Transactions of the **13th International Conference on Structural Mechanics in Reactor Technology** (SMiRT 13), Escola de Engenharia - Universidade Federal do Rio Grande do Sul, Porto Alegre, Brazil, August 13-18, 1995

Seismic evaluation of a hot cell structure

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ABSTRACT: The evaluation of the structural capacity of and the seismic demand on an existing hot cell structure in a nuclear facility is described. Modal analysis of a finite–element model of the cell, treating the walls as plates and the floor and ceiling as a system of discrete beams, showed that the fundamental frequencies of the cell walls lie far above the earthquake frequency range. Based on an equivalent static analysis it was demonstrated that the hot cell structure would readily withstand the evaluation basis earthquake.

1 INTRODUCTION

The objective of the work reported here was to evaluate the effect of seismic excitation on the building structure housing a nuclear facility, known as the Alpha-Gamma Hot Cell, and to ascertain that the demand from seismic loading was within the capacity of the structure.

The DOE Standard DOE-STD-1020-94 (1994), requires that for this category of facilities, a dynamic analysis be performed to determine the seismic demand on the structure. It was decided to perform a preliminary analysis that included elements of a dynamic analysis without actually doing a full-blown finite-element dynamic response analysis.

2 DESCRIPTION OF THE CELL STRUCTURE

The Hot Cell, located within a reinforced concrete (RC) frame building, is a two-story concrete structure. The first story is below grade level and consists of reinforced concrete foundation walls on a mat. The second story, referred to as the cave, consists of concrete walls with steel liners. The plan of the walls at the two story levels is shown in Fig. 1. The rectangle enveloping the entire plan has a dimension of about 49 ft by 38 ft, with the longer dimension parallel to the north-south direction. Most of the walls have a thickness between 36 in and 12 in. While the foundation walls do not have any steel liners, all the cave walls have 1/4-in thick steel liners on both faces.

The floor and ceiling of the cave consist of a framework of steel girders and joists, all encased in concrete. The beam framing for the floor and ceiling of the cave is shown in Fig. 2. The total thickness of the concrete floor is 21 in, and that of the ceiling is 36 in. A 1/2-in thick steel plate, welded to the top of the steel joists of the floor framing, lines the top surface of floor concrete. Similarly, a 1/4-in thick steel plate is welded to the

bottom of the ceiling beams and lines the bottom surface of ceiling concrete. A number of rectangular openings in the cave walls are provided for doorways, windows and transfer ports.

3 ASSUMPTIONS FOR STRUCTURAL MODELING OF HOT CELL

For the purpose of the preliminary analysis, the cell structure is considered completely isolated from the surrounding building and is treated as an independent structure. In view of the approximate nature of the calculations, soil-structure interaction is neglected. The cave is assumed to be fixed at the base, i.e. fixed to the foundation mat. The site-specific horizontal seismic ground motion is assumed to be applied at this elevation.

Composite action between concrete and steel liners is assumed for cave walls. However, concrete in the cave floor and ceiling is assumed not to contribute to structural action. The absence of any kind of arrangement to provide for shear transfer between concrete and steel beams/girders in the floor and ceiling would require such an assumption. Therefore the only load-bearing system in the floor and ceiling consists of built-up steel beams and girders. For calculating beam stiffness, the effective width of the flange was determined according to the rule given by Timoshenko (1956).

In computing the section stiffness for the finite element analysis, the gross thickness of walls is used. For members subjected to pure bending, the stiffness may be calculated by neglecting the concrete under tensile stresses. The walls, however, are subjected to combined axial load and bending. It is likely that the tension in concrete is limited to small values and therefore it is justifiable to include all the concrete for computing section stiffness.

4 DYNAMIC ANALYSES FOR DETERMINATION OF NATURAL FREQUENCIES

The natural frequencies of the cave were determined as a necessary pre-requisite to seismic response determination. A finite-element model of the cave was subjected to a modal analysis with the ANSYS code. All the walls of the cave were modeled as plate elements. The floor and ceiling framing was modeled with a number of discrete beams. Figure 3 shows the typical finite-element mesh for the walls.

