Topics II-2.A, III-2, III-4.A, and III-7.B, R. E. Ginna Nuclear Power Plant, Docket No. 50-244
   April 22, 1983

2. J. E. Maier, Rochester Gas and Electric Corporation
   Letter to D. M. Crutchfield, Chief, Operating Reactor Branch No. 5, USNRC
   Subject: Structural Reanalysis Program, SEP Topics II-2.A, III-2, III-4.A, and III-7.B, R. E. Ginna Nuclear Power Plant, Docket No. 50-244
   May 19, 1983

3. J. E. Maier, Rochester Gas and Electric Corporation
   Letter to D. M. Crutchfield, Chief, Operating Reactor Branch No. 5, USNRC
   Subject: SEP Topic III-7.B, Design Codes, Design Criteria, and Load Combinations, R. E. Ginna Nuclear Power Plant, Docket No. 50-244
   May 27, 1983

4. Gilbert Commonwealth calculations for the Ginna Nuclear Power Plant Unit 1 on the following subjects:

   a. Integrity of structural walls against punching shear (5.6, Attachment 3 of Reference 3). Specific example: Main steam penetration under postulated LOCA.

   b. Integrity of elements loaded in shear with no diagonal tension (5.3, Attachment 3 of Reference 3). Specific example: Shear capacity of beam pockets supporting the intermediate building floor.

   c. Development length of lapped splices in columns (5.1, Attachment 3 of Reference 3). Specific example: Column group which includes control room column.

   d. Coped beams (4.2.6 of Reference 2). Specific example: Integrity of roof beams (if coped) under extreme environmental load.

e. Steel embedments (4.2.9 of Reference 2). Specific example: Frame columns under low roof of the auxiliary building.

Before undertaking licensee report reviews, FRC prepared tabular forms to be used as a working tool during the review process and also to document the review work and its findings when the review was completed.

These tables are intended to:

1. establish a systematic and comprehensive review procedure
2. standardize, as much as possible, the review process for all licensees
3. present a relatively compact overview of each licensee's SEP Topic III-7.B compliance status.

Two such forms were prepared, one related to design code changes and the other to the differences between loads and load combinations used for design and loads and load combinations current today.*

The form sheets provide space to summarize key information reported in licensee responses. Certain items (such as descriptions of Scale A code changes, conclusions, and comments) frequently are not adaptable to abbreviated summary. For such items, the form sheets refer the reader either to sections of this TER where the matter is developed more fully or to an extended note list compiled on separate sheets. The note list, although detached from the main table in order to allow a fuller discussion, accompanies each table and should be regarded as an integral part of it.

The form sheet consists of four major columnar sections which:

1. identify each Scale A item
2. state the action that the licensee took or the logic that the licensee presented to resolve the item
3. provide an assessment of engineering conclusions that may be reasonably drawn from the evidence provided

*The tables for load and load combinations do not appear in this report because RG&E plans to address these matters fully and in due course as part of their "Structural Reanalysis Program." However, for each Seismic Category I structure, RG&E listed currently appropriate load combinations; this is discussed later.

4. summarize the licensee's compliance status with respect to the item.

Items listed on the tables are designed code changes (or itemized load combinations) designated Scale A. This list is drawn directly from TER-C5257-322, the earlier report on this topic [1].

Licensees may choose to address potential concerns stemming from Scale A items in two ways:

1. generically, i.e., on an overall basis which resolves the concern for all plant structures collectively, or

2. on a structure-by-structure basis.

The form sheets are compiled in a manner matching the licensee's approach, with one form sheet containing generically treated matters and with structure-specific form sheets for each structure-specific matter.

Form sheets summarizing the review findings concerning the licensee's compliance status with respect to the implementation of SEP Topic III-7.B aspects related to design code changes follow in Section 4. A discussion of the review findings concerning the licensee's compliance status with respect to load and load combination changes is presented in Section 5.

4. TABULAR SUMMARY OF REVIEW FINDINGS OF LICENSEE COMPLIANCE STATUS CONCERNING IMPLEMENTATION OF SEP TOPIC III-7.B IMPACT OF DESIGN CODE CHANGES

Form sheets summarizing the review findings concerning technical aspects with respect to the implementation of SEP Topic III-7.B as related to design code changes follow.

SUMMARY OF LICENSEE COMPLIANCE STATUS --
IMPACT OF DESIGN CODE CHANGES

CODE CHANGE CITED AS SCALE A IN TER-5257-322			LICENSEE'S ACTION TO RESOLVE POTENTIAL CONCERN			EVALUATION OF LICENSEE'S ACTION			LICENSEE STATUS	
REFERENCED CODES AND PARAGRAPH		DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE		APPROACH	IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
CURRENT	DESIGN		DOCUMENT	PAGE NUMBER						
AISC 1980	AISC 1963									
1.11.4	1.11.4	Shear connectors in composite beams (2.1)	Ref. 2	p. 4-2 App. A	Calculations and construction drawings were reviewed for the use of shear connectors for composite beams.	Yes	Yes	C-1	Resolved	None
1.11.5	--	Composite beams or girders with formed steel deck (2.2)	Ref. 2	p. 4-2 App. A	Calculations and construction drawings were reviewed for composite beams with steel decking. Selected beams were analyzed for loads shown on the drawings.	Yes	See Notes 1 & 2	C-2	OK for loads shown on drawings.	Further investigation required for C and D service conditions.
1.10.6	1.10.6	Hybrid girders (2.3)	Ref. 2	p. 4-3 App. A	Construction drawings and specifications were reviewed for the existence of hybrid girders.	Yes	Yes	C-3	Not applicable	None
1.9.1.2 and App. C	1.9.1	Compression elements having width/thickness ratio greater than specified in 1980 Code (2.4)	Ref. 2	p. 4-3 App. A	The plant structural model was reviewed to determine where tee sections were used in compression. These were evaluated under normal operating load combinations.	Yes	See Notes 1 & 2	C-4	OK for normal operating load combinations.	Further investigation required for C and D service conditions.
1.14.2.2	--	Tension members, when load is transmitted by bolts or rivets (2.5)	Ref. 2	p. 4-6 App. A	Using the formulas and allowables for each code, the structural capacity of a generic design example was computed and compared.	Yes	Yes	C-5	Resolved	None

Notes: 1. The Licensee has not yet considered this code change in conjunction with current accident and faulted service loading conditions.
2. Paragraph 4.1 in Appendix A of Reference 2 states, "The effects of seismic loads are not a part of the code comparison of this report."

