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October 24, 1985
ST-HL-AE-1428
File No.: G9.17

Mr. George W. Knighton, Chief
Licensing Branch No. 3
Division of Licensing
U. S. Nuclear Regulatory Commission
Washington, DC 20555

South Texas Project
Units 1 and 2
Docket Nos. STN 50-498, STN 50-499
Responses to NRC Question
Questions 220.04N, 220.08N, 220.13N, 220.25N, and 220.30N

Dear Mr. Knighton:

The attachments enclosed provide STP's response to Draft Safety Evaluation Report (DSER) or Final Safety Analysis Report (FSAR) items.

The item numbers listed below correspond to those assigned on STP's internal list of items for completion which includes open and confirmatory DSER items, STP FSAR open items and open NRC questions. This list was given to your Mr. N. Prasad Kadambi on October 8, 1985 by our Mr. M. E. Powell.

The attachments include mark-ups of FSAR pages which will be incorporated in a future FSAR amendment unless otherwise noted below.

The items which are attached to this letter are:

Attachment	Item No.*	Subject
1	Q220.04N, Q220.08N, Q220.09N, Q220.13N, Q220.25N and Q220.30N	Revised response to NRC Structural Engineering Branch Questions

Please note that only Q220.08N, Q220.13N and Q220.030N appear on the list identified above.

* Legend
D - DSER Open Item C - DSER Confirmatory Item
F - FSAR Open Item Q - FSAR Question Response Item

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If you should have any questions concerning this matter, please contact Mr. Powell at (713) 993-1328.

Very truly yours,

M. R. Wisenburg
Manager, Nuclear Licensing

MEP/bl

Attachments: See above

L1/DSER/ak

cc:

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Revised 9/25/85

L1/DSER/ak

Question 220.04N

For concrete barriers against turbine generated missile, the modified Petry formula is used. Compare the penetration depths using modified NDRC formula and make sure that you have provided barriers having equivalent conservatism to that required by the modified NRC formula. A copy of the revised SRP Section 3.5.3 covering the subject (Attachment 1) is provided for your information.

Response

~~The modified Petry formula is not utilized for concrete barriers against turbine generated missiles. For concrete barriers against turbine generated missiles, the Chang SAF formula for scabbing and the CEA EDF formulation for perforation are used. The results calculated using these formulations provide conservative (and consistent) results. (See "NRC Review of Impact Damage," Seminar on Turbine Missile Effects in Nuclear Power Plants," October, 1982, published by EPRI.) A comparison of the predications of the BRL, Chang SAF, and CEA-EDF is also made in the EPRI publications. A comparison of scabbing and perforation thickness using modified NDRC and the Chang Formula (SAF) is presented in "Impact of Solid Missiles on Concrete Barriers" by Wen S. Chang, ASCE Journal of the Structural Division, February, 1981.~~ 36

See revised response - Insert A

Insert A

The modified Petry formula is not utilized in the STP to establish the thicknesses or probability of failure of concrete barriers against turbine-generated missiles. In the STP the Chang Semi-Analytical Formula (SAF) for scabbing and the CEA-EDF Formula for perforation were used for the evaluation of barriers against the turbine-generated missiles. As explained in the following paragraphs, that approach is consistent with the intent of the SRP Section 3.5.3, II.1.a which states that "For turbine missile barriers, penetration and scabbing predictions should be based on empirical equations such as the modified NDRC formula or the results of a valid test program".

The Chang SAF for predicting scabbing resistance is based upon the principles of engineering mechanics with coefficients which are determined from test data. Reference (1) provides detailed information regarding the derivation of the formula and the determination of the coefficients used with the formula. The accuracy of the CEA-EDF formulations for perforation resistance are evaluated in reference (2).

The test data used in reference (1) were the only suitable data available at the time. These data were obtained based on cylindrical steel missiles impacting on concrete barriers. Later, full scale and reduced scale turbine missile tests were conducted by Sandia Laboratories and SRI International, respectively. Results from these tests, as well as data from tests conducted in France were utilized in an indepenendent assessment of the accuracy of the existing formulas. The results were reported in Reference (2) where it is concluded that the Chang SAF and the CEA-EDF formulas provide accurate predictions of scabbing and perforation, and that any deviation from the tests results are on the conservative side.

