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An Exelon/British Energy Company

10 CFR 50.90

November 30, 2001
2130-01-20244

U.S. Nuclear Regulatory Commission
Attn: Document Control Desk
Washington, DC 20555

Subject: Oyster Creek Generating Station
Docket No. 50-219
Facility Operating License No. DPR-16
Technical Specification Change Request No. 281
Supplement 2 to Response to Request for Additional Information

References: 1) AmerGen Letter No. 2130-01-20042 dated April 4, 2001, "Technical Specification Change Request No. 281"

2) NRC Letter dated August 24, 2001, "Oyster Creek Nuclear Generating Station – Request for Additional Information on Technical Specification Change Request No. 281 – Heavy Loads Over Irradiated Fuel (TAC No. MB1747)"

3) AmerGen Letter No. 2130-01-20211 dated October 12, 2001, "Technical Specification Change Request No. 281, Response to Request for Additional Information"

4) AmerGen Letter No. 2130-01-20241 dated November 28, 2001, "Technical Specification Change Request No. 281, Supplement to Response to Request for Additional Information"

In Reference 1 AmerGen Energy Company, LLC (AmerGen) requested a change to the Technical Specifications contained in Appendix A to the Facility Operating License regarding restrictions on handling heavy loads over irradiated fuel in the spent fuel storage pool at Oyster Creek. Reference 3 provided AmerGen's response to the additional information requested in Reference 2. Enclosure 1 to this letter provides an additional supplement to Reference 3 and additional information concerning the AmerGen responses to NRC Questions 1, 5 and 6 in

A001

Reference 4 requested by and discussed with the NRC staff on November 29, 2001. Enclosure 2 to this letter contains revised pages for Enclosure 1 of Reference 3. Areas of revision are indicated by vertical lines in the right-hand margin.

Should you have questions or require additional information please contact Mr. Paul F. Czaya at 609-971-4139.

I declare under penalty of perjury that the foregoing is true and correct.

Very truly yours,

11-30-01
Executed On

Ron J. DeGregorio
Vice President
Oyster Creek

Enclosures: 1) Response to Request for Additional Information Supplement 2
2) Revised Pages for October 12, 2001 AmerGen Letter 2130-01-20211

c: H. J. Miller, Administrator, USNRC Region I
L. A. Dudes, USNRC Senior Resident Inspector, Oyster Creek
H. N. Pastis, USNRC Senior Project Manager, Oyster Creek
File No. 01037

Enclosure 1

Oyster Creek Generating Station

Technical Specification Change Request No. 281

Supplement 2 to Response to Request for Additional Information

NRC Questions 1, 5 and 6 below refer to the questions contained in Enclosure 1 to Reference 4 identified in the cover letter. They are supplemented by additional information requested by the NRC staff on November 29, 2001.

NRC Question 1

Attachment G, Page G3: Is the live load value of 7 psf, which includes snow load, consistent with the reactor building roof snow load documented in the FSAR?

AmerGen Response

The Oyster Creek Updated Final Safety Analysis Report does not specifically address allowable snow load on the reactor building roof. A roof load of 20 pounds per square foot (psf) was considered in the finite element dynamic analysis. Roofing materials account for 13 psf. This leaves 7 psf for other possible loads, including snow. The American Institute of Steel Construction (AISC) Manual of Steel Construction specifies a recommended design live load of 12 psf. This exceeds the 7 psf snow load considered for the roof. A further static analysis of the reactor building roof structure, with the additional 5 psf live load, determined it to be adequate for loading combinations of dead load plus live load plus the operating basis earthquake (most limiting case). The roof member interactions are summarized in the revised response to NRC Question 9(d) in Enclosure 1 and Attachment G (pages 4 – 7 are revised) of Reference 3, which are contained in Enclosure 2 of this letter. Note that the revised table of interaction ratios computed in the original analysis has been expanded and corrected. The previous values for roof truss chords and bracing were transposed and rounded (the 1.00 value should have been 0.99).

NRC Questions 5 and 6

Attachment H, Page H10: Discuss the behavior of the member in question in relation to its intended function.

AmerGen Response

The function of the end truck connector member shown in Figure 1 of Reference 4 is to maintain a constant horizontal separation distance between end trucks. The member was conservatively modeled in the finite element model with no end releases. This member should have been modeled as pinned at each end. A pinned end truck connection member would only be subjected to axial tensile and compressive forces. The high interaction ratio was the result of high bending stresses due to rotational restraint at the member ends. With pinned connections the high bending stresses will not be present and thus the results from the tension or compression stresses is acceptable.

