SEISMICALLY-INDUCED SOIL AMPLIFICATION AT THE DOE PADUCAH GASEOUS DIFFUSION PLANT SITE*

David W. Sykora and Mary E. Hynes
Geotechnical Laboratory
U.S. Army Engineer Waterways Experiment Station
Vicksburg, Mississippi 39180-6199

William R. Brock and R. Joe Hunt, Technical Operations
Karen E. Shaffer, Uranium Enrichment
Martin Marietta Energy Systems, Inc.
Oak Ridge, Tennessee 37831-7294

ABSTRACT

A site-specific earthquake site response (soil amplification) study is being conducted for the Department of Energy (DOE), Paducah Gaseous Diffusion Plant (PGDP). This study is pursuant to an upgraded Final Safety Analysis Report in accordance with requirements specified by DOE.

The seismic hazard at PGDP is dominated by the New Madrid Seismic Zone. Site-specific synthetic earthquake records developed by others were applied independently to four soil columns with heights above baserock of about 325 ft. The results for the 1000-year earthquake event indicate that the site period is between 1.0 and 1.5 sec. Incident shear waves are strongly amplified at periods of motion greater than 0.3 sec. The peak free-field horizontal acceleration, occurring at very low periods, is 0.28 g.

INTRODUCTION

The Paducah Gaseous Diffusion Plant (PGDP), located near Paducah, Kentucky, (Fig. 1) is one of two operating Department of Energy (DOE) gaseous diffusion plants. Considered a non-reactor nuclear facility, uranium hexaflouride is enriched in the U-235 isotope as an integral part of the nuclear fuel cycle.

* This manuscript has been authored by a contractor of the U.S. Government under Contract No. DE-AC05-84OR21400. Accordingly, the U.S. Government retains non-exclusive royalty-free license to publish or reproduce the published form of this contribution or allow others to do so for U.S. Government purposes. Research was sponsored by the Office of the Assistant Secretary for Nuclear Energy, U.S. Department of Energy, under Contract with Martin Marietta Energy Systems, Inc., and under Inter-Agency Agreement No. DE-AI05-91OR21971 with the U.S. Army Waterways Experiment Station.

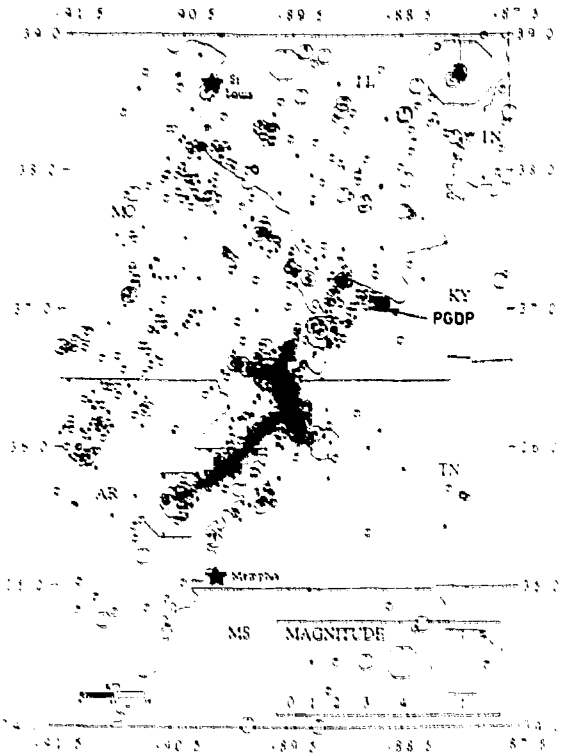

Fig. 1: General location of PGDP showing seismic activity in a 189-month period between 1974 and 1990 (courtesy of Saint Louis University)

Engineering analyses and other scientific studies required to upgrade the existing PGDP Final Safety Analysis Report (FSAR) are presently on-going in accordance with current DOE guidelines and requirements. These efforts include the evaluation of the effects of natural phenomena hazards, i.e., seismic events, extreme winds, and floods. Situated on a deep soil profile within close proximity (60-100 km) to a strong earthquake source, the New Madrid Seismic Zone (NMSZ), PGDP has been categorized as a moderate hazard facility with respect to natural phenomena hazard classifications [1]. Consistent with this classification, the 1000-year event was of particular interest in this study.

Synthetic earthquake records corresponding to rock outcrop motions were developed from probabilistic assessments using an extended-source seismic hazard analysis and were represented by uniform risk response spectra for the 500, 1000, and 5000-year recurrence intervals. Acceleration records which enveloped the uniform risk response spectra were generated for each of three mutually-orthogonal directions for each earthquake record. Two horizontal components were used for the site response analysis reported in this paper.