Recently the authors of Reference (2) investigated the damage probability of turbine missile impact as reported in Reference (3) and selected the Chang SAF as the most applicable formula for concrete scabbing prediction and quantified the conservatism contained in the SAF.

For information purposes the STP has performed supplementary analyses to determine the probability of damage to concrete barriers of the entire plant using the modified NDRC formulas for scabbing and perforation predictions. The resulting critical probability from those analyses would be 1.26×10^{-7}, if the governing concrete walls and roof slabs of the Diesel Generator Building were considered to be 29 inches instead of the actual 24 inches.

Insert A (continued)

ATTACHMENT 1
ST-HL-AE-1428
PAGE 3 OF 23

This recalculated probability is introduced simply as a reference value to assess implications of using the alternate NDRC formula which has been demonstrated by tests to be ~~unnecessarily~~ conservative. The critical probability using the Chang SAF and the CEA-EDF formulations as reported in the FSAR Section 3.5, Table 3.5-8 is 0.83×10^{-7}. This probability value is maintained as the design-basis for the STP since the value ~~satisfied the prescribed limit of 1.0×10^{-7} and~~ is determined on a sound analytical basis through accepted formulations proven to be reliable by observed test results, and it satisfies the prescribed limit of 1.0×10^{-7}.

References

1. Chang, W. S., "Impact of Solid Missiles on Concrete Barriers", Journal of the Structural Division, ASCE, February 1981.

2. Wolde-Tinsae, A. M., et al., "NRC Review of Impact Damage", Proceedings of Seminar on Turbine Missile Effects in Nuclear Power Plants, published by Electrical Power Research Institute, Palo Alto, Calif., October 1982.

3. Gopalakrishna, H. S. and Wolde-Tinsae, A. M., "Damage Probability of Turbine Missile Impact", Journal of the Structural Division, ASCE, December 1984.

NOTES TO TABLE Q220.08N-1

1) Frequency above 4 Hz, out of range - No effect

2) FEM spectra envelopes the EHS spectra for MEAB, where equipment is located - No effect 30

3) Generic design is based on seismic acceleration levels in the range of peak amplification, which is not increased by EHS spectra - No effect.

4) Enough margin in existing design - No effect

5) Enough margin was found in the existing support embedment - No effect; ~~Some structural members in the bridge truss appear to be marginal and hence may need to be reinforced. A confirmatory analysis appears to be warranted~~.

See Insert B

A definitive analysis has confirmed that the members are adequate,

Insert B

~~The preliminary structural evaluation suggested that some members in the bridge truss could be marginal and hence may need to be reinforced. A definitive confirmatory analysis has been completed by Bechtel. The analysis established that the members are adequate,~~ and that the structural integrity of the equipment is not compromised by the EHS augmented seismic response spectra.

Question 220.09N

State what kind of maximum relative displacement you expect due to earthquake and other applicable loads among supports of Category I structures, systems and components, and what considerations have been given in this respect. Confirm that the staff position stipulated in SRP Section 3.7.3 is fully complied with.

Response

Maximum relative displacements due to earthquake and settlement among principal power block structures are as follows:

	Max. Relative Displacement (in)		
Interface	OBE	SSE	Long Term Diff. Settlement *
RCB/MEAB	0.14	0.23	~~0.2 - 1.0~~ 1.0
RCB/FHB	0.22 **	0.44 **	~~0.2 - 0.4~~ 1.0
DGB/MEAB	~~0.03~~ 0.17	~~0.06~~ 0.34	~~0.0 - 0.5~~ 1.0
FHB/MEAB	0.16	0.28	1.0

30

RCB: Reactor Containment Building
FHB: Fuel Handling Building
MEAB: Mechanical-Electrical Auxiliary Building
DGB: Diesel Generator Building
OBE: Operating Basis Earthquake
SSE: Safe Shutdown Earthquake

As stated in the Sections 3.7.3.8 and 3.7.3.9, the effect of maximum relative displacements is included in the analysis of systems which interconnect structures.

For piping systems, the analysis is in accordance with the requirements of the ASME Boiler and Pressure Vessel Code, Section III. The procedure used is in compliance with SRP Section 3.7.3.

