UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D.C. 20555-0001

February 28, 1994

Docket No. 52-004

Mr. Patrick W. Marriott, Manager
Licensing & Consulting Services
GE Nuclear Energy
175 Curtner Avenue
San Jose, California 95125

Dear Mr. Marriott:

SUBJECT: REQUEST FOR ADDITIONAL INFORMATION (RAI) REGARDING THE SIMPLIFIED BOILING WATER REACTOR (SBWR) DESIGN (Q220.1-Q220.57)

The staff has determined that it needs additional information to support its review activities related to the SBWR design certification. Some additional information on the civil/structural engineering design described in Chapter 3 of the SBWR standard safety analysis report (SSAR) is needed (Q220.1-Q220.57).* Please provide a written response to the enclosed questions within 90 days of the date of this letter.

You have previously requested that portions of the information submitted in the August 1992, application for design certification of the SBWR plant, as supplemented in February 1993, be exempt from mandatory public disclosure. The staff has not completed its review of your request in accordance with the requirements of 10 CFR 2.790; therefore, that portion of the submitted information is being withheld from public disclosure pending the staff's final determination. The staff concludes that this RAI does not contain those portions of the information for which you are seeking exemption. However, the staff will withhold this letter from public disclosure for 30 calendar days from the date of this letter to allow GE the opportunity to verify the staff's conclusions. If, after that time, you do not request that all or portions of the information in the enclosure be withheld from public disclosure in accordance with 10 CFR 2.790, this letter will be placed in the NRC's Public Document Room.

*The numbers in parentheses designate the tracking numbers assigned to the questions.

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This RAI affects nine or fewer respondents, and therefore, is not subject to review by the Office of Management and Budget under P.L. 96-511.

If you have any questions regarding this matter, please contact me at (301) 504-1178 or Mr. Son Ninh at (301) 925-1125.

Sincerely,

(Original signed by)

Melinda Malloy, Project Manager
Standardization Project Directorate
Associate Directorate for Advanced Reactors
and License Renewal
Office of Nuclear Reactor Regulation

Enclosure:
RAI on the SBWR Design

cc w/enclosure:
See next page

Distribution (w/ enclosure):

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*To be held for 30 days

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DATE	2/25/94	2/25/94	2/28/94	2/28/94

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Docket No. 52-004

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REQUEST FOR ADDITIONAL INFORMATION (RAI) ON THE
SIMPLIFIED BOILING WATER REACTOR (SBWR) DESIGN

Civil/Structural Engineering Design

220.1 In Section 2.3.1 of the standard safety analysis report (SSAR), the applicant states, "The missile spectra is per Spectrum I of Standard Review Plan 3.5.1.4. Missile velocity is 35 percent of the maximum horizontal wind speed with an altitude of 9.1 m (30 ft) above grade for large soft and rigid missiles. Small rigid missiles are postulated at all elevations." The following items should be clarified:

a. What is the maximum horizontal wind speed? Is it the design basis wind or the design basis tornado?

b. What is the definition of large soft and rigid missiles?

c. Standard Review Plan (SRP) Section 3.5.1.4 does not specify the elevation requirement for an 1800 kg automobile and a 125 kg 8" armor-piercing artillery shell and they are assumed to impact at normal incidence. Provide the basis for the 9.1 m (30 ft) elevation requirement imposed for them.

d. What are the vertical velocities for Spectrum I?

220.2 The following items are missing from SSAR Section 3.3.3 regarding combined operating license (COL) license information:

a. Effect of remainder of plant structures, systems, and components not designed for wind loads.

b. Identification of potential accident situations in the vicinity of the plant and the bases upon which these potential accidents were or were not accommodated in the design. If the site-dependent blast loads are not accommodated in the standard design, the design resistance for blast loading needs to be characterized.

220.3 In SSAR Section 3.5.3, the modified Petry formula is applied for missile penetration in concrete. However, Table 1 in SRP Section 3.5.3 specifies the minimum acceptable barrier thickness requirement for local damage prediction against tornado generated missiles. Discuss in the SSAR how the minimum wall thickness criteria per the SRP guideline is being incorporated into the plant design.

220.4 Why are SSAR Sections 3.5.4.2 and 3.5.4.3 identical? Should one section describe the COL item for the missiles generated by other natural phenomena?

220.5 In SSAR Section 3.7.1, provide the analysis method and design criteria for seismic Category II structures, systems, and components.

Enclosure

220.6 Regarding SSAR Section 3.7.2.5, the NRC will review the direct spectra generation method of developing floor response spectra based on Reference 3.7-1 (if used by GE) and accept it on a case-by-case basis. Therefore, the actual method GE proposes to use should be described in the SSAR for staff review and approval.

220.7 In SSAR Section 3.7.2.10, indicate the rationale for using equivalent vertical static factors, even though all seismic Category I structures and the reactor pressure vessel (RPV) are subjected to a vertical dynamic analysis. Provide the justification that the structure is rigid in the vertical direction.

220.8 If modular construction is used, SSAR Section 3.7.2 should be revised to include a discussion on seismic behavior and the corresponding design analysis methods for the SBWR modular elements.

220.9 In SSAR Section 3.7.3.2 for the seismic fatigue evaluation, provide the rationale for using two low-level earthquake events [lesser magnitude than the safe shutdown earthquake (SSE)] rather than two SSEs.

220.10 In SSAR Section 3.7.3.2, provide a discussion for the following items:

a. Qualification by design rule.

b. Analysis procedure for non-seismic structures in lieu of dynamic analysis.

c. Interaction of other structures, systems, and components with seismic Category I structures, systems, and components.

220.11 In SSAR Section 3.7.3.12, is there any seismic Category I buried piping in the SBWR? If so, identify the lines and discuss how inservice inspection will be performed on the welds?

220.12 In SSAR Section 3.8.1, provide functional requirements in the description for the reinforced concrete containment.

220.13 In SSAR Section 3.8.1.1, provide more details in the descriptions of the containment, including the foundation, containment wall, liner plate, top slab, and location and typical arrangement of reinforcements.

