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PWR REACTOR BUILDING RESPONSE FROM HIGH ACCELERATION, SHORT DURATION EARTHQUAKES

by

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SUMMARY

Inelastic response of a typical PWR reactor building designed for low to moderate seismicity regions subjected to a near field, moderate magnitude earthquake has been investigated. Such earthquakes have resulted in large peak accelerations but only minor damage. This paper demonstrates that the structure quickly shakes down to stable pseudo-elastic behavior with permanent deformation substantially less than the yield displacement. It is demonstrated that even though the structure is softened during limited inelastic excursions, there is not enough energy content in the earthquake to significantly damage the structure. Analyses performed consider soil structure interaction, nonlinear soil structure interaction effects due to foundation mat uplift and nonlinear degrading stiffness hysteresis in shear.

STRUCTURE AND EARTHQUAKE GROUND MOTION CONSIDERED

The typical PWR reactor building considered consists of a prestressed concrete cylindrical external containment building and roof dome and a reinforced concrete internal structure on a common foundation mat. The internal structure is an assemblage of concentric cylindrical walls and connecting radial walls. This structure has been designed for a broad band earthquake response spectrum scaled to a peak acceleration of 0.2g. Although various soil conditions have been considered, this paper concentrates on seismic response of the structure founded on competent rock.

The earthquake ground motion considered is the accelerogram recorded at Melendy Ranch Barn on September 4, 1972 (N61E component) scaled to a maximum acceleration of 0.5g (the actual maximum acceleration of this accelerogram was nearly 0.5g). This was a magnitude 4.7 earthquake which occurred along the San Andreas fault near Hollister, California. Only one horizontal component of earthquake time history is considered in this study and vertical ground motion is not considered. The Melendy Ranch Barn earthquake record is characterized by high peak acceleration but very short duration. The response spectrum for this earthquake record has a narrow amplified response region and minimal energy content in the low frequency range.

METHOD OF ANALYSIS

Dynamic analyses have been performed by direct integration of the

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equations of motion by means of a version of the computer program DRAIN-2D (Ref. 1). DRAIN-2D performs seismic analyses of inelastic planar structures. The structure has been represented by a model consisting of lumped masses concentrated at nodes interconnected by beam elements. In addition, spring elements attached to the foundation mat are used to incorporate soil structure interaction by representing the stiffness of the underlying soil or rock. The model representing the exterior containment building wall, the dome, the crane, the foundation mat, the internal structure and the soil stiffness is illustrated in Figure 1 superimposed over a cross-section of this reactor building.

Soil-structure interaction has been modeled by a lumped parameter stiffness model in which the resisting forces developed when the structure moves relative to the surrounding soil mass are incorporated into the analysis by means of compliance functions represented as equivalent springs and dashpots connecting the foundation mat to the ground. In addition to the soil springs shown in Figure 1, corresponding soil dashpots are included to incorporate the damping of the soil in the soil-structure model. Soil damping is generally considered to be composed of two types of damping; 1) material damping associated with energy losses within the soil due to hysteretic or viscous effects; and 2) radiation damping associated with the loss of energy through propagation of elastic waves from the immediate vicinity of the foundation (i.e., feedback of energy from the structure to the surrounding soil. Values for soil spring stiffnesses and dashpot constants for radiation damping have been calculated based on formulas proposed by Novak (Refs. 2 and 3). These formulas are approximate analytical solutions for the condition of a rigid foundation mat resting on an elastic half-space and partially embedded in an overlying elastic layer.

Radiation damping in translation is very large (on the order of 55 percent of critical damping). Radiation damping in soil rocking and material damping of the structure and soil are much smaller (on the order of 5 to 10 percent of critical damping). For the analyses performed, the damping matrix was assumed to be of the following form:

$$[C] = \alpha[M] + \beta[K] + \lceil C_D \rfloor$$

The mass and stiffness proportional constants, α and β were selected to provide 5 to 10 percent of critical damping over the frequency range of interest and correspond to soil and structure material damping and radiation damping in the soil rocking mode. The $\lceil C_D \rfloor$ portion of the damping matrix corresponds to the concentrated dashpot values which incorporate the high radiation damping in the soil translational mode. DRAIN-2D was modified to accommodate concentrated dashpots.