Insert C

Insert C for Page Q&R 3.7-38N

*These values represent the established design criteria for differential movement. The values are derived from differential settlement projections, and are subject to ongoing monitoring to assure consistency with the periodically measured settlements of controlled locations. To date, the actual, measured settlements agree with the predicted settlements. The differential settlement criteria is discussed at length in Section 2.5.4.11 and predicted and actual differential settlement values are reported n Appendix 2.5.C.

**These design-basis values are slightly lower than the corresponding values obtained from the confirmatory single-step finite element seismic analysis. However, it has been determined that the difference in relative displacements from the analytical techniques does not affect the seismic design of interconnecting piping anchored in the FHB and RCB.

Question 220.13N

To account for the effect of accidental torsion, NRC staff's position requires that an additional eccentricity of 5 percent of the maximum building dimension at the level under consideration shall be assumed over the actual geometrical eccentricity of Category I structures. Copy of revised SRP 3.7.2 (Attachment 2) is provided for your reference. Confirm that this staff position is fully complied with in your Category I structural design and analysis.

Response

geometric was initial

The additional 5 percent eccentricity was not included in the initial design, only ~~but~~ the actual eccentricity between the center of mass and the center of rigidity ~~has been~~ considered in the ~~current~~ seismic analyses. ~~The design of each structure will be reevaluated to include the additional 5 percent eccentricity to determine if the existing design meets the recommended staff position.~~ Insert D

Insert D

Subsequent analyses for Category I structures have been performed to account for the effect of accidental torsion in accordance with the NRC position. The results of the confirmatory analyses summarized in Table 220.13N-1 indicate that the existing design of all Category I structures is adequate.

TABLE 220.13N-1

Results of the Confirmatory Analyses

Building/Structure Description of Key Shear Walls	Calculated Shear Force, k/ft., 5% Accidental Torsion Excluded	Calculated Shear Force, k/ft., 5% Accidental Torsion Included	Allowable, Shear Load, k/ft. Concrete Alone, No Reinforcing	Reinforcement For In-Plane Shear Sq.-in./ft. Required Vertical	Required Horizontal	Available Vertical	Available Horizontal
Reactor Containment Building (RCB)							
Primary Shield Walls							
Elev. (-) 13'-3 to 12'-1	128	130	290	None	None	-	-
12'-1 to 19'-0	246	249	422	None	None	-	-
19'-0 to 38'-6 1/2	332	334	390	None	None	-	-
Secondary Shield Walls							
Southwest, Elev. (-) 5'-3	122	124	125	None	None	2.6	7.6
North, Elev. (-) 5'-3	443	445	179	4.42	4.42	5.2	7.6
Containment Shell							
Elev. (-) 13'-3 (basemat)	38	40	138	None	None	2.1	1.7
Elev. 38'-9	31	33	0	None	0.84	2.8	0.90
Elev. 74'-9	25	27	0	None	0.44	2.8	0.90
Elev. 133'-0 (Springline)	8.8	9.7	151	None	None	2.8	1.5

TABLE 220.13N-1

Results of the Confirmatory Analyses

Building/Structure Description of Key Shear Walls	Calculated Shear Force, k/ft., 5% Accidental Torsion Excluded	Calculated Shear Force, k/ft., 5% Accidental Torsion Included	Allowable, Shear Load, k/ft. Concrete Alone, No Reinforcing	Reinforcement For In-Plane Shear Sq.-in./ft. Required	Reinforcement For In-Plane Shear Sq.-in./ft. Provided
Mechanical Electrical Auxiliary Building (MEAB)					
East Exterior Wall Elev. 10'-0	72	78	86	1.03 (Min.)	1.60
North Exterior Wall Elev. 35'-0	59	66	75	0.90 (Min.)	1.60
East Exterior Wall Elev. 60'-0	22	24	75	0.90 (Min.)	1.60
East Exterior Wall Elev. 80'-0	19	21	75	0.90 (Min.)	1.60
Fuel Handling Building (FHB)					
East Exterior Wall Elev. (-) 29'-0 to 4'-0	135	153	85	1.30	2.40
North Exterior Wall Elev. (-) 29'-0 to 4'-0	184	198	46	2.90	3.10
South Exterior Wall Elev. 21'-11 to 30'-0	124	130	62	1.30	1.70
South Exterior Wall Elev. 4'0- to 21'-11	140	142	62	1.60	1.70
South Exterior Wall Elev. 68'-0 to 119'-0	114	119	46	1.40	1.60