SUMMARY OF LICENSEE COMPLIANCE STATUS --
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PLANT: Ginna
STRUCTURE: All steel structures
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CODE CHANGE CITED AS SCALE A IN TER-5257-322			LICENSEE'S ACTION TO RESOLVE POTENTIAL CONCERN			EVALUATION OF LICENSEE'S ACTION			LICENSEE STATUS	
REFERENCED CODES AND PARAGRAPH		DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE		APPROACH	IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
CURRENT	DESIGN		DOCUMENT	PAGE NUMBER						
AISC 1980	AISC 1963									
1.5.1.2.2	--	Beam end connection with top flange coped, if subject to shear (2.6)	Ref. 2	p. 4-5 App. A	Steel fabrication drawings were reviewed for major members with bolted connections and coped top flanges. Lightly loaded girts, platforms, stair stringers, etc. were not included. The block shear capacity of each beam was compared with: 1. loads shown on the construction drawings 2. the shear capacity of the bolts, or 3. the maximum allowable load for the beam span.	Yes	See Notes 1 & 2 (Sheet 1)	C-6	OK for loads shown on the construction drawings.	Further investigation required for C and D service conditions.
1.15.5.2 through 1.15.5.4	--	Column web stiffeners for connections carrying moment or restrained member connection (2.7)	Ref. 2	p. 4-6 App. A	Construction and fabrication drawings were reviewed for use of moment connections. Only screenhouse roof beams were so designed. These were checked against the AISC 1980 Code using the original applied loads.	Yes	See Notes 1 & 2 (Sheet 1)	C-7	OK for original applied loads.	Further investigation required for C and D service conditions.

PLANT: Ginna
STRUCTURE: All steel structures
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SUMMARY OF LICENSEE COMPLIANCE STATUS --
IMPACT OF DESIGN CODE CHANGES

CODE CHANGE CITED AS SCALE A IN TER-5257-322			LICENSEE'S ACTION TO RESOLVE POTENTIAL CONCERN			EVALUATION OF LICENSEE'S ACTION			LICENSEE STATUS	
REFERENCED CODES AND PARAGRAPH		DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE		APPROACH	IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
CURRENT	DESIGN		DOCUMENT	PAGE NUMBER						
AISC 1980	AISC 1963									
2.9	2.8	Spacing of lateral supports of members designed using plastic design methods (2.8)	Ref. 2	p. 4-6 App. A	Available calculations and the Ginna FSAR were reviewed for evidence of plastic design methods.	Yes	See Notes 1 & 2 (Sheet 1)	C-8	OK for all loadings when reactions remain elastic at beam supports.	No action required unless plastic logic is subsequently used to justify the integrity of the existing structures under Scale A loading combination. If so, Licensee-stated conclusions must be reexamined.

SUMMARY OF LICENSEE COMPLIANCE STATUS --
IMPACT OF DESIGN CODE CHANGES

PLANT: Ginna
STRUCTURE: All concrete structures

CODE CHANGE CITED AS SCALE A IN TER-5257-322			LICENSEE'S ACTION TO RESOLVE POTENTIAL CONCERN			EVALUATION OF LICENSEE'S ACTION			LICENSEE STATUS	
REFERENCED CODES AND PARAGRAPH		DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE		APPROACH	IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
CURRENT	DESIGN		DOCUMENT	PAGE NUMBER						
ACI 349-76	ACI 318-63									
11.13	--	Short brackets and corbels (not on the containment shell) (2.9)	Ref. 3	p. 17 Sect. 5.2 Att. 3	Concrete outline drawings and available original calculations were reviewed to determine where brackets and corbels were used. Twelve corbels were found. Significantly loaded corbels having similar geometry were grouped. A corbel from each group (judged to have the worst load) was evaluated for compliance with ACI 349-76 configuration requirements. If all requirements were met, the capacity of the corbel was calculated in accordance with ACI 349-76. If a corbel did not conform to configuration requirements, the concrete shear stresses was computed, taking no credit for reinforcing.	Yes	Yes	C-9	Resolved	None

SUMMARY OF LICENSEE COMPLIANCE STATUS --
IMPACT OF DESIGN CODE CHANGES

PLANT: Ginna
STRUCTURE: All concrete structures

CODE CHANGE CITED AS SCALE A IN TER-5257-322			LICENSEE'S ACTION TO RESOLVE POTENTIAL CONCERN			EVALUATION OF LICENSEE'S ACTION			LICENSEE STATUS	
REFERENCED CODES AND PARAGRAPH: CURRENT	DESIGN	DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE: DOCUMENT	PAGE NUMBER	APPROACH	IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
ACI 349-76	ACI 318-63									
11.16.1 through 11.16.6	--	Shear walls used as primary load-carrying members (2.10.1)	Ref. 3	p. 20 Sect. 5.4.1 Att. 3	A total of 187 shear walls was identified. The walls in each building were taken as a group, and further classified as either interior or exterior. One wall representative of each classification was evaluated. For the controlling load combination, in-plane vertical, in-plane horizontal, and lateral loads on the wall were evaluated to code provisions. (Shear walls in the screenhouse were evaluated by comparison with auxiliary building walls.)	Yes	Yes	C-10	Resolved except for diesel generator building.	RG&E has committed to make modifications to the diesel generator building.
11.16.7	--	Punching shear stress for walls (2.10.2)	Ref. 3	p. 22 Sect. 5.4.2 Att. 3	Load sheets from the Ginna Seismic Upgrade Program were reviewed to determine punching loads from pipe and equipment supports. For pipe supports, the most severe loads found were applied to the thinnest wall, using a 6-in square area of application. The capacity of the wall calculated in accordance with the ACI 349-76 provisions was determined. Equipment punching loads were individually treated.	Yes	Yes	C-11	Resolved	None

