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Vogtle Project

February 14, 1985

Director of Nuclear Reactor Regulation
Attention: Ms. Elinor G. Adensam, Chief
Licensing Branch #4
Division of Licensing
U. S. Nuclear Regulatory Commission
Washington, D.C. 20555

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NRC DOCKETT NUMBERS 50-424 and 50-425
CONSTRUCTION PERMIT NUMBERS CPPR-108 and CPPR-109
VOGTLE ELECTRIC GENERATING PLANT - UNITS 1 AND 2
SUPPLEMENTAL INFORMATION-STRUCTURAL AUDIT
AND DESIGN REVIEW

Dear Mr. Denton:

Attached please find for your review five (5) copies of supplemental information requested by your staff during the structural audit and design review conducted on December 4-6, 1984. As noted in the attachment, information appropriate for the FSAR will be included in FSAR Amendment 14 scheduled for submittal on February 15, 1985.

If your staff requires any additional information, please do not hesitate to contact me.

Sincerely,

J. A. Bailey

J. A. Bailey
Project Licensing Manager

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Attachment

VOGTLE ELECTRIC GENERATING PLANT - UNITS 1 AND 2
DOCKET NOS. 50-424 AND 50-425

NRC SER AUDIT MEETING
STRUCTURAL AND GEOTECHNICAL ENGINEERING BRANCH

Tuesday; December 4, 1984

1. Include as part of the FSAR, Appendix 3D, all items of the November 13, 1978 GPC letter to the NRC (confirmatory study, sensitivity study, and methodology to account for torsion caused by the seismic wave propagation effects). In addition, include therein a comparison of the VEGP design in-structure response spectra (e.g., envelope of N-S and E-W response spectra considered applicable for any two mutually orthogonal horizontal directions) with the response spectra provided in the confirmatory study, and the resulting conclusions.

Response

FSAR Appendix 3D, revised to include the above information, will be provided in Amendment 14.

2. Provide justification for using the component factor method (1.0, 0.4, 0.4) in lieu of the square root of the sum of the squares (SRSS) method for consideration of three component earthquake effects.

Response

Closed out at meeting on 12/6/84

3. Provide the basis for the equation used in determining the rotational mass moment of inertia in the containment model.

Response

Closed out at meeting on 12/6/84

4. Provide the basis for concluding that, for the containment basemat design, the combination including 100 percent of the vertical seismic loads in the component factor method does not control over the other combinations including 100 percent of the seismic loads in either horizontal direction.

Response

For the VEGP containment basemat design, the total structural response for seismic loading is computed by combining the maximum codirectional responses from the two horizontal and the vertical load cases. The combining is done according to the "component factor method" as follows:

$$R_{total} = \mp R_i \mp .4R_J \mp .4R_k$$

where R_i R_j, and R_k are the set of three codirectional responses due to the individual excitation in three directions. To satisfy the component factor method, the following responses must be considered:

Eq. 1) $R_t = \mp R_x \mp .4R_y \mp 4R_z$

Eq. 2) $R_t = \mp .4R_x \mp 1.0\ R_y \mp .4R_z$

Eq. 3) $R_t = \mp .4R_x \mp .4R_y \mp 1.0R_z$

where R_t is the total seismic response (shear, membrane force or moment). R_x, R_y, and R_z are the seismic response due to excitation in the two horizontal (x and y and vertical (z) directions.

It was concluded that equation 3 does not control over equations 1 and 2. The basis for this conclusion is an examination of the shears, membrane forces, and moments resulting from the individual x, y, and z seismic load cases for the elements that control the design. The examination showed that the maximum response (shear, membrane or moment) due to seismic forces in the horizontal directions are greater than the maximum response due to seismic forces in the vertical direction. Thus, equation 3 would yield smaller values than equations 1 and 2 and, therefore, does not control.

Wednesday; December 5, 1984

5. Provide justification for the use of 25 percent of the design live load in the containment internal structure design for the load combination involving earthquake load effects. Provide similar justification for the control building basemat.

Response

The maximum live load intensity for any given grating area, floor slab panel or basemat panel is identified as the Design Live Load. It is based on the maximum probable load enveloping operation, shutdown, and maintenance periods. Live load accounts for only those loads that vary with intensity and occurrence, whereas dead load includes all permanently attached items (e.g., substructures, platforms, equipment piping, raceway, conduit, HVAC, and electrical control panels and consoles). In addition, the design live load is based on loads that are likely to occur only in localized areas and are therefore even more conservative when applied to the whole panel area.

