SEISMIC ANALYSIS OF A LIQUEFIED NATURAL GAS (LNG) STORAGE TANK USING STRAIN-DEPENDENT SOIL PROPERTIES

by

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ABSTRACT

A seismic soil-structure interaction response analysis of a liquefied natural gas [LNG] storage tank was conducted using a finite element model and strain-dependent soil properties. The effect of the LNG in the tank and frozen soil around the tank on the seismic response was considered. Three design conditions were analyzed: [1] empty tank surrounded by unfrozen soil, [2] full tank surrounded by unfrozen soil, and [3] full tank surrounded by frozen soil. Based on the results of the study, it may be concluded that: [1] the frequency content of the ground motions (reflected by response spectra) does not change when the soil around the tank is frozen, [2] calculated maximum absolute accelerations are lower when the soil around the tank is frozen and are slightly lower when the liquid stored inside the tank is considered in the analysis compared to maximum absolute accelerations for the empty tank and unfrozen soil, and [3] the predominant frequency of the ground motion of the frozen soil around the tank matches the predominant frequency of the ground motion of the adjacent unfrozen soil deposit.

INTRODUCTION

The demand for pollution-free energy has led to accelerated development of natural gas fields and increased imports of liquefied natural gas [LNG]. Since natural gas is toxic and highly flammable, strict safety measures must be observed when the gas is stored. Underground storage tanks are used to reduce hazards associated with leakage and the cost of acquiring large storage areas. When the LNG storage structure is massive, embedded in the ground, and subjected to earthquake loadings, a soil-structure interaction response analysis must be conducted to reflect the effect of the soil on the response of the structure and the effect of the structure on the response of the soil.

Fig. 1 shows an underground LNG storage tank at the Negishi base in Japan (1). The LNG is stored at a temperature of approximately -160°C, thus, a zone of frozen soil will develop around the tank. In recognition of the need to demonstrate interaction effects between the unfrozen soil, frozen soil and the concrete containment structure, a seismic analysis was conducted using a finite element model and strain-dependent soil properties. FLUSH (2), a computer program for approximate analysis of three-dimensional soil-structure interaction problems, was employed for three design conditions: [1] empty tank surrounded by unfrozen soil, [2] full tank surrounded by unfrozen soil, and [3] full tank surrounded by frozen soil. Acceleration response spectra at designated nodal points were obtained for each design condition. Response spectra and maximum ground accelerations were compared to determine the influence of the frozen soil around the tank on the seismic response of the tank and the surrounding soil. The results of these studies are reported herein.

COMPUTATIONAL METHOD

The computational method used in this study, proposed by Hwang, et al (3), is a pseudo three-dimensional method (or "simplified" three-dimensional method). The method employs a plane strain finite element model to which viscous dampers are added along the plane of the model, parallel to the plane, to simulate energy dissipation in the third dimension due to the radiation of waves generated by the vibration of the structure. The viscous boundary approach is based on the established analogy between the dynamic response of a uniformly loaded elastic half space (loaded with a uniform normal stress or a uniform shear stress) and a viscous dashpot.

Further, in the finite model of the soil-structure system fictitious boundaries are specified. The bottom boundary is assumed to be rigid. If the soil deposit is resting on much stiffer, rock-like material, the bottom boundary is set at the interface between the soil and the rock. When dealing with a deep soil deposit, the bottom boundary must be placed far enough from the structure to insure the boundary will not reflect any energy back to the structure and, hence, affect its response to a seismic excitation. Transmitting boundaries which absorb any wave effects emanating from the structure at the ends of the model are provided. They simulate the effects of an extensive soil deposit and allow the dimensions of the model to be drastically reduced. The viscous boundaries specified along the planar surfaces of the soil slice absorb or transmit a train of plane body waves.

