GENERAL ELECTRIC

NUCLEAR ENERGY

ENGINEERING

DIVISION

GENERAL ELECTRIC COMPANY, P.O. BOX 460, PLEASANTON, CALIFORNIA 94566

September 21, 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 Request for Additional Information - General Electric Test Reactor - Docket 50-70

Reference: (1) Letter from Robert W. Reid (NRC) to R. W. Darmitzel (GE), dated August 16, 1979

Dear Mr. Reid:

Attachment 1 provides responses to all eight (8) items contained in Reference 1.

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

Very truly yours,

R W Darmitzel

R. W. Darmitzel
Manager
Irradiation Processing Operation

Attach.

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External Distribution
Response to 8 Structural Modifications Questions
of August 16, 1979

9/21/79

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Advisory Committee on Reactor Safeguards

Request No. 1

Describe the procedure used to envelope the computed floor response spectra.

Response to Request No. 1

As described previously in Response to Request No. 7 in Reference 7 and in the Response to Request No. 19 in Reference 8, a very conservative procedure was used to develop envelope design spectra from the raw computed floor response spectra presented in Reference 1. The influences of potential uncertainties and variabilities in the soil properties and the modeling techniques, as well as the influence of potential nonlinearities due to sliding and uplift (overturning due to rocking), on the magnitude and the location of the peaks in the floor response spectra were conservatively estimated. The peaks were then raised and broadened to take these influences into account. As a result, the peak of the horizontal floor response spectrum was raised by about 60 percent, and was broadened by about 30 percent into the lower frequency range and 60 percent into the higher frequency range. Similarly, the peak of the vertical floor response spectrum was raised by about 60 percent, and was broadened by about 30 percent into the lower frequency range, and 100 percent into the higher frequency range. This procedure used for developing the envelope design spectra was consistent with but considerably more conservative than the procedure recommended by the U. S. Nuclear Regulatory Commission (Ref. 2 and Ref. 6).

Request No. 2

Verify that an earthquake smaller than the design 0.8g earthquake would not produce a larger structural response than the design earthquake, taking into consideration the changes in damping values and other factors. Provide the level of vibratory ground motion for which the reactor will be shut down and detailed investigations carried out to determine that no damage has occurred to safety related systems, prior to resuming operations.

Response to Request No. 2

The main seismic analyses were performed for an effective peak ground acceleration (epga) of 0.8g. In addition, however, similar analyses were also performed for an epga of 0.6g to investigate the influence on the response of the reactor building of a lower level earthquake. It was found that the smaller earthquake produced lower structural response. Other, still smaller earthquakes will not produce higher responses.

The reactor is automatically shut down (scrammed) whenever the peak ground acceleration exceeds .01-.03g. Whenever the peak ground acceleration* exceeds a predetermined low value (to be specified by General Electric at a later date), detailed investigations will be completed on all non-seismic safety related systems to assure that no damage has occurred. Whenever the peak ground acceleration exceeds 0.4g*, detailed investigations will also be completed on the seismic safety related systems to assure that no damage has occurred. The seismic safety related systems, which are designed to withstand 0.8g peak ground acceleration, include all systems necessary to mitigate the maximum postulated seismic event. These seismic safety related systems are listed in Table 1 of Reference 4.

* As measured by the reactor strong motion recorder.

Request No. 3

Provide information detailing how the modifications to the computer programs used in the analysis of the reactor building were qualified.

Response to Request No. 3

EDAC computer programs are controlled, documented, and protected to prevent unauthorized and unverified modifications, and to ensure that the current version of a program is used on all projects. For each EDAC computer program, a Quality Assurance File is maintained. The program documentation, which includes a detailed description of its development, list of modifications, sample calculations, and a program source listing, is retained in the Quality Assurance File. Each EDAC program is identified with a program name, revision number, level number, and revision modification date. This information is printed at the beginning of each computer output produced by the program to ensure that the current version of the program is used.