ATTACHMENT I
ST-HL-AE-1428
PAGE 9 OF 23

TABLE 220.13N-1

Results of the Confirmatory Analyses

Building/Structure Description of Key Shear Walls	Calculated Shear Force, k/ft., 5% Accidental Torsion Excluded	Calculated Shear Force, k/ft., 5% Accidental Torsion Included	Allowable, Shear Load, k/ft. Concrete Alone, No Reinforcing	Reinforcement For In-Plane Shear Sq.-in./ft. Required	Reinforcement For In-Plane Shear Sq.-in./ft. Provided
Diesel Generator Building (DGB)					
West Exterior Wall Elev. 25'-0 to 55'-0	30	35	58	0.72 (Min.)	1.60
South Exterior Wall Elev. 25'-0 to 55'-0	80	81	55	0.72 (Min.)	1.60
Essential Cooling Water Intake Structure (EWIS)					
East Exterior Wall Elev. 10'-0	37	42	31	0.72 (Min.)	1.20
West Exterior Wall Elev. 34'-0	47	54	46	1.08 (Min.)	1.20
North or South Exterior Wall Elev. 34'-0	41	47	46	1.08 (Min.)	1.20
Interior Wall East to West Elev. 10'-0	32	33	31	1.08 (Min.)	1.20

ATTACHMENT I
ST-HL-AE-1428
PAGE 10 OF 23

Question 220.25N

State if the concrete is assumed to be cracked under any load combination involving axisymmetric and non-axisymmetric loadings. If so, by what method have you considered the cracking and the basis thereof?

Response

~~Concrete is assumed to be cracked whenever tensile stresses are present. The cracked section analysis was performed on a section-by-section basis after the section design forces were obtained from a linear elastic analysis.~~

~~In cracked section investigation, the thermal effect may be reduced considering the actual state of stress and the compatibility conditions. This approach is consistent with Standard Review Plan (SRP) Section 3.8.1.II.4.d. This is discussed in revised Section 3.8.1.4.~~ 40

Insert E →

Insert E

The structural analyses for the determination of design moments, forces, and shears under all loads are performed on the basis of linear elastic analysis. Non-linear analyses involving iterative processes to account for concrete cracking are not used under any load combination involving axisymmetric or non-axisymmetric loadings. The cracking of concrete is considered in the design of the individual concrete sections, for which the amount of reinforcing steel is provided without relying on the concrete to resist any tension. For the design of reinforced concrete sections under thermal loading, the state of stress under non-thermal loads is determined first, and then, if necessary, reductions in thermal stresses are calculated based on concrete cracking, reinforcement yielding (within allowable limits), compatibility of state of stress and strain, and boundary conditions. The foregoing reductions of thermal stresses operate on the design loads calculated by linear elastic analyses, and do not represent non-linear iterative analyses devised to account for concrete cracking.

Question 220.30N

The staff presently accepts the use of ACI-349 as augmented by Regulatory Guide 1.142 in the design of Category I concrete structures other than containment. FSAR Sections 3.8.3, 3.8.4, and 3.8.5 have mentioned the use of ACI-318 Code for Concrete Structure. Evaluate and assess the impact of using ACI-349 as augmented by Regulatory Guide 1.142. Identify specific deviations from the staff position and the areas where use of ACI-318 Code results in less conservative design. Also discuss specific means for disposition of these less conservative design areas or justify their design adequacy.

Response

Insert F

The only significant ~~differences~~ difference between the ACI-318 and ACI-349 Codes ~~are:~~

~~1) Provisions regarding quality assurance (QA).~~
~~2) Loading combinations.~~
~~3) Provisions of Appendices A, B, and C of the ACI-349 Code (these appendices are not included in the ACI-318 Code).~~

With regard to Item 1, STP criteria require compliance with the applicable QA requirements including 10CFR50 Appendix B which is referenced in ACI-349, and no discrepancy arises with respect to the ACI-349 Code.