A combination of geophysical and geotechnical data obtained at four sites located around the periphery of PGDP were used to derive four soil columns. These soil columns are about 325 ft thick and represent alluvium (Pleistocene-age Continental Deposits) and Tertiary-age interbedded sands, silts, and clays. The bedrock is limestone of Mississippian Age. Seismic wave velocities were measured at each location using crosshole and downhole geophysical techniques. Other downhole geophysical techniques were used to infer the variation of density with depth.

This paper summarizes site-specific earthquake response analyses conducted by the U.S. Army Engineer Waterways Experiment Station (WES) for the 1000-year design earthquakes and represent a key element of the seismic studies for PGDP. Analyses for 500- and 5000-year events are in progress. Site-specific spectra were developed for free-field (soil) response.

The results presented in this paper correspond to the an initial set of assumed conditions where the potential effects of high confining pressures on shear modulus and damping ratio relationships are not yet addressed. The analyses that include the effects of confining pressure and parametric variations are mentioned briefly.

The results of analyses presented in this paper are expected to be of interest to the technical community because the analysis of strong motion in deep soil deposits in the central United States has not been widely reported. The most difficult aspect appears to be a lack of resources to obtain material properties for the full height of the soil profile.

EARTHQUAKE HAZARDS AND DESIGN MOTIONS

The earthquake hazards and design motions used in the site response analysis for PGDP are summarized in a recent report [2] and also a paper to this conference. A brief description is also presented in this section.

Three separate hazard analyses were considered for the overall earthquake hazard study: the Electric Power Research Institute/Seismicity Owners Group (EPRI/SOG) analysis, the Lawrence Livermore National Laboratory (LLNL) analysis, and the extended-source analysis. The EPRI/SOG analysis used the input data and methodology developed by EPRI, under the sponsorship of SOG, for the evaluation of seismic hazard in the central and eastern United States. The LLNL analysis used the input data and methodology developed by LLNL for the Nuclear Regulatory Commission (NRC).

Both the EPRI/SOG and LLNL methodologies treat earthquakes as point sources. This assumption is not directly applicable to PGDP because of the possibility of large earthquakes occurring in the NMSZ. As a result, an extended-source seismic hazard analysis was performed that modeled the NMSZ as a system of parallel faults running in a north-northeasterly direction. The results from the EPRI/SOG and LLNL analyses were only used to develop baseline results and to understand the dominant seismic sources and the uncertainties in their parameters.

The seismic hazard results from the extended-source analysis are shown in Figures 2 and 3. Figure 2 shows the mean, 15th, 50th, and the 85th percentile peak horizontal rock accelerations versus the annual probability of exceedance. Figure 3 shows the median (50th percentile) uniform hazard response spectra for rock at 5 percent damping, for annual probabilities of exceedance of 0.002, 0.001, and 0.0002.

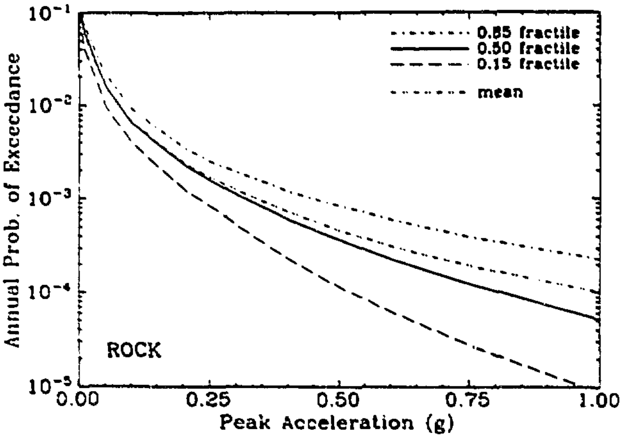

Fig. 2: Peak ground acceleration hazard curves [2]

Earthquake magnitude, distance, duration, and synthetic acceleration records were calculated which represent the uniform hazard response spectra. Figure 4 shows the two components of horizontal motion corresponding to rock outcrop for the 1000-year median uniform hazard response spectra.