SUMMARY OF LICENSEE COMPLIANCE STATUS --
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PLANT: Ginna
STRUCTURE: All concrete structures
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CODE CHANGE CITED AS SCALE A IN TER-5257-322			LICENSEE'S ACTION TO RESOLVE POTENTIAL CONCERN			EVALUATION OF LICENSEE'S ACTION			LICENSEE STATUS	
REFERENCED CODES AND PARAGRAPH		DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE			IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
CURRENT	DESIGN		DOCUMENT	PAGE NUMBER	APPROACH					
ACI 349-76	ACI 318-63									
11.15	--	Structural elements loaded in shear where it is inappropriate to consider shear as a measure of diagonal tension (shear friction) (2.11)	Ref. 3	p. 18 Sect. 5.3 Att. 3	Review of concrete outline drawings and available calculations revealed 203 shear-friction conditions from a variety of beam and slab supports and other situations. Similar configurations were grouped together in 15 categories. Taking credit only for reinforcement meeting ACI 349-76 provisions, the shear capacity of one member (the most heavily loaded) of each group was determined. This capacity was checked against a code-required factor of safety or (failing this) against actual failure.	Yes	Yes, but see Note C-11 and status comment	C-12	OK for loads stated in example given in Reference 5.b.	Further investigation may be required for C and D service conditions.

SUMMARY OF LICENSEE COMPLIANCE STATUS --
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PLANT: Ginna
STRUCTURE: All concrete structures
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CODE CHANGE CITED AS SCALE A IN TER-5257-322			LICENSEE'S ACTION TO RESOLVE POTENTIAL CONCERN			EVALUATION OF LICENSEE'S ACTION			LICENSEE STATUS	
REFERENCED CODES AND PARAGRAPH		DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE			IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
CURRENT	DESIGN		DOCUMENT	PAGE NUMBER	APPROACH					
ACI 349-76	ACI 318-63									
Appendix A	--	Concrete regions subject to high-temperature time-dependent and position-dependent temperature variations (2.12)	Ref. 3	p. 23 Sect. 5.5 Att. 3	In buildings where a possible thermal differential condition of consequence could occur, drawings and calculations were reviewed to determine thermal conditions. Six situations were found. Of these, the cable tunnel condition was judged to be the worst case and evaluated. Using the most severe loading combination, the moments in the cable tunnel were determined and compared to the corresponding moment capacities.	Yes	Yes	C-13	Resolved	None
7.10.3	805	Column with spliced reinforcement subject to stress reversal (2.13)	Ref. 3	p. 15 Sect. 5.1 Att. 3	Drawings and calculations were reviewed to determine columns with spliced reinforcing; 57 were found. All use lap splices at the bottom of the column. These were grouped, according to their reinforcing details and sizes, into nine categories. One heavily loaded column from each group was chosen for evaluation to to ACI 349-76 provisions. The splice	Yes	Yes, for the loads considered in the computations	C-14	OK for loads considered in the report, but the report does not clearly state that all extreme load cases have been considered for all column groups.	Further investigation may be required for C and D service conditions.

SUMMARY OF LICENSEE COMPLIANCE STATUS --
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PLANT: Ginna
STRUCTURE: All concrete structures

CODE CHANGE CITED AS SCALE A IN TER-5257-322			LICENSEE'S ACTION TO RESOLVE POTENTIAL CONCERN			EVALUATION OF LICENSEE'S ACTION			LICENSEE STATUS	
REFERENCED CODES AND PARAGRAPH		DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE		APPROACH	IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
CURRENT	DESIGN		DOCUMENT	PAGE NUMBER						
ACI 349-76	ACI 318-63									
7.10.3 (Cont.)					capacity was calculated. If the splices did not have the minimum required splice length to fully develop the bar, splice capacities were reduced in proportion to their length.					
Appendix B	--	Steel embedment used to transmit load to concrete (2.14)	Ref. 2	p. 4-7 Sect. 4.2.9	From a total population of 194 columns, 46 (having concrete anchorage) were selected as a statistical sample for evaluation. These column anchorages were checked for ductile failure and other requirements of the ACI 349-80 Code. If code requirements were met, the anchorage was deemed acceptable. If not, the ultimate capacity of the anchorage was compared to the normal design load combinations.	Yes	See Notes 1 & 2 (Sheet 1)	C-15	OK for normal design loads using current load combinations.	Further investigation may be required for C and D service conditions.
Appendix C	--	Elements subject to impulsive and impactive loads, whose failure must be precluded (2.15)	--	--	This item will be addressed in the Structural Upgrade Program.					

SUMMARY OF LICENSEE COMPLIANCE STATUS --
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STRUCTURE: All concrete structures
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CODE CHANGE CITED AS SCALE A IN TER-5257-322			LICENSEE'S ACTION TO RESOLVE POTENTIAL CONCERN			EVALUATION OF LICENSEE'S ACTION			LICENSEE STATUS	
REFERENCED CODES AND PARAGRAPH		DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE		APPROACH	IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
CURRENT	DESIGN		DOCUMENT	PAGE NUMBER						
ASME B&PV Code Section III Div 2, 1980	ACI 318-63									
CC-3421.5	--	Containment transmitting inplane shear (2.16)	Ref. 3	p. 2 Sect. 1.2 Att. 3	Resolved in SER transmitting TER-C5257-322 to RG&E (Section 1.2, Attachment 3 of Reference 3).					
CC-3421.6	1707	Region of the containment shell subject to peripheral shear (2.17)	Ref. 3	p. 24 Sect. 5.6 Att. 3	A total of 126 penetrations was identified, and grouped by sleeve diameter into ten categories. Some groups were adequate by inspection; for others a "worst case" penetration from each group was chosen and the shell capacity of these penetrations was evaluated. Actual factors of safety were calculated and compared to the factor of safety required by the code.	Yes	Yes	C-16	Resolved	No further action required.
CC-3421.7	921	Region of containment shell subject to torsion (2.18)	Ref. 3	p. 26 Sect. 5.7 Att. 3	Structural drawings were reviewed to identify penetrations which rely upon concrete capacity to resist torsion. Only the main steam and feedwater penetrations were found to be so designed.	Yes	Yes	C-17	Resolved	No further action required.