COMPUTATIONAL MODEL

The representation of the LNG tank considered in the study (re. Fig. 1) is shown in Fig. 2. Owing to symmetry it was only necessary to model one-half of the tank. The model consists of displacement-compatible isoparametric quadrilateral solid elements (2). Very soft, light solid elements were used to model the void inside the tank. The lower bound-

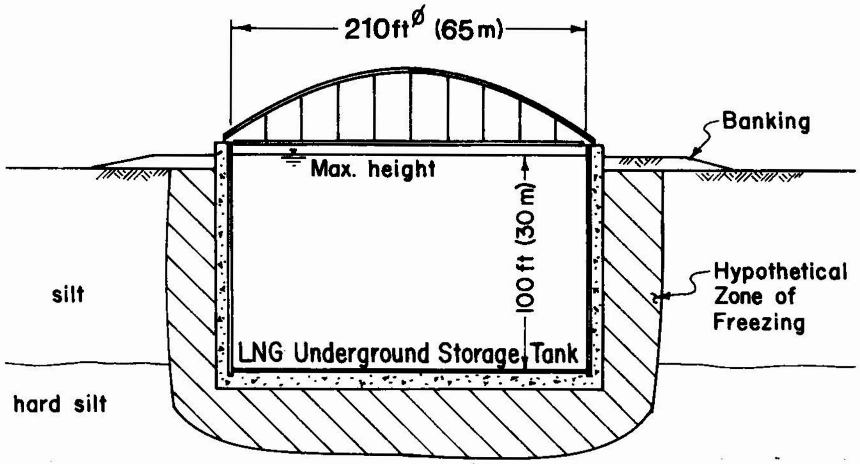

FIGURE 1 - LNG UNDERGROUND STORAGE TANK with HYPOTHETICAL ZONE of FREEZING at NEGISHI BASE, JAPAN

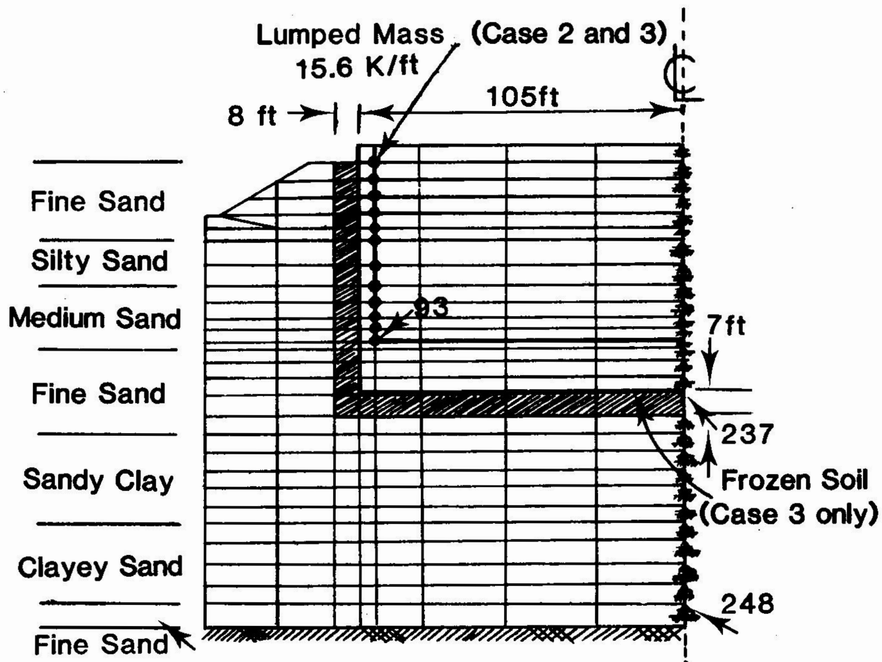

FIGURE 2 - COMPUTER MODEL for CASE 1, 2 and 3

ary of the model is assumed to be rigid and to translate according to a horizontally acceleration time history $\ddot{y}(t)$. The three-dimensional effect was approximated using viscous boundaries along both sides of the model slice. When viscous boundaries are used each solid element must be associated with a soil layer in the free field. Nodes on the line of symmetry were restricted from translating vertically because the study was performed for a horizontal excitation.