Modifications to any EDAC computer program are performed strictly following the procedures set forth in the EDAC Quality Assurance Manual. Only qualified and experienced personnel are assigned to make any modifications to an EDAC computer program. After the modifications are made, the validity of the computer program is verified by the use of sample problems for which either theoretical solutions or analytical solutions from other computer codes are available. After modifications, the program is identified by its name and modification date. The modified version of the program and the verification problems are documented, and the documentation is kept in the program Quality Assurance File.

The Quality Assurance procedure described above was used for the modifications in the computer programs used in the analyses of the reactor building (Ref. 1).

Request No. 4

You stated the re-analysis of the piping and RPV system was based on the original analytical model. Did the model used in the piping and RPV systems re-analysis reflect the as-built conditions?

Response to Request No. 4

The model used in the primary piping and RPV systems re-analysis represents the conditions as they now exist. The piping restraints which were added to the system were included in the mathematical model for the re-analysis.

Request No. 5

The calculation for the polar crane impact structures are for the end of the crane impacting the structure. In some conditions (perpendicular to the position shown in figure 2 of the document, "Structural Analysis of Third Floor Missile Impact System, General Electric Test Reactor") the polar crane would be supported by the structure over the reactor itself and then only on one side of the bridge girder. Also, the polar crane can be supported by only one bridge girder on bent 1. Provide your calculations to show the impact structures are capable of withstanding the impact of the polar crane for this condition.

Response to Request No. 5

The GETR polar crane bridge assembly is normally supported at each end by a rail system mounted on top of a perimeter box beam. This box beam is supported from the third floor by crossbraced columns as discussed in Appendix 9 of Reference 4 and shown in Figure 1 of Reference 5.

Because of the possibility that the polar crane bridge assembly could derail during a postulated seismic event, polar crane impact structures have been installed to prevent impact of the polar crane bridge assembly with other safety related systems, components, and structures. These polar crane impact structures are described in Appendix 9 of Reference 4 and in Reference 5.

If derailment/dislocation is postulated, one end of the polar crane bridge could lose support and drop until it impacts one of the polar crane impact structures, the 3rd floor elevator structure, or both. Because of the various configurations in which dislocation could be postulated, a scale model of the third floor area was constructed to carefully examine the reaction of the polar crane bridge assembly. Photo No. 1 shows the model as depicted in Figure 2 of Reference 5. The four structural bents (which comprise the polar crane impact structures), the elevator structure, the bridge crane and trolley are shown.

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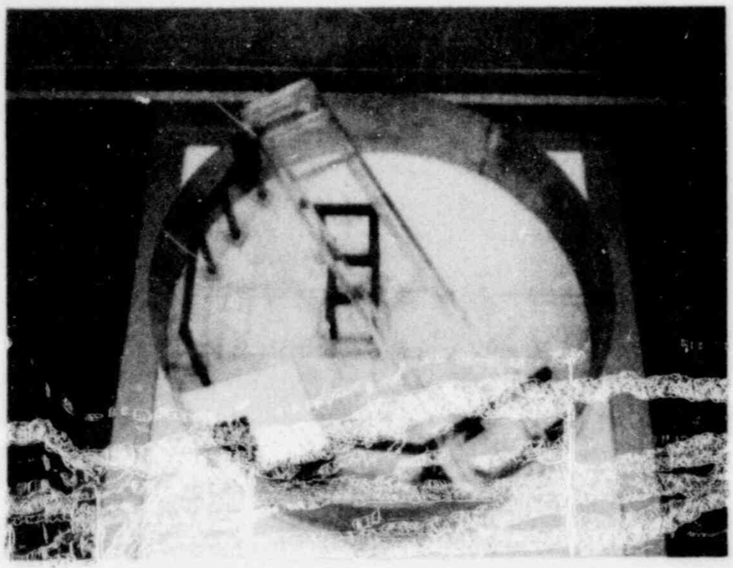

