GENERAL ELECTRIC

NUCLEAR ENERGY

ENGINEERING

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September 5, 1979

Mr. Robert W. Reid, Chief
Operating Reactors Branch #4
Office of Nuclear Reactor Regulation
U. S. Nuclear Regulatory Commission
Washington, D.C. 20555

Subject: Response to Recent Requests for Additional Information - General Electric Test Reactor - Docket 50-70

References: (1) Letter from Robert W. Reid (NRC) to R. W. Darmitzel (GE) dated June 27, 1979

(2) Letter from Robert W. Reid (NRC) to R. W. Darmitzel (GE) dated July 9, 1979

Dear Mr. Reid:

Attachment 1 provides responses to Items #6 and #7 contained in Reference 1. All other items in Reference 1 have previously been responded to. Attachment 2 provides responses to all 28 items contained in Reference 2.

If we can be of further assistance in this matter, please let me know.

Very truly yours,

DLGilliland for

R. W. Darmitzel
Manager
Irradiation Processing Operation

Attach.

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External Distribution
Response to 28 Structural Modification Questions

9/5/79

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Advisory Committee on Reactor Safeguards

ATTACHMENT 1

GENERAL ELECTRIC RESPONSE TO
THE NRC REQUEST FOR ADDITIONAL
INFORMATION DATED JUNE 27, 1979

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RESPONSES TO NRC REQUEST FOR INFORMATION BASED ON
GETR SITE VISIT HELD 18 JUNE 1979
AND LETTER DATED JUNE 27, 1979

Request No. 6

Discuss how base plate flexibility was considered in the design of additional supports and in the evaluation of original supports for all safety-related systems, equipment, and components.

Response to Request No. 6

Almost all base plates for both new and original supports for the safety-related systems, equipment, and components are symmetrically constructed. The forces in the anchor bolts for these supports were determined based on symmetry and statically determinate analysis. The flexibility of the base plates was considered in the design of primary piping system supports. When the base plate is thin enough to be considered flexible, an additional prying force may be produced in the anchor bolts. This prying force was computed using Equation 5-3, page 5-37, of "Structural Steel Designers' Handbook," Edited by F. S. Merritt, McGraw-Hill, Inc., 1972.

For two base plates which are statically indeterminate, analyses were performed to determine the distribution of forces in the anchor bolts. In these analyses base plate and anchor flexibilities were combined based on principles of mechanics to obtain the base plate forces. It was found in these analyses that the base plates were essentially rigid and that base plate flexibility did not affect the distribution of forces.

Remaining base plates were considered to be rigid since the unstiffened distance between the member welded to the base plate and the anchor bolts is less than twice the thickness of the plate.

Request No. 7

Provide the factors of safety assumed for all new and original concrete expansion anchor bolts used in the support system for all safety-related systems, equipment, and components. Justify the adequacy of the factors of safety.

Response to Request No. 7

The factors of safety for all concrete expansion anchors (wedge anchors) used in the support systems for all safety-related systems, equipment, and components exceed the value of five. In addition, almost all restraints exceed the value of seven, and many restraints have even higher factors of safety.

The computations for the factors of safety for the wedge anchors were based on conservative assumptions and included the following factors:

- Concrete strength
- Spacing between anchors
- Depth of anchor embedment
- Pre-existing holes in concrete
- Minimum edge distance
- Combined shear-tension interaction
- Base plate/anchor flexibili

The computed factors of safety are adequate because they were based on conservative calculations and equal or exceed the manufacturer's recommended values.

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ATTACHMENT 2

GENERAL ELECTRIC RESPONSE TO
THE NRC REQUEST FOR ADDITIONAL
INFORMATION DATED JULY 9, 1979

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GENERAL ELECTRIC RESPONSE TO THE NRC REQUEST
FOR ADDITIONAL INFORMATION DATED JULY 9, 1979

Request No. 1

Discuss how extensive the cracking of the floor slabs is due to the overturning moments. Verify that there is no impact on safety related components due to spalling or cracking.

Response to Request No. 1

It was shown in EDAC Report 117-217.03 (Reference 1) that overturning moments resulting from postulated vibratory ground motions would cause the concrete core structure to rotate primarily as a rigid body over the foundation soil. It was also shown in the same reference (Table 2-12) that the induced stresses in the slabs within the concrete core structure would be less than the concrete cracking threshold capacities. Therefore, there will be no cracking of the floor slabs within the concrete core structure. It was also shown that there may be some cracking in the floor slabs at lower floor levels exterior to the concrete core.

All safety-related systems, components and structures within the reactor building, except portions of the fuel flooding system (FFS) lines and portions of the polar crane impact structures, are located within the concrete core structure (Reference 2). Since there will be no cracking of floor slabs within the concrete core structure, there will be no impact on the safety-related components located within the core structure due to spalling or cracking of floor slabs. The FFS lines outside the core structure are protected against miscellaneous impinging objects by steel protective covers. The polar crane impact structures are located on and above the third floor level. The possible cracking or spalling of concrete would therefore not have any impact on the polar crane structures, especially considering that there would be no cracking or very minor cracking in the third floor (exterior to the concrete core structure). Also, the analyses described in Reference 1 showed that induced moments in the third floor slab would be slightly smaller than its yield capacity and the corresponding elastic deformations would be very small. Thus, imposed loads on the polar crane structures due to floor deformation would be very small.

Request No. 2

Verify, with a detailed discussion addressing generated missiles, pipe and support deformation capabilities, structural stiffness and strength degradation, and reactor building leaktightness integrity; that the extensive cracking and failure resulting from surface rupture will not impact safety-related components or systems.

Response to Request No. 2

As discussed below and in the Response to Request No. 3, the response of the reactor building and subsequent potential damage during the postulated surface rupture offset will not affect the polar crane structures, FFS lines, or other safety-related items (a list of all safety-related systems, components, and structures is given in Reference 3).

All safety-related systems, components, and structures in the reactor building except portions of the fuel flooding system (FFS) lines and some of the polar crane impact structures are located within the concrete core structure outlined in Figure 3-7 of Reference 1 (Phase 2 Report). The concrete core structure will respond in the elastic range and thus will not crack or generate concrete missiles.

In addition to the concrete core structure, the protected area includes the third floor slab on the south side of the reactor building where part of the polar crane impact structures are located. The circumferential wall in the basement beneath this area will support the slabs above it.

Potential generated missiles at the third floor level of the reactor building are listed and discussed in Appendix 9 of Reference 2. Modifications are being made to prevent damage to safety-related equipment from these missiles. As discussed above, the postulated surface rupture offset will not affect the concrete which supports the polar crane impact structures.

In regard to potential missiles caused by cracking and spalling of the concrete due to the postulated surface rupture offset, all safety-related equipment,

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except the FFS lines, are either located on the third floor or within the concrete core structure which protects them from damage. The FFS lines attached to the concrete core structure are protected against falling concrete pieces by steel protective covers. In order to provide maximum protection for the north FFS line from penetration 19 to the concrete core structure wall, the flexible steel pipe with steel braid covering will be additionally protected by a steel cover. The FFS line on the southwest side between the penetration and the concrete core structure is protected by a steel braid covering. Since there is no potential for falling concrete pieces in this area, no steel cover will be installed. The possibility of generated missiles impacting the FFS is discussed in EDAC Report 117-217.08 (FFS Report) (Reference 4). Note that the FFS lines have been designed with sufficient flexibility to accommodate any deflections of the containment shell which may occur (see Response to Request No. 5, Reference 5).