The walls are represented by their mid-planes. The beams (in the model) spanning between the mid-planes would become longer than their actual spans by an amount equal to the thickness of the walls. Since the walls are relatively very thick in the present case, the unrealistically longer spans would cause a (spurious) increase in the stresses in the beams. To avoid such an overestimate of beam stresses, relatively rigid horizontal plate elements were attached to the top of the walls and the beam ends were connected to the rigid plates. The width of the rigid plate elements were adjusted so as to make the beam spans correspond to their actual values. These rigid elements are only a computational device and hence are not shown in the sketches. A total of 2837 elements and 3003 nodes comprised the model. All the nodes at the base of the vertical wall elements were completely restrained. A total of 1500 master degrees-of-freedom was specified in such a manner as to include all the modes of interest in the frequency range of 0 to 33 Hz.

There are hundreds of modes in the frequency range of interest because each beam acts independently. Since there are many beams of similar properties, multiple modes occur at many of the frequencies. The fundamental mode occurs at 11.6 Hz, which is the first beam mode for deformation in the vertical plane. There are no wall modes in the seismic frequency range of 0 to 33 Hz.

The latter conclusion was verified by making approximate estimates for the natural frequency of the cave walls. One of the relatively thinner walls, of thickness 16 in, height 288 in, and width 228 in, was assumed to be free on top and fixed at all other edges. For this condition the fundamental frequency of the wall was estimated to be about 575 Hz, on the basis of Case 15, Table 11-4 of Blevins (1979). Another wall of thickness 36 in, height 288 in and width 420 in, was found to have a fundamental frequency of 626 Hz.

Since the wall modes are all well above the seismic range of 33 Hz, only a 'rigid' analysis is required to calculate the wall seismic response. This is accomplished by performing a static analysis for a loading equivalent to a horizontal acceleration at the ZPA level.

The beams, which respond primarily in the vertical deformation modes, have frequencies within the seismic range. However, computationally it is not considered cost-effective to perform such a dynamic response analysis in view of the large number of beams each of which has many modes in the seismic range. Sec. 3.2.5.3. of ASCE Standard ASCE 4-86 (1986) permits the equivalent static method for simple structures in which the maximum response results from the loads in the same direction. Since the beams of this model are indeed simple structures in that sense, an equivalent static analysis is deemed to give more cost-effective, yet conservative results.

5 LOADS AND LOAD COMBINATIONS

Besides the seismic loading, dead load, D, and live load, L, need to be considered. Based on consultation with facility operators and the general recommendations in the ASCE Standard ANSI/ASCE 7-88 (1990), the live load, L, is taken to be a uniformly distributed load with an intensity of 200 psf.

The seismic loading is based on a PGA of 0.15 *g*. Per the site specific spectrum, the peak amplification occurs at 4.4 Hz. The vertical component of the seismic load is assumed to be two-thirds of the horizontal component for the entire frequency range. The seismic loading, E, was calculated on the above basis and the requirement of Sec. 3.1.4.2 of ASCE Standard ASCE 4-86 (1986) that the inertial mass include an appropriate part of the live load. For determining E, the mass of the floor and ceiling are increased by an amount equivalent to 10% of the live load. Equivalent-static earthquake load is determined to be 0.417 *g* according to the provisions of DOE-STD-1020-94 (1994).

Since the fundamental frequency of the cave walls lie far above the cut-off frequency of 33 Hz, only 'rigid' response need be considered. Consequently, the final static calculations are made with a static acceleration of 0.15 *g* in each of the two horizontal directions per Sec. 3.2.5.3, ASCE Standard ASCE 4-86 (1986).

The fundamental frequency of the beams in the floor and ceiling lie in the frequency range above 11 Hz. The beam modes are primarily vertical modes. At frequencies above 11 Hz, the spectral acceleration is lower than the peak value which occurs at 4.4 Hz. However, in accordance with Sec. 3.2.5.3 of ASCE Standard ASCE 4-86 (1986), a factor of 1.5 is applied to the peak acceleration of the vertical spectrum to obtain the equivalent static load. Since the vertical component is only two-thirds of the horizontal, the static load is based on a vertical acceleration of 0.417 *g*.