Enclosure 2

Oyster Creek Generating Station

Technical Specification Change Request No. 281

Revised Pages
for
Enclosure 1
of
AmerGen Letter No. 2130-01-20211
dated
October 12, 2001

AmerGen Response

The response spectra used in the analysis were the Oyster Creek licensing basis in-structure response spectra at the operating floor of the reactor building.

Q9(d) In a tabular form, provide the maximum analysis results (i.e., stress and deformation) and the corresponding locations on the building structure.

AmerGen Response

The analysis was a linear elastic analysis. The stresses in the structure elements were checked against normal AISC allowable stress for DL+LL+OBE, and 1.6 times normal AISC allowable stress for DL+LL+SSE per the Standard Review Plan. Element deformations were not checked explicitly as they would necessarily be within the elastic range.

The maximum interaction ratios and computed displacements for the various elements of the steel superstructure are shown in Section 2 of Attachment G as revised:

deleted

Q9(e) Indicate whether the crane structure was included in this superstructure analysis. If not, provide responses to items (a), (b) and (d) above for the crane structural analysis.

The design load of the building siding is 10 psf. This plus the weight of the secondary framing members was distributed to the column joints.

The dead load of the main building members included in the model was calculated and distributed by the computer program.

The dead weight of the crane was distributed as shown on Figure 6. The 105 ton live load on the hook was added to the trolley weight for the vertical direction. Crane impact loads for starting and stopping of the crane motors are short duration transient load, and they are not considered to be coincident with the maximum earthquake loads in the steel structure.

A static analysis was performed to calculate joint displacements and element stresses. Load factors for dead and live load were unity.

A separate static analysis of a roof truss was performed to assess the effect of a total live load of 12 psf.

Seismic Analysis

The seismic stresses in the building elements were calculated by a linear elastic response spectrum analysis.

A modal analysis of the model was performed using the Load Dependent Ritz Vector method, and 50 modes were computed. The modal frequencies ranged from 2.35 Hz to 71.94 Hz. There were 46 frequencies below 33 Hz. The computed modes gave more than 98% mass participation in each direction.

Response spectrum analyses were performed for the operating basis earthquake (OBE) and the safe shutdown earthquake (SSE). The modal damping ratios were 4% for OBE and 7% for SSE, from the OCGS Updated Final Safety Analysis Report (UFSAR) for bolted steel structures. 50 modes were included in the analysis, giving mass participation exceeding 98% in each direction.

The seismic inputs were the OBE and SSE floor response spectra from the OCGS UFSAR, at 4% (OBE) and 7% (SSE) damping, for the reactor building operating floor at El. 119'-3". There are separate smoothed and peak broadened spectra for the N-S, E-W and vertical directions.

Individual modal element stress contributions were initially combined by the absolute sum method for each direction of excitation, and directional responses were then combined by SRSS in accordance with Regulatory Guide 1.92. If an individual member stress exceeded the acceptance criteria using the absolute sum results, the member stress was recalculated using the 10% grouping method of Section 1.2.1 of Regulatory Guide 1.92.

Acceptance Criteria

The acceptance criteria were based on review of the OCGS UFSAR and NUREG-0800 (SRP). There were two load combinations:

1. DL + LL + OBE

   Acceptance criteria are from Table 3.8-5 of the OCGS UFSAR

   Tension: 0.60 Fy on net section
   Shear: 0.40 Fy on gross section
   Compression: varies with slenderness ratio
   Combined axial and bending: 0.60 or 0.66 Fy, reduced for members with excessive unbraced compression flange length in accordance with AISC specification

   The above conforms to AISC 9th Edition without 33% increase in allowable for earthquake load.

   Allowable stress for A325 bolts from AISC Table J3.2, with threads excluded from shear plane:

   Tension: 44 ksi (0.62 Fy)
   Shear: 30 ksi (0.42 Fy)

2. DL + LL + SSE

   Acceptance criteria are from NUREG-0800, Section 3.8.4, Equation 3.c.ii.a.1.

   Stresses are compared to 1.6 times normal allowable stress of Part 1 of AISC Specification.

Results

The following table shows the maximum stress results for the key sections of the model when 7 psf live load is considered. They are in terms of the AISC interaction ratio for combined axial and bending stress.