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SUMMARY OF LICENSES COMPLIANCE STATUS --
IMPACT CF DESIGN CODE CHANGES

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CODE CHANGE CITED AS SCALE A IN TER-5257-322			LICENSEE'S ACTION TO RESOLVE POTENTIAL CONCERN			EVALUATION OF LICENSEE'S ACTION			LICENSEE STATUS	
REFERENCED CODES AND PARAGRAPH: CURRENT	DESIGN	DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE: DOCUMENT	PAGE NUMBER	APPROACH	IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
ASME B&PV Code Section III Div 2, 1980	ACI 318-63									
CC-3440 (b), (c)	--	Elements subject to transient thermal loading (2.18)	--	--	--	--	--	--	Not addressed, but is considered in NUREG/CR-2580.	Per resolution of NUREG/CR-2580 findings.
CC-3532.1.2	--	Areas of containment shell subject to biaxial tension (2.19)	Ref. 3	p. 27 Sect. 5.9 Att. 3	Containment concrete drawings were examined to identify the areas where main reinforcing bars are terminated. Nine areas we e found where the mai reinforcing bars in the wall and dome are terminated and seven additional areas where supplem ntary bars are terminate . Thirteen of the 16 areas were individually evaluated. The tensile development lengths required for the controlling load combination were compared to the development lengths provided.	Yes	Yes	C-18	OK, for load combination considered.	Licensee should provide assurance that all containment service loads were considered in this evaluation.

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STRUCTURE: All concrete structures
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SUMMARY OF LICENSEE COMPLIANCE STATUS --
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CODE CHANGE CITED AS SCALE A IN TER-5257-322			LICENSEE'S ACTION TO RESOLVE POTENTIAL CONCERN			EVALUATION OF LICENSEE'S ACTION			LICENSEE STATUS	
REFERENCED CODES AND PARAGRAPH		DESCRIPTION OF CODE CHANGE (See Indicated Report Section)	REFERENCE			IS METHOD VALID AND APPROPRIATE?	IS SUFFICIENT EVIDENCE REPORTED TO JUSTIFY CONCLUSIONS?	CONCLUSIONS AND COMMENTS (SEE NOTE)	STATUS WITH RESPECT TO THIS CODE CHANGE	FURTHER ACTION REQUIRED
CURRENT	DESIGN		DOCUMENT	PAGE NUMBER	APPROACH					
ASME B&PV Code Section III Div 2, 1980	ACI 318-63									
CC-3421.8	--	Brackets and corbels in containment shell (2.21)	Ref. 3	p. 27 Sect. 5.8 Att. 3	Drawings and calculations for the containment shell were reviewed to determine where corbels were used.	--	--	--	No brackets or corbels were found in the containment shell.	None

NOTES:

In the following notes, the Licensee's conclusion is presented first, followed by the reviewer's comments, if any, in brackets.

C-1. The review showed that the steel beam section was adequate to carry the applied loads and that composite action was not relied upon.

C-2. The analysis showed that composite design was not required for these beams and the Licensee surmised that the existing shear connectors were provided to preclude lateral torsional buckling in the top flange.

C-3. The review showed no use of hybrid girders in plant structures.

C-4. The review showed that, under normal load combinations, none of the tee section failed the code check for members in compression.

C-5. The results of the generic review showed that, for the structural materials used in the Ginna plant, the AISC 1963 Code provides a more conservative design.

[For this design code change, the conservatism or nonconservatism of the 1963 AISC Code is material dependent. For the Ginna plant, where all structural members are of A-36 steel, the conclusion that the 1963 AISC Code is more conservative is correct. However, this is not necessarily true of plants which also use other construction materials, particularly the higher strength steels.]

C-6. In all cases, it was found that the beam capacity was controlled by one of the three other loading limits cited and not by the block shear capacity.

[Positive evidence that coping will not reduce safety margins is provided for those beams which pass comparison tests 2 and 3. For such beams, the critical section controlling beam capacity is not through the coping but elsewhere.

Determination of coping acceptability by test 1 shows that safety margins (although smaller than formerly perceived to be) are still code acceptable for the loads that the Licensee considered, i.e., normal operating conditions.

In any case, since to date only normal operating loads have been considered in the Licensee's review of this item, any structural concern about the acceptability of coped members at the Ginna plant is relegated to the review of member acceptability under the portion of the III-7.B topic devoted to loads and load combinations.]

C-7. It was determined that no column web stiffeners are required to safely carry the original applied loads.

C-8. No evidence was found of plastic design methods being used.

C-9. Evaluation of the twelve corbels showed:

a. Six of seven corbels supporting primary structural elements met code requirments. One did not conform with the minimum reinforcing requirements but its stresses were too small to be of concern.

b. The five corbels which support secondary elements did not comply with the code requirements for reinforcing. However, all five have insignificant stresses.

C-10. The evaluation showed:

a. The shear walls in the auxiliary building, intermediate building, control building, containment interior structures, and screenhouse met the code requirements.

b. The shear walls in the diesel generator building did not meet current code criteria because of the new code provision for in-plane shear.

C-11. The evaluation found that, in all cases, the walls met the code required factor of safety for punching shear.