220.14 In SSAR Section 3.8.1.1, for steel liners, provide the materials to be used, stiffening methods, thicknesses at various locations (e.g., major structural attachments, such as penetration sleeves, structural beam brackets, RPV pedestal, and diaphragm floor connections to the containment wall), and erection methods.

220.15 SSAR Figure 6.2-1 shows the ASME Boiler and Pressure Vessel Code Section III Subsection CC jurisdictional boundaries for the concrete containment design. However, more descriptive information is needed to establish that the primary structural aspects and elements relied upon to perform the containment function have been adequately defined in SSAR Section 3.8.1.1.

220.16 In SSAR Section 3.8.1.5, provide any construction loads which are applied to the containment (as applicable).

220.17 In SSAR Section 3.8.1.5, provide the justification for the inclusion of 25 percent live loads in the calculation of design seismic forces. Indicate whether or not 100 percent of the live loads will be considered for local stresses.

220.18 SSAR Section 3.8.1.6 references Table 3.8-1. What does "OT" in the "Events" column of Table 3.8-1 stand for?

220.19 In SSAR Section 3.8.1.6, provide the loads and load combination to be used for the design of the steel liner and liner anchors.

220.20 In SSAR Section 3.8.1.7, discuss the extent to which the STARDYNE computer code was validated for application to models being analyzed for the SBWR standard plant design.

220.21 In SSAR Section 3.8.1.7, with regard to the design and analysis procedures utilized for the containment, provide the following information for the staff to review:

a. Axisymmetric and nonaxisymmetric loads for cylindrical wall, top slab, and foundation mat

b. Major penetrations

c. Variation of physical material properties

d. Corrosion prevention

e. Containment ultimate capacity

f. Welding method

g. Testing and inservice inspection requirements

(1) Structural integrity pressure test
(2) Preoperational and inservice integrated leak rate test

220.22 In SSAR Section 3.8.2, provide a design description for and analysis procedures to be used for steel components of the containment which resist pressure and are not backed by structural concrete, such as personnel air locks, equipment hatches, penetrations, and the drywell head, if applicable.

220.23 In SSAR Section 3.8.3, provide the following information:

a. Special construction techniques to be used

b. Welding methods and acceptance criteria for structural and building steel

c. Testing and inservice inspection requirements

220.24 In SSAR Section 3.8.3.1, describe the functions of the containment internal structures.

220.25 In SSAR Section 3.8.3.1, clarify the statement "...The containment internal structures are of structural steel construction." Are all the containment internal structures constructed of structural steel only?

220.26 In SSAR Section 3.8.3.1, indicate if the following structures are included in the containment internal structures:

a. Reactor pedestal

b. Other internal structures

(1) Miscellaneous platforms
(2) Lower drywell equipment tunnel
(3) Lower drywell personnel tunnel

220.27 In SSAR Section 3.8.3.2, provide commitments to the following regulatory guides (RGs) or propose other suitable methods: RGs 1.10, 1.15, 1.55, 1.57, 1.94, and 1.142.

220.28 The staff positions on the soil pressure in embedded structures (see Appendix A) and the use of ANSI/AISC N690 for steel internal structures (see Appendix B) should be incorporated in SSAR Section 3.8.3.4.

220.29 In SSAR Section 3.8.3.4, indicate the load combination technique [e.g., square root of the sum of the squares (SRSS), absolute sum] being used. Add "SRSS" to the list of acronyms in the SSAR.

220.30 In SSAR Section 3.8.3.4, provide the design and analysis procedures for the following structures:

a. Diaphragm floor

b. Reactor pedestal

c. Reactor shield wall

d. Drywell equipment and pipe support structure

e. Other internal structures

(1) Miscellaneous platforms
(2) Lower drywell equipment tunnel
(3) Lower drywell personnel tunnel

220.31 Because the internal structures use concrete as well as steel, the load combination, the design analysis procedures, and the acceptance criteria for concrete structures should also be included in SSAR Section 3.8.3.6.

220.32 In SSAR Section 3.8.4, provide the following information on loads, load combinations, design and analysis procedures, structural acceptance criteria, and welding and weld acceptance criteria, as applicable:

a. Seismic Category I cable trays, cable tray supports, conduit, and conduit supports

b. Seismic Category I heating, ventilation, and air conditioning ducts and supports

220.33 In SSAR Section 3.8.4, provide the following information pursuant to SRP Section 3.8.4:

a. Special construction techniques to be used

b. Testing and inservice inspection requirements

220.34 In SSAR Section 3.8.4.2, the staff positions on the soil pressure in embeded structures (see Appendix A), the use of ANSI/AISC N690 for steel internal structures (see Appendix B), and the use of ACI Code 349 for steel embedments (see Appendix C) should be incorporated in the SSAR and cross referenced, as appropriate.

220.35 In SSAR Section 3.8.5, provide the following information:

a. Special construction techniques to be used

b. Testing and inservice inspection requirements

220.36 In SSAR Section 3.8.5.3, indicate how the live loads in load combinations 1 and 3 are evaluated and provide the justification for including the live load pursuant to SRP Section 3.8.5.

220.37 In SSAR Section 3.8.5.3, indicate if the "F" in the listed load combinations means buoyant force of design groundwater or buoyant force of design basis flood.

220.38 In SSAR Section 3.8.5.3, define the "H" in the listed load combinations.

220.39 In SSAR Section 3.8.5.4, provide a detailed description of the analytical and design methods for the foundation mat.

220.40 In SSAR Section 3.8.5.4, identify and evaluate the available conservatism in the foundation stability calculations against sliding.

220.41 In SSAR Section 3.8.5.4, provide the basis for excluding wind and tornado loadings for overturning and sliding in Section 3E.3.4 and Table 3E.7-26.

220.42 In SSAR Section 3.8.6, provide the following as COL license information:

a. Foundation waterproofing

b. Site-specific physical properties and foundation settlement

c. Structural integrity test pressure results

d. Identification of seismic Category I structures

e. Structural modular constructions

220.43 Throughout SSAR Appendix 3E, care should be taken in the conversion from psig to kPa. The psig should be changed to psia first and then to kPa. Provide the revised kPa conversions throughout this appendix.