~~With regard to Item 2, STP loading combinations comply with Standard Review Plan (SRP) requirements which are the same as ACI-349 loading combinations as modified by RG 1.142.~~

~~STP criteria regarding thermal considerations (ACI-349, Appendix A), and impulsive and impactive effects (ACI-349, Appendix C) are either the same as those included in the Code or more conservative.~~

~~Finally, STP embed design criteria will be reviewed for compliance with the ACI-349, Appendix B provisions. Where the criteria differ from those of the Code, adequate justification will be provided for the existing design.~~

~~In summary, the STP design bases (using the ACI-318 Code as supplemented by other project criteria and upon completion of Appendix B compliance review) are equivalent to full compliance with the ACI 349 Code.~~

→ Insert G

Insert F

is in the load combination equations. The STP load combinations comply with the Standard Review Plan (SRP) requirements, which are the same as the ACI-349 loading combinations as modified by R. G. 1.142. Therefore, the STP structural design satisfies the current NRC acceptance criteria.

Other differences are:

(1) Provisions regarding quality assurance (QA)

(2) Provisions of Appendix A, B, and C of the ACI-349 Code (these appendices are not included in the ACI-318 Code).

Insert G (1 of 11)

With regard to item (2), the STP criteria for thermal considerations and for impulsive and impactive effects are the same as, or more conservative tnan those prescribed in the Code in Appendix A and Appendix C, respectively.

With regard to Appendix B of the Code, the STP design criteria differs from the Code provisions in the following respects:

(a) For the welded anchor studs of standard embedded plates used for miscellaneous supports, and for ductile-type undercut expansion anchors (Drillco Maxibolts), the interaction equation prescribed by the STP criteria for combined tension and shear is:

$$\left(\frac{t}{T}\right)^{5/3} + \left(\frac{s}{S}\right)^{5/3} \leq 1.0$$

instead of the linear equation implied by the Code (subsection B.6.3.2):

$$\frac{t}{T} + \frac{s}{S} \leq 1.0$$

where:

t, s = design tension load and shear load, respectively

T, S = allowable tension load and shear load, respectively (allowable loads based on ultimate loads)

(b) For grouted rock-bolts (Williams), the interaction equation prescribed by the STP criteria for combined tension and shear is:

$$\left(\frac{t}{T}\right)^2 + \left(\frac{s}{S}\right)^2 \leq 1.0$$

instead of the linear equation implied by the Code.

(c) Anchor bolts for certain applications are allowed to be provided with embedment lengths that result in ultimate load capacities that satisfy the required design load with the prescribed load factors, but do not necessarily satisfy the generic Code provision to develop the full tensile strength of the steel bolts, implied in Subsection B.4.2.

Discussion

Items (a) & (b)

The foregoing interaction equations (with 5/3 and 2.0 exponents) are allowed by the STP criteria only for the cases where the tension and shear ultimate loads of the studs/bolts represent ductile behavior governed by the steel material strength. For the Maxibolts, the hole drilled into the concrete is undercut (conically enlarged in diameter at its base) in order to provide a positive mechanical anchorage for the expanded head of the bolt. This positive anchorage, plus the prescribed deep embedment and wide separation between bolts, preclude slippage and/or concrete cone failure so that the full strength of the steel bolt is invariably developed as demonstrated by tests. Similarly, for the rock-bolts, the combination of an effective head expanded by torquing upon initial installation, followed by grouting by injection of a high-strength non-shrink mix through an axial hole in the bolt, plus the prescribed deep embedment, assures the development of the full strength of the steel bolt. That means that the concrete ultimate load capacities, which are calculated in accordance with the ACI-349 Code for the above bolts with a specific embedment and spacing, exceed and fully develop the steel material ultimate load of the bolts. Therefore, in these cases the relevant interaction mechanism is that applicable to steel bolts as opposed to the interaction associated with concrete anchorage or cone failure represented by the linear interaction. For steel studs in concrete the ultimate load capacities and interaction behavior have been extensively evaluated by tests as reported in References (1) and (3), where the interaction equation with 5/3 exponents is recommended. For steel bolts the interaction behavior recognized by the AISC in Reference (2) is defined by an elliptical relationship which is equivalent to the interaction equation with an exponent of 2.0. It is noted that the interaction equation with exponents of 2.0 (AISC) is the upper bound analytical expression derived from tests, and it envelops the more conservative equation with 5/3 exponents.