In the first case study considered the tank was empty and the surrounding soil was unfrozen. The soil and material properties employed are summarized in Table 1 (4). Non-linear soil behavior was approximated through equivalent linearization procedures (5). This approach requires an iterative linear analysis with a linear visco-elastic material model, which is defined by two elastic constants, shear modulus and Poisson's Ratio, and a damping factor. Reduction factors for shear modulus and damping proposed by Seed and Idriss (6) for sand and clay were incorporated in the computational effort to reflect the strain dependency of the dynamic soil properties.

The second case study considers the influence of the LNG stored in the tank on dynamic reponse. The total mass of LNG was equally distributed at nodal points along the inner walls of the tank as shown in Fig. 2. The lumped masses, rigidly connected to the tank, moves with the tank walls. The soil in the vicinity of the tank was considered to be unfrozen, simulating a condition immediately following construction and initial filling of the tank or a condition in which a heating system surrounding the tank is employed to prevent ground freezing. The soil properties are the same as those used for Case 1. The third case study considers the tank filled with LNG with a 7 ft. zone of frozen soil surrounding the tank. The dynamic properties of the frozen soils incorporated in the analysis were obtained from Vinson, et al (7) and are summarized in Table 1. The soil properties for the unfrozen zone are the same as those used in Case 1 and 2. Reduction factors for shear modulus and damping were developed and incorporated in the computational effort to reflect the strain dependency of the dynamic frozen soil properties.

DESCRIPTION OF SEISMIC INPUT MOTION

The seismogram from the San Luis Obispo earthquake of June 27, 1966, N36W component (maximum acceleration = 0.014g and predominant period = 0.15 sec.) was modified for use in the study. The original record consists of a horizontal time history of acceleration digitized at N = 1484 points at a time interval Δt = 0.02 sec.

The design earthquake magnitude and the epicentral distance chosen for the study were as follows: Richter Magnitude M = 7.0 and epicentral distance = 10 miles. The design rock motion characteristics (8) associated with the Richter magnitude and distance are: maximum acceleration (a_{max}) = 0.4g and predominant period (T_p) = 0.32 sec (predominant frequency = 3.1Hz). To reflect the design earthquake motion, the original San Luis Obispo record was modified (9) to yield a_{max} = 0.4g, T_p = 0.32 sec, and Δt = 0.04 sec. Further, to reduce the computational effort, the original record was decimated (this process consists of throwing away excess points in a sequential order). Decimation led to the use of a modifed San Luis Obispo record digitized at N = 368 points. Also, high frequencies in the control motion may be neglected without affecting the accuracy of the solution. A cutoff frequency of 8Hz for earth dams and 25Hz for nuclear power plants is reasonable (2). A cutoff frequency of 8.5Hz was employed in this study.

The smoothed spectrum for the control motion was compared to the mean plus 1 standard deviation spectrum obtained by Seed, et al (10) for rock motions based on 28 records. The smooth spectrum for the control motion used in the study was found to be in good agreement with the mean and mean plus one spectrum.

RESPONSE OF UNDERGROUND LNG STORAGE TANK TO EARTHQUAKE LOADINGS

Acceleration response spectra and maximum ground accelerations were obtained for nodal point 93 (side of the tank), nodal point 237 (base of the tank on the centerline), and nodal point 248 (base of computational model on the centerline) (refer to Fig. 2). Acceleration response spectra at nodal point 93 corresponding to the three case studies considered are shown in Fig. 3a. Results from Case 1 and Case 2 indicate that Case 1 (unfrozen soil, empty tank) gave spectral amplifications slightly higher than the amplifications for Case 2 (unfrozen soil, full tank). At the predominant frequency of the ground motion, f = 3.1Hz, the spectral acceleration for Case 1 was 0.48g compared to 0.46g for Case 2. The location of the peaks and the overall shape of the response spectra, corresponding to Case 2 (unfrozen soil, full tank) and Case 3 (frozen soil, full tank) are in good agreement. However, it was noted that at frequencies greater than 1.0Hz, Case 2 gave spectral amplifications higher than those corresponding to Case 3. At the first natural frequency of the soil deposit, f = 3.1Hz, spectral accelerations for Case 2 and Case 3 were 0.46g and 0.38g, respectively.