220.44 The staff positions on soil pressure in embeded structures (see Appendix A), the use of ACI Code 349 for steel embedments (see Appendix C), and the use of ANSI/AISC N690 for steel internal structures (see Appendix B) should be incorporated in SSAR Appendix 3E.

220.45 In SSAR Section 3E.3.3.2, provide the acceptance criteria for both the concrete and steel parts of the RPV pedestal.

220.46 In SSAR Section 3E.3.3.2, most containment internal structures are made of structural steel filled with concrete. The strength of the filler concrete seemed to be neglected. For the stress analysis due to static loads, such an approach should be conservative. But for dynamic analysis, the filler concrete might contribute not only the mass but also to the stiffness. Depending on the quality of the filler concrete, cracks can develop in the filler concrete. Both of these can affect the natural frequency of the modular structure and, thus, its dynamic behavior. Therefore, it is essential to take such conditions into consideration in the design. Provide the acceptance criteria for the dynamic analysis of these structures.

220.47 In SSAR Section 3E.3.4, provide the basis for excluding wind and tornado loads in the calculations for overturning and sliding safety factors of reactor building foundations.

220.48 In SSAR Section 3E.3.4, provide information on material properties for concrete, reinforcing bar, liner plate, and other materials.

220.49 In SSAR Figure 3E.3-5, the chugging pressure ratio at 3.5 m is 2.1:1.0 = 1:0.476 for the ABWR and 1:0.4 for the SBWR. Provide an explanation for this difference.

220.50 In SSAR Section 3E.4.1.3.1, indicate if the strength of in-fill concrete is considered.

220.51 Provide the descriptions for the intermediate steel frame in SSAR Section 3E.4.1.4.

220.52 In SSAR Section 3E.4.2.2, for the diaphragm floor and the vent wall, provide the descriptions for the connections between steel frame (quadrilateral and/or triangular plate elements) and in-fill concrete (cube elements) in the analytical model.

220.53 In SSAR Section 3E.4.3, provide the actual values to be used for the vertical and horizontal soil spring constants, K_z and K_x, and maximum soil bearing stress.

220.54 In SSAR Section 3E.4.5, provide the validation package for the CECAP computer code for staff review.

220.55 In SSAR Section 3E.4.5, provide a flow chart, including STARDYNE, CECAP, load combinations, etc., to show how the rebar stress, concrete stress, and liner plate stra are obtained.

220.56 In SSAR Section 3E.7.6, provide the following information:

a. The detailed energy balance approach to calculate the factor of safety against overturning

b. The procedures to calculate the factor of safety against floatation

c. Provide the complete procedure to compute F_s.

d. Identify and evaluate the available conservatism in the foundation stability calculations against sliding

220.57 Make the following editorial changes in the indicated sections of the SSAR:

a. Sections 2.4.2 and 2.4.3 - The staff believes these two sections are mixed up. Section 2.4.2 should say that the probable maximum flood (PMF), as defined in ANSI/ANS 2.8, is 0.3 m (1 ft) below grade or lower and there should not be any requirement in the Section 2.4.3 for the PMF on streams and rivers.

b. Section 2.4.10 - Should reference Section 2.4.2, not 2.4.3.

c. Section 3.3.3 - Reference 1 (ANSI Standard A58.1) should be updated to ANSI/ASCE 7-88.

d. Section 3.5.1.4 - Interface requirements are described in both Sections 3.5.4.2 and 3.5.4.5, not just Section 3.5.4.2 as indicated.

e. Section 3.7.1 - In the second bullet on page 3.7-1, was "in a safe condition" intended to read "in a safe shutdown condition?"

f. Section 3.7.2.5 - In the last paragraph, change "internals" to "intervals."

g. Section 3.8.1.2 - It is the staff's understanding that T_o is for thermal effects and loads during normal operating, startup, or shutdown conditions and T_t is for during structural integrity testing. Should "testing startup or shutdown conditions" read "testing, startup, or shutdown conditions?"

h. SSAR Appendix 3E

   (1) On page 3E.3-2, change "ASCE-7" to "ANSI/ASCE 7-88" in precipitation (for roof design).

   (2) In Figures 3E.3-4 and 3E.3-5, should the 3.5 m be from the top of pool surface instead of from the bottom of the pool?

   (3) In Section 3E.4.1.2, change the top slab diameter of 30' 21/2" to read either 30' 2½" or 30'-2.5" and the RPV pedestal inside radius of 17'-101/2" to read either 17'-10½" or 17'-10.5".

   (4) In Tables 3E.7-2 through 3E.7-10, since the square root of the sum of the squares is used for the calculation of E_{ss}, the 2's and ½'s in the equation for E_{ss} should be superscripted.

DYNAMIC LATERAL EARTH PRESSURES ON EARTH RETAINING WALLS AND EMBEDDED WALLS OF NUCLEAR POWER PLANT STRUCTURES

INTRODUCTION

In the design of earth retaining walls and embedded exterior walls of nuclear power plant structures it is important to include the loads due to seismically induced lateral earth pressures. Standard Review Plan (SRP) Section 2.5.4 which deals with the stability of subsurface materials and foundations does not provide specific review criteria regarding acceptable procedures to determine the dynamic lateral earth pressures. However, it makes a generic statement that the applicant should satisfy the requirements of applicable codes and standards in designing the structures, systems, and components (per 10 CFR Part 50.55a). In addition, this SRP section states that state-of-the-art methods are to be used to design the structures. Section 3.5.3 of ASCE Standard 4-86 (Ref. 1), which is currently being revised by ASCE, identifies certain analytical methods to be used to establish dynamic lateral soil pressures for the design of retaining walls or structures founded below grade surface (Refs. 2 & 3). These methods are based on the original analysis of this problem by Mononobe and Okabe (M-O) in the 1920s (Ref. 1).