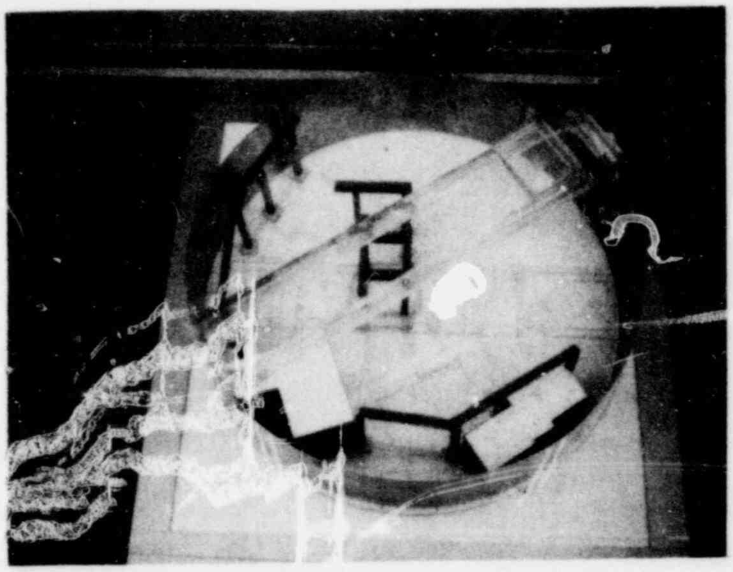

Photo #1. 3rd Floor Model (Bridge Assembly Orientation same as shown in Figure 2 of Reference 5)

Photo #2. 3rd Floor Model (Bridge Assembly Perpendicular to Photo #1 Orientation)

For most orientations the bridge assembly will not fall from the polar crane impact structures after the initial impact; however, there are some orientations where a subsequent fall is possible. The polar crane impact structures are designed to assure that these postulated fall trajectories are always away from the central core area of the reactor building. In this way, postulated missiles are directed away from any safety related items located on the third floor.

For example, Photo No. 2 shows the bridge assembly model rotated 90^{o} from the position shown in Photo No. 1. In this rotated position, the west end of the bridge assembly is over both the elevator and bent no. 1, and the center of the bridge assembly is over the bent structure covering the missile shield

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If the end of the bridge assembly over the elevator and bent no. 1 is postulated to derail and move radially outward, the opposite end of the bridge assembly could lose peripheral support causing at least one of the bridge girders to impact the missile shield bent. If the other bridge girder did not make contact, the bridge assembly would then rotate away from the central core area as shown in Photo No. 3. The loads imposed on the missile shield bent because of the foregoing postulated derailment case are of less consequence than the case selected for analysis in Reference 5. For the derailment case discussed above, only half of the polar crane bridge assembly impacts the bents, which produces significantly lower impact loading than the case considered in reference 5. Due to the structural configuration of the missile shield bent, the loading associated with the case postulated above would cause lower stresses as compared to the case analyzed in reference 5. Similarly, if an impact on the south bent is postulated, the resulting loadings and impact configurations would be less critical than those considered in reference 5.