It was found that the floor slabs on the east side of the core structure may yield downward due to gravity load if the lateral soil pressure deforms the basement wall inward. Also, it was reported in the FFS Report that the circumferential basement wall on the east side has considerable reserve strength beyond the yield point and will not collapse when subjected to the postulated one-meter surface rupture offset. The maximum slab rotations due to the postulated surface rupture offset were computed and it was determined that the FFS lines which pass through the floor slabs at the north corner of the east wall of the core will not be damaged since the holes in the slabs are being drilled large enough to accommodate the displacement which may occur.

Stresses in the concrete core structure itself (due to the postulated surface rupture offset) are less than the concrete cracking strength. Deformations within this region are well within the elastic range; thus significant relative support deformations of the piping supports and piping will not occur.

The structural stiffness and strength of limited portions of the reactor building may be degraded if the postulated surface rupture offset occurs. However, any stiffness or strength degradation will occur in the portions of the structure away from the concrete core structure, which will not affect the safety-related items. Stiffness and strength degradations could affect subsequent response to vibratory ground motion. Accordingly, the effects of an aftershock on the reactor building were investigated and it was found that the safety-related portion of the structure would be stable and that stresses induced for the post-offset conditions would be considerably less than the stresses from the linear elastic analyses performed for the postulated main shock. The post-offset analyses are discussed in Chapter 4 of the Phase 2 Report (Reference 1).

After completion of the modifications to protect the safety-related systems, components, and structures, those portions of the reactor building concrete and enclosed items which are required for safety will continue to function in the event of the postulated surface rupture offset.

Request No. 3

Verify that the extensive failure resulting from surface rupture will not compromise the integrity of the interior radial wall, the circumferential wall connection, or the ability of the containment to support the required loadings without impacting the integrity of any safety related components, systems, or equipment. Discuss the extent of the predicted containment damage in detail to substantiate your statement that its deformations are acceptable. Specifically, address the possibility of a punching mode of failure.

Response to Request No. 3

Based on clarifying discussions with an NRC Technical Reviewer in regard to this request for additional information, it is our understanding that the NRC is primarily concerned that the safety-related portion of the concrete above the basement level may be affected by postulated failure of concrete in the basement area. In addition, details concerning the boundary loads on the finite element model were requested, and the question was asked whether any safety-related equipment is located in the basement area. The following response integrates the formal request for additional information given above and the concerns expressed in the follow-up discussion.

Results of analyses of the reactor building concrete for the effects of the postulated surface rupture offset are presented in EDAC reports 117-217.02 (Phase 1 Report), 117-217.03 (Phase 2 Report), and 117-217.08 (FFS Report) (References 6, 1, and 4). A summary of the effects on the interior radial walls, the circumferential wall, and the concrete which supports the integrity of safety related systems, components, and structures is given below. The details of the finite element model boundary conditions, the extent of reactor building concrete damage, and the possibility of a punching mode of failure are also discussed.

Since there are no safety related systems, components, or structures in the basement area there is no possibility of damage to a safety-related item if a surface rupture offset occurs and the circumferential basement wall deforms inward due to lateral soil pressure.

An analysis of the circumferential wall on the west side of the reactor building in the basement area was performed to determine the response to lateral soil pressure caused by the thrust component of the postulated surface rupture offset. It was concluded in the Phase 1 Report that this wall could yield inward between two of the radial walls due to lateral soil pressure. Based on this finding two additional investigations were conducted: first, the interior radial walls in the basement area were analyzed to determine their capacity to resist the lateral load transmitted by the circumferential wall just prior to its yielding; and second, the entire concrete core structure above the basement area was analyzed to determine whether it could accommodate the yielding of the circumferential wall.

In the first investigation a simplified stress analysis was performed to determine the shear stresses in the interior radial walls in the basement area caused by the soil pressures on the circumferential wall. Results of this analysis, which are reported in the Phase 1 Report, indicate that the shear stresses are well below the allowable values.

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In the second investigation two separate analyses were performed. In the analysis conducted for the Phase 1 Report, the exterior wall just above the basement circumferential wall, which was conservatively assumed to have yielded, was analyzed as a deep beam spanning between the two adjacent radial walls. Because of the strength and stiffness of the deep beam, the slight eccentricity of the upper circumferential wall will not cause additional stresses due to large deformations of the circumferential wall beneath it. This analysis was made in conjunction with the cantilever support case analysis performed for the reactor building (see Phase 1 Report). It was found and reported in the Phase 1 Report that the maximum shear stresses in the concrete core structure, including the circumferential wall just above the yielded basement wall, were low and the concrete will remain uncracked. The results of the analyses presented in the Phase 2 Report verified the Phase 1 Report conclusions. In this second analysis the concrete core structure (including the radial and circumferential walls) was modeled using three-dimensional finite elements. Stresses in the concrete core structure, including the circumferential wall just above the basement wall (again conservatively assumed to have yielded), were computed based on conservative support boundary conditions at the foundation level and edge moments and shear forces at the floor levels where the floor slabs connect to the concrete core structure. A schematic representation of the support and edge boundary loads is shown in Figure 3-11a and b of the Phase 2 Report. A description of the basis for these boundary conditions is given below.

On the east side of the reactor building it was assumed that because of the weight of the concrete and equipment, the first, second, and third floor slabs would yield downward if the basement circumferential wall yields inward. Since the floor slabs on the east side may not be capable of cantilevering from the core walls, yield moments and corresponding shear forces were conservatively assumed to be transmitted to the concrete core structure. Note that no moment was applied at the third floor level since there is no top steel anchored into the core walls in the third floor slab. The floor slabs on the south side of the reactor building will not yield downward since the circumferential basement wall on the south side will support the slabs located above it. In addition, the slabs in this area have sufficient strength to carry their own weight as cantilever sections. However, it was conservatively assumed that the boundary edge moments on the core walls were equal to the yield moment capacities of the floor slabs.

Because of the construction detail between the foundation mat and the basement slab, the boundary edge loads at the foundation mat level will not affect the response of the safety-related concrete above. In the Phase 2 analysis, conservative boundary edge loads at this level were selected based on an assumed conservative response due to the postulated surface rupture offset. Because of the construction detail between the foundation mat and the rest of the reactor building, the foundation mat could be free to cantilever off the edge of the supporting soil (see soil pressure block shown in Figure 3-11a of the Phase 2 Report). In general if the foundation mat yielded downward, it would not apply any loads to the reactor building above it. Thus, the edge boundary moments on the east side of the soil pressure block were assumed equal to the moment capacity of the basement slab. However, along the south edge of the concrete core structure, foundation mat yield moments and corresponding shear forces were assumed to act upward. These forces would be due to the area of vertical soil pressure underneath the reactor building which extends beyond the concrete core structure (see Figure 3-3 of the Phase 2 Report). This is a conservative interpretation of the soil pressure diagram and in reality the pressures would be more uniformly distributed. On the south side, it was assumed that the soil will push up on the foundation mat and the net result will be an upward shear force and moment at the south boundary of the model. Edge effects due to twisting of the foundation mat were also included in the model.

Based on the analyses of both the simplified and finite element models, (including the circumferential core wall just above the circumferential basement wall which may yield inward), the induced stresses in the concrete core structure are much lower than the conservative cracking capacities. The

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concrete core structure (minus the circumferential basement wall) will respond as a rigid block during the postulated surface rupture offset, but will remain intact and uncracked. Thus none of the radial or circumferential walls will punch into the concrete required to protect the safety-related systems, components, or structures.

Request No. 4

Justify the material properties used for the soil spring model, including damping values and Poisson's ratio. To what level in the actual subgrade do these values correspond? Discuss the impact on the factors of safety provided for the forces and floor accelerations if the "most realistic case" of subgrade parameters (as described in your Phase 2 Seismic Analysis Report) is not present. That is, what safety considerations are provided in the event that the actual response is greater than predicted by Case 1 parameters?