Seed and Whitman presented a classical state-of-the-art report (Ref. 3) at an ASCE Specialty conference in 1970 on "Lateral Stresses in the Ground and Design of Earth-Retaining Structures". They presented data to show that seismic lateral pressure coefficients for cohesionless backfills computed by the M-O method agreed reasonably well with the values developed in small scale (model) tests. Subsequently, several researchers have made significant contributions to this important subject area (Refs. 4 through 8). In November 1992, the US Army Corps of Engineers acting as a consultant for the US Naval Civil Engineering Laboratory, published a comprehensive technical report (with about 30 example problems and solutions) on the seismic design of waterfront retaining structures (Ref. 9). This report (prepared with input from a team of experts in the USA and Canada) summarizes the procedures recommended for computing dynamic lateral soil pressures and grouping them according to the expected displacement of the backfill and wall during seismic events. The Department of Energy is currently (March 1993) engaged in research and development work related to the area of dynamic lateral soil pressures. This brief summary of work done in the area of lateral pressures is not, by any means, complete; however, it gives a good indication of the apparently large uncertainties that appear to be unresolved in this area.

Bechtel Power Corporation, a consultant for General Electric for the ABWR standardized design of Category I structures, has calculated the dynamic lateral earth pressures on retaining walls and embedded exterior walls of structures, using the M-O method mentioned above. Bechtel Corporation's Topical Report BC-TOP-4, Rev. 4 (Ref. 10) states that the M-O method was modified, where necessary, by procedures suggested by Wood in 1973 (Ref. 2), and by Nazarian & Hadgian in 1979. Judging from the large amount of work reported in this area after 1979 (Refs. 4-8), it appears that the procedures recommended in BC-TOP-4, Rev. 4 may not fully reflect the advances made in the state-of-the-art in this area since 1979. The objective of this brief paper is to review as many significant research papers available in the literature as possible, and comment on the appropriateness of Bechtel's calculation procedures for dynamic lateral soil pressures, for the staff guidance in the review of the Advanced Light Water Reactor Standard Design.

REVIEW OF CURRENT ANALYTICAL PROCEDURES

Mononobe and Okabe proposed a somewhat complicated equation to calculate the dynamic lateral soil pressures due to both horizontal and vertical earthquake accelerations. Their method, developed for dry cohesionless backfill materials, was essentially based on the classical Coulomb's theory of earth pressures with the following assumptions:

1) the wall yields sufficiently to produce minimum active earth pressures;

2) a soil wedge behind the wall is at the point of incipient failure and the maximum soil shear strength is mobilized along the potential sliding surface which passes through the toe of the wall; and

3) the soil wedge behind the wall acts as a rigid body so that seismic accelerations may be considered uniform throughout the mass.

Seed and Whitman (Ref. 3) state that Mononobe and Okabe apparently assumed that the total pressure computed by their analytical approach would act on the wall at the same position as the initial static pressure, i.e., at one-third the height of the wall above the base. Subsequent researchers, however, found that this assumption was not correct, and that the dynamic lateral force increment acted at about the middle height of the wall (Refs. 2 and 3). In view of the complex nature of the M-O equation that gives the total dynamic lateral pressure, Seed and Whitman also proposed a simplification of the M-O method to calculate the dynamic active lateral force increment. Kapila (Ref. 3) in 1962 described methods of determining both active and passive lateral pressures by the M-O method utilizing graphical construction.

While the M-O method was developed for yielding retaining walls, Wood (Refs. 2 and 3) found a solution for non-yielding walls using elastic theory and assuming that material properties are constant with depth. Wood's solution predicted that the dynamic lateral force increment would act at about 0.63 times the height of the wall, which corresponded approximately to a parabolic distribution of earth pressure unlike M-O's inverted triangular distribution. Wood's theoretical work was corroborated by experimental shake table tests conducted by others who found that the measured lateral pressures on non-yielding walls exceeded those predicted by the M-O method by a factor of 2 to 3 (Ref. 4). Finite element analyses in which the soil modulus increased with depth resulted in 5 percent to 15 percent smaller dynamic lateral pressures with the resultant acting closer to 0.5 times the height of the wall (Ref. 4).

According to Whitman (Ref. 4), Richards and Elms made a major advance in the area of dynamic lateral pressures by formulating a displacement-oriented solution which used the concept of allowable permanent movement of the gravity retaining walls (Ref. 5). Their approach, called the displacement-controlled method, differs from that of the M-O method which is strength-controlled. Whereas some traditional designers using the M-O method are reported to have assumed less than the maximum design earthquake, the displacement-controlled approach of Richards and Elms permits the selection of a proper design acceleration coefficient (Ref. 4). Further, their method, based on Newmark's sliding block analogy and retaining the M-O equation, permits an evaluation of permanent displacement of retaining walls following an earthquake (Ref. 6).

Based on a review of several researchers in this area, Whitman concluded that model test results have given continuing support for the use of the M-O equation for design of relatively simple walls, 30 ft or less in height; however, for higher walls and non-yielding walls, he recommends more careful analysis (Ref. 4). Regarding basement walls, Whitman, in his second state-of-the-art paper in 1991 (Ref. 6), states that the use of Wood's theory (Ref. 2) for unyielding walls may seem logical, if the basement rests directly on hard rock and if the outside walls of the basement are well-braced by floors. He further states that actual peak acceleration should be used if any yielding or cracking of the walls is to be avoided. These requirements, according to Whitman (Ref. 6), can lead to quite large lateral earth pressures.

Chang, et al. (Ref. 7) described a study which evaluated the uncertainties of several analytical solutions by comparing the computed and recorded dynamic lateral earth pressures on the embedded wall of the Lotung, Taiwan 1/4-scale model structure during several moderate earthquakes. In this study, a 1/4-scale reactor containment model structure was embedded at a depth of 4.57 meters (15 ft) below the ground surface. The analysis of recorded data showed that the magnitude of dynamic lateral earth pressures was significantly lower than that predicted by published elastic solutions (Ref. 1 and 2). The recorded dynamic lateral pressure increments were similar to, or lower than, those calculated by the M-O method. Based on the results of this study, Whitman concluded that it may suffice to use the M-O equation together with the actual expected peak acceleration (Ref. 6).