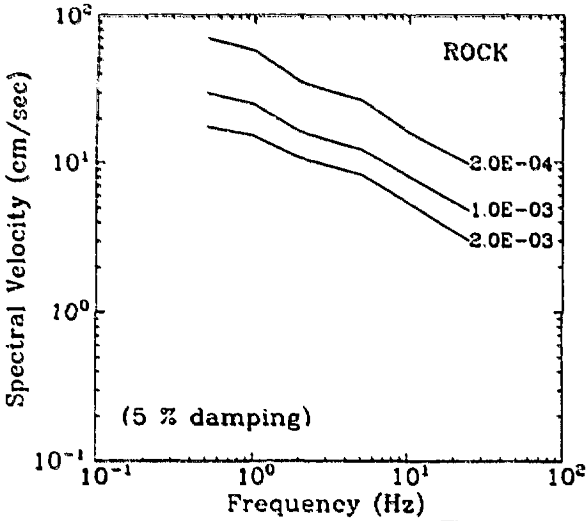

Fig. 3: Equal hazard spectra for extended source model [2]

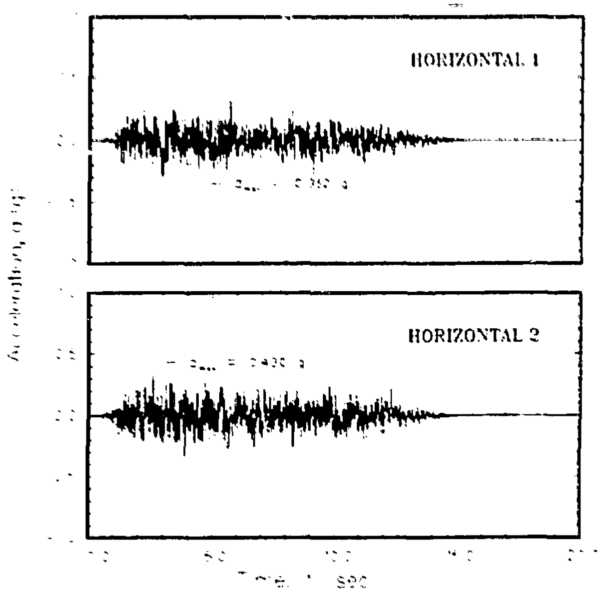

Fig. 4: Synthetic acceleration records for 1000-year event [2]

SOIL COLUMNS

Four individual soil columns were provided to WES as basic input to the free-field seismic response analysis at PGDP. [3] A soil column is a one-dimensional idealization of a layered soil deposit. The information contained in these soil columns was the product of a borehole drilling and in-situ geotechnical and geophysical data acquisition program conducted during FY90. [3,4,5]

The approach of analyzing the columns independently and then combining results differs from another method that involves deriving an "average" stratigraphy and then averaging in-situ densities and seismic velocity measurements and assigning an appropriate shear modulus and damping ratio curve for each layer. The authors believe that separate evaluation will provide a more effective and realistic bracket of the potential range of site conditions that could be produced. The procedure of averaging is subjective and will deemphasize, and may overlook, strong response produced by isolated stiff layers.

The locations of the four sites are shown in Figure 5. At Sites 1 and 2, three shallow boreholes were drilled to depths ranging from 70 to 125 ft. Four boreholes were drilled at each of the other two sites (3 and 4), three of which were extended to depths of approximately 125 ft. The final borehole at each of Sites 3 and 4 terminated in bedrock encountered at depths of 364 and 322 ft, respectively.

The four sites are separated by great distances and are not particularly close to critical structures. Noted variations in the evaluation of stratigraphic sections provides evidence that the extent of possible conditions in terms of height of column and individual material types may be adequately addressed with existing columns, however, and should adequately bracket the range of expected site response. Potential variations were addressed by parametric analyses.