The foregoing approach, whereby the implied linear interaction is recognized for the design of anchor studs/bolts which are proportioned to fully develop the steel material strength so that slippage and/or concrete cone failure do not govern, is also mentioned in Reference (3). In this reference paper the elliptical shear/tension interaction is recognized as valid, but it is conditionally recommended for the reassessment of existing designs rather than

for generic use in new designs. This is actually the case for the STP since the designs affected by the elliptical interaction equations are mostly the earlier designs based on the original STP criteria established prior to the ACI-349 Code. The subsequent new designs for embedded plate anchors performed by Bechtel are in accordance with the Code.

For the cases of anchor studs/bolts where the concrete ultimate load capacity governs because of allowed reductions in embedment and/or spacing, the STP criteria reverts to the linear interaction equation implied by the ACI-349 Code.

Therefore, based on the foregoing clarifications and on consideration of Reference (3), it is regarded that the interaction equations as prescribed by the STP criteria are adequate to assure the structural integrity of the anchor studs/bolts under combined tension and shear, and are consistent with an interpretation of the Code supported by the ASCE paper of Reference (3).

References

(1) Design Data 10 - Embedment properties of headed studs by TRW, Nelson Division, 1977. (Refer to sections 1.0 and 6.0, and references cited therein)

(2) Commentary on the specification for the design, fabrication and erection of structural steel for buildings, AISC, November 1, 1978. (Refer to subsection 1.6.3)

(3) State-of-the-art report on steel embedments, by ASCE Nuclear Structures and Materials Committee, June 1984. (Refer to subsections 3.3.3.2 and 4.1.2.3)

Item (c)

In some instances, the anchor bolt size (diameter) provided for equipment mounting is based on the bolt hole size specified in the equipment manufacturer drawings. The resultant bolt size is verified by the STP engineer to be adequate for the calculated design loads, and the bolt anchorage into the concrete (as governed by the bolt embedment, spacing and head or anchor plate at the end of the bolt) is designed to satisfy the calculated design loads. Often in these cases the bolt size as derived from the manufacturer's standardized drawings is actually oversized with respect to the calculated loads. Therefore, it is not ~~meaningful~~ necessary to extend the overdesign into the bolt anchorage by attempting to fully develop the ultimate tensile strength of bolts whose function does not demand loads close to the ultimate load range. In these cases of oversized bolts, it is considered sufficient to design the bolt anchorage (using the ACI-349 Code formulations) to develop the calculated factored design load for the specific bolts rather than to develop the generic bolt ultimate load.

In view of the above discussion, the design procedures and construction practices used in the STP ensure that the structures are adequate for the specified conditions prescribed by the current NRC criteria.

Insert G (continued) (4 of 11)

ATTACHMENT 1
ST-HL-AE-1428
PAGE 16 OF 23

In accordance with the request made during the NRC structural audit, the impact of the NRC positions as stated in R. G. 1.142 has been evaluated. The following table compares the NRC and STP positions on the twelve items included in R. G. 1.142.

Table 220.30N-1 ← Global Charge

STP Positions on Regulatory Guide 1.142
"Safety-Related Concrete Structures for Nuclear Power Plants
(Other than Reactor Vessels and Containments)"

Regulatory Position	STP Position
1. Structures required to withstand pressures and to maintain a certain degree of leak-tightness during operating and accident conditions will be reviewed in accordance with the specific provisions of Standard Review Plan 3.8.3.	1. The requirements for leaktightness specified in Standard Review Plan 3.8.3 are applicable to PWR ice-condenser containment internal structures and to BWR containment internal structures, and therefore are not applicable to STP, which has PWR dry containment internal structures.
2. When concrete structures are used to provide radiation shielding, provisions of ANSI/ANS 6.4-1977 (see Appendix A) are applicable to the extent that they enhance the radiation shielding function of these structures. Reduction in shielding effectiveness due to embedments, penetrations, and openings should be fully evaluated.	2. Concrete structures which are used as radiation shields are analyzed for shielding effectiveness utilizing the methods addressed in FSAR Section 12.3. Reductions of shielding effectiveness such as shielding discontinuities, penetrations and openings (e.g., doors and access hatches), are reviewed for impact on radiation dose rate zoning. Additional shielding in the form of penetration seals or labyrinths is provided as necessary to ensure operating personnel exposures are maintained ALARA.
3. The Code lacks specific requirements to ensure the ductility of concrete moment frames. Adherence to the requirements of Appendix A to ANSI/ACI 318-77 is acceptable.	3. The STP Category 1 structures do not utilize concrete moment frames, and therefore this position is not applicable to STP design.