While the above conclusion may be generally true, it appears that Whitman's conclusion did not cover certain additional field data and discussions provided by Chang, et al.(Ref. 7). These relate to: (1) the effect of variation of the backfill shear modulus with depth and (2) the effect of the rocking motion on the dynamic lateral pressure distribution which were measured at the Lotung site. The soil shear modulus is generally smaller at the ground surface due to low confining pressure and gradually increases with depth, contrary to the constant modulus assumption in elastic solution. Probably due to this factor the recorded dynamic earth pressures were substantially smaller than the those given by the elastic solutions (Ref. 7). Based on a detailed study of the Lotung site data, Chang, et al. (Ref. 7) have concluded that the dynamic earth pressures acting on an embedded symmetrical structure are related primarily to soil-structure interaction (SSI) and that this phenomenon is different from that of a yielding retaining wall being acted upon by an active earth pressure. Thus the concept of limiting equilibrium used in the M-O method is not strictly applicable to the dynamic earth pressures on embedded structures.

Soydemir (Ref. 8) has also recommended caution in using the M-O method indiscriminately. He points out that the M-O method is being used without checking whether the retaining structures yield or not, and whether the conditions assumed in the M-O analysis are satisfied. Soydemir states that, even though the M-O equation for active earth pressure conditions is quite appropriate for yielding walls, it may underestimate the dynamic lateral pressure acting on rigid, non-yielding earth retaining walls, or structures.

Section 4.5 of Bechtel Topical Report BC-TOP-4A Rev. 4 states that, presently, the M-O method is used to evaluate the seismically induced lateral earth pressures in the earthquake resistant design of both the retaining walls and the embedded portions of exterior walls of nuclear power plant structures (Ref. 10). The topical report further states that, when the wall does not experience sliding or rotation, the elastic solution (Ref. 2) becomes more appropriate. In such cases, in addition to the "at rest" static pressures, all the resulting dynamic forces are to be increased by a factor of 2 for consideration of such non-yielding conditions, e.g., the embedded walls of massive structures. The report states that the value of 2 is based on the findings of Wood (Ref. 2) and also on the fact that "at rest" pressures are of the order of twice the active pressures. Since the factor 2 is for an infinitely long backfill, the Topical report says that the appropriate elastic solution can be used for shorter lengths of backfills (Ref. 10). Section 4.5 of BC-TOP-4 is silent about the seismic lateral pressures due to submerged backfill for which procedures are available in the literature (Ref. 11).

CONCLUSION/RECOMMENDATIONS

Based on a review of the several papers and reports cited above, and also based on conversations with experienced engineers working in this area at Universities, Industry, and Government agencies, it appears that the calculation procedures suggested in Bechtel Topical Report BC-TOP-4 are generally adequate for walls with shallow embedment. However, the topical report does not specifically address several factors such as the effect of depth of embedment of exterior walls of nuclear power plant structures which have embedments ranging from 40 to 85 ft in the case of advanced light water reactors.

The following recommendations, based on my literature review and discussions with professionals specializing in this field, will show that additional research is needed to develop specific acceptance criteria for the staff review of the applicant's design of embedded walls of Category I structures:

1. In determining the dynamic lateral soil pressures, it is necessary to distinguish three different types of structures each of which may require a distinct analysis/evaluation. They are: (a) gravity retaining walls and sheetpile walls, etc., with level or sloping backfill starting at the same elevation as the top of the retaining wall, (b) basement walls in buildings with the superstructure above the ground (e.g., embedded walls of nuclear power plant structures), and (c) completely buried underground structures (e.g., tunnels, underground tanks).

2. For rigid walls with shallow embedment, it seems appropriate to use the M-O method using the peak ground acceleration coefficient.

3. For deeply embedded basement walls with massive superstructure above ground which may experience rocking components of motion, and for rigid gravity walls which may undergo rotational displacements about the

vertical axis, the use of the M-O method does not seem appropriate. For such cases, the procedures recommended in BC-TOP-4, Rev. 4 need to be modified, in view of the extensive amount of more recent work done in this area. Proper consideration should be given to the actual conditions (e.g., variations of soil properties and seismic accelerations with depth, flexibility and expected deformations of embedded walls, etc.) while determining the appropriate method to calculate the lateral earth pressures, as the US Army Report (Ref. 9) has attempted to do. In such complex cases, the lateral earth pressures derived from the results of a soil-structure interaction analysis may be used in conjunction with the pressures predicted by the M-O method to determine a range of dynamic lateral pressures that could be expected to act on the embedded walls. These results may also be compared, as a check, with the lateral earth pressures that could be estimated by using the Uniform Building Code provisions for the base shear. In case an applicant wishes to use the elastic solution proposed by Wood (Ref. 2), a case-by-case justification for the factor 2 for non-yielding walls mentioned in BC-TOP-4, Rev. 4 must be provided by the applicant.

References:

1. ASCE Standard 4-86, "Seismic Analysis of Safety Related Nuclear Structures and Commentary on Standard for Seismic Analysis of Safety Related Nuclear Structures," 1986

2. Wood, J. H. "Earthquake-Induced Soil Pressures on Structures," Report No. EERL 73-05, Earthquake Engineering Research Laboratory, California Institute of Technology, Pasadena, CA, Aug. 1973

3. Seed, H.B., and Whitman, R.V., "Design of Earth Retaining Structures for Dynamic Loads," a state-of-the-art paper presented at the ASCE Specialty Conference on "Lateral Stresses in the ground and Design of Earth Retaining Structures," Cornell University, Ithaca, NY, 1970.

4. Whitman, R. V., "Seismic Design and Behavior of Gravity Retaining Walls," Proc. of ASCE Conf. on Design and Performance of Earth Retaining Structures," Cornell Univ., Ithaca, NY, 1990.