Response to Request No. 4

Soil parameters shown in Table 3-1 (Page 3-16) of Reference 1 were derived on the basis of the Seed-Idriss relationships (Reference 7). A confining pressure of 3,000 psf was used since it represents the mean confining pressure 20 to 30 ft below the ground surface. A soil shear strain of 0.2 percent was estimated for the free field maximum postulated seismic event (particle velocity = 1 ft/sec which corresponds to 0.8g pga). Hence, to be conservative, an assumed shear strain of 0.1 percent was used to calculate modulus and damping parameters. Higher, more realistic soil strains will not significantly affect the modulus but would increase damping, and would generally reduce response. Lower soil strains (which would increase response by increasing the frequency and reducing damping) were determined to be incompatible with free field or structure-induced soil strains.

A Poisson's ratio of 0.4 was selected on the basis of review of literature applicable to the specific foundation soils at the site (stiff saturated gravely clays) and the rapid undrained loading condition which would occur during seismic shaking.

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The linear elastic analyses, termed the "most realistic case" in the following discussions, were based on conservative values of subgrade parameters, namely, a soil shear modulus of 1000 ksf and an average area of contact between the base slab and the soil of 75 percent of the total area. Parametric analyses were performed to determine the influence of the variation in the subgrade parameters on the structural response. Table 2-4 in Reference 1 gives the effects of the variation in subgrade parameters on the reactor building response. An extremely conservative upper bound soil shear modulus (i.e., 3000 ksf) was selected which corresponded to very low strains which would occur only during a very low peak ground acceleration seismic event. This upper bound soil shear modulus was used in conjunction with an area of contact of 100 percent. As given in Table 2-4, the use of these extremely conservative subgrade parameter values increased the response of the structure. It should be emphasized however, that a value of 3,000 ksf for the soil shear modulus is unrealistic for a ground motion with peak ground acceleration of 0.8g (which would cause higher levels of soil strains) and thus the calculated responses for this case will not occur. The parametric analyses also included the case of shear modulus equal to 1,000 ksf and area of contact equal to 50 percent of the total area. Use of these subgrade parameters showed that the forces in the structure decreased by 2 percent, the floor accelerations decreased by 5 percent, and the spectral accelerations decreased by 10 percent.

In addition to the conservative linear elastic analyses, nonlinear analyses were performed (Reference 1) to determine the influence of nonlinearities due to potential uplift (overturning) and sliding. These nonlinear analyses provided a more realistic representation of the actual response which would occur for ground motions with a peak ground acceleration of 0.8g. Two nonlinear models (A and B) were used for this purpose. As shown in Tables 2-6, 2-8, and 2-10, and discussed on pages 2-10, 2-11, and 2-12 of Reference 1, the forces in the structure will be reduced by about 10 to 35 percent (from the elastic analysis forces) due to uplift, and by about 20 to 30 percent due to sliding. These results represent actual reductions in forces below those obtained from the conservative "most realistic case" linear elastic analyses.

Therefore, on the basis of the results of the linear parametric analyses and nonlinear analyses, the actual response will not be greater than that predicted by the linear elastic analyses using the "most realistic case" parameters.

Request No. 5

Verify that the calculations of the sliding and overturning resistance have accounted for the reduction of the weight of the building due to vertical uplift. If uplift due to vertical excitation is not considered, justify the appropriateness of the unconservative analysis.

Response to Request No. 5

The nonlinear analyses for sliding and uplift (overturning due to rocking) were performed for horizontal excitation; vertical excitation was not included. However, the analyses were conservative for the reasons described below.

The analyses did not include the resisting effects of the 20 ft embedment. As shown in Table 2-7 of Reference 1, if embedment resistance to sliding were included, the building would not slide. Similarly, if resisting embedment effects were included in the uplift (overturning due to rocking) analysis, the vertical displacement would be smaller due to the frictional force between the reactor building and the surrounding soil. Thus, any effect of the vertical excitation would be compensated by the resisting effect of embedment.

As discussed in the Response to Request No. 4, the effect of including the nonlinearities due to sliding and uplift would be to significantly reduce the stresses in the concrete structure. Therefore, in the implausible case that the effects of vertical excitation were not compensated by the embedment effects and the resulting nonlinearities were more pronounced, the overall result would be to further reduce the stresses in the concrete structure.

Furthermore, as discussed in Response to Request No. 10, there are significant safety margins available against sliding and instability due to uplift. For uplift, a maximum angle of rotation of 0.09 degrees was obtained from the nonlinear analysis A (Table 2-6, Reference 1) which is roughly 1/300 of the theoretical angle at which the building could "tip over". (It should be noted that the concept of "tipping" is one which is based on static design procedures and is overly conservative when applied to dynamic oscillatory loadings.) Similarly, as shown in Table 2-7 of Reference 1, the force required to slide the building is more than twice the induced seismic force obtained from the conservative linear elastic analysis. The building would therefore be stable even in the unlikely event that the nonlinear effects were higher due to vertical excitation.

Thus, consideration of the constraining effects of embedment, reduction of response due to nonlinear effects, and adequacy of overall building stability demonstrates that the nonlinear analyses performed for horizontal excitation were conservative.

Request No. 6

Verify that the maximum sliding displacement of 1.3 inches results in no failure of safety-related piping, components, or equipment.

Response to Request No. 6

As discussed in Response to Request No. 2, either all safety-related piping, components, and equipment are contained within the core area of the reactor building or are protected as a result of recent modifications or by the concrete structure on the south side of the reactor building. The only components which potentially could be affected by the maximum sliding displacement of 1.3 inches are the two fuel flooding system (FFS) lines, which penetrate the containment shell and run to the pool and canal. Since several feet of slack in the lines is provided on both sides of the containment shell, there is sufficient flexibility to accommodate a 1.3 inch sliding displacement.

Request No. 7

In your Seismic Analysis of Reactor Building - Phase 2 Report, you state that "there is no structural continuity between the foundation mat and the rest of the reactor building." Describe how this is represented in the mathematical model. Provide the properties of the member between the foundation mat and the basement slab, and describe how they were determined. (Provide the terms of the local stiffness matrix). Also, verify that the results in Table 2-9 for sliding at the interior concrete - foundation slab interface reflect these properties and the bounding case considering response variations due to all potential variations considered in your analyses. If relative motion is predicted, discuss the impact on the results of your analyses.

Response to Request No. 7

The connection between the foundation mat and the basement slab was represented in the mathematical model by properties of a rigid member (very large values of stiffness), since it was found from preliminary computations that sliding will not occur at the interface between the basement slab and the foundation mat due to frictional resistance. Even if there were sliding at this interface, the forces would be reduced in the concrete structure in a manner similar to the effect of hypothetical sliding between the foundation mat and soil, which resulted in the reduction of forces by about 20 to 30 percent (Table 2-8, Page 2-22 of Reference 1). Thus, it was conservative to neglect the potential for sliding at the interface of the basement slab and the foundation mat in the reactor building model.

Request No. 8

Describe the procedures utilized in the determination of the soil spring boundary conditions in your Model B nonlinear analysis. Also demonstrate that this type of representation of the subgrade is appropriate considering soil depth, layering, etc. Discuss the acceptability of your modeling as opposed to using the current finite element techniques.

Response to Request No. 8

The properties of the soil springs in nonlinear model B were determined based on the assumption that the soil cannot take any tension. In compression, a bilinear force-displacement model was used which was based on the assumption that the soil will respond linearly as a half-space model until it begins yielding at the soil yield capacity. This model realistically represents the actual site in regard to depth and soil layering, since the site is generally uniform with no distinct layers.

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The analytical techniques were selected to be consistent in detail and accuracy with the objectives of the analyses, the simplicity of the structure, the uniformity of the soil materials, and the precision of parameters such as nonlinear material properties. Other analytical techniques, such as the finite element approach, were considered but not used, since they would have involved their own set of approximations and assumptions and would not have provided greater accuracy. Thus the technique used for these analyses was appropriate.