With sufficient overturning moment during seismic response, there is a tendency for a portion of the foundation mat to lift off the underlying supporting soil. When only a portion of the foundation mat is in contact with the soil, there is softening of the soil-structure system. This behavior can be simulated by developing a nonlinear moment-rotation relationship for the base rocking spring which accounts for the appropriate soil-structure interaction for any amount of contact between the foundation mat and underlying soil. Softening of the base translation spring has not been considered. A previous study (Ref. 4) indicated that uplift effects on this highly damped mode of deformation did not significantly influence either structure response or in-structure response spectra. The DRAIN-2D computer program was modified to allow nonlinear elastic behavior of the base rocking spring and thus incorporate foundation mat uplift effects.

The inelastic behavior of reinforced and prestressed concrete members such as those making up this typical PWR reactor building has been simulated by an elastic beam element with inelastic hinge elements at each end representing flexural stiffness and a short inelastic spring element representing shear stiffness. The beam element and inelastic flexure and shear springs act in series. Note that for prestressed concrete elements the inelastic shear spring is excluded because inelastic shear behavior is not feasible as the vertical and circumferential prestress provide confinement which prevents shear cracks from developing.

For this structure subjected to the Melendy Ranch Barn earthquake record, the only inelastic behavior is that due to shear in the lower region of the internal structure. The primary loading curve assumed for inelastic shear spring elements representing shear flexibility of reinforced concrete members includes two linear segments. The stiffness of the first segment corresponds to shear being carried totally by the concrete. The stiffness of the second segment corresponds to the concrete being unable to take additional shear and added shear is taken by the reinforcing steel. Thus inelastic shear behavior is considered to occur when the concrete load capacity is exceeded and the concrete cracks but the steel behaves elastically. If the shear force would reach a level in which the steel reached yield stress, a brittle failure is conservatively assumed to occur. It is assumed that a member in shear exhibits softer unloading stiffness and degrading stiffness during reloading because the concrete cracks do not heal during unloading and the concrete begins to deteriorate. During unloading and reloading, the inelastic shear springs will follow rules for hysteresis under load reversals taken from Takeda (Ref. 5). The DRAIN computer program was modified to include the Takeda hysteresis rules for members in shear.

SEISMIC RESPONSE

Maximum displacements and shears throughout the containment building and internal structure as determined from both elastic and inelastic time history analyses are illustrated in Figures 2 and 3. Both the elastic and inelastic analysis response shown includes the nonlinear effects of foundation mat uplift. Inelastic behavior locally increases displacements by a factor of about 1.3 over values determined from elastic analysis. The increased displacements are seen only in the region of inelastic behavior near the bottom of the internal structure and elsewhere in the structure there is little difference in displacements from elastic or inelastic analysis. Inelastic behavior reduces response shears in the internal structure by a factor of 1.6 to 1.7 from values determined from elastic analysis. In addition, shears in the containment building which remains elastic are reduced by about 10 percent.

In-structure response spectra at locations in the internal structure and containment building are presented in Figures 4 and 5. Spectra are shown for elastic analysis ignoring nonlinear effects of foundation mat uplift, for elastic analysis including the nonlinear effects of uplift and inelastic analysis including the nonlinear effects of uplift. From these figures, it may be seen that accounting for the nonlinear effects of uplift produces a small reduction from elastically calculated spectra. Accounting for inelastic behavior produces a large reduction in the peak value of the spectra as well as a reduction of the frequency of this peak for locations in the internal structure. In the containment building, there is only a small reduction from the elastically calculated spectrum due to inelastic behavior. It may be noted that there may be a small increase in the spectra in the high frequency region when either inelastic behavior or nonlinear effects of uplift are included.

The force-deflection response of the inelastic shear spring element which undergoes the greatest inelastic behavior is illustrated in Figure 6. The response is shown for 1.6 seconds of earthquake shaking. From this figure, it may be seen that there are only two large inelastic excursions. Also degrading stiffness behavior for the cyclic response is evident. It is demonstrated in Figure 6 that this structure shakes down to stable pseudo-elastic behavior even including degrading stiffness behavior. Also, the permanent deformation resulting from this seismic excitation is less than the yield displacement.