The influence of the presence of the tank and the frozen soil zone may be evaluated by comparing Figures 3a and 4a. Note that the position of nodal point 93 corresponds to layer number 9. Figure 4a shows the free-field spectrum at the top of layer number 9. At the predominant frequency of the control motion, f = 3.1Hz, the free-field spectral acceleration was slightly higher, 0.47g versus 0.46g. Profound differences were observed when Case 3 (frozen soil, full tank) was compared to the free-field response at the top of layer number 9. At f = 3.1Hz, the spectral acceleration from Case 3 was 0.38g compared to 0.47g for free-field.

Maximum absolute accelerations from the 1st, 2nd and 3rd Case study at nodal point 93 are presented in Table 2. It should be noted that truncation of the control motion at 8.5Hz reduced the maximum acceleration from 0.40g to 0.33g. This magnitude of reduction is to be expected (2). It was observed that in Case 1 and Case 2 the maximum accelerations are equal. It was also noted that the maximum acceleration for Case 1 or 2 is higher than Case 3, 0.17g versus 0.14g.

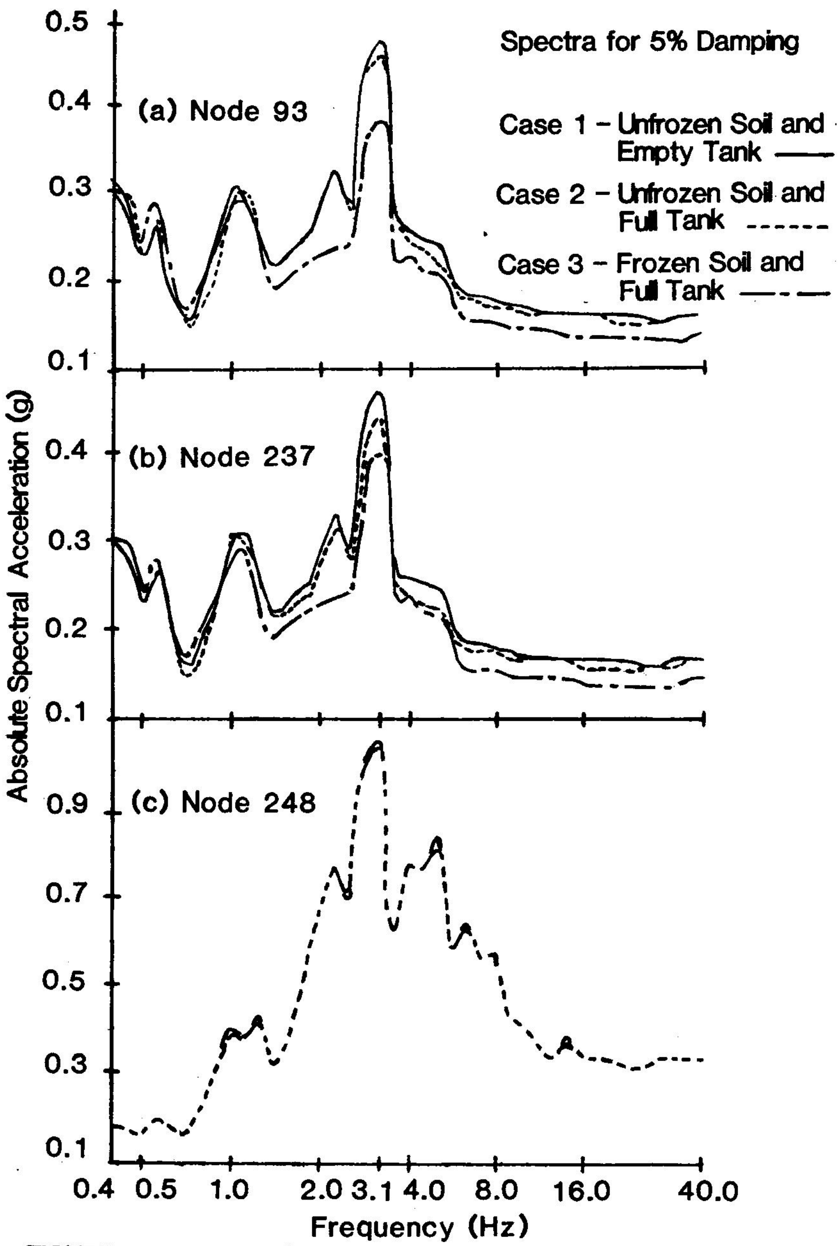

FIGURE 3 - RESPONSE SPECTRA at NODES 93, 237 and 248

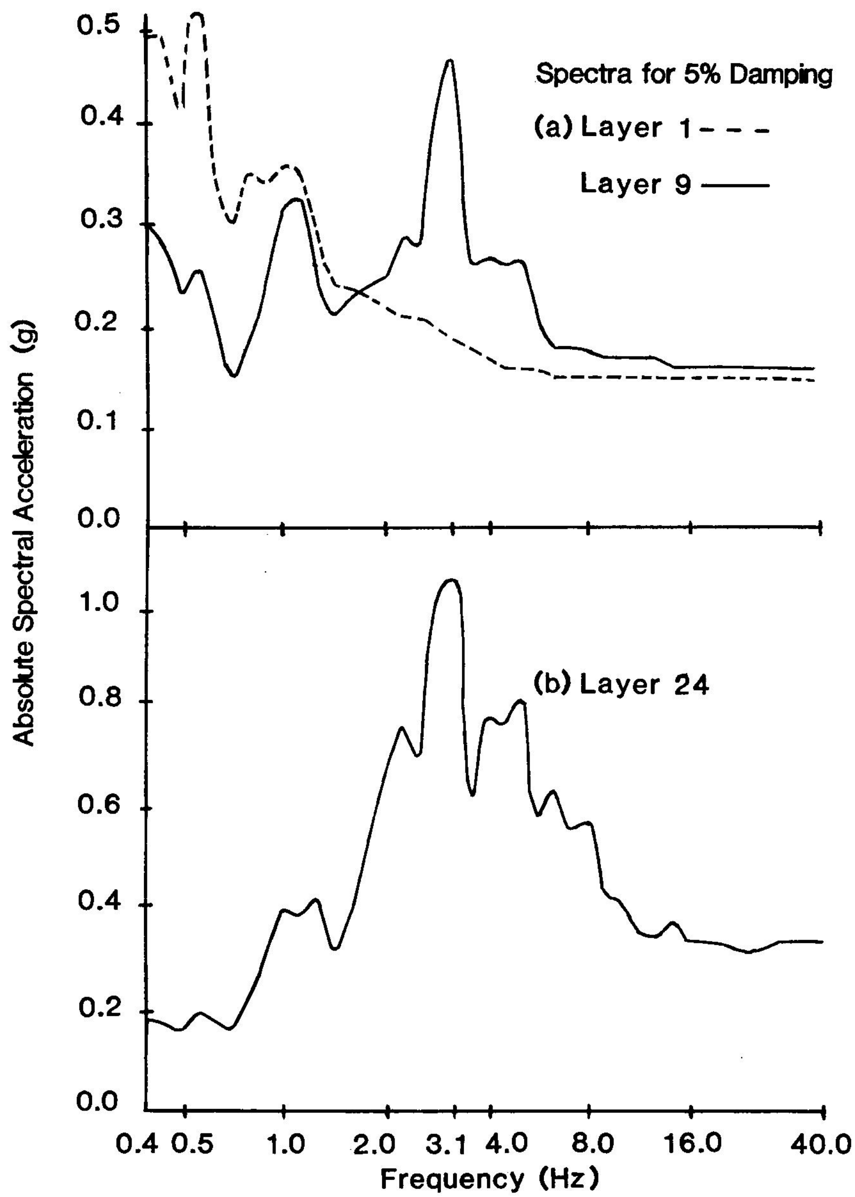

FIGURE 4 - FREE-FIELD RESPONSE SPECTRA
at TOP of LAYER 1, 9 and 24

The results shown in Fig. 3a indicate that the frequency content of the ground motion (reflected by response spectra) at nodal point 93, did not change when the frozen soil zone (around the tank) was considered.

Acceleration response spectra at nodal point 237 corresponding to the three case studies are presented in Fig. 3b. The results are similar to those obtained at nodal point 93. Maximum spectral accelerations corresponding to Case 1 and Case 2 were 0.47g and 0.45g, respectively. The