Request No. 9

Provide a description of the core structure displacements associated with the yielding and settlement of the foundation mat. Verify that these displacements were considered in the design of the core structure and safety related components, systems and equipment, and that the integrity of these safety related items is not compromised.

Response to Request No. 9

Based on clarifying discussions with an NRC Technical Reviewer in regard to this request for additional information, it is our understanding that the NRC is concerned that the reactor building may slide or separate from the foundation mat and subsequently impact the mat during the postulated surface rupture offset. As stated in EDAC Report 117-217.03 (Phase 2 Report) (Reference 1), the cantilever case would not occur (primarily because of the downward frictional forces produced by the soil which rests against the basement wall). However, for purposes of analysis, the very conservative assumption was made that the cantilever case could occur. In addition, it was observed based on a survey of published field data that fault displacements take place over a considerable span of time (Reference 8). Thus, impact due to the postulated surface rupture offset would not occur.

In Section 3.1 of the Phase 2 Report the building displacements and rotations associated with the hypothetical cantilever support case are discussed. The criteria used in the surface rupture offset analysis consist of a postulated one meter surface rupture offset considered to be at an angle of 15 degrees. A summary of the displacements and rotations are given below.

- If the postulated cantilever support case occurred the concrete core structure could be lifted vertically a maximum of 10 inches and would rotate (tilt) approximately two degrees toward the west before the building again makes contact with the soil.

- Since the coefficient of friction between the basement slab and the foundation mat and the supporting soil is much greater than the tangent of the tilt angle there will be no slippage due to rotation of the concrete core structure.

Based on the information presented above, integrity of safety related systems, components and structures is not compromised.

Request No. 10

In the post-offset analyses, provide the acceptance criteria for the seismic displacements and forces. Also, provide the factors of safety against sliding and overturning for this condition and summarize how they were determined.

Response to Request No. 10

The peak ground acceleration (pga) criteria for a postulated post-offset event is 0.15g. However, the post-offset analyses (for the reactor building) were performed for a pga of 0.8g (to demonstrate without question that the structure is adequate).

The forces obtained from the conservative post-offset analyses, given in Table 4-1 of Reference 1, are significantly smaller than the forces shown in Table 2-3 of the same reference obtained from the linear elastic analyses for the hypothetical 0.8g pga event. Since the stresses corresponding to the forces from the linear elastic analyses (for 0.8g pga) were found to be smaller than the concrete cracking threshold capacities (which was the acceptance criteria), as shown in Table 2-12, it was concluded (page 4-3) that the stresses from the post-offset analyses (0.15g) would be much smaller than the concrete cracking threshold capacity.

Table 4-1 of Reference 1 also shows that the maximum vertical uplift of the base slab would be only 1.60 inches and the corresponding maximum rotation would be 0.29 degrees for a 0.8g aftershock. The theoretical angle at which the building could "tip over" would be 31 degrees, which is roughly 100 times the calculated angle. (It should be noted that the concept of "tipping" is one which is based on static design procedures and is overly conservative when applied to dynamic oscillatory loadings.) Thus, there will be a large margin of safety available against uplift (overturning).

The force required to cause sliding (without embedment), calculated in a manner similar to the results of the analysis given in Table 2-7 of Reference 1, would be about 1.14 times the corresponding shear force obtained from the analyses for the postulated post-offset vibratory ground motion. If embedment resistance is included, the force required to cause sliding would be approximately 2.44 times the shear force obtained from the main event analyses. Thus, there will be a significant safety margin available against sliding.

Request No. 11

Describe in detail the methods by which the allowable shear and tensile stresses were determined from the referenced test data. Justify the correspondence between the GETR walls and these test samples since the PCA tests were flange reinforced specimens. Verify that the stresses calculated via your finite element representation of the GETR are directly comparable to your stated allowables. Provide the bases for your statements. Include a discussion of how construction joints were considered in your evaluation and the possibility of degradation of these joints due to water seepage weakening the shear transfer across the joint.

Response to Request No. 11

The allowable shear and tensile stresses were determined based on data obtained from a Portland Cement Association (PCA) report (Reference 9) and two reports from Stanford University (References 10 and 11). A summary of the data analysis, discussion of the methods used in the analyses, and the results are given in the submittal to the NRC in response to a request for additional information on the Phase 2 Report (Reference 12).

In the analysis of the PCA data, principal tension stress values from tests B1-1, B2-1, B3-2, B4-3, B5-4, B6-4, and B7-5 (Reference 9) were used to obtain a mean value of 6.1 $\sqrt{fc^1}$ and a standard deviation of 0.6 $\sqrt{fc^1}$ (where fc^1 = compressive concrete strength). This data corresponds to wall aspect ratios (i.e., height to width ratio between 0.25 and 0.50. The individual test values ranged from 5.5 $\sqrt{fc^1}$ to 7.1 $\sqrt{fc^1}$.

The six Stanford tests were performed for walls with an aspect ratio of 0.3. The mean first crack strength was 8 $\sqrt{fc^1}$ with a standard deviation of 1.5 $\sqrt{fc^1}$ and a range from 5.6 $\sqrt{fc^1}$ to 9.9 $\sqrt{fc^1}$. An analysis of the composite data set from both PCA and Stanford tests produced a mean value of 7.0 $\sqrt{fc^1}$ with a standard deviation of 1.4 $\sqrt{fc^1}$.

The value of 6 $\sqrt{fc^1}$ recommended in the PCA report (Reference 9) was used in the analysis of the GETR concrete. This value is substantiated by both the PCA and the Stanford data as shown by the results of the statistical analyses.

The floor slabs of the GETR reactor building are generally several feet thick. These slabs would act as flanges for the walls during a seismic event, and thus the reactor building structural configuration is similar to the specimens tested by PCA and at Stanford.

The stresses computed in the finite element model used in the Phase 2 study were maximum principal shear, tension, and compression stresses. These values were compared to the allowable value of 6 $\sqrt{fc^1}$ which was selected based on the statistical analysis of principal stress values obtained from the PCA and Stanford tests and the recommendation in the PCA report. Hence, computed stress values were compared to allowable values which are also principal stress values. Note that all the GETR walls have a height to width ratio of 0.5 or less so that the computed stresses are directly comparable to the 6 $\sqrt{fc^1}$ values.

The Stanford test specimens did not have construction joints; however the specimens tested at PCA had construction joints at the top and bottom of the walls. The construction joints at GETR have an uneven appearance based on visual observations of the wall surfaces, which helps provide shear interlock between the wall sections on each side of each construction joint. During inspection of reinforcing steel in the reactor building, concrete at a construction joint was chipped back to the outside curtain of reinforcement (Reference 13). This inspection was conducted at a joint where water seepage had occurred in the past. No evidence of deterioration to the concrete or corrosion to the exposed reinforcing bar was observed. Based on the above factors, the construction joints at GETR will have no detrimental effect on the strength of the reactor building concrete core structure.

Request No. 12

Verify that the effects of the primary piping which is anchored to the concrete structure, have been considered in the seismic analysis and design of the concrete structure.

Response to Request No. 12

In regard to the effects of the primary piping on the overall response of the structure, the weights of all piping and equipment were included in the weights at the appropriate nodal points of the mathematical model. The stiffnesses of the piping and equipment are very small relative to those of the concrete structure and were therefore ignored.

The local effects of the primary piping on the walls and floors of the structure have also been considered. All applicable walls and slabs have been evaluated and have sufficient capacity to resist the applied loads.