5. Richards, R. Jr., and Elms, D. G., "Seismic Behavior of Gravity Retaining Walls," ASCE Journal, GT Division, Vol. 105, April 1979

6. Whitman, R.V., "Seismic Design of Earth Retaining Structures," a state-of-the-art paper presented at Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, MO, March 11-15, 1991.

7. Chang, C.Y., Power, M.S., Mok, C.M., Tang, Y.K., and Tang, H.T., "Analysis of Dynamic Lateral Earth Pressures Recorded on Lotung Reactor Containment Model Structure," 4th U.S. National Conference on Earthquake Engineering, Palm Springs, CA, Vol. 3, May 20-24, 1990.

8. Soydemir, C., "Seismic Design of Rigid Underground Walls in New England," 2nd International Conf. on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, MO, March 11-15, 1991.

9. Ebeling, E. M. and Morrison, Jr., E. E., "The Seismic Design of Waterfront Retaining Structures," US Army Technical Report ITL-92-11, November 1992

10. Bechtel Power Corporation Topical Report No. BC-TOP-4A, Rev. 4, "Seismic Analysis of Structures and Equipment for Nuclear Power Plants."

11. Matsuzawa, H., Ishibashi, I, and Kawamura, M., "Dynamic Soil and Water Pressures on Submerged Soils," ASCE Journal of Geotechnical Engineering, Vol. 111, No. 10, October 1985

STAFF POSITION ON THE USE OF STANDARD
ANSI/AISC N690
NUCLEAR FACILITIES-STEEL SAFETY RELATED STRUCTURES

The use of the Standard ANSI/AISC N690 (1984 Edition) for the design, fabrication and erection of safety related structures in ABWR and ABB-CE System 80+ is acceptable when supplemented by the following provisions:

1. In Section Q1.0.2, the definition of secondary stress should apply to stresses developed by temperature loading only.

2. Add the following notes to Section Q1.3.6:

   "When any load reduces the effects of other loads, the corresponding coefficient for that load should be taken as 0.9, if it can be demonstrated that the load is always present or occurs simultaneously with other loads. Otherwise, the coefficient for that load should be taken as zero."

   "Where the structural effects of differential settlement are present, they should be included with the dead load 'D'."

   "For structures or structural components subjected to hydrodynamic loads resulting from LOCA and/or SRV actuation, the consideration of such loads should be as indicated in the Appendix to SRP Section 3.8.1. Any fluid structure interaction associated with these hydrodynamic loads and those from the postulated earthquake(s) should be taken into account."

3. The stress limit coefficients (SLC) for compression in Table Q1.5.7.1 should be as follows:

   1.3 instead of 1.5 [stated in footnote (c)] in load combinations 2, 5 and 6.

   1.4 instead of 1.6 in load combinations 7, 8, and 9.

   1.6 instead of 1.7 in load combination 11.

4. Add the following note to Section Q1.5.8:

   "For constrained (rotation and/or displacement) members supporting safety related structures, systems or components the stresses under load combinations 9, 10 and 11 should be limited to those allowed in Table Q1.5.7.1 as modified by provision 3 above. Ductility factors of Table Q1.5.8.1 (or provision 5 below) should not be used in these cases."

5. For ductility factors 'μ' in Sections Q1.5.7.2 and Q1.5.8, substitute provisions of Appendix A, II.2 of SRP Section 3.5.3 in lieu of Table Q1.5.8.1.

6. In load combination 9 of Section Q2.1, the load factor applied to load P_a should be $1.5/1.1 \approx 1.37$, instead of 1.25.

7. Sections Q1.24 and Q1.25.10 should be supplemented with the following requirements regarding painting of structural steel:

   - Shop painting to be in accordance with Section M3 of Reference 1.

   - All exposed areas after installation to be field painted (or coated) in accordance with the applicable portion of Section M3 of Reference 1.

   - The quality assurance requirements for painting (or coating) of structural steel to be in accordance with Reference 2 as endorsed by Regulatory Guide 1.54, "Quality Assurance Requirements for Protective Coatings Applied to Water Cooled Nuclear Power Plants."

References:

1. Load and Resistance Factor Design Specification for Structural Steel Buildings and Its Commentary, Published by AISC, Chicago, Sept. 1, 1986.

2. ANSI N101.4, "Quality Assurance for Protective Coatings Applied to Nuclear Facilities," American Institute for Chemical Engineers, New York, N.Y.

TECHNICAL BASIS FOR INTERIM STAFF POSITION
ON THE USE OF ANSI/AISC N690 STANDARD

1. The Standard defines the "secondary stress" as: "any normal stress or shear stress developed by the constraint of adjacent material or by self-constraint of the structure. The basic characteristic of a secondary stress is that it is self limiting due to deformation-limited effects." This definition has been interpreted by some to be applicable to the stresses generated by mechanical (i.e., non-thermal) loads at the structural discontinuities. The position clarifies the staff's interpretation.

2. These notes provide guidance to the users regarding consideration of additional load effects in designing the steel structures. The notes are parts of SRP Sections 3.8.3 and 3.8.4.

3. The research done in the last twelve years on the strength and stability of compression members indicates that the base curve (SSRC curve in Figure 1) used in arriving at the stress limit coefficients (SLCs) in SRP Sections 3.8.3, 3.8.4, and in the Standard does not reflect the results of the available test data. In developing the AISC Building Specification based on Load and Resistance Factor Design (LRFD) concept, the AISC changed the formulation for compression members to reflect the results of the test data. The LRFD curve (with ϕ=1.0) is also shown in Figure 1. Based on the test-data, this curve has the minimum reliability index, β^* of 2.6 (Ref. 1,. The LRFD specification requires ϕ=0.85 in establishing the resistance of compression members.

   Figure 1 shows the curves reflecting the SLCs of 1.0, 1.4, 1.5, 1.6 and 1.7 as applied to the stresses specified for allowable stress design (ASD) of the AISC. Based on the comparison with the LRFD curve (ϕ=1.0), the following SLCs are recommended in the interim position:

   SLC of 1.6 ($\phi \approx 0.95$) for load combinations 10 and 11. This is reasonable for load combinations containing the effects of the two low probability events, i.e., SSE+LOCA.