location of the peaks and the overall shape of the response spectra corresponding to Case 2 and Case 3 were in good agreement. For frequencies higher than 1.0Hz, Case 3 (frozen soil, full tank) gave spectral amplifications that were lower than the spectral amplifications obtained from Case 2.

Refering to Table 2, it may be observed that in Case 2 the maximum absolute acceleration is higher than in Case 3, 0.17g versus 0.14g. From Fig. 3b may can be noted that the frequency content of the ground motions (reflected by response spectra) at nodal point 237 did not change when the frozen zone was included in the analyses. Further, the predominant frequency of the ground motion of the frozen soil zone around the tank matches the predominant frequency of the ground motion of the unfrozen soil deposit.

Fig. 3c presents the response spectra at nodal point 248 for the three case studies. The response corresponding to Case 1 matches the reponse corresponding to Case 2. Case 3 (frozen soil, full tank) gave spectral amplifications that are slightly higher than the spectral amplifications obtained from Case 2 (unfrozen soil, full tank). At the predominant frequency of the ground motion, f = 3.1Hz, the spectral acceleration corresponding to Case 2 was 1.1g compared to 1.16g for Case 3. This difference may be explained by comparing the spectra shown in Figures 3c and 4b. Figure 4b represents the free-field response spectrum at the top of layer number 24. Note that the position of nodal point 248 with respect to the free-field layered system, corresponds to layer number 24. It was observed that the response corresponding to Case 2 matches the response corresponding to the free-field motion at the top of layer number 24. It was also noted that Case 3 gave peak spectral accelerations