Request No. 13

Discuss the procedures used to determine the location of impact of the cask drop on the canal slab which produces the maximum moment on the slab. Provide this moment, and verify that the slab is capable of withstanding this load. Also, verify that spalling of concrete due to the cask drop on the canal slab does not impact any safety related items.

Response to Request No. 13

The potential trajectories of a postulated dropped cask were investigated, and the impact area was determined to be located along a nine foot length of the canal starting at the facility pit. In selecting this area, it was recognized that the possible locations of impact on the canal floor slab due to a postulated cask drop are limited by the refueling bridge which is positioned and fixed to the third floor slab (above the fuel storage tanks) prior to and during all cask handling operations.

The basis, procedure, and results of the analysis for the postulated cask drop impact on the canal floor slab are presented in EDAC Report 117-217.04 (Reference 14). In the analysis, the minimum energy-absorbing capacity for each postulated structural failure mode was first determined and it was found that all energy-absorbing capacities exceeded the maximum potential energy of the largest (700 series) cask. Then, for each postulated structural failure mode (Reference 14) the location of the cask was systematically varied until the maximum ductility required to absorb the maximum potential energy of the cask was determined. It was found that the beam-yield mode requires the largest ductility to absorb the energy associated with the postulated cask drop. The corresponding point of impact is located four feet from the outside surface of the facility pit wall.

Note that the maximum moment in the slab (8,750 Kip-feet positive moment and 4,090 Kip-feet negative moment) corresponds to the yield level of the reinforcing bars. Once this moment level is reached, the reinforcing steel will yield and the slab will deflect until the total potential energy is absorbed.

It was concluded, based on detailed analyses of the different postulated structural failure modes, that the reinforcing steel and canal floor slab can adequately resist the effects of the postulated cask drop without failure.

It is possible that pieces of spalled concrete may fall from the bottom of the canal floor slab if a cask drop occurs. Since the concrete cover under the reinforcing steel is only about three inches thick, the spalled pieces would be small and thus would not damage the primary system piping and associated components which are located below the canal.

A cask drop accident may cause the canal floor slab to deflect downward several inches. Even though GE believes that the current modifications are adequate to protect the primary piping system, the trapeze hanger supports for the primary piping will be modified so that they are not directly connected to the canal floor slab in order to provide additional assurance that the piping system, and hence the reactor pressure vessel, are protected.

Request No. 14

If a cask drop results in damage to the liner and cracking of the concrete, verify that adequate canal water is maintained.

Response to Request No. 14

The design of the new fuel storage tanks and associated structures is based on the conservative assumption that the canal liner may crack if the postulated cask drop accident occurs. The new storage tank system consists of three inner leaktight tanks located within an outer leaktight tank. This arrangement provides two leaktight barriers for containment of coolant water in the highly unlikely event water is lost from the canal. A description of the new fuel storage tanks and associated structures is given in the "Updated Response to NRC Order to Show Cause," June 1978 (Reference 3).

Thus, canal water can be lost without impacting the health and safety of the public.

Request No. 15

Provide justification that a non-mechanistic lower head nozzle rupture occurs with sufficiently low probability to assure the acceptability of the consequences of this event. Provide similar justification for rupture of the reactor pool. Include a discussion, in terms of radiation levels and stress levels, verifying that no embrittlement occurs, such as to preclude postulating the above failures.

Response to Request No. 15

Bottom Head Nozzles

The consequences of postulated failure of one of the bottom head nozzles are presented in the new GETR Safety Analysis Report (NEDO 12622). Non-mechanistic bottom head nozzle failure was postulated (in NEDO-12622) for the express purpose of creating a loss of coolant event and bounding fuel melt condition. The containment building was then evaluated to assure it could function to safely limit the radiological consequences associated with this fuel melt accident. In other words, this accident was hypothesized in order to evaluate the design capability of the containment building, not because the accident was considered credible.

In fact, failure of a bottom head nozzle is not considered credible. Simultaneous failure of a nozzle coupled with a severe earthquake is also not considered credible. Reasons for this position were discussed in Reference 15 and more specifically in General Electric's Response to Request No. 16 of Reference 16. These reasons are reiterated below.

1. The bottom head nozzles (which are made of 304 stainless steel) experience nominal environmental conditions during normal or off-normal operation, as noted below:

Normal nozzle temp (with reactor at 50 MWt)	$< 170^{o}F$
Maximum nozzle temp.	$< 180^{o}F$
Minimum nozzle temp.	$\geq 50^{o}F$
Normal internal pressure	$\sim$ 108 psig
Maximum internal pressure	150 psig
Maximum longitudinal stress	690 psi
Maximum hoop stress	1220 psi

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Fluids in contact with nozzles

Internal	High purity demineralized primary water
External	Air
Neutron fluence to date (all energies above 0.17 ev)	$< 1.9 \times 10^9$ n/cm^2

The combination of normal environmental conditions (i.e., temperature, stress, and high purity water) have not and will not produce any known long-term degradation of the nozzles. The maximum stress (1220 psi) is well below the maximum allowable stress for type 304 stainless steel operating below 200^{0}F (the maximum allowable stress is 15,600 psi for normal operation and 37,400 psi for earthquake conditions). The neutron fluence is at least 9 orders of magnitude below a value which would begin to promote embrittlement and subsequent brittle failure.

2. The maximum stress in the bottom head nozzles does not increase appreciably during the maximum postulated seismic event. The maximum stress during the seismic event is approximately 2600 psi. This is significantly below the allowable stress (37,400 psi).

For the above reasons, nozzle failure due to long-term degradation, or over-stress during normal or accident conditions (including a seismic event) is not considered credible.

Reactor Pool

The integrity of the reactor pool concrete and liner has previously been discussed in Attachment 2 of Reference 13, Attachment 3 of Reference 13, Section 5 of Reference 1, and in General Electric's Response to Request No. 9 of Reference 12.

The comprehensive response contained in Reference 12 demonstrated that:

- o high pool integrity is not required
- o even though integrity is not required, it will be maintained (i.e., the concrete that forms the pool will not crack and the liner will not fail).

The effects of neutron radiaton on pool liner integrity were not previously discussed because the fluence levels are inconsequential. The current peak neutron fluence level (for all energies above 0.17 ev) is less than 2.8×10^{12} n/cm^2. The fluence will no more than double during the remaining life of the plant. This fluence is several orders of magnitude below a value which would begin to promote embrittlement.

All the work done to date and discussed in the above listed references verifies that pool integrity will be maintained. The liner and concrete operate under nominal environmental conditions. Stress and neutron fluence levels are low, and stress levels remain low during the maximum postulated seismic event. Massive failure of the pool liner and concrete (which could produce rapid drainage of the pool), is not credible.

Request No. 16

Specify the maximum inner and outer fuel storage tank displacements, and verify that these maximum displacements are obtained using the more realistic configuration where sliding is permitted. Verify that these displacements do not adversely impact the safety related functions of the tanks. Discuss the consequences of a 1.4 inch inner tank maximum rocking deflection. Also, verify that sliding of the tanks does not result in impact to the canal liner.

Response to Request No. 16

The maximum deflections associated with the inner and outer tank analyses are, respectively, 0.36 inches and 1.4 inches. In the case of the inner tank, the maximum deflection value is associated with a side wall and represents a relative displacement between the top and the base of the wall. In the case of the outer tank, the maximum deflection is associated with a divider wall and again represents a relative displacement between the top and base of the divider. In both analyses the base is assumed fixed, hence the assumption of sliding has no effect on these values.

The assumption of a fixed base in both the inner and outer tank analyses is conservative since it requires that all of the kinetic energy due to seismic motion be reacted by straining of structural members rather than by rigid body motion. Rigid body motion of the fuel storage system was considered in an independent sliding analysis (Reference 17, page 4), and there it is shown that the maximum displacement due to sliding is 0.16 inches. A displacement of this magnitude coupled with the maximum structural deflections given above cannot result in an impact between the fuel storage tanks and the canal walls.