C-12. The results showed:

a. Six groups representing 26 conditions had safety factors that were equal to or greater than the code-required factor of safety, considering only code-satisfying reinforcing.

b. Five groups representing 108 conditions met the code-required factor of safety, considering code-satisfying reinforcing plus taking credit for any additional well-anchored reinforcing installed.

c. Two groups representing three conditions met the code-required factor of safety for shear stresses in unreinforced concrete.

d. One group representing six conditions (beam pockets for beam supporting the intermediate building floor) had an actual factor of safety less than the code requires, but greater than unity against ultimate failure.

[Computations for the beam pockets for beams supporting the floor at elevation 271 ft in the intermediate building were examined during of the review. This was one of several sample calculations arbitrarily chosen by the reviewers and provided by the Licensee to serve as examples typical of computations made by the Licensee in support of its conclusions. It was noted that the loading combination used in this computation was the most severe of the operating loads (which included the operational basis earthquake). However, a more severe loading would appear to occur under accident conditions (for example, a load combination including the safe shutdown earthquake).

If the same procedure used for the check computations made by the Licensee were applied to the latter loading, it appears that the beam pockets would exhibit a factor of safety less than 1. However, the check computation is conservative. It relies on the shear capacity of the concrete alone and takes no credit for additional shear resistance provided by existing bearing plate anchors and other reinforcement that may also be present.]

e. One group representing two conditions met the code required factor of safety assuming an in-situ concrete strength (f'c) of 3300 psi, as opposed to the 28-day strength of 3000 psi. This in-situ strength is judged to be reasonable.

   [The reviewers concur that it is reasonable to expect a long-term strength increase of at least this much.]

f. The results for the screenhouse show the safety factors are greater than those required by the code.

C-13. The factor of safety found for the cable tunnel was greater than the code requires. Based on this "worst case," the remaining five elements were judged to meet the current code requirements.

C-14. The evaluation found all concrete columns examined met the code required factor of safety.

C-15. Results of the evaluation:

a. Of the 46 column anchorages evaluated, 22 did not meet the ACI 349-80 Code.

b. Of the 22 that did not meet the code, 5 anchorages were unacceptable for the applied loads.

Using statistical projection, at a 95% confidence level, no more than 27% of the population of 194 column anchorages would have unacceptable margins of safety for normal load combinations.

C-16. The results of the evaluation:

a. For penetration groups with 6-in, 12-1/2-in, and 14-1/4-in diameter sleeves, the code-specified punching shear capacity of the concrete exceeded the ultimate axial load of the pipe penetration.

b. For penetration groups with 24-in and 54-in diameter sleeves, the shell capacity was judged to be adequate, since no significant punching shear loads were identified.

c. At equipment and personnel locks, significant punching shear loads occur under containment internal pressure only. Under the abnormal loading condition (1.5 Pa), adequacy against punching failure local to the penetration was demonstrated by calculations.

d. For the groups with 10-in and 24-1/4-in diameter sleeves, the shell capacity was shown adequate.

e. For the 29-in and 45-1/4-in diameter sleeve groups (feedwater and mainstream penetrations), the shell was found not to meet the current code-required factor of safety using pipe rupture loads from the original plant design calculations. However, the actual factor of safety is greater than 1.0.

C-17. A torsional shear stress check was not required.

C-18. In all of the 13 areas evaluated, the provided tensile development lengths exceeded ASME Code requirements.

5. REVIEW FINDINGS - LOADS AND LOAD COMBINATIONS

This section presents, on a structure-by-structure basis, the review findings concerning the Licensee's compliance status with respect to the loads and load combination aspects of SEP Topic III-7.B.

5.1 CONCRETE CONTAINMENT SHELLS

The reviewers concur with the RG&E conclusion (see Page 10, Attachment 3, Reference 9) that the following set of loads is, as reduced by building-specific considerations, a proper loading combination under current criteria.

1. D + L + F + Pv + To + Ro
2. D + L + F + Pv + To + Eo + Ro
3. D + L + F + Pv + To + W + Ro
4. D + 1.3L + F + Pv + To + 1.5Eo + Ro
5. D + 1.3L + F + Pv + To + 1.5W + Ro
6. D + L + F + Pv + To + Ess + Ro
7. D + L + F + Pv + To + Wt + Ro
8. D + L + F + 1.5Pa + Ta + Ra
9. D + L + F + Pa + Ta + 1.25Ra
10. D + L + F + 1.25Pa + Ta + 1.25Eo + Ra
11. D + L + F + 1.25Pa + Ta + 1.25W + Ra

*12. D + L + F + To + Eo

*13. D + L + F + To + W

14. D + L + F + Pa + Ta + Ess + Ra + Yr + Yj

TER-C5257-322 had cited load combinations 7, 8, and 14 as Scale A_x.

RG&E has demonstrated in Section 1, Attachment 2 of Reference 3 that load combinations 7 and 8 may be removed from Scale A_x classification. This conclusion is based on the results of SEP Topics II-2 and III-6.

Based on the conclusions drawn in NUREG/CR-1821 (substantiating the seismic adequacy of the containment to withstand SSE considered as acting alone) and the findings of NUREG/CR-2580 (where seismic stresses were considered in combination with other loadings), the Scale A_x rating may also be removed from load combination 14.

*The Licensee's response references a single load combination (designated as License No. 12) representing the combined load combinations 12 and 13.

5.2 CONTAINMENT LINER

Based on the information provided by RG&E (Section 2, Attachment 2, Reference 3), the following set of loads appears to be a proper loading combination under current criteria when reduced by plant-specific considerations.

1. D + L + F + To
2. D + L + F + To + Eo
3. D + L + F + To + W
4. D + L + F + To + Eo
5. D + L + F + To + W
6. D + L + F + To + Ess
7. D + L + F + To + Wt
8. D + L + F + Pa + Ta + Ra
9. D + L + F + Pa + Ta + Ra
10. D + L + F + Pa + Ta + Eo + Ra
11. D + L + F + Pa + Ta + W + Ra

*12. D + L + F + Ha + To + Eo

*13. D + L + F + Ha + To + W

14. D + L + F + Pa + Ta + Ess + Ra

Load combinations 7, 8, and 14 are cited in TER-C5257-322 as Scale A_x.