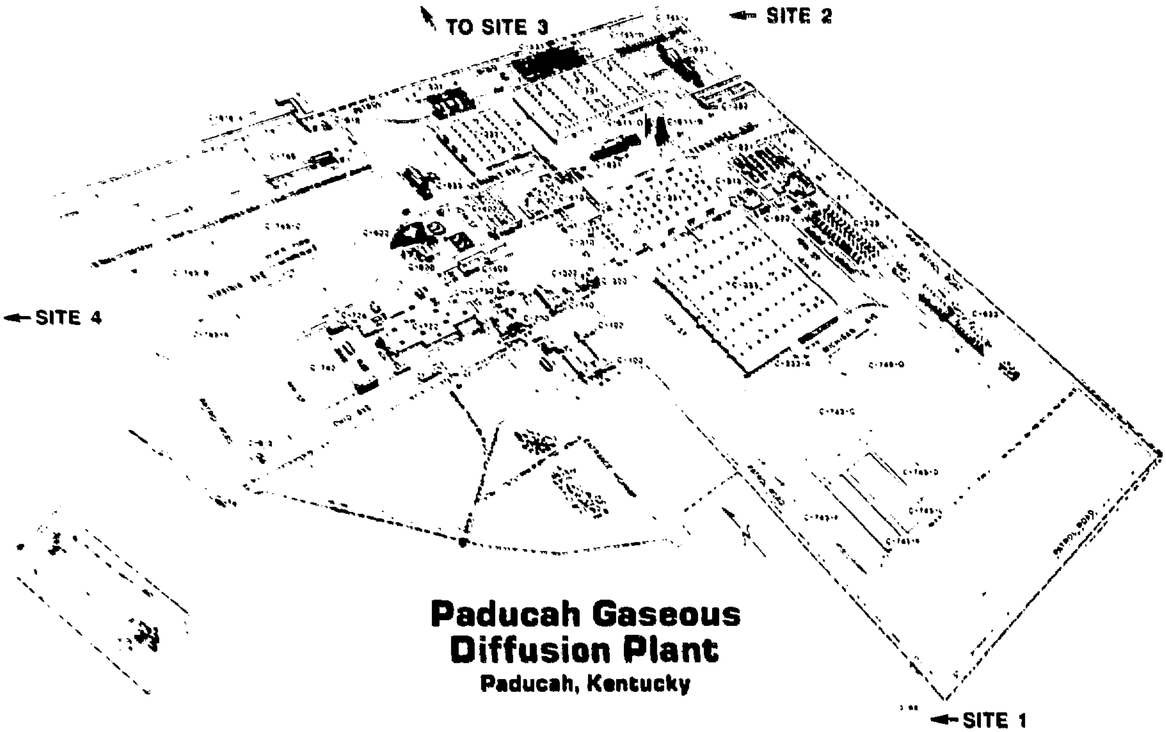

Fig. 5: Oblique plan view of PGDP and locations of sites used to derive soil columns

During the drilling activity, typical Standard Penetrometer Tests (SPT) were performed at frequent intervals in one of the shallow boreholes at each site and in the deeper borehole, where present, from a depth of around 125 ft to bedrock. The soil classification as a function of depth was generally determined from observations and drill cutting evaluations. In addition, a small number of split spoon and "undisturbed" samples were also obtained.[3]

The subsurface materials consist of a thin loess veneer overlying Continental Deposits of Pleistocene age then Tertiary-Age deposits of the Clayton-McNairy Formations. The area is underlain by hard limestone of Mississippian Age. At Site 1, 35 ft of loess and Continental Deposits overly 84 ft of the Porter's Creek Formation and the remaining 203 ft of soil are from the Clayton-McNairy Formation. At Sites 2, 3, and 4, the Continental Deposits have a thickness of about 112 ft and the Clayton-McNairy is about 210 thick. At Site 3, there is an additional layer of "rubble" above bedrock.

After completion of the drilling activity at a site, a suite of geophysical surveys were performed. Crosshole and downhole seismic methods were used to measure shear wave and compression wave velocities as a function of depth at each of the four sites. [4]

The soil column developed for Site 2 is shown in Figure 6 and generally reflects the stratigraphy and material properties for the other three columns. Solid lines represent demarcations between soil types whereas dashed lines represent depths at which material properties changed within a particular soil stratum. The depth of the phreatic surface at all sites, including Site 2, was determined from measurements in nearby monitoring wells. At Site 2, the depth of the phreatic surface is about 58 ft.