CONCLUSIONS

The basic conclusion of this study is that the typical PWR reactor building considered subjected to a near field, low magnitude, high acceleration earthquake will experience some localized inelastic behavior. However, this inelastic behavior is limited such that the structure is not expected to experience significant damage or loss of function. This PWR reactor building subjected to the Melendy Ranch Barn earthquake accelerogram scaled to a maximum acceleration of 0.5g does not undergo a great deal of inelastic deformation during seismic response as no cracking of the exterior containment building is expected and no plastic hinges are expected to be formed in the internal structure. The only inelastic behavior during this earthquake would occur due to shear in the bottom eight meters of the internal structure.

This reactor building undergoes inelastic behavior to the extent that the system ductility factor is 1.8 and the maximum story inelastic deflection is 3.3 times the effective yield displacement. For reinforced concrete nuclear plant structures which behave primarily in shear, it is recommended for design that the system ductility factor be limited to the range of 1.3 to 2.0 and the maximum story inelastic deflection be limited to under 3.0 times the effective yield displacement (Refs. 6, and 7). For this PWR reactor building subjected to the Melendy Ranch Barn earthquake record, the system ductility factors are below 2.0 but the inelastic displacement exceeds 3.0 times the effective yield displacement. For large magnitude design earthquakes, this PWR reactor building is one in which the system ductility factor should be limited to near 1.3 as this structure unloads at a lower stiffness than loading and has localized inelastic behavior. However, for a moderate magnitude earthquake such as the Melendy Ranch Barn earthquake, it has been demonstrated in this study that this structure can safely withstand the seismic excitation with a system ductility factor of 1.8 and inelastic displacement 3.3 times the effective yield displacement. For the Melendy Ranch Barn earthquake, this structure undergoes only one inelastic excursion to peak displacement with no other vibration cycles near peak displacement. This structure quickly shakes down to stable pseudo-elastic behavior with permanent deformation less than the yield displacement. It is demonstrated that even though the structure is softened by the inelastic excursion, there is not enough energy content in this earthquake to damage the structure.

In many conventional design analyses of reinforced concrete structures, the elastically calculated shear is compared with the ultimate shear to judge the adequacy of the structure and an inelastic analysis is not performed. This approach is most likely conservative because the inelastic energy absorption capacity of the structure is not considered. However, this approach does not account for softening of the structure and the corresponding reduction in natural frequency or permanent set which result when the concrete load capacity is exceeded. If elastic design analyses are performed in which the response shear exceeds the concrete load capacity, a reduced effective stiffness should be used instead of the uncracked concrete stiffness in order to accurately represent the natural frequency of the structure.

Inelastic behavior has the effect of reducing the in-structure response spectra for most frequencies from elastically calculated spectral values. For locations in the internal structure, it is concluded that:

a. There is a reduction in peak spectral acceleration roughly corresponding to $1/\mu$ where μ is the system ductility factor

b. There is a reduction in the frequency of the peak spectral acceleration roughly corresponding to $\sqrt{1/\mu}$

c. There can be an increase in spectral acceleration in the high frequency regime. This potential increase is uncertain and difficult to predict, but is not significant in terms of potential damage or loss of function.

For locations in the containment building which remained elastic, the inelastic in-structure response spectrum was reduced by a lesser amount.

Inelastic behavior also had the effect of reducing the seismic response accelerations, moments and shears in the internal structure below elastically calculated values. The amount of reduction was approximately equal to the system ductility factor or the same reduction as seen for in-structure response spectra. Also there is a small reduction in containment building response due to inelastic behavior of the internal structure. Inelastic response displacements are locally greater than elastically calculated displacements in the region of inelastic behavior near the bottom of the internal structure. Elsewhere throughout the containment building and internal structure, inelastic response displacements are essentially the same as elastically calculated values.