Although the concrete shell and liner form an intergral structure and are currently designed to the same code provisions, the liner was given individual attention in the Topic III-7.B study because of the special considerations associated with it. Primary among these considerations is maintenance of liner integrity. Loading cases 7, 8, and 14 are retained as Scale A_x pending:

1. Resolution under Topic III-6 of effects associated with pipe reactions occurring under accident or faulted service conditions.

2. Resolution of concerns for dome liner integrity raised in NUREG/CR-2580.

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*See footnote on page 33.

5.3 SPENT FUEL POOL (CONCRETE)

The reviewers concur with the RG&E conclusion that the following set of loads is, as reduced by building-specific considerations, a proper load combination under current criteria.

1. 1.4D + 1.7L
2. 1.4D + 1.7L + 1.9Eo
3. 1.4D + 1.7L + 1.7W
4. 0.75 (1.4D + 1.7L)
5. 0.75 (1.4D + 1.7L + 1.9Eo)
6. 0.75 (1.4D + 1.7L + 1.7W)
7. 1.2D + 1.9Eo
8. 1.2D + 1.7W
9. D + L + Ess
10. D + L + Wt
11. D + L
12. D + L + 1.25Eo
13. D + L + Ess

TER-C5257-322 had cited load combinations 10 and 13 as Scale A_x.

RG&E has demonstrated in Section 3, Attachment 2 of Reference 3 that load combinations 10 and 13 may be removed from the list.

This was based on the Licensee's response concerning loading case 10 which states:

> "It was shown in the SER's for SEP Topics III-2 and III-4.A, that the Spent Fuel Pool would not be affected by wind and tornado (including missile) loadings."

Concerning loading case 13, the Licensee stated:

> "The spent fuel pool was shown to be adequate to withstand SSE loads, per NUREG/CR-1821. Temperature variations as the result of failures in the Spent Fuel Pool Cooling system were considered, and found acceptable, in the NRC's SER for SEP Topic IX-1, 'Fuel Storage', dated January 27, 1982."

The original analysis of the spent fuel pool treated earthquake loadings using static equivalent forces; current practice requires dynamic analysis. However, because the pool is a massive structure and because of its location, it is expected to respond to earthquake loads without appreciable amplification or structural deformations. Consequently, it seems reasonable to expect that static and dynamic treatment should not produce widely divergent results.

On this basis, for Topic II-7.B objectives, the review finds that pool adequacy has been demonstrated.

5.4 AUXILIARY BUILDING (CONCRETE)

The reviewers concur with the RG&E conclusion that the following set of loads is, as reduced by building-specific considerations, a proper loading combination under current criteria.

1. 1.4D + 1.7L
2. 1.4D + 1.7L + 1.9Eo
3. 1.4D + 1.7L + 1.7W
4. 0.75 (1.4D + 1.7L + 1.7Ro)
5. 0.75 (1.4D + 1.7L + 1.7Ro + 1.9Eo)
6. 0.75 (1.4D + 1.7L + 1.7Ro + 1.7W)
7. 1.2D + 1.9Eo
8. 1.2D + 1.7W
9. D + L + Ro + Ess
10. D + L + Ro + Wt
11. D + L + Ra
12. D + L + Ra + 1.25Eo
13. D + L + Ra + Ess

TER-C5257-322 had cited load combinations 10 and 13 as Scale A_x.

The A_x classification for both of these loading combinations is retained pending:

1. resolution of issues related to masonry walls, and
2. establishment of embedment strength needed to ensure that all columns can withstand loadings found during SEP seismic review and also from tornado loadings at wind magnitudes satisfying SEP objectives.

RG&E states that loading combination 13 reduces to loading combination 9, a case treated in the original analysis of the auxiliary building. Except for regions local to pipe penetrations or pipe supports (or the like), this equivalency does exist. However, it should be made clear that absence of a Scale A citation of a previously analyzed load combination does not necessarily reflect tacit agreement that existing analytical results are in full accord with current criteria. It merely indicates that some other loading combination was deemed likely to be more significant.

5.5 AUXILIARY BUILDING (STEEL)

Based on the information provided by RG&E (Section 5, Attachment 2 of Reference 3), the following set of loads appears to be a proper loading combination under the current criteria.

1. D + L
2. D + L + E
3. D + L + W
4. D + L + Ro
5. D + L + Ro + E
6. D + L + Ro + W
7. D + L + Ro + E'
8. D + L + Ro + Wt
9. D + L + Ra
10. D + L + Ra + E
11. D + L + Ra + E'

TER-C5257-322 had cited load combinations 10 and 13 as Scale A_x.

The A_x classification for both of these loading combinations is retained pending results from the RG&E Structural Reanalysis Program.

5.6 CONTROL BUILDING

The reviewers concur with the RG&E conclusion that the following set of loads is, as reduced by building-specific considerations, a proper loading combination under current criteria.

1. 1.4D + 1.7L
2. 1.4D + 1.7L + 1.9Eo
3. 1.4D + 1.7L + 1.7W
4. 0.75 (1.4D + 1.7L)
5. 0.75 (1.4D + 1.7L + 1.9Eo)
6. 0.75 (1.4D + 1.7L + 1.7W)
7. 1.2D + 1.9Eo
8. 1.2D + 1.7W
9. D + L + Ess
10. D + L + Wt
11. D + L
12. D + L + 1.25Eo
13. D + L + Ess

TER-C5257-322 had cited load combinations 10 and 13 as Scale A_x.

The east wall of the control building incorporates masonry block construction. Although this wall is reinforced and has received analytical attention, criteria acceptable to the NRC are not available as a basis for establishing its acceptability. Consequently, the Scale A_x rating has been retained for both loading cases 10 and 13.

5.7 INTERMEDIATE BUILDING (CONCRETE)

The reviewers concur with the RG&E conclusion that the following set of loads is, as reduced by building-specific considerations, a proper loading combination under current criteria.