slightly higher than the peaks corresponding to the free-field response at the top of layer number 24. Therefore, it may be concluded that the influence of the presence of the tank and the frozen soil zone reach the assumed rigid base of the finite element model. However, a weak reflection appears to occur at this boundary as evidenced by the slightly higher peaks in the response corresponding to Case 3.

CONCLUSIONS

Based on the results presented in this paper, the following conclusions may be drawn: [1] the frequency content of the ground motions (reflected by response spectra) does not change when the soil around the tank is frozen, [2] calculated maximum absolute accelerations are lower when the soil around the tank is frozen, [3] calculated maximum absolute accelerations are slightly lower when the liquid stored inside the tank is considered in the analysis compared to maximum absolute accelerations for the empty tank and unfrozen soil, [4] the predominant frequency of the ground motion of the frozen soil zone around the tank matches the predominant frequency of the ground motion of the adjacent unfrozen soil deposit, and [5] for the assumed depth of the frozen soil zone (extending to 8 ft below the base of the tank) and for the size of the tank analyzed (height = 86 ft and diameter = 210 ft) specifying rigid boundary conditions at the base of the finite model at a depth of -138 ft was reasonable. In the event that the frozen soil zone is extended deeper from the base of the tank, the authors suggest taking the rigid base of the finite element model at a level lower than the depth assumed in the present study. This will insure that the rigid boundary will not reflect energy back to the finite element model and, hence, affect the predicted response of the tank.