Request No. 17

In the inner fuel storage tank rocking analyses, provide justification for the use of a factor of 67% to reflect the energy losses and fluid inertia effects.

Response to Request No. 17

In the analysis method used to calculate stresses and deflections of the fuel storage system, no credit was taken for the energy losses due to inelastic collision and fluid damping effects, except through the employment of a 67% factor (described on page 23 of Reference 17). This factor was taken as a reasonable approximation of combined losses due to inelastic impact and fluid damping effects.

Inelastic deformations are inherent in the analysis method used (which was taken from ASME Section III, Appendix F), and these inelastic deformations limit the loading that can be transferred to the tank wall. In addition, the fluid between the inner and outer tanks acts as a hydraulic damper and significantly reduces the kinetic energy developed during rocking. The combined effects of these two forms of energy loss is taken to be one-third of the total kinetic energy developed due to seismic motion. It should be noted that in the development in Appendix B of Reference 17, the kinetic energy is not explicitly determined, but the maximum value is given by the maximum potential energy.

Request No. 18

In the inner fuel storage tank rocking analysis, describe how the 123.25 lb/in. live load on the outer tank was resolved into the concentrated loads applied at nodes 22, 23, 24, and 25.

Response to Request No. 18

Figure 3.3 of Reference 17 shows the analytical model used to calculate outer tank stresses. Note that, because of symmetry, only half the tank wall was modeled. In the model, nodes 22, 23 and 24 are nodes on one side of the wall, and node 25 represents the centerline node. For a symmetric model, the 123.25 lb/in. line load (page 23, Reference 17) must be distributed over nodes 22, 23, 24 and 25 as follows:

Load Bearing Distance	=	30.84"
Line Loading	=	123.25 lb/in.
Total Load	=	(123.25)(30.84) = 3801 lbs
Load/Node (for nodes 22, 23 and 24)	$\cong$	1086 lbs
Load/Node (Node 25)	$\cong$	1086/2 = 543 lb.

Request No. 19

Explain how the response spectra for three percent damping used in the seismic analysis of the primary cooling system and RPV envelop the response spectra obtained for one percent damping, by a factor of 1.2. (See EDAC Report 117-217.05, page 2-2.)

Response to Request No. 19

The Envelope Floor Response Spectra shown in Figures 2-11, 2-12, and 2-13 of EDAC Report 117-217.03 (Phase 2 Report, Reference 1), and in Figures 2-1, 2-2, and 2-3 of EDAC Report 117-217.05 (Reference 15) were developed early in the investigation of the GETR Reactor Building (during the Phase 1 analyses prior to completion of the Phase 2 analyses). These Envelope Floor Response Spectra were developed for damping ratios of one and three percent. Conservative procedures were used in the development of these Envelope Floor Response Spectra to ensure that they would exceed the final confirmatory spectra to be obtained from the subsequent Phase 2 analyses.

As a part of the completion of the Phase 2 analyses, the Envelope Floor Response Spectra were compared with the results of the Phase 2 linear elastic analysis (i.e., compare Figures 2-11, 2-12, and 2-13 with corresponding Figures 2-8, 2-9, and 2-10 of the Phase 2 report). It was found that the Envelope Floor Response Spectra for three percent damping exceed the final confirmatory computed Phase 2 floor response spectra for one percent damping by a factor of at least 1.2 for all frequencies greater than 7 cps, which corresponds to the range of frequencies of interest for the primary cooling system.

Floor response spectra for two percent damping represent the appropriate seismic input for analysis of the primary cooling system and RPV as specified by USNRC Reg. Guide 1.61 (Reference 18). It was thus concluded that it was conservative to use the Envelope Floor Response Spectra for three percent damping for the analysis of the primary cooling system, since these spectra exceeded the final confirmatory spectra for both one and two percent damping.

Request No. 20

Describe the piping displacements resulting from the analysis of Run 1 and Run 2. Provide the design and acceptance criteria for pipe displacements, and verify that the maximum displacements are within design allowables. Also verify that seismic excitation does not result in impact between piping systems and any safety related equipment or components.

Response to Request No. 20

The evaluation of the primary cooling system Run 1 and Run 2 was based on the allowable stress acceptance criteria specified in EDAC Report 117-217.05 (Reference 15). The piping displacements resulting from the analyses of Runs 1 and 2 are relatively small. For Run 1, the maximum seismic horizontal displacement is 0.17 inch at Node 69 (near Expansion Joint 101) and the maximum seismic vertical displacement is 0.63 inch at Node 62 (near PRI-150). For Run 2, the maximum seismic horizontal and vertical displacements are 0.58 inch at Node 90 (top of Standpipe) and 0.23 inch at Node 48 (near PRI-190), respectively. These displacements produced stresses within allowable limits and were therefore acceptable.

The magnitude of the above displacements indicates that these piping systems will not impact between themselves. The smallest clearance between adjacent portions of the primary piping was measured to be 1.75 inch. This clearance is between valve PRI-150 and the 6-inch diameter emergency recirculation line, both on Run 1. The maximum displacement at PRI-150 (Node 62) was computed to be 0.65 inch (SRSS of three directions) and the maximum displacement of the 6-inch by-pass line (at Node 68) was computed to be 0.56 inch (SRSS of three directions). Thus a conservative estimate of the maximum relative displacement is 1.21 inch, which is less than 1.75 inch clearance.

The distance to all other safety-related systems, components, and structures exceeds the maximum seismic displacement, thus seismic excitation will not result in impact between the primary cooling system and any safety-related item.

Request No. 21

Discuss how the effects of a surface rupture offset have been considered, and verify that they will not compromise the integrity of the primary cooling system and reactor pressure vessel.

Response to Request No. 21

The results of the analysis for surface rupture offset are presented in References 6, 1 and 4. The response to NRC Requests No. 2, 3, and 9 explain how the effects of surface rupture offset were considered and provide verification that the effects of a surface rupture offset will not compromise the integrity of the primary cooling system and reactor pressure vessel.

Request No. 22

List the types of restraint anchorages used for the GETR piping and equipment, and describe the procedures used in the design of these anchorages. Verify that cyclic loads have been considered, and describe and justify the anchor bolt and rock bolt cyclic load design requirements. Describe any inservice inspections which are planned for the bolts and justify the extent of the program.

Response to Request No. 22

The type of restraint anchorage used to secure the GETR piping and equipment are the following:

- o Cast in place concrete anchor bolts
- o Phillips 3/8-inch and 3/4-inch concrete wedge anchors
- o Williams hollow core rebar rock bolts

Almost all of the anchorages are new and consist of either concrete wedge anchors or hollow core rebar rock bolts. The capacities of the few existing concrete anchor bolts were based on the lesser of the strength of the bolt or the capacity of the concrete to resist the applied forces. Conservative values were used since the forces in these components were found to be low.

The basis for the design of the concrete wedge anchors is discussed in the responses to requests for information No. 6 and 7 made by the USNRC as the result of the GETR site visit held 18 June 1979 (Reference 5). Recommendations for strength given by the anchor manufacturer were used in the analysis. The factors of safety for all wedge anchors exceed the value of 5.

Similar analyses were performed for the rock bolts. The allowable value for tension was established to be 0.70 of the ultimate strength of the bolt. Because of the deep embedment of rock bolts, the mode of failure observed in tests occurs in the bolt rather than the concrete, thus it is appropriate to base the design capacity on the properties of the bolt material.

Based on a review of the history of reactor operations during the last five years, it is estimated that less than 1000 cycles of start-up and shut-down will occur during the next 20 years. Because of the relatively few cycles which will occur the cyclic loads will not affect the capacity of the anchors. The in-service surveillance program for the wedge anchors is discussed in the Response to Request 27 below.