   SLC of 1.4 ($\phi \approx 0.84$) for load combinations 7, 8 and 9. This is appropriate for combinations containing the effects of the single low probability events, i.e, SSE, Tornado or LOCA.

   SLC of 1.3 in load combinations 2, 5 and 6 is recommended when the secondary stresses due to T_o are included in the load combinations. This is consistent with the current position of allowing higher stresses under the effects of operating temperature.

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* β is defined as a ratio of $\ln (R_m/Q_m)$ to $(V_R^2+V_Q^2)^{1/2}$; where--
R_m = median value of resistance, Q_m= median value of load;
and, V_R, V_Q are the corresponding coefficients of variation.

4. Neither the SRPs nor the Standard provide any guidance regarding the tolerable deformation of the constrained steel members when they are subjected to temperature growth under sustained T_a or other LOCA loads. Statistically meaningful test data simulating the inelastic behavior of such constrained members under representative load combinations (including T_a and E_s) are not available. This provision ensures against the instability condition arising from the effects of T_a or other LOCA loads under load combinations 9, 10 and 11.

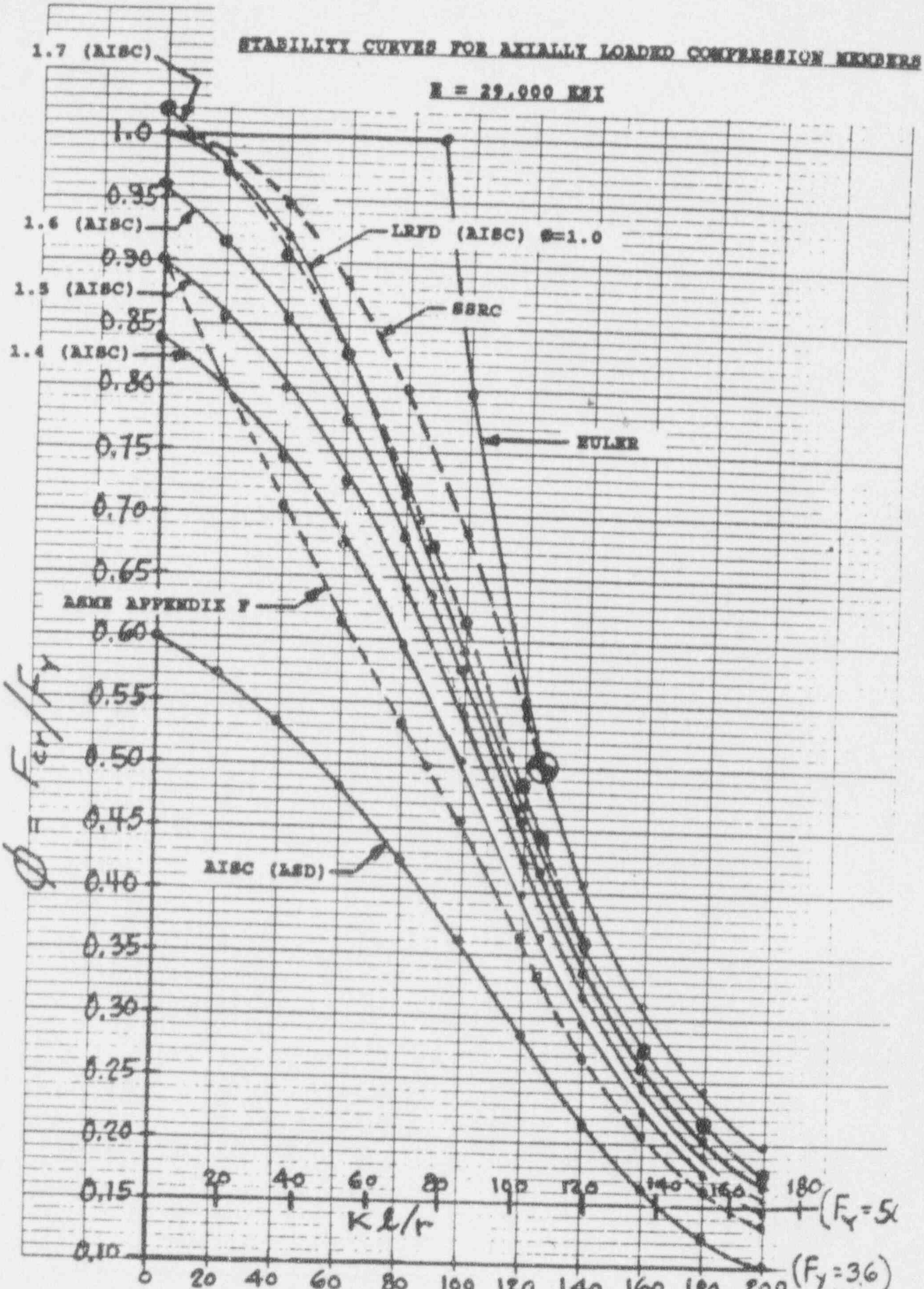
5. The ductility factors provided in Table Q1.5.8.1 are either more liberal than those in the SRP Appendix A of SRP Section 3.5.3(e.g., μ for compression members), or involve some inconsistencies in definitions and interpretation of the formulas (e.g., formulas in 2.D of the Table) given in the Table. Therefore, until sufficient test based justification for ductility factors listed in Table Q1.5.8.1 is provided, the staff position as stated in the Appendix is recommended for use.

6. This provision makes the load combination consistent with that in the SRPs.

7. Additional provision regarding painting of structural steel is provided.

Reference:

1. Load and Resistance Factor Design Specification for Structural Steel Buildings and Its Commentary, Published by AISC, Chicago, September 1, 1986.