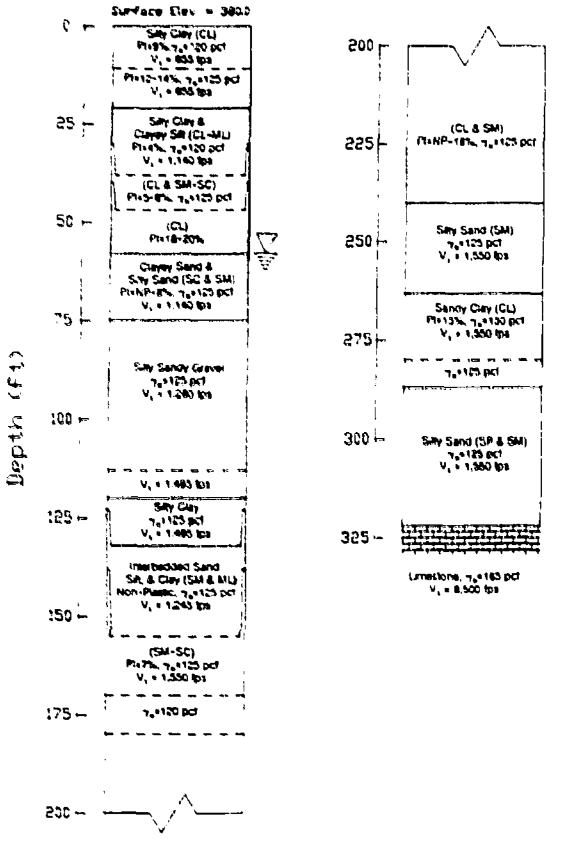

Fig. 6: Soil column for Site 2 (modified from [3])

The variations of shear moduli and damping ratios corresponding to materials encountered at PGDP were estimated using soil classifications based on several published results of laboratory studies.[6,7,8,9,10] These relationships are widely used by the geotechnical engineering profession when site-specific results of cyclic laboratory testing are unavailable and are shown in Figures 7 and 8.

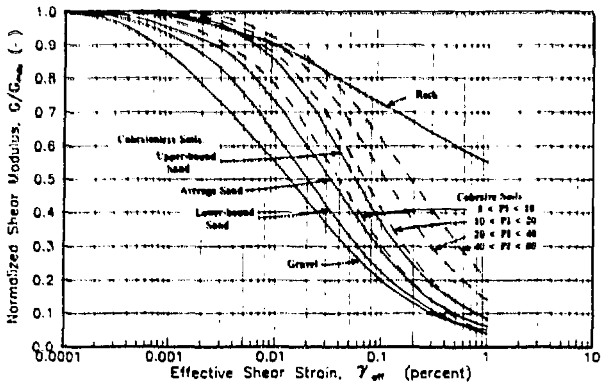

Fig. 7: Variations of shear moduli used for soil layers

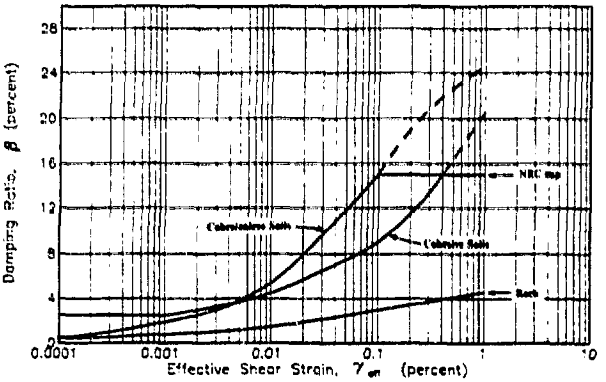

Fig. 8: Variations of damping ratios used for soil layers

METHOD OF CALCULATION

The computer program *SHAKE* [11] was used to calculate site-specific free-field response produced by the synthetic horizontal records assumed to represent horizontally-polarized shear waves propagating vertically. Calculations were made using the U.S. Army CRAY Y-MP supercomputer operated at WES. The methodology and algorithms incorporated in this program are fairly simple and straight-forward and quite adequate for the purpose intended. The wide acceptance of *SHAKE* and standardized modulus and damping curves is based primarily on the success in matching measured and calculated responses during moderate and strong ground motions.[e.g.,12,13]

The basic assumptions are:

1. Layers are horizontal;
2. Layers extend to infinity;
3. Layers are defined by shear modulus and damping as a function of shear strain, thickness, and unit weight;
4. Cyclic strength is adequately represented by a visco-elastic model implemented using the equivalent-linear method;
5. Incident earthquake motions travel vertically and contain shear waves only.

In general, these assumptions are consistent with site conditions at PGDP and are widely accepted by the geotechnical earthquake engineering profession as being suitable for most applications.

RESULTS FOR 1000-YEAR EVENT

The results of site response analysis for the 1000-year event are presented below. The shear modulus and damping ratio relationships used for this initial set of calculations correspond to best estimates based on soil classification. Results are presented in the form of pseudo-velocity response spectra in tripartite format (5 % damping), the ratio of absolute acceleration spectra for free field to outcrop, and a profile of acceleration records. The response of Site 2 to the Horizontal 2 component is used as an example of individual column response and then the results for all four sites and both horizontal components are combined.

The calculated pseudo-velocity spectra for five damping levels (2,5,7,10,and 12%) at Site 2 for the Horizontal 2 component are shown in Figure 9. The spectra are independent of damping ratio and smooth at periods below 0.06 sec. Significant variations with damping ratio occur at periods greater than 0.15 sec. The natural period at Site 2 is about 1.1 sec.