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REFERENCES

(1) ____________, "LNG Underground Storage Tanks," Technical Report 12, Shimizu Construction Co. Ltd., Tokyo, Japan, 1977.

(2) Lymser, J., Udaka, T., Tsai, C., and Seed, H.B., "FLUSH - A Computer Program for Approximate 3-D Analysis of Soil-Structure Interaction Problems", Report No. EERC 75-30, University of California, Berkeley, 1975.

(3) Hwang, R., Lysmer, J., and Berger, E., "A Simplified Three-Dimensional Soil-Structure Interaction Study", Proceedings of the Second ASCE Specialty Conference on Structural Design of Nuclear Plant Facilities, Vol. I-A, , New Orleans, December 1975, pp. 786-808.

(4) Takewaki, N., Kurahashi, K., Nakahi, S., and Ishii, K., "Dynamic Reponse Analysis and Earthquake Observation of In-Ground Tank", Shimizu Construction Co. Ltd., Tokyo, Japan, 1978.

(5) Seed, H.B., and Idriss, I., "Influence of Soil Conditions on Ground Motions During Earthquakes", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 94, No. SM1, January 1969.

(6) Seed, H.B., and Idriss, I., "Soil Moduli and Damping Factors for Dynamic Response Analysis", Report No. EERC 70-10, University of California Berkeley, 1970.

(7) Vinson, T.S., Czajkowski, R., and Li, J., "Dynamic Properties of Frozen Cohesionless Soils Under Cyclic Triaxial Loading Conditions," Report No. MSU-CE-77-1, Div. of Engineering Research, Michigan State University, 1977.

(8) Seed, H.B., Idriss, I., and Kiefer, F., "Characteristics of Rock Motions During Earthquakes", Report No. EERC 68-5, University of California, Berkeley, 1968.

(9) Schnabel, P., Lysmer, J., and Seed, H.B., "SHAKE - A Computer Program of Earthquake Response Analysis of Horizontally Layered Sites", Report No. EERC 2-12, Univ. of California, Berkeley, 1972.

(10) Seed, H.B., Ugas, C. and Lysmer, J., "Site-Dependent Spectra for Earthquake Resistant Design," Report No. EERC 74-12, University of California, Berkeley, 1972.

Table 1 - Concrete and Soil Properties for Finite Element Model

		Unfrozen			Frozen		
Depth Interval (ft)	Soil Type	Poisson's Ratio	Unit Weight (pcf)	Shear Modulus at Low Strain (kips/sq ft)	Poisson's Ratio	Unit Weight (pcf)	Shear Modulus at Low Strain (kips/sq ft)
0-9	Fine Sand	0.47	120.0	785	0.28	118.3	17,700
9-24	Silty Sand	0.49	116.5	471	0.28	118.3	17,700
24-44	Medium Sand	0.49	126.2	1,523	0.3	126	207,000
44-74	Fine Sand	0.47	123.6	6,613			
74-103	Sandy Clay	0.48	106.0	3,403			
103-129	Clayey Sand	0.47	123.0	5,940			
129-138	Fine Sand	0.46	126.0	9,722			
---	Concrete	0.17	155.0	187,221			

Table 2 - Maximum Absolute Accelerations for Case Studies

Nodal Point	X-Acceleration (Case 1)	X-Acceleration (Case 2)	X-Acceleration (Case 3)
93	0.17 g	0.165 g	0.14 g
237	0.17 g	0.165 g	0.14 g
248	0.34 g	0.34 g	0.34 g