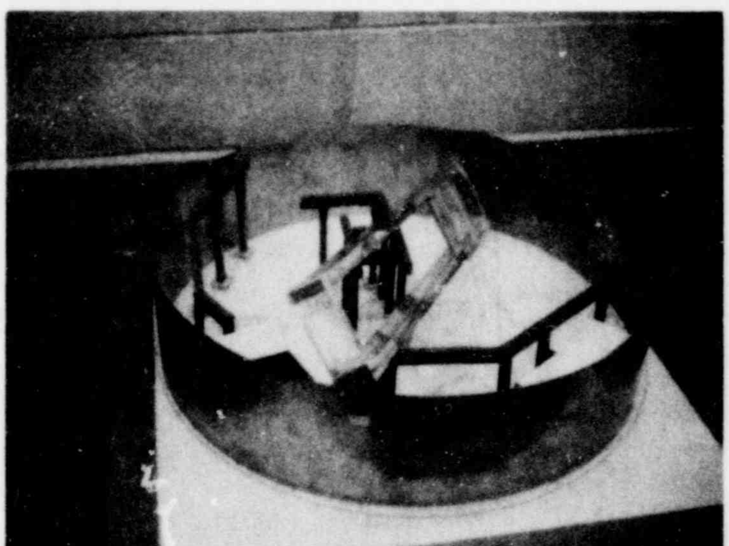

Photo #3. Resulting Bridge Position

Alternatively, if the end of the bridge assembly over the elevator is postulated to derail and move radially inward, the end of the bridge crane will impact bent 1 and the elevator structure as shown in Photo No. 4. Again, this bent loading is of less consequence than the case considered in reference 5. The effect on the elevator structure is analyzed in Appendix 1 of reference 5 and it is shown therein that the postulated impact of the polar crane assembly can be taken by the elevator structure without excessive deformation.

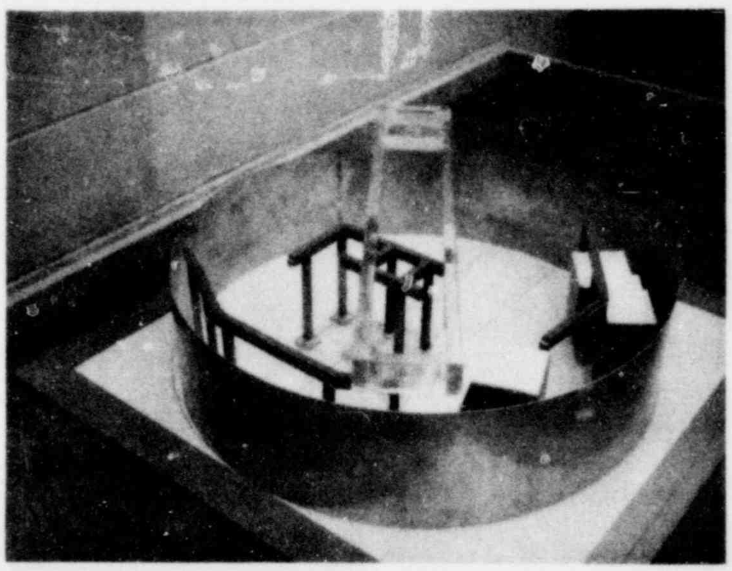

Photo #4. Postulated Dislocation of the Bridge Over the Elevator & Bent 1

Request No. 6

It appears that the polar crane can be in such a position that only one of the bridge girders will impact the polar crane impact structure. Show that the polar crane will not topple off the impact structure and the structure will support the crane with only one bridge girder resting on the supporting structure.

Response to Request No. 6

The scale model of the GETR third floor area and polar crane bridge assembly (discussed in the Response to Request No. 5) was constructed to insure that loss of crane support in any orientation was fully examined. There are some orientations in which postulated dislocation and derailment would result in the impact of only one girder of the bridge assembly on the bent system. In all cases of single girder impact, the third floor model clearly demonstrates that all fall trajectories would be away from the central core area of the reactor building. Since all safety related items (Fuel Flooding System, Fuel Storage System, etc.) are located in the central core area, a fall of the bridge crane would not impact or damage safety related equipment.

Request No. 7

The elastic half space theory used to determine soil springs assumes that the foundation slab is rigidly welded to the elastic half space. Provide your bases for using a fraction of the contact area for computing the soil springs. In addition, discuss how the failure of portions of the embedded structure (e.g., foundation walls) affects the elastic half space soil springs developed to account for the embedment effects. Provide a justification for including the effects of total embedment, and discuss the affects on the resulting structural responses if embedment effects were not considered.