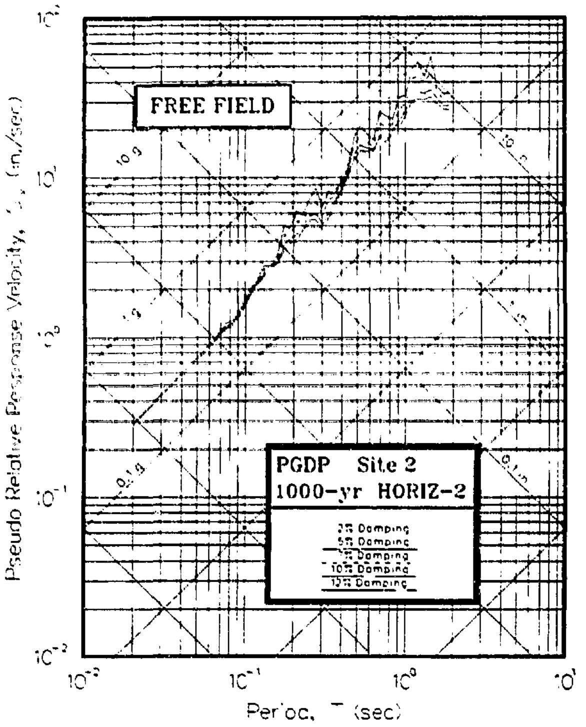

Fig. 9: Response spectra calculated for Site 2

The ratios of response acceleration spectra for free field to rock outcrop for Site 2 and the Horizontal 2 component are shown in Figure 10. Motions are amplified for periods greater than 0.3 sec. The greatest amplification occurs between periods of 1.0 and 1.3 sec.

The general effect of the attenuation of motion from the soil profile on the propagation of shear wave energy can be seen in Figure 11 where the calculated acceleration records at the top of various layers are shown. In general, amplitudes decrease and periods decrease as the waves propagate vertically.

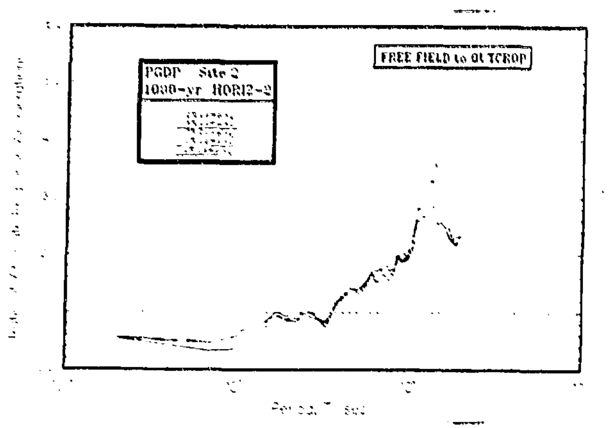

Fig. 10: Ratio of acceleration spectra for free field to rock outcrop motions at Site 2

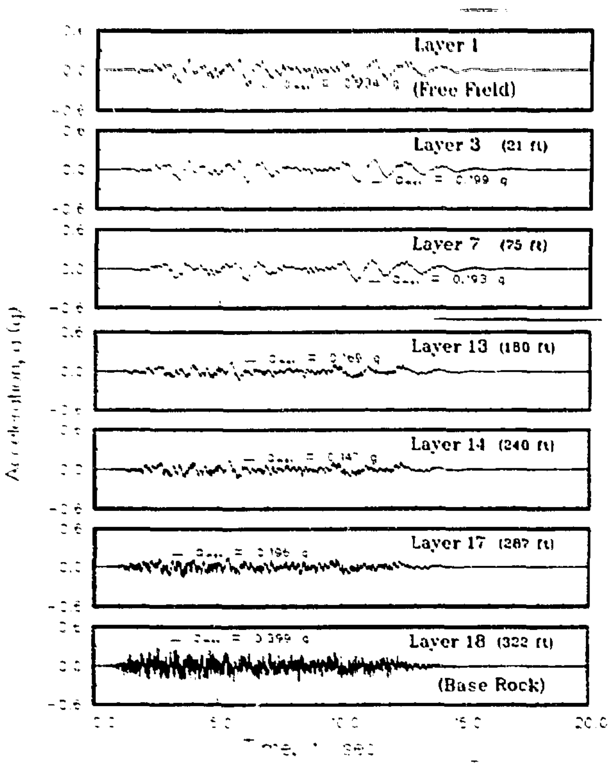

Fig. 11: Profile of calculated acceleration records at Site 2

The set of results for all four sites and both horizontal components at five percent damping are shown in Figures 12 and 13. The data indicate that strong free-field motions should be expected. The range of results is narrow except for two spectra that fall considerably below the pack at periods greater than 0.5 sec. These two responses were calculated for Site 3 and are attributed to the "rubble zone" noted earlier. This zone attenuated a significant portion of the energy for this initial analysis.