Of the load combinations to be considered per Sec. 9.2.1 of the code, ACI 349-85 (1986), the only applicable one is given by D + L + E, since the evaluation basis earthquake is considered equivalent to the safe shut down earthquake and there is no earthquake equivalent to operating basis earthquake for this facility. This load combination actually results in two cases, i.e. D + L + E and D + L - E.

6 DEMAND (RESPONSE) CALCULATION

The seismic load was applied as a body force. The inertial mass of the structure reflected not only the dead load but also an addition equivalent to 10% of the live load on the floor and ceiling. An acceleration of 0.15 *g* was applied in each of the two horizontal directions, and the vertical acceleration applied was 0.417 *g* as noted above. Loading in each direction constituted a separate load step. The resulting response from each load step was subsequently combined by the square-root-of-sum-of-squares (SRSS) method. This resultant response is the seismic demand, denoted as D_S. No inelastic energy absorption factor was applied for the sake of conservatism.

The response to dead load and live load were determined in separate load steps. In each case the load was converted to an equivalent body force and applied as an acceleration for the appropriate inertial mass. The response due to dead and live load are simply added to arrive at the total non-seismic demand, D_{NS}.

The total demand, D_{TI} is obtained by enveloping the results of the two load combinations, $D_{NS} + D_S$ and $D_{NS} - D_S$. The significant results of this envelope are summarized below.

Wall deformations are very small, i.e. of the order of a hundredth of an inch. The largest deformation occurs in an east wall, of magnitude 0.0084 in, in the vertical direction. The asterisk in Fig. 3 shows the location where the maximum deformation occurs. The peak deflection of the wall normal to its plane is smaller at 0.0069 in. The peak stresses also occur at the same location as the peak deformation. After converting the ANSYS results obtained as force and moment per inch length to the demand per foot length of the composite wall section, the two pairs representing the governing demand are determined as: (1) P = 6.3 kips, M = 61.6 inch kips and (2) P = 39.2 kips, M = 412.8 inch kips, where P is the direct compressive force per foot length and M is the bending moment per foot length of wall.

The highest stresses in the beams of the floor framing occur at the locations indicated by C (for compressive stress) and T (for tensile stress) in Fig. 2. The peak compressive stress is -21155 psi and the peak tensile stress is 13290 psi. In the ceiling the peak compressive and tensile stresses occur at the same beam element, marked by an asterisk. The peak beam stresses in the ceiling are -10612 psi in compression and 8649 psi in tension.

7 SEISMIC DEMAND VS. CAPACITY

At the point where the most severe stresses occur in the wall, the wall thickness is 24 in. The wall is made up of a composite section with 1/4-in steel plates lining both faces. In addition, #5 reinforcing bars are placed at 12-in centers near both faces, both vertically and horizontally. A 12-in wide section of the wall is considered. Since the wall section is subjected to an axial compression and bending moment, the section capacity is defined by an interaction diagram (Nilson and Winter, 1986). A strength reduction factor of 0.7 is applied to the moment and axial load capacities calculated, per Sec. 9.3.2 (b) of the code, ACI 349-85 (1986). An additional capacity reduction factor of 0.85 is applied to the axial force capacity per Sec. 10.3.5.1 of ACI 349-85 (1986).

Figure 4 shows the capacity vs. demand for the cave wall at the location of the most severe stresses and deformation. It is clear that the combined demand is far below the capacity of the wall. At other locations, the margin against failure is even greater.

Regarding the floor and ceiling beams, it is not necessary to compare demand with capacity because the largest stress in the beams (21155 psi) is well below the yield stress of 33000 psi for the structural steel. (The moment capacity of beams is reached

only after the peak stresses reach yield point values.) Because the beams are encased in concrete, the potential for buckling does not exist.

Based on the above comparisons, it is concluded that no structural failure is likely to occur in any part of the cave structure when it is subjected to the evaluation basis earthquake.

8 CONCLUDING REMARKS

Although only typical engineering methods were used, the subject reported here is of interest as a case history of a non-typical structure. The original design of the hot cell, dated about 1958, was characterized by two factors that differed from those of the current analysis: no credit was taken for composite action, and seismic load was not considered.