Request No. 23

Verify that the piping restraints and anchors are in the correct locations, as designed.

Response to Request No. 23

The correct location of the piping restraints and anchors is being verified. The method for assuring proper installation is described below.

Each major task, (such as the primary pipe seismic restraint addition, for example) is assigned to a Project Engineer. The Project Engineer has complete responsibility for the design, fabrication, installation and checkout of equipment related to his task. The Project Engineer develops the design basis specifications, fabrication and installation specifications, and other engineering definition documentation. The Project Engineer also assures that the applicable quality assurance plan is adhered to throughout the project. During the project, the Project Engineer works closely with the design engineer(s), drafting, the fabrication shop(s), nuclear safety and quality assurance/control personnel to assure compliance with task requirements. Thus, the Project Enginer is completely involved from concept to completion and provides a high level of continuity. This extremely desirable project control arrangement is possible because the GETR is relatively small, related projects are relatively small, and the work is performed on a 40-hour per week basis.

As the installation of new or modified hardware (such as the new restraints) is completed, the Project Engineer personally verifies that all components are properly installed. In addition, the design engineer(s) and quality assurance personnel will inspect various aspects of each installation to assure conformance with design parameters and documented installation requirements.

Request No. 24

Verify that thermal loads and fluid transients were considered in the analysis and testing of the valves.

Response to Request No. 24

The testing and analysis procedures and test specifications used in the seismic qualification of the safety-related valves located in the GETR reactor building are given in EDAC Report 117-217.09 (Valve Report, Reference 19). The procedures used in the qualification tests were developed to evaluate the ability of the valves to resist the effects of the maximum postulated seismic event and perform the required safety-related function. Conservative fluid pressures were applied during the tests to simulate the operating pressure conditions which would exist at the time of the maximum postulated seismic event. Thermal or fluid transient loads were not applied during the valve tests since these loads are either small or do not exist. The maximum normal or off-normal temperature for any of the valves is less than 150°F. Thus the valve qualification test conservatively represented the conditions which could occur.

Request No. 25

In your Structural Analysis of Third Floor Missile Impact System, as discussed on page 17, discuss how and why the normal impact loads are applied to the bent in the Z-direction, instead of the y-direction which appears to be consistent with a resultant force Px. Also, discuss why the lateral loading was applied in the x-direction. Verify that a lateral load applied perpendicular to the x-direction is not a more critical case. Provide bent allowable stresses, including buckling stresses, if appropriate, to verify that design stresses are small. Explain the inconsistencies of Figure 5 and 6 notations.

Response to Request No. 25

The finite element code applied to the structural analysis of the third floor missile impact system (Reference 20) utilizes two distinct coordinate systems. A global coordinate system (shown in Figure 6 of Reference 20) is used to define the geometry of the structure and external loads are applied in terms of this global system. A local coordinate system (shown in Figure 5 of Reference 20) is defined by the nodes that form each element. All reaction forces and moments are calculated in this local coordinate system. Therefore, loads are applied in the global z-direction, and the associated reactions are shown as p_x, M_y, and M_z in the local system.

Loading due to impact in the global z-direction corresponds to the x-direction in the local system. Lateral forces in either the global x or y direction are limited by the honeycomb lateral crush strength and friction forces. These forces were considered in the analysis (Reference 20, page 21).

The allowable bent loadings are given in Appendix 4 of Reference 20 using methods prescribed in the AISC code. Examination of Table A4-1 in Appendix 4 (in which maximum bent stress resultants are listed) indicates that the bent element loadings are well below the allowables for all loading conditions. The bent cross-section is of sufficient size to preclude any local or general instabilities.

Request No. 26

Verify that maximum tensile force in the base plate bolts due to lateral bent loading with upward seismic motion (and no normal impact loading) have been considered and are within allowables.

Response to Request No. 26

The rock bolt and base plate analyses were performed using bent support reactions taken from Table 3.6.1 (Reference 20). These reactions are for the bent under seismic loading combined with the polar crane lumped mass. In Table 3.6.1, the maximum Px reaction force in the upward direction occurs at node 1. This force is 9800 lbs, and produces maximum rock bolt tension. Table 3.4.3 (Reference 20) lists the reactions for the bent system in the freestanding condition. A comparison of the node 1 value in Table 3.4.3 and Table 3.6.1 shows that the freestanding bent is not the limiting case for base plate and bolt loading, thus this case has been adequately considered.

Request No. 27

Discuss the in-service surveillance programs which will be conducted on all safety-related components.

Response to Request No. 27

The in-service surveillance program for all safety-related systems, components, and structures is discussed in the following text. These systems, components, and structures are listed in Table 1 of the "Updated Response to NRC Order to Show Cause Dated 10-24-77", June 1978 (Reference 3) and are listed below in the same order.

1. Reactor Concrete Structure

   The reactor concrete structure consists of the central concrete mass in the containment building. It provides support for other safety related systems, components and structures and containment for the pool and canal.

The concrete structure is massive and under very little stress. Damage to the concrete in areas required to support and/or protect safety related items will not occur. The reactor concrete structure rebar has also been inspected and found to be in excellent condition. Thus, there is no need for routine surveillance of the concrete structure.

2. <u>Reactor Primary Piping and Associated Restraints</u>

The reactor primary coolant system was recently modified and new pipe restraints were added to further assure that seismic induced stresses transmitted to the reactor pressure vessel would be below acceptable levels. Additionally, standpipes were added to the emergency cooling check valves located on the reactor outlet piping to assure that reactor water level (in the pressure vessel) would remain above the reactor fuel even in the highly unlikely event that the pool is drained.

In-service surveillance for the emergency cooling check valve standpipes is discussed in the Response to Request No. 10 of Reference 5.

In-service surveillance of the primary pipe restraints and concrete wedge anchor bolts will be in accordance with the information contained in the Response to Request No. 12 of Reference 5.

3. <u>Reactor Pressure Vessel and Associated Restraints</u>

The reactor pressure vessel and associated restraints consist of the reactor pressure vessel and its lateral restraints. The restraints prevent high stress levels from developing in the reactor pressure vessel (which contains water coolant necessary to keep the reactor fuel cool).

In-service surveillance for the reactor pressure vessel is described on page 4-22 of Reference 21.

In-service surveillance for the pressure vessel lateral supports consists of the following:

a. A visual inspection that the clevis pin and ball lock are properly installed.
b. A test that the ball lock is securely in position.
c. A visual inspection that the clevis pins and cotter keys are properly installed.
d. A visual inspection for any anomolous conditions associated with the lateral supports.

The surveillance will be performed annually.

4. Primary Heat Exchanger Restraints

The primary heat exchanger restraints consist of restraints which prevent heat exchanger movement which could induce excessive load on the primary piping and associated restraints.

In-service surveillance of heat exchanger restraints and concrete wedge anchor bolts will be in accordance with the information contained in the Response to Request No. 12 of Reference 5.

5. Pool Heat Exchanger Restraints

The pool heat exchanger restraints consist of restraints which prevent heat exchanger movement which could result in potential damage to the primary piping and associated restraints.

In-service surveillance of heat exchanger restraints and concrete wedge anchor bolts will be in accordance with the information contained in the Response to Request No. 12 cf Reference 5.

6. Reactor Seismic Scram and Trip System

The reactor seismic scram and trip system consists of the sensors and circuitry to dete:t and initiate 1) a rapid shutdown and depressurization of the reactor, and 2) opening of the Fuel Flooding System admission valves.