The calculated responses at 5 percent damping for Sites 1, 2 and 4 are similar. The natural period is between 1.0 and 1.5 sec with a (peak) spectral velocity of about 60 in/sec. A secondary resonance peak exists at a period of about 0.5 sec. The maximum horizontal acceleration is 0.28 g.

Earthquake motions may be amplified at periods greater than 0.15 sec although large amplification occurs at periods greater than 0.3 sec. The ratios of free-field to rock outcrop response are between 3.5 and 4.0.

FURTHER ANALYSES & PARAMETRIC STUDIES

Following the initial set of calculations conducted by asigning shear modulus and damping ratio curves on the basis of soil classification alone, the effects of confining stress will be applied to the set of analyses. High confining stresses, experienced at great depths, tend to reduce the amount of modulus reduction and amount of damping ratio for a given shear strain. [9,10] The effect of this consideration on the response spectra is likely to increase the free-field motions (acceleration and displacement) and decrease natural period moderately.

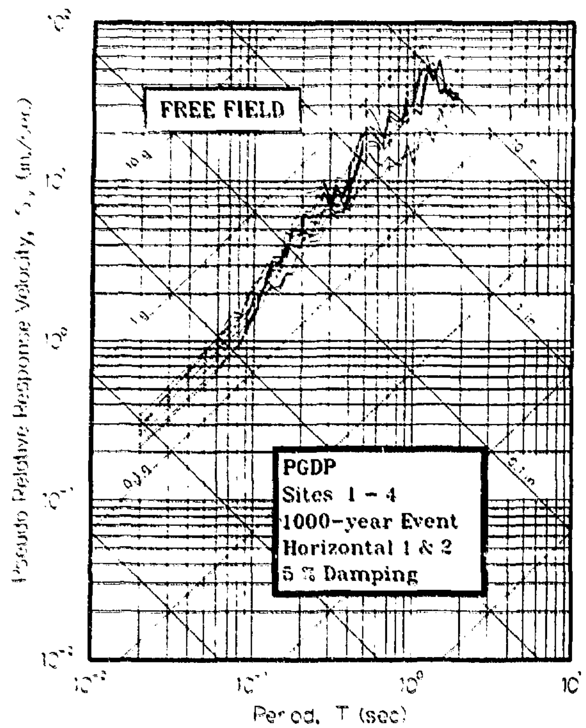

Fig. 12: Combined response spectra for four sites and two horizontal components

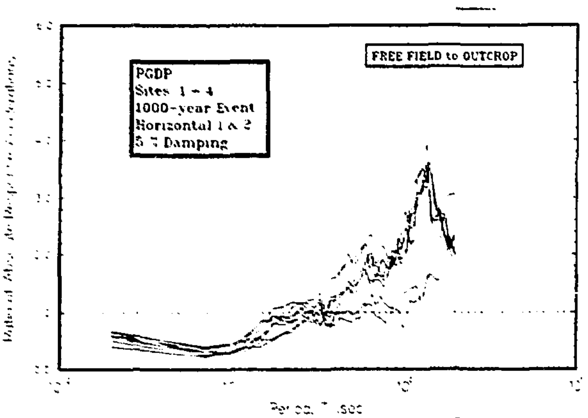

Fig. 13: Combined acceleration ratios of acceleration spectra for four sites and two horizontal components

Parametric studies are also in progress for the 1000-year design event to evaluate the sensitivity of various input parameters to the response calculations. The basis for upper and lower bounds of variation are based on potential measurement inaccuracies and natural variations. Parameters being considered are maximum shear modulus (shear wave velocity and density), depth to bedrock, and the modulus and damping ratio curves assigned to individual layers.

The results of parametric analyses completed for the 1000-year event show that the calculations tend to be insensitive to the depth of bedrock and the exact soil classification (used to assign shear modulus and damping ratio relationships). The calculations are moderately sensitive to the magnitude of shear modulus (shear wave velocity).