ATTACHMENT 1
ST-HL-AE-1428
PAGE 17 OF 23

Sheet 6 (continued)

5 of 11

STP Positions on Regulatory Guide 1.142
"Safety-Related Concrete Structures for Nuclear Power Plants
(Other than Reactor Vessels and Containments)"

Regulatory Position	STP Position
4. In addition to the requirements of Section 1.3.1 of the Code, the inspectors should have sufficient experience in reinforced and prestressed concrete practice as applied to the construction of nuclear power plants. The inspectors should be thoroughly familiar with the acceptable ACI and ASTM standards. The examiners/inspectors qualified to Appendix VII of Section III, Division 2, of the ASME Boiler and Pressure Vessel Code (ACI 359) are acceptable as inspectors.	4. Inspectors involved with concrete related work on the STP are qualified in accordance with ANSI N45.2.6, "Qualifications of Inspection, Examination, and Testing personnel for Nuclear Power Plants," a widely accepted standard in nuclear construction.
5. In lieu of the frequency of compressive strength testing required by Section 4.3.1 of the Code or that required by ANSI N45.2.5 as endorsed by Regulatory Guide 1.94, the following is acceptable: Samples for strength tests of concrete should be taken at least once every shift for each class of concrete placed or at least once for each 100 cu yd of concrete placed. When the standard deviation for 30 consecutive tests of a given class is less than 600 psi, the amount of concrete placed between tests may be	5. Concrete for the STP is tested every 100 cu. yds. (or at least once a day during production). This test frequency meets or exceeds both ACI 318 and ANSI N45.2.5 requirements. The provisions to reduce testing frequencies outlined in this position have not been exercised.

ATTACHMENT 1
ST-HL-AE-1428
PAGE 18 OF 23

Sheet C (continued)
6 of 11

STP Positions on Regulatory Guide 1.142
"Safety-Related Concrete Structures for Nuclear Power Plants
(Other than Reactor Vessels and Containments)"

Regulatory Position	STP Position
increased by 50 cu yd for each 100 psi the standard deviation is below 600 psi, except that the minimum testing rate should not be less than one test for each shift when concrete is placed on more than one shift per day or less than one test for each 200 cu yd of concrete placed. The test frequency should revert back to each 100 cu yd placed as soon as the test data of any 30 consecutive tests indicate a higher standard deviation than the value controlling the decreased test frequency.	
6. The load factors used in Section 9.3.1 of the Code are acceptable to the staff except for the following: a. In load combinations (9), (10), and (11), $1.3T_o$ should be used in place of $1.05T_o$. b. In load combination (6), $1.5P_a$ should be used in place of $1.25P_a$.	6. The load combinations and the associated load factors used in the STP design meet the minimum requirements specified in Standard Review Plant Sections 3.8.3 and 3.8.4 and therefore are consistent with the modifications to the load factors outlined in this position.

ATTACHMENT 1
ST-HL-AE-1428
PAGE 19 OF 23

Insert G
(continued)
(7 of 11)

STP Positions on Regulatory Guide 1.142
"Safety-Related Concrete Structures for Nuclear Power Plants
(Other than Reactor Vessels and Containments)"