1. 1.4D + 1.7L
2. 1.4D + 1.7L + 1.9Eo
3. 1.4D + 1.7L + 1.7W
4. 0.75 (1.4D + 1.7L + 1.7Ro)
5. 0.75 (1.4D + 1.7L + 1.7Ro 1.9Eo)
6. 0.75 (1.4D + 1.7L + 1.7Ro + 1.7W)
7. 1.2D + 1.9Eo
8. 1.2D + 1.7W
9. D + L + Ro + Ess
10. D + L + Ro + Wt
11. D + L + Ra

*12. D + L + Ra + 1.25Eo

*13. D + L + Ra + Ess

TER-C5257-322 had cited load combinations 10 and 13 as Scale A_x.

The A_x scale ratings for load combinations 10 and 13 are retained pending resolution of issues relating to:

1. the wind load magnitude in compliance with SEP objectives, and
2. the structural integrity of the intermediate building's masonry block walls.

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*Current requirements that the effects of an instantaneous guillotine pipe break be considered in these load combinations have been waived. The Licensee stated:

> "As noted in SEP Topic III-5.B, an inservice inspection program has been instituted by RG&E, and accepted by the NRC, which would prevent full diameter breaks in the steam and feedwater piping systems. Thus, only crack breaks in the main piping, or full-diameter breaks in the small branch lines, need to be postulated. The modifications implemented by RG&E as a result of the review of postulated piping failures in the intermediate building (e.g., jet shields and missile barriers) consider the effects of the resultant piping dynamic loads."

5.8 INTERMEDIATE BUILDING (STEEL)

Based on the information provided by RG&E (Section 8, Attachment 2, Reference 3) the following set of loads appears to be a proper loading combination under current criteria.

1. D + L
2. D + L + E
3. D + L + W
4. D + L + Ro
5. D + L + Ro + E
6. D + L + Ro + W
7. D + L + Ro + E'
8. D + L + Ro + Wt
9. D + L + Ra

*10. D + L + Ra + E

*11. D + L + Ra + E'

Load combinations 8 and 11 are cited in TER-C5257-322 as Scale A_x.

A Scale A_x rating is retained on load combination 8 pending determination of the wind speed magnitude deemed necessary to comply with SEP objectives.

A Scale A_x rating is also retained on load combination 13 based on the following consideration. NUREG/CR-1821 found the intermediate building column system, as presently constructed, to be "marginally acceptable" under SSE. Modifications to the intermediate building are currently anticipated in order to provide structural integrity under tornado. Assurance should be provided that such modifications also enhance the structure's earthquake resistance or at least do not detract from it due to an altered dynamic response.

5.9 CABLE TUNNEL

The reviewers concur with the RG&E conclusion that the following set of loads is, as reduced by building-specific considerations, a proper loading combination under current criteria.

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*See footnote for corresponding items for intermediate building concrete structures (Section 5.7).

1. 1.4D + 1.7L
2. 1.4D + 1.7L + 1.9Eo
3. 1.4D + 1.7L + 1.7W
4. 0.75 (1.4D + 1.7L)
5. 0.75 (1.4D + 1.7L + 1.9Eo)
6. 0.75 (1.4D + 1.7L + 1.7W)
7. 1.2D + 1.9Eo
8. 1.2D + 1.7W
9. D + L + Ess
10. D + L + Wt
11. D + L + Ta + 1.5Pa
12. D + L + Ta + 1.25Pa + 1.25Eo
13. D + L + Ta + Pa + Ess

TER-C5257-322 had cited load combination 13 as Scale A_x.

Based on conclusions reached in NUREG/CR-1821, the Scale A_x rating for loading combination 13 may be removed, and the structural integrity of the cable tunnel may be considered demonstrated.

5.10 SCREENHOUSE

The reviewers concur with the RG&E conclusion that the following set of loads is, as reduced by building-specific considerations, a proper loading combination under current criteria.

1. 1.4D + 1.7L
2. 1.4D + 1.7L + 1.9Eo
3. 1.4D + 1.7L + 1.7W
4. 0.75 (1.4D + 1.7L)
5. 0.75 (1.4D + 1.7L + 1.9Eo)
6. 0.75 (1.4D + 1.7L + 1.7W)
7. 1.2D + 1.9Eo
8. 1.2D + 1.7W
9. D + L + Ess
10. D + L + Wt
11. D + L

*12. D + L + 1.25Eo

*13. D + L + Ess

TER-C5257-322 had cited load combinations 10 and 13 as Scale A_x.

*Alternative methods of achieving safe shutdown are proposed under SEP Topic III-5.B in the event of postulated pipe breaks in the screenhouse.

A Scale A_x ranking is retained for load combination 10 pending resolution of wind speed magnitudes deemed satisfactory to assure compliance with SEP objectives under tornado loadings.

The Licensee observes the equivalence, when reduced by building-specific considerations, of load combination 13 (ranked Scale A_x) and load combination 9 (for which an original analysis was made). The original analysis was based on representation of earthquake loading by an equivalent static g load; current criteria presume dynamic methods of analysis.

The Scale A_x ranking is retained pending demonstration that the original analytical methods are adequately conservative.

5.11 DIESEL GENERATOR BUILDING (CONCRETE)

The reviewers concur with the RG&E conclusion that the following set of loads is, as reduced by building-specific considerations, a proper loading combination under current criteria.

1. 1.4D + 1.7L
2. 1.4D + 1.7L + 1.9Eo
3. 1.4D + 1.7L + 1.7W
4. 0.75 (1.4D + 1.7L)
5. 0.75 (1.4D + 1.7L + 1.9Eo)
6. 0.75 (1.4D + 1.7L + 1.7W)
7. 1.2D + 1.9Eo
8. 1.2D + 1.7W
9. D + L + Ess
10. D + L + Wt
11. D + L
12. D + L + 1.25Eo
13. D + L + Ess

TER-C5257-322 had cited load combinations 10 and 13 as Scale A_x.