The magnitude of live load on a panel area should be consistent with the anticipated level of plant activity for that particular panel area, as well as be consistent with the general plant condition (e.g., operating or shutdown condition). The plant activity for certain floor areas or slab panels (e.g., the control room and locker areas in the control building, and the concrete operating deck laydown areas in the containment building) warrants the application of 100 percent of the design live load due to either an active occupancy function or laydown requirement regardless of load combination. However, the use of 25 percent is equally justified as a conservative level of live load intensity for panels that do not have any specific occupancy and/or laydown function under normal operating conditions (e.g., containment building grating areas and control building basemat). Accordingly, the use of 25 percent was exercised for load combinations involving earthquake load effects. Similarly for the shutdown condition, the 25 percent level of live load intensity is also justified, provided that the integrity of the panel during this plant condition can not affect plant safety. Otherwise the panel or structural element is designed using 100 percent of design live load (e.g., reactor head laydown area panels).

The methodology used on VEGP to utilize the appropriate intensity level of the design live load (i.e., 100 percent or 25 percent), for load combinations involving earthquake, is a reasonable and conservative design practice since it accounts for the anticipated level of plant activity for any given floor area or panel, at the same time recognizing the plant operating condition and the potential impact on plant safety during the shutdown condition. This methodology is applied through an engineering evaluation process that considers all of the above factors.

6. Select the worst case for the containment internal structure steel beam-to-column connections and demonstrate the adequacy of the connection and column design considering the moment resistance introduced by the connecting gussets.

Response

An evaluation was performed to account for the moment resistance introduced by the gusset plates connecting diagonal bracing. The evaluation, which included the worst case loading and joint configuration, verifies that each component of the connection is adequate to resist the additional induced forces.

7. Provide the basis for the conclusion that the OBE loading combination governs the design of slabs in the auxiliary building, rather than the SSE condition.

Response

The basis for concluding that for the auxiliary building slab design under normal, severe environmental and extreme environmental conditions, the load combinations with OBE earthquake load control over the load combinations with SSE earthquake load is provided below. The areas subjected to abnormal loads (e.g., main steam isolation valve room) were investigated for both OBE and SSE earthquake load combinations.

The equations of load combinations are shown on Table B.2 of VEGP - Auxiliary Building Design Report (See Attachment No. 1). The equations which include OBE and SSE earthquake loads are equation numbers 3, 6, 7, 10 and 11. Equations 10 and 11 address abnormal loading conditions in combination with OBE and SSE.

As stated earlier, all areas subject to abnormal loading conditions are designed for both EQ 10 and EQ 11. In the absence of abnormal loads, equations 6 and 7 envelop equations 10 and 11. Therefore, for areas not subject to abnormal loads, only the following load combinations need be considered:

(EQ 3) $U = 1.4D + 1.7L + 1.9E$
(EQ 6) $U = 1.05D + 1.275L + 1.275\ T_o + 1.425E + 1.275R_o$
(EQ 7) $U = D + L + T_o + E' + R_o$
where E = OBE loads and E' = SSE loads

In the VEGP auxiliary building design, load combinations represented in EQ 3 and EQ 6 are addressed in one equation as follows:

(EQ. 6A) $U = 1.4D + 1.7L + 1.3T_o + 1.9E + 1.3R_o$

The absence of T_o and R_o is also considered if their effects reduce the effects of other loads.

By comparison of equations 6A & 7 it is observed that except for the load factors and earthquake terms (E and E'), all other terms are identical. The load factors for loads other than seismic for EQ 6A are greater than the corresponding load factors in EQ 7. Therefore, by comparing the effects of 1.9E (OBE Conditions) and 1.0E' (SSE Conditions), the controlling loading case can be determined.

Table 1 provides the comparison of OBE accelerations multiplied by its load factor 1.9 with the SSE accelerations multiplied by its load factor 1.0.

As observed from this comparison, the auxiliary building OBE accelerations with 1.9 load factor are greater than the SSE accelerations with 1.0 load factor at all levels. Therefore, under other than abnormal loading conditions, the OBE loading combination (equation 6A), rather than the "SSE" load combination (equation 7), governs the design of slabs in auxiliary building.