ACKNOWLEDGMENTS

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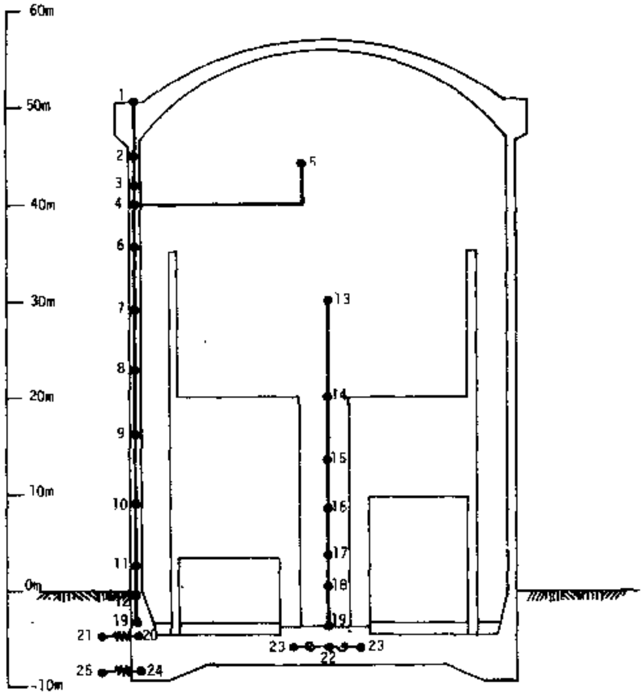