Regulatory Position	STP Position
c. In load combination (7), $1.25P_a$ and $1.25E_o$ should be used in place of $1.15P_a$ and $1.15E_o$, respectively.	
d. In load combinations (2) and (10), $1.9E_o$ and $1.4E_o$ should be used in place of $1.7E_o$ and $1.3E_o$, respectively.	
7. When the lateral and vertical pressures of liquids are due to the normal ground water variation in the soil surrounding the structure, the load factors of H loading of Section 9.3.1 should be applied to these forces or their related internal moments and forces.	7. In the STP design, the load factors used to compute the water pressure resulting from the ground water table are, as a minimum, those applicable to the dead load of the structure. The design water table used to calculate hydraulic forces on structures which extend below the water table is based on a high water table elevation. Since the unit weight of water is well defined and the design is based on the high water table elevation corresponding to 1 foot below grade, the ground water loads are actually defined with a high level of certainty and are not subject to adverse variation. Therefore the load factors applicable to well defined loads, such as dead load, are considered appropriate.
8. In Section 9.3.2 the effects of differential settlement should be included in load combinations (1) through (11).	8. In the STP design, the effects of differential settlement would have been included, had significant differential settlement been anticipated. However, the natural ~~soul~~ soil and Category I backfill, which

ATTACHMENT 1
ST-HL-AE-1428
PAGE 20 OF 23

Sheet G (continued)

8 of 11

STP Positions on Regulatory Guide 1.142
"Safety-Related Concrete Structures for Nuclear Power Plants
(Other than Reactor Vessels and Containments)"

Regulatory Position	STP Position
	support all Category I structures, have been investigated and evaluated to ensure that differential settlements within structures will remain within tolerable limits. As part of an ongoing program, settlement in the structures is monitored to ensure that this is the case. Differential settlements within structures observed to date are considered negligible.
9. The consideration of loads due to pool dynamics for the concrete structures in pressure-suppression containments will be evaluated on a case-by-case basis.	9. Because STP does not utilize pressure-suppression containments, this position is not applicable to the STP design.
10. The local exceedance of section strengths in accordance with Appendix C of the Code is acceptable in analyses for impactive or impulsive effects of Y_r, Y_j, and Y_m in load combinations (7) and (8), and those of tornado-generated missiles in load combination (5) except for the following: a. The deformation and degradation of the structure resulting from such an analysis will not cause loss of function of any safety-related structures, systems, or components.	10. The STP design is in compliance with this position.

ATTACHMENT 1
ST-HL-AE-1428
PAGE 21 OF 23

Insert G (continued)

9 of 11

STP Positions on Regulatory Guide 1.142
"Safety-Related Concrete Structures for Nuclear Power Plants
(Other than Reactor Vessels and Containments)"

Regulatory Position	STP Position
b. The section strengths should be adequate to satisfy these load combinations without the impactive or impulsive effects.	
c. In Section C.3.4, the permissible ductility ratios (u) when concrete structure is subjected to a pressure pulse due to compartment pressurization or external explosion (blast) loading should be as follows: (1) For the structure as a whole $u \leq 1.0$. (2) For a localized area in the structure $u \leq 3.0$.	
d. In Section C.3.7, where shear controls the design, the permissible ductility ratios should be as follows: (1) When shear is carried by concrete alone, $u \leq 1.0$. (2) When shear is carried by combination of concrete and stirrups or bent bars, $u \leq 1.3$.	

Insert G
continued
10 of 11

STP Positions on Regulatory Guide 1.142
"Safety-Related Concrete Structures for Nuclear Power Plants
(Other than Reactor Vessels and Containments)"

Regulatory Position	STP Position
11. The local exceedance of section strengths in accordance with Appendix C of the Code is also acceptable under the impactive and impulsive loadings associated with aircraft impact, turbine missiles, and a localized pressure transient during an explosion, subject to the applicable exceptions of regulatory position C.10.	11. The STP design is consistent with this position.
12. The generic criteria of Appendix A, "Thermal Consideration," of the Code are acceptable for the analysis of structures under loads T_o and T_a.	12. The STP design considers thermal effects for Category 1 reinforced concrete structures. In general, the OPTCON computer code is used for determining the thermal effects on the design of reinforced concrete sections. Even though the method outlined in Appendix A of the ACI 349 code has not been used, OPTCON reflects the state-of-the-art methodology in reinforced concrete design, incorporating an equally acceptable procedure for computing the thermal effects. OPTCON is one of the modules of the Bechtel Structural Analysis Program, Post Processor described in Appendix 3.8.A. (Refer to the response to Question 220.25N for a more detailed discussion on the consideration of cracked sections in the STP structural analyses.)

ATTACHMENT 1
ST-HL-AE-1428
PAGE 23 OF 23

Insert G (continued)
(11 of 11)