STABILITY CURVES FOR AXIALLY LOADED COMPRESSION MEMBERS
E = 29,000 KSI
1.7 (AISC)
1.6 (AISC)
1.5 (AISC)
1.4 (AISC)
LRFD (AISC) Φ=1.0
SSRC
EULER
ASME APPENDIX F
AISC (ASD)
Φ = Fcr/Fy
Kℓ/r
(Fy = 50)
(Fy = 36 KSI)
1.0
0.95
0.90
0.85
0.80
0.75
0.70
0.65
0.60
0.55
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0 20 40 60 80 100 120 140 160 180 200
20 40 60 80 100 120 140 160 180

STAFF POSITION
ON
STEEL EMBEDMENTS

A. INTRODUCTION

General Electric Company (GE) in its Standard Safety Analysis Report (SSAR) proposed the use of the ACI 349 Code for the design of seismic Category I structures for the Advanced Boiling Water Reactor facilities. The staff had reviewed the Appendix B to the ACI 349 (up to 1985 Edition), "Code Requirements For Nuclear Safety Related Concrete Structures," and test data for anchor bolts both from United States and foreign countries. Based on the above review, the staff has taken exceptions to Appendix B to the ACI 349 Code as stated below. This proposed position was developed as an aid for the review of the advanced reactors. The staff's fundamental concerns regarding Appendix B to the ACI 349 Code are discussed below and exceptions to the use of Appendix B are indicated.

The staff's fundamental problem with the Appendix B to the ACI 349 Code is the use of a basic assumption of the 45 degree concrete failure cone. The choice of this assumption might have resulted from convenience. However, this assumption has been proven to be wrong by tests even for single anchors. The problem become greater (less conservative) when an anchor is located near the free edge of the concrete or anchors are closely spaced.

Appendix B to the ACI 349 Code is deficient in that the code has no provisions for anchor strength reduction when the anchor is located in cracked concrete, such as in the tension zone of a concrete slab. The Uniform Building Code has provisions for anchor strength reduction when an anchor is located in the tension zone.

B. EXCEPTIONS TO THE APPENDIX B TO THE ACI 349 CODE

1. Section B.4.2 - Tension and Figs. B.4.1 and B.4.2

This section and the figures specify that the tensile strength of concrete for any anchorage can be calculated by a 45 degree failure cone theory. The staff has disseminated the German test data questioning the validity of the 45 degree failure cone theory to licensees, A/Es, bolt manufacturers, and the code committee members in its meetings with them. The data indicated that the actual failure cone was about 35 degree and the use of the 45 degree cone theory could be unconservative for anchorage design, especially for anchorage of groups of bolts. The Code Committee, having gone through some research of its own, recently agreed with the staff's position. Changes to this section are in the making by the Code Committee. In the meantime, the staff position on issues related to this Section is to ensure adoption of design approaches consistent with the test data through case by case review.

2. B.5.1.1 - Tension

This section states a criterion for ductile anchors. The criterion is that the design pullout strength (force) of the concrete as determined in Section B.4.2 exceeds the minimum specified tensile strength (force) of the steel anchor. Any anchor that meets this criterion is qualified as a ductile anchor and, thus, a low safety factor can be used. The staff believes that the criterion is deficient in two areas. One is that the design pullout strength of the concrete so calculated is usually higher than the actual strength, which has been stated in Section B.4.2 above. The other is that anchor steel characteristics are not taken into consideration. For example, the Drillco Maxi-Bolt Devices, Ltd. claims that its anchors are ductile anchors and, thus, can use a low safety factor. The strength of the Maxi-Bolt is based on the yield strength of the anchor steel, which is 105 ksi. The embedment length of the anchor, which is used to determine the pullout strength of the concrete, is based on the minimum specified tensile strength of the anchor steel of 125 ksi. The staff believes that the 19-percent margin (125/105) for the embedment length calculation is insufficient considering the variability of parameters affecting the concrete cone strength. The staff also questions the energy absorption capability (deformation capability after yield) of such a high strength anchor steel. Therefore, in addition to the position taken with regard to Section B.4.2 above, the staff will review vendor or manufacturer specific anchor bolt behaviors to determine the acceptable design margins between anchor bolt strengths and their corresponding pullout strengths based on concrete cones.

B.5.1.1(a) - Lateral bursting concrete strength

This section states that the lateral bursting concrete strength is determined by the 45 degree concrete failure cone assumption. Since this assumption is wrong and likely to be replaced as stated before, the staff believes that the lateral bursting concrete strength determination is also wrong and needs to be replaced. The staff will review the lateral bursting concrete strength provided by the concrete cover around anchor bolts and lateral bursting force created by the pulling of anchor bolts against test data to determine if adequate reinforcement against lateral bursting force needs to be provided on a case by case basis.

3. B.5.1.2.1 - Anchor, Studs, or Bars

This section states that the concrete resistance for shear can be determined by a 45 degree half-cone to the concrete free surface from the centerline of the anchor at the shearing surface. Since the 45 degree concrete failure cone for tension has been found to be incorrect, the staff believes that the use of the 45 degree half-cone for shear should be re-examined. In the meantime, the staff will review the adequacy of shear capacity calculation of concrete cones on a case by case basis with emphasis on methodology verification through vendor specific test data.

4. B.5.1.2.2(c) - Shear Lugs

This section states that the concrete resistance for each shear lug in the direction of a free edge shall be determined based on the 45 degree half-cone assumption to the concrete free surface from the bearing edge of the shear lug. This is the same assumption as used in Section B.5.1.2.1 and the staff has the same comment as stated in that section. Therefore, the staff position related to the design of shear lugs is to perform case-by-case reviews. The staff review will emphasize methodology verification through specific test data.

5. B.7.2 - Alternative design requirements for expansion anchors

This section states that the design strength of expansion anchors shall be 0.33 times the average tension and shear test failure loads, which provides a safety factor of 3 against anchor failure. The staff position on safety factor for design against anchor failure is 4 for wedge anchors and 5 for shell anchors unless a lower safety factor can be supported by vendor specific test data.

6. Anchors in tension zone of supporting concrete

When anchors are located within a tensile zone of supporting concrete, the anchor capacity reduction due to concrete cracking shall be accounted for in the anchor design.