In-service surveillance of the reactor seismic scram and trip system consists of the following:

a. The seismic sensors are each functionally tested prior to every reactor cycle startup (average 5 weeks). The test consists of manually tripping the sensor and verifying loss of control rod magnet power, opening of the Fuel Flooding System admission valves and the emergency cooling automatic valves, and closure of the required pressurizer valves. This test is performed with alternate halves of the redundant circuit in bypass each time. Redundant circuit components are checked, then, every second cycle.

b. The seismic sensors are calibrated annually.

7. <u>Control Rods and Associated Drives</u>

The control rods and associated drives assure rapid shutdown of the reactor and serve as part of the water containing boundary needed to keep the reactor fuel cool.

In-service surveillance of the control rods consists of the following tests and checks prior to every cold reactor startup:

a. All switches on the control rod drives are checked. This includes the drive lower limit, drive upper limit, rod seated and rod engaged switches.
b. Rod scram checks are performed where each control rod is raised one at a time and scrammed by tripping a different scram sensor for each rod.
c. A latch integrity test is performed to verify the three control rod components are properly latched. After the control rod components are installed and engaged to the drive, an upward vertical force is applied which exceeds the weight of the components. Improperly latched components will separate.

Visual inspections of control rod components are performed at the following frequencies:

a. Poison sections each time the poison section is removed from the core.
b. Fuel followers prior to initial installation.
c. Shock sections and guide tubes at least once every 15 months.
d. All components when removed from the pressure vessel.
e. Accessible parts of the control rod drives by mechanical technicians quarterly.
f. The water retaining boundary of the control rod drive is inspected prior to every reactor cycle startup.

Drop times for each control rod are measured and recorded after replacement, disassembly or maintenance on any control rod component or the control rod drive or at least once per operating cycle. Drop times are measured both with and without the primary system pressurized and water circulating.

Preventive maintenance is performed on each control rod drive unit at least every 15 months. Preventive maintenance involves disassembly, inspection and rebuilding of the drive by a qualified technician.

The control rod bank reactivity worth is routinely checked at least annually.

8. Reactor Pressure Vessel and Pool Drain Lines and Poison Injection Line and Associated Restraints

The reactor pressure vessel drain line and the poison injection (secondary shutdown) line are connected directly to the reactor pressure vessel (and thus form part of the primary water coolant boundary).

The pool drain line and pool level transmitter tubing are connected directly to the pool and, if failed, could result in loss of pool water. The associated restraints of this small piping protect these lines.

In-service surveillance of the small piping restraints and concrete wedge anchor bolts will be in accordance with the information contained in the Response to Request No. 12 of Reference 5.

9. Canal Fuel Storage Tanks and Associated Structure

The canal fuel storage tanks and associated structures contain the water coolant necessary to keep stored fuel cool. The inner fuel storage tanks are fabricated from 1/4" thick stainless steel and the outer tank is 1/2" thick stainless steel. Under non-seismic conditions, the fuel storage tanks are under no load and cannot be damaged by the light service work performed in the canal. The old fuel storage tanks constructed of 1/4" thick stainless steel were in excellent condition after 10 years of service. Consequently, no in-service surveillance is required.

10. Third Floor Missile Impact System

The third floor missile impact system includes the polar crane impact structures, trolley restraints, pool missile shield restraints and the refueling bridge restraints. The third floor missile impact system protects the reactor pressure vessel, the primary system piping in the pool, canal fuel storage tanks and the Fuel Flooding System supply lines.

In-service surveillance for the third floor missile impact system consists of the following semi-annual visual inspections:

a. The polar crane impact structure columns, column base plates, and honeycomb for general condition, corrosion, and tightness.

b. The crane trolley restraints for general condition, wear, corrosion and tightness.

c. The reactor refueling bridge restraints for general condition, wear, corrosion and tightness.

d. The missile shield lateral restraints for lack of defects and good thread condition.

11. Canal Impact Pad

The canal impact pad is a stainless steel structure installed between the canal fuel storage baskets and the cask loading area in the fuel storage canal. The purpose of this pad is to prevent fuel storage tank damage as a result of a hypothetical cask tip-over. The pad is a heavy stainless steel structure. It is entirely passive and is under no stress. Consequently, no in-service surveillance will be required.

12. Fuel Flooding System

In-service surveillance for the Fuel Flooding System is discussed in the following submittals:

a. Reference 2, Appendix 10

b. Reference 5, Request 10

c. Reference 22, Item 9

13. Valves

Several valves at the GETR are considered to be safety related. These valves, their purpose and in-service surveillance are listed below:

a. The primary system anti-siphon valves prevent the reactor coolant from siphoning in the event of a postulated primary system pipe break.

In-service surveillance is performed semi-annually. The test consists of lowering the water in the reactor pressure vessel to the anti-siphon valve level and visually confirming proper operation.

b. The emergency cooling check valves open the primary system to the pool and provide the inlet path for makeup water from the Fuel Flooding System.

   In-service surveillance consists of testing the check valve disc assembly for freedom of motion and measuring the amount of disc travel. These tests are performed prior to every reactor cold startup.

c. The emergency cooling power-operated valves provide rapid depressurization of the primary system.

   In-service surveillance consists of a functional test every reactor shutdown. The valves are verified to open when the primary pump is shut down. Prior to each cold reactor startup, the valves are reliability tested by tripping open five times each.

d. The primary pressurizer safety related valves include the valve which isolates the pressurizer from the primary system and the valve which isolates the nitrogen supply from the pressurizer.

   In-service surveillance of these safety-related valves consists of a functional test as described in paragraph 6 above, and an annual leak test.

e. The redundant liquid poison system check valves prevent back-leakage of primary coolant following postulated failure of any upstream liquid poison system component.

   In-service surveillance consists of leak checking the valves annually.

f. The reactor pressure vessel drain line valve prevents leakage of primary coolant. The drain line is restrained and capped. In-service surveillance consists of inspecting the water retaining boundary for signs of deterioration or leakage prior to every reactor cycle startup.

g. The capsule coolant system (CCS) anti-siphon valves prevent possible siphoning of pool coolant following postulated failure of an upstream CCS line.

   In-service surveillance consists of leak testing each redundant check valve on four CCS lines every six months. Valve testing is rotated until all valves have been tested in the current test cycle before retesting any valves.

h. The pool drain line valve prevents leakage of pool water. The drain line is restrained and capped. In-service surveillance consists of inspecting the water retaining boundary for signs of deterioration or leakage prior to every reactor cycle startup.

i. The canal emergency recirculating system (ECRS) anti-siphon valves prevent possible siphoning of the canal water following postulated failure of the upstream system piping.

   In-service surveillance consists of a leak test of each redundant valve. The test is performed either in place or on a test bench with the valve removed. This test is performed semi-annually.

j. In-service surveillance for the Fuel Flooding System admission valves, check valves and anti-siphon valves is referenced in paragraph 12 above.

14. The permanent pool shielding restraints prevent postulated failure of the pool shielding and damage to the reactor pressure vessel or primary piping in the pool.

In-service surveillance consists of an inspection for general condition, corrosion and tightness.

Because this shield has been in place for several years under reactor operating conditions, no increased surveillance during reactor restart is necessary. The permanent pool shielding restraints will be inspected annually.

Request No. 28

Justify the acceptability of bolted base plates where the jam nut is placed inside of the main nut. Specifically, verify that the system will not fail at the jam nut when loaded due to vibratory motion, thus unlocking the main nut and allowing it to back off.

Response to Request No. 28

Reference 23 (pages 318, 319 and 320) states that three full threads are all that are required to develop full bolt strength. The jam nuts used on the bolted base plates contain 3-1/2 to 4 threads. Thus, it is acceptable to allow either the jam nut or the main (standard) nut to take the entire load due to vibratory motion. The jam nut may be placed on either side of the standard nut.

REFERENCES

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