TABLE 1

AUXILIARY BUILDING SEISMIC ACCELERATION VALUES

Level	Elevation	Floor Accelerations (g's) (1)					
		1.0 X SSE(2)			1.9 X OBE(2)		
		E-W	N-S	Vert.	E-W	N-S	Vert.
Level D	119'-3"	0.18	0.19	0.29	0.21	0.23	0.34
Level C	14[illegible] -6"	0.19	0.19	0.29	0.2[illegible]	0.23	0.36
Level B	170'-6"	0.21	0.22	0.30	0.25	0.27	0.36
Level A	195'-0"	0.22	0.25	0.30	0.27	0.30	0.36
Level 1 (grade level)	220'-0"	0.24	0.28	0.30	0.29	0.34	0.38
Level 2 East Wing	240'-0"	0.26	0.33	0.36	0.30	0.40	0.44
Level 2 West Wing	240'-0"	0.26	0.33	0.36	0.30	0.40	0.44
Level 3 East Wing	260'-0"	0.26	0.34	0.36	0.32	0.42	0.44
Level 3 West Wing	260'-0"	0.26	0.34	0.36	0.32	0.42	0.44
Level 4	288'-2"	0.38	0.36	0.36	0.48	0.46	0.44

(1) The actual acceleration values used in the design of the structure may be higher than the values shown.

(2) The SSE and OBE acceleration values are provided in Table 1 of Auxiliary Building Design Report.

8. Check the effect of tornado depressurization on the Category I tank wall together with hydrostatic pressure.

Response

A comparison has been made of the radial loadings on the refueling water storage tank wall resulting from the design tornado wind pressure and depressurization loads and hydrostatic loads with those resulting from the hydrostatic and seismic effects.

In the tank design, the peak governing OBE load of 970 psf (including hydrodynamic and wall inertia loads) is conservatively applied as an enveloping axisymmetric load to derive the design hoop tension and vertical moment profiles. This load is clearly greater than the equivalent peak tornado load of 495 psf (governed by $W_t = W_{tq} + .5 W_{tp}$). This difference is increased when combined with the hydrostatic load in the appropriate load combination (i.e., a higher load factor for the hydrostatic load is used in the governing OBE load combination). It is, therefore, concluded that the load combination containing hydrostatic and tornado depressurization effects do not govern the design.

TABLE B.2[a][f]

CONCRETE DESIGN LOAD COMBINATIONS
STRENGTH METHOD

	EQN	D	L	P_a	T_o	T_a	E	E'	W	W_t	R_o	R_a	Y_j	Y_r	Y_m	M	B	Strength Limit
Service Load Conditions																		
	1	1.4	1.7															U
(See note b.)	2	1.4	1.7						1.7									U
(See note c.)	3	1.4	1.7				1.9											U
	4	1.05	1.275		1.275						1.275							U
	5	1.05	1.275		1.275				1.275		1.275							U
	6	1.05	1.275		1.275		1.425				1.275							U
Factored Load Conditions																		
	7	1.0	1.0		1.0			1.0			1.0							U
(See note d.)	8	1.0	1.0		1.0					1.0	1.0							U
	9	1.0	1.0	1.5		1.0						1.0						U
(See note e.)	10	1.0	1.0	1.25		1.0	1.25					1.0	1.0	1.0	1.0			U
(See note e.)	11	1.0	1.0	1.0		1.0		1.0				1.0	1.0	1.0	1.0			U
	12	1.0	1.0		1.0						1.0						1.0	U
	13	1.0	1.0		1.0						1.0					1.0		U

a. See Appendix A for definition of load symbols. U is the required strength based on strength method per ACI 318-71.
b. Unless this equation is more severe, the load combination 1.2D+1.7W is also to be considered.
c. Unless this equation is more severe, the load combination 1.2D+1.9E is also to be considered.
d. When considering tornado missile load, local section strength may be exceeded provided there will be no loss of function of any safety-related system. In such cases, this load combination without the tornado missile load is also to be considered.
e. When considering Y_j, Y_r, and Y_m loads, local section strength may be exceeded provided there will be no loss of function of any safety-related system. In such cases, this load combination without Y_j, Y_r, and Y_m is also to be considered.
f. Actual load factors used in design may have exceeded those shown in this table.