SUMMARY AND CONCLUSIONS

The results of site-specific earthquake response analyses at the Paducah Gaseous Diffusion Plant, near Paducah, Kentucky, for a 1000-year design event have been presented. Four soil columns with heights of 322 and 364 ft derived for this study using the results of in-situ measurements were evaluated for site response separately. Two horizontal components of motion were propagated through each column.

The compilation of results indicates that strong free-field motions should be expected. Peak horizontal free-field accelerations were calculated to be about 0.28 g. The natural period is expected to be between 1.0 and 1.5 sec and large amplification is expected for periods of motion greater than 0.3 sec. A secondary resonance peak exists at a period of about 0.5 sec.

The results using *SHAKE* are considered to be reasonable and realistic based on previous validation studies published in engineering literature. Evaluations using more conservative assumptions about inputs affecting material properties and parametric analyses are underway.

ACKNOWLEDGEMENTS

The calculations and analyses described, unless otherwise noted, were sponsored by the U.S. Department of Energy, Oak Ridge Operations, under Contract DE-AC05-84OR21400 with Martin Marietta Energy Systems, Inc., and IAG No. DE-AI05-91OR21971 with the U.S. Army Engineer Waterways Experiment Station. Permission to publish this information was obtained from DOE, Martin Marietta Energy Systems, Inc., and the Chief of Engineers.

BIBLIOGRAPHY

[1] R. Kennedy, S. Short, J. McDonald, M. McCann, Jr., R. Murray, and J. Hill,"Design and Evaluation Guidelines for Department of Energy Facilities Subjected to Natural Phenomena Hazards," UCRL-15910, Lawrence Livermore Nat'l Lab, Livermore, CA, 1990.

[2] Risk Engineering, Inc., "Seismic Hazard Evaluation for the Paducah Gaseous Diffusion Plant, Paducah, Kentucky," Rpt to Martin Marietta Energy Systems, Inc., July, 1991.

[3] ERCE,"Recommended Soil Columns for Use in Amplification Studies," Unpub. Rpt to Martin Marietta Energy Systems, Inc., ERCE File No. B672, Knoxville, TN, 26 Nov 1990.

[4] ASG, Inc., "Geophysical Study of Subsurface Conditions in the Vicinity of the Paducah Gaseous Diffusion Plant," ASG/U-101, Final unpub. data package submit. Martin Marietta Energy Systems, Inc., Oak Ridge, TN, 14 Feb 1991.

[5] W. Staub, J. Wong, and R. Selfridge, "Assessment and Interpretation of Cross- and Downhole Seismograms at the Paducah Gaseous Diffusion Plant," Oak Ridge Nat'l Lab, Oak Ridge, TN, 1991.

[6] B. Hardin & V. Drnevich, "Shear Modulus and Damping in Soils: Design Equations and Curves," Journal, Soil Mechanics and Foundation Engr., ASCE, vol. 98, no. SM7, pp 667-692, 1972.

[7] P. Schnabel, "Effects of Local Geology and Distance from Source on Earthquake Ground Motions," PhD Thesis, Univ Calif, Berkeley, CA.

[8] H. Seed and I. Idriss,"Soil Moduli and Damping Factors for Dynamic Response Analysis," Rpt EERC-87/15, Earthquake Engr Res Cntr, Berkeley, CA, 1970.

[9] H. Seed, R. Wong, I. Idriss, and K. Tokimatsu, "Moduli and Damping Factors for Dynamic Analysis of Cohesionless Soils," Journal, Geotech. Engr. Div., ASCE, vol. 112, no. 11, pp 1016-1032, 1986.

[10] J. Sun, R. Golesorkhi, & H. Seed, "Dynamic Moduli and Damping Ratios for Cohesive Soils," Rpt EERC-88/15, Earthquake Engr Res Cntr, Berkeley, CA, 1988.

[11] P. Schnabel, J. Lysmer, and H. Seed,"*SHAKE*: A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites," Rpt EERC-72/12, Earthquake Engr Res Cntr, Berkeley, CA, 1972.

[12] R. Seed, S. Dickenson, and I. Idriss, "Principal Geotechnical Aspects of the 1989 Loma Prieta Earthquake," Soils and Foundations, vol. 31, no. 1, pp. 1-26, March 1991.

[13] H. Seed, M. Romo, J. Sun, A. Jaime, and J. Lysmer, "Relationships Between Soil Conditions and Earthquake Ground Motions in Mexico City in the Earthquake of Sept. 19, 1985," Rpt EERC-87/15, Earthquake Engr. Res. Cntr, Berkeley, CA, 1987.