The structural connection between the concrete walls and the liners justified assuming composite action. In view of the large margin against failure, indicated by Fig. 4, it is likely that the walls would have adequate margin against failure even if no composite action in the walls occurred. The floor and ceiling framing beams, although encased in concrete, could not be considered as composite members. The large amount of concrete was used for shielding purposes and it is apparent that no effort was made to take structural advantage of the concrete by ensuring composite action.

It is interesting to note that in spite of the fact that the original design was not based on resistance to seismic loading, the structure has a large capacity against seismic failure. The reason for this is that the massive concrete walls result in the fundamental frequency occurring well beyond the seismic range. Although the massive concrete in the walls was intended for shielding, it helps to reduce seismic stresses also.

ACKNOWLEDGMENTS

This work was supported by the U. S. Department of Energy, under Contract Number W–31–109–Eng–38.

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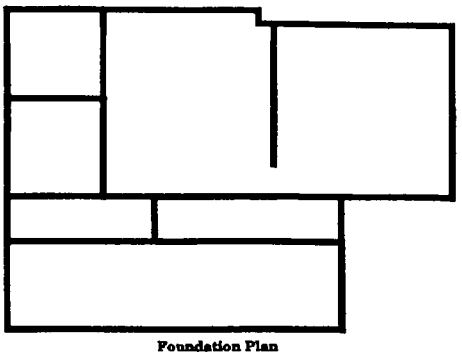

Fig. 1. Plan of Hot Cell Structure.

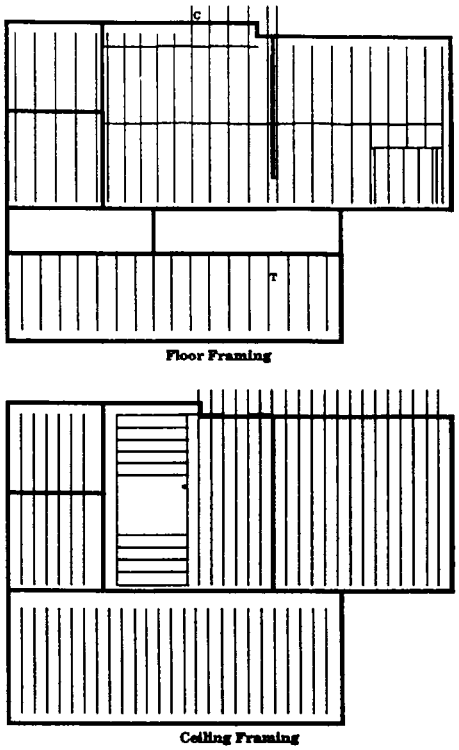

Fig. 2. Beams and Girders of Hot Cell Structure.

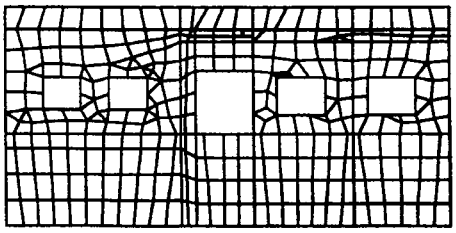

Fig. 3. Typical Finite–Element–Mesh: East Walls.

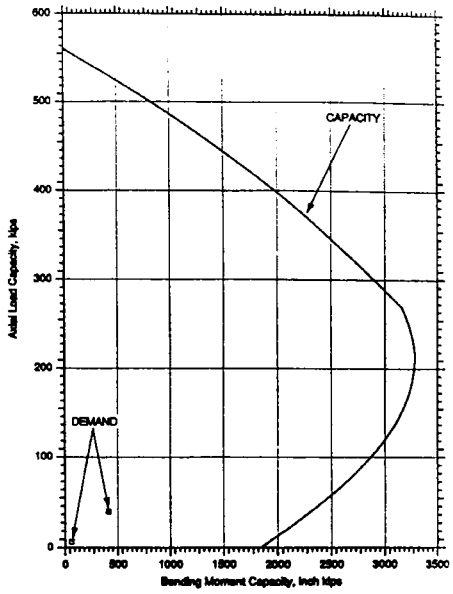

Fig. 4. Comparing Demand to Capacity of a 12 in. Wide Section of Wall at the Point of Peak Stresses in Wall.