FIGURE 1 PWR REACTOR BUILDING AND ANALYTICAL MODEL

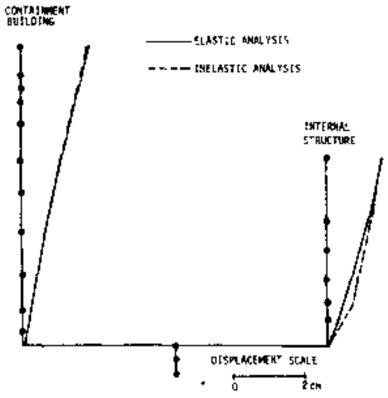

FIGURE 2 SEISMIC RESPONSE DISPLACEMENTS

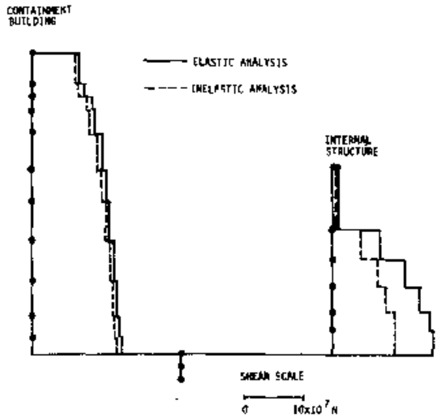

FIGURE 3: SEISMIC RESPONSE SHEARS

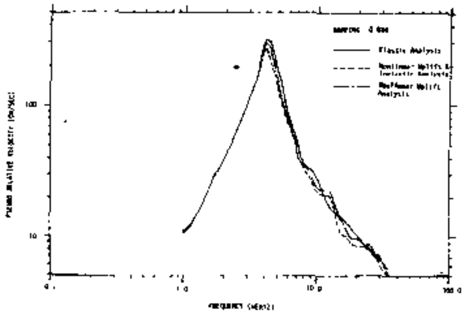

FIGURE 4: FLOOR RESPONSE SPECTRA, CONTAINMENT BUILDING

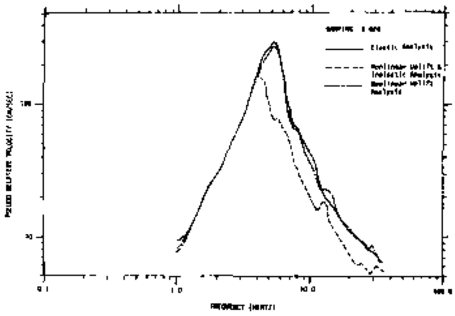

FIGURE 5: FLOOR RESPONSE SPECTRA, INTERNAL STRUCTURE

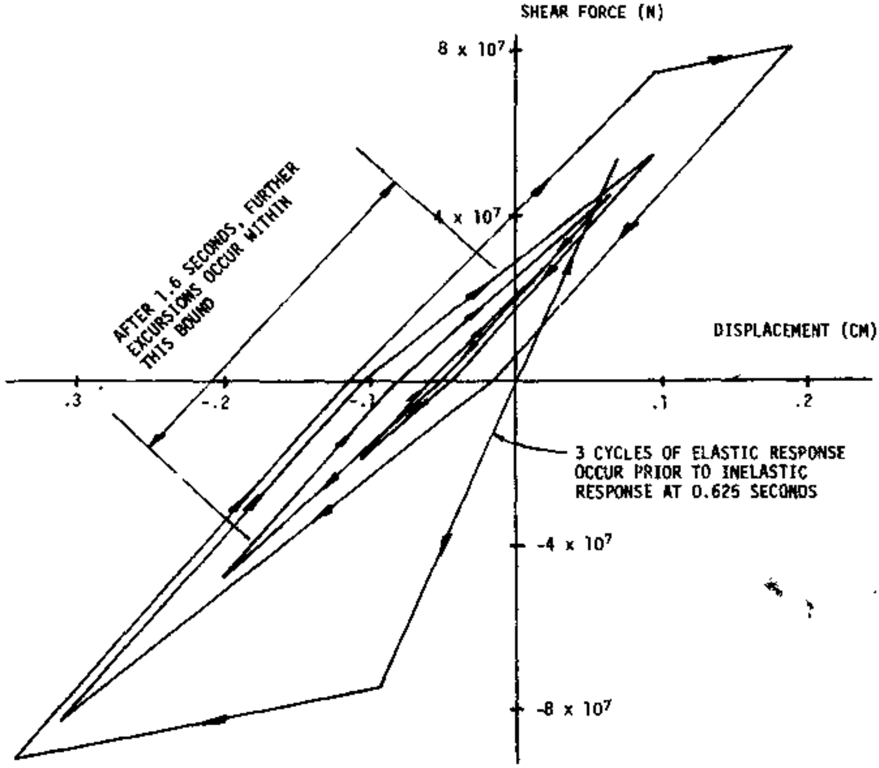

FIGURE 6: SHEAR-DISPLACEMENT RESPONSE, BOTTOM OF INTERNAL STRUCTURE

SEISMIC ANALYSIS OF THE TURBINE BUILDING OF A BWR NUCLEAR POWER PLANT

by

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SYNOPSIS

Considering the requirements imposed by the Swiss Regulations, the preliminary seismic design of the turbine building of NPP Graben-Switzerland has been carried out by using dynamic seismic analysis methods. The turbine building of the GE-Mark III BWR NPP is an extremely complicated structure. The main difficulty for the analysis of the turbine building lies with the idealization of such a complicated structure into a 3D mathematical model with a reasonable number of degrees-of-freedom, yet providing suitable amount of response information that could allow a detailed stress analysis. The mathematical model created has 330 active degrees-of-freedom. The responses are calculated by employing the response spectrum program included in E+B's Dynamic Analysis Package.

INTRODUCTION

In many of the countries where nuclear power plant technology is under establishment, the US NRC Regulations are adapted almost without any alteration. According to these regulations, nuclear power plant structures, systems and components important to safety should be designed to withstand the effects of earthquakes without loss of capability to perform necessary safety functions. In US NRC R.G. 1.29 and SRP Section 3.2.1, those plant structures, systems and components that are designed to remain functional if the SSE occurs are designated SEISMIC CATEGORY I. SRP sections 3.7.1, 2 and 3 require sophisticated dynamic analysis methods for design of Seismic Category I structures, systems and components.

The rest of the structures, systems and components are designated NON-SEISMIC CATEGORY I and the use of conventional seismic design methods specified in local building codes (such as UBC) are allowed for these structures, systems and components.

According to the above mentioned US NRC regulations, the turbine building is classified as a Non-Seismic Category I structure and in those countries where US NRC Regulations are adapted without any alteration, no special attention is paid for the seismic design of the turbine building.

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