	DL+LL+OBE	DL+LL+SSE
Columns	0.94≤1.0	1.16≤1.6
Column Bracing	0.77≤1.0	1.05≤1.6
Side Panel Girts	0.72≤1.0	0.98≤1.6
Top Edge Beams	0.20≤1.0	0.27≤1.6
Crane Rail Girder	0.96≤1.0	1.15≤1.6
Roof Trusses:		
E-W Top Chords	0.53≤1.0	0.68≤1.6

E-W Bottom Chords	0.43≤1.0	0.54≤1.6
E-W In-Plane Bracing	0.80≤1.0	1.03≤1.6
N-S Top Chords	0.98≤1.0	1.37≤1.6
N-S Bottom Chords	0.87≤1.0	1.21≤1.6
N-S In-Plane Bracing	0.98≤1.0	1.09≤1.6
Horizontal Bracing	0.99≤1.0	1.23≤1.6

The following are the maximum computed displacements (N-S, E-W, Vertical) in inches relative to the operating floor (absolute sum of modal displacements):

	OBE	SSE
Roof Truss Joints	0.39, 0.44, 0.26	0.51, 0.64, 0.37
Crane Runway/Bridge	0.95, 1.07, 0.48	1.22, 1.45, 0.61

An additional static analysis was performed to compute stresses in the superstructure due to an additional distributed load on the roof of 5 psf. This was to assess the superstructure roof truss system for a minimum live load of 12 psf, which is 5 psf more than the live load included in the seismic analysis. Since the roof load is primarily carried by the E-W trusses, a typical truss line was analyzed and AISC interaction ratios computed for the members. These interaction ratios are shown below and are combined with the interaction ratios from the table above (DL+LL+ OBE is limiting.)

	Added 5 psf live load	DL+LL+OBE
Columns	0.02	0.96≤1.0
E-W Top Chords	0.10	0.63≤1.0
E-W Bottom Chords	0.05	0.48≤1.0
E-W In-Plane Bracing	0.07	0.87≤1.0

The column bases (refer to attached drawing BR 4200) were checked as follows:

- All base plates have shear lugs to transfer base shear in the N-S direction. The maximum base shears in the N-S direction were 70.9 kip OBE and 96.9 kip SSE. The allowable shears in the shear lugs were 95 kip OBE and 153 kip SSE. The weld of the shear lug to the base plate controlled.

- Base plate types I, Ia and III have shear lugs to transfer base shear in the E-W direction. The maximum base shears in the E-W direction were 74.2 kip OBE and 95.8 kip SSE. The allowable shears in the shear lugs were 95 kip OBE and 153 kip SSE. The weld of the shear lug to the base plate controlled.

- Shear in the E-W direction of base plate types II, IV, IVa and V are transferred by the anchor bolts. The maximum base shears for these columns were 22.3 kip OBE and 26.7 kip SSE. The allowable shear in the anchor bolts was 70.8 kip

OBE and 1.6x70.8 kip SSE based on AISC Table I-D for 1½-inch diameter A307 bolts in single shear.

- Vertical uplift loads are transferred by the anchor bolts in tension. The only base plates with uplift are types I, Ia, IV and IVa. The maximum uplift was 74.7 kip OBE and 126.6 kip SSE. The allowable tension load on the anchor bolt group was 105 kip OBE and 168 kip SSE, determined from AISC nominal capacity for 1½-inch diameter A307 bolts and reduced for spacing by the overlapping shear cone method. The bolt embedment was sufficient to develop the bolt nominal capacity. The base plate thickness was sufficient to preclude prying action.

- Shear-tension interaction for column anchor bolts transferring both shear and tension loads was checked using the tri-linear AISC shear-tension interaction formulation. The worst case shear force combined with the worst case tension force was found to lie within the bounds of the diagram.

The structural steel connections were checked by comparing the bounding DL+LL+OBE or DL+LL+SSE bolt load for each connection detail to the normal AISC bolt allowable bolt load. The bounding cases were:

- Column cross-bracing: W14x43 with ten ⅞-inch diameter A325 bolts per connection with threads excluded from the shear plane. Maximum bolt shear force is 8.7 kip versus AISC normal allowable of 18 kip.

- Roof truss framing: 5x3x⅜ double angle with four ⅞-inch A325 bolts. Maximum bolt shear force is 15.7 kip versus AISC normal allowable of 18 kip.

- Crane runway girder to column (bottom): four bolt stiffened beam seat connection (see Figure 2) with 1-inch A325 bolts. Maximum bolt shear is 10.7 kip versus AISC normal allowable of 23.6 kip, maximum bolt tension is 23.8 kip versus AISC normal allowable of 30.8 kip using AISC Table J3.3 for allowable tension with shear.

- Crane runway girder to column (top): a tie plate connects the top of the crane runway girder to the upper column section (element type 20 in Figure 2). The tie plate stress exceeded the acceptance criteria. A modification to the tie plate was designed so it would meet the criteria (Figures 7 and 8). After the design was completed, the model was revised to reflect the modified tie plate. The modification had no effect on the modal frequencies of the model or the resultant forces in the tie plate.

The elements representing the crane end truck wheels and the trolley wheels were checked for tension loads, which would indicate that the wheels would lift off the rails. The element forces were checked for the cases of DL+LL+OBE (up) and DL+LL+SSE (up). All of the forces were compressive, indicating that the wheels would not lift off.