Response to Request No. 7

The elastic half-space theory used to determine soil springs employed for the linear elastic analyses described in Ref. 1 assumes that the foundation slab is rigidly connected to the elastic half space. On the basis of preliminary calculations, it was found that when subjected to a high magnitude earthquake, the reactor building could undergo small rocking oscillations with computed soil pressures falling to zero or near zero on portions of the foundation slab, resulting in an effectively reduced area of contact between the foundation slab and the underlying soil. This area of contact could vary with time as the amplitude of earthquake motion varied.

For the linear elastic analyses described in Ref. 1, it was assumed that the average "effective" area of contact between the foundation slab and the underlying soil would be 75 percent of the total area. This implied that at all times the area of contact between the foundation slab and the underlying soil was 75 percent of the total area of contact, and this reduced area was considered to be rigidly connected to the soil foundation. Detailed nonlinear analyses were performed using nonlinear models A and B (described in Ref. 1) to determine the influence on the structural response of a potential reduction in the contact area between the foundation slab and the underlying soil, and to validate the use of the assumed contact area (i.e., 75 percent). These nonlinear analyses showed that the linear elastic analyses were conservative.

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The effects of embedment were included in the linear elastic analyses. The influence of the exclusion of embedment effects on the response of the reactor building structure was, however, investigated as part of the parametric studies. It was found, as discussed on page 2-9 of Reference 1, that with the complete exclusion of the embedment effects, the forces in the structure could increase by approximately 10 to 12 percent. However, it is unrealistic to assume that there would be no embedment effects at all. At the very least, partial embedment resistance will always be present. The influence of the inclusion of such partial embedment effects (as compared to the total embedment effects) on the response of the reactor building structure would be less than about 5 percent and would therefore be negligible.

Request No. 8

Recent tests conducted by PCA ("Tests to Evaluate Coefficient of Static Friction Between Steel and Concrete, February 1979") indicate that the coefficient of friction between steel and concrete is lower than the value used in your analyses, and that the measured coefficient of friction for wet and dry concrete were different. Verify that the reactor building is stable and will not slide considering these reported friction values, and discuss the appropriateness of using the wet or dry values.

Response to Request No. 8

The recent tests conducted by Portland Cement Association (PCA), as presented in Ref. 3, indicate that the coefficient of friction between steel and concrete varies between 0.57 to 0.7 for different levels of normal stress, as well as for wet and dry concrete. In the analyses presented in Table 2-9 of Ref. 1, a value of 0.7 was used for the coefficient of friction between concrete and steel. If the lower values of the coefficient of friction obtained by PCA are used instead of a value of 0.7, the conclusions do not change. With the use of lower PCA values of the coefficient of friction, "the sliding force available" would still be smaller than the "sliding force required", and there would be no sliding at the interior concrete-foundation slab interface. These conclusions are valid for both wet and dry values of the coefficient of friction.

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REFERENCES

1. Engineering Decision Analysis Company, Inc., "Seismic Analysis of Reactor Building, General Electric Test Reactor -- Phase 2, EDAC-117-217.03", prepared for General Electric Company, 1 June 1978.

2. United States Nuclear Regulatory Commission, Standard Review Plan 3.7.2, June 1975.

3. Rabbat, B. G., and H. G. Russell, "Tests to Evaluate Coefficient of Static Friction Between Steel and Concrete," prepared by Construction Technology Laboratories, Portland Cement Association, Skokie, Illinois, February 1979.

4. "Updated Response to NRC Order to Show Cause Dated 10-24-77", submitted by General Electric Company to the NRC on July 20, 1978.

5. "Structural Analysis of Third Floor Missile Impact System, General Electric Test Reactor" - Structural Mechanics Associates, June 1978, by Dr. H. Durlofsky.

6. United States Nuclear Regulatory Commission, Regulatory Guide 1.122, September 1976.

7. General Electric Response to NRC Request for Additional Information on the Phase II Report, dated 26 July 1978.

8. General Electric Response to NRC Request for Additional Information, dated 5 September 1979.