The Scale A_x rating is retained for load combination 10 pending resolution of tornado wind speed magnitudes deemed necessary to comply with SEP objectives.

The Scale A_x rating may be removed from load combination 13 based on conclusions stated in NUREG/CR-1821.

6. SUMMARY OF REVIEW FINDINGS

Number of Scale A and Scale A_x Rankings for Unresolved Items for Ginna Seismic Category I Structures

Issues	Scale A Code Changes: AISC 1963 vs. AISC 1980	ACI 318-63 vs. ACI 349-76	ACI 318-63 vs. ASME B&PV Sect. III Div. 2 1980
Raised by TER-C5257-322	8	8[a]	6
Resolved	3	3	4
Remaining	5	5	2
Planned Resolution per Structural Reanalysis Program[b]	5	5	2

Issues	Scale A_x Load Combinations	
Raised by TER-C5257-322	23	
Resolved	7	
Remaining	16	
Planned Resolution per Structural Reanalysis Program[b]	16	(All structural elements except masonry walls)
Open Issues	6	(Masonry walls only)

a. Appears in TER-C5257-322 as seven items. The Licensee provided rational treatment of code shear provisions (Section 11.16) as two separate items.

b. Presumes that RG&E concurs with general recommendations (see Section 7 of this report) and that SEP structural acceptance criteria satisfactory to NRC are adopted in the Structural Reanalysis Program.

7. CONCLUSIONS AND RECOMMENDATIONS

The review disclosed that Rochester Gas and Electric Corporation has undertaken a substantial engineering effort responsive to the objectives of Topic III-7.B and that RG&E has supported its findings concerning Ginna Unit 1 with a considerable body of analytical evidence developed during the course of its review of this topic.

A number of items were found to be unresolved and these are cited in sections of this report dealing with the review findings.

The remaining items primarily relate to the assessment of effects that currently defined loads and loading combinations for extreme environmental and faulted service conditions may have on perceived margins of safety in building structures that are determined to be essential to safe shutdown, especially when these are taken in conjunction with Scale A design code changes.

RG&E plans to address these items in due course under their structural reanalysis program. All plant modifications that may be found necessary to comply with the objectives of the Systematic Evaluation Program are to be constructed to current design codes and to currently specified loads and loading combinations. Thus, for all modified plant structures, Topic III-7.B will be fully resolved.

It is anticipated, however, that some structures determined to be essential to safe shutdown will be found acceptable as built. It is likely that determination of acceptability will be based primarily on a demonstration that the general sizing of major structural elements in these buildings is adequate to sustain current loads and load combinations. A number of the design code changes, however, relate to the adequacy of specific structural details. It is therefore recommended that a review of remaining Topic III-7.B items for essential structures which are retained as-built be incorporated as a specific aspect of RG&E's structural reanalysis program.

8. REFERENCES

1. Franklin Research Center, Technical Evaluation Report
Design Codes, Design Criteria, and Loading Combinations (SEP Topic II-7.B) Rochester Gas and Electric Corporation, Robert Emmett Ginna Nuclear Power Plant Unit 1, TER-C5257-322
May 28, 1982

2. "Specification for Design, Fabrication, and Erection of Structural Steel for Buildings," Sixth Edition
American Institute of Steel Construction, Inc.
New York, NY
1963

3. "Specification for Design, Fabrication, and Erection of Structural Steel for Buildings," Eighth Edition
American Institute of Steel Construction, Inc.
New York, NY
1980

4. "Code Requirements for Nuclear Safety Related Concrete Structures" (ACI 349-76)
American Concrete Institute, Detroit, MI

5. "Building Code Requirements for Reinforced Concrete" (ACI 318-63)
American Concrete Institute, Detroit, MI

6. ASME Boiler and Pressure Vessel Code, Section III, Division 2
"Code for Concrete Reactor Vessels and Containments"
New York, NY
1980

7. J. E. Maier, Rochester Gas and Electric Corporation
Letter to D. M. Crutchfield, Chief, Operating Reactor Branch No. 5, USNRC
Subject: Structural Reanalysis Program, SEP Topics II-2.A, III-2, III-4.A, and III-7.B, R. E. Ginna Nuclear Power Plant, Docket No. 50-244
April 22, 1983

8. J. E. Maier, Rochester Gas and Electric Corporation
Letter to D. M. Crutchfield, Chief, Operating Reactor Branch No. 5, USNRC
Subject: Structural Reanalysis Program, SEP Topics II-2.A, III-2, III-4.A, and III-7.B, R. E. Ginna Nuclear Power Plant, Docket No. 50-244
May 19, 1983

9. J. E. Maier, Rochester Gas and Electric Corporation
Letter to D. M. Crutchfield, Chief, Operating Reactor Branch No. 5, USNRC
Subject: SEP Topic III-7.B, Design Codes, Design Criteria, and Load Combinations, R. E. Ginna Nuclear Power Plant, Docket No. 50-244
May 27, 1983

10. T. C. Stilwell (FRC)
    Letter to D. Persinko (NRC)
    Subject: Topic list for NRC/RG&E/GC/FRC meeting of June 21, 1983
    June 15, 1983

11. Hand-carried Gilbert Commonwealth calculations, in response to Reference 10, on the following subjects:

    a. Integrity of structural walls against punching shear (5.6, Attachment 3 of Reference 9). Specific example: Main steam penetration under postulated LOCA.

    b. Integrity of elements loaded in shear with no diagonal tension (5.3, Attachment 3 of Reference 9). Specific example: Shear capacity of beam pockets supporting the intermediate building floor.

    c. Development length of lapped splices in columns (5.1, Attachment 3 of Reference 9). Specific example: Column group which includes control room column.

    d. Coped beams (4.2.6 of Reference 8). Specific example: Integrity of roof beams (if coped) under extreme environmental load.

    e. Steel embedments (4.2.9 of Reference 8). Specific example: Frame columns under low roof of the auxiliary building.