SOIL - STRUCTURE INTERACTION EFFECTS ON HIGH LEVEL WASTE TANKS

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ABSTRACT

High Level Waste Tanks consist of steel tanks located in concrete vaults which are usually completely embedded in the soil. Many of these tanks are old and were designed to seismic standards which are not compatible with current requirements. The objective if this paper is to develop simple methods of modeling SSI effects for such structures and to obtain solutions for a range of parameters that can be used to identify significant aspects of the problem.

INTRODUCTION

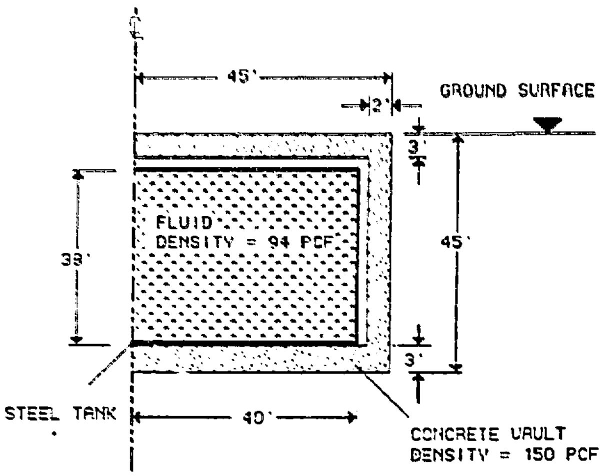
The Department of Energy (DOE) operates many tanks which are used to store radioactive waste material. These tanks were constructed over a long time span and the original design of the tanks was often based on criteria which did not meet current seismic codes. As a result DOE is undertaking a comprehensive review of the adequacy of these facilities to meet current seismic standards.

Two methods are available for performing seismic response analyses of structures. The first is the simpler of the two and couples a discrete model of the structure to the free field with a lumped parameter model of the soil-structure interaction (SSI) process. The second is based on continuum models of the structure and free field which are coupled through continuity conditions at the soil-structure interface.

The general objective of the work reported in this paper is to investigate the vault characteristics that are important to include in their seismic evaluations. These results will then be used to develop criteria that can be used to establish minimum requirements for the analyses used to determine the seismic response of the tanks. The results presented in this paper are based on the lumped parameter method. The methodology is developed, and numerical results are obtained to achieve the objective.

ANALYSIS

Consider the tank/vault system shown in Fig. 1. The dimensions shown in Fig. 1 are typical of those found in the DOE vaults and are those used for the numerical solutions shown below. The analysis is developed, however, for arbitrary values of these parameters. For the purposes of this analysis the vault is assumed to be rigid, and the impulsive mass of the fluid is modeled at the appropriate elevation. The impulsive fluid mass and its elevation is computed based on the methodology contained in [1] assuming that the tank is also rigid. The vaults for most of the DOE tanks are sufficiently stiff so that the rigid vault assumption is reasonable. While the tanks are probably not rigid, the influence of tank flexibility on the seismic response of the vault is likely to be small.



VAULT/TANK CHARACTERISTICS

VAULT MASS	= 278.2 K-SEC^2/FT
FLUID MASS	= 555.6 K-SEC^2/FT
IMPULSIVE FLUD MASS	= 289.2 K-SEC^2/FT
HEIGHT OF IMPULSIVE	= 14.25 FT
VAULT MASS + IMPULSIVE MASS	= 567.5 K-SEC^2/FT
VAULT AND IMPULSIVE MASS CG	= 19.8 FT
ROTARY INERTIA ABOUT BASE	= 367,000 K-SEC^2-FT

Fig. 1 VAULT/TANK GEOMETRY

The vault model shown in Fig. 1 is coupled to the soil media with lumped spring/damper elements modeling soil-structure-interaction (SSI) effects at the base and sidewalls. Beredugo and Novak [2] have developed such a lumped parameter methodology for computing the response of partially buried facilities to seismic disturbances. The characteristics of the spring/damper model are developed based on analytical solutions to the case of a rigid structure vibrating in an elastic half space. The base force (F) and moment about the base (M) resulting from the SSI forces are shown on Fig. 2 and given in [2] as:

$$F = K_{uu}\,(U-Z) + K_{u\phi}\,\Phi + C_{uu}\,(\dot{U}-\dot{Z}) + C_{u\phi}\,\dot{\Phi}$$

$$M = K_{u\phi}\,(U-Z) + K_{\phi\phi}\,\Phi + C_{u\phi}\,(\dot{U}-\dot{Z}) + C_{\phi\phi}\,\dot{\Phi} \qquad (1)$$

where,

Z = soil displacement in absence of vault

$$K_{uu} = G\,R\,[\,4.83 + 4.05\,G_s\,\delta\,/\,G]$$

$$K_{\phi\phi} = G\,R^3\,[\,2.50 + 4.83\,(H/R)^2 + 2.50 G_s\,\delta/G + 4.05\,G_s\,\delta\,\{\delta^2/3 + (H/R)^2 - \delta\,(H/R)\}/\,G]$$

$$K_{u\phi} = -G\,R\,[4.83\,H + 4.05\,G_s\,\delta\,\{H - L/2\}\,/\,G]$$

$$C_{uu} = (\rho\,G)^{1/2}\,R^2\,[\,3.00 + 9.90\,\delta\,(\rho_s\,G_s/\,\rho\,G)^{1/2}\,]$$

$$C_{\phi\phi} = (\rho G)^{1/2} R^4 [\, \mathbf{0.43 + 3.00\, (H/R)^2} + \delta\, (\rho_s G_s / \rho G)^{1/2} \{ \mathit{1.80} + \{\delta^2/3 + (H/R)^2 - 9.90\, \delta\, (H/R)\}]$$

$$C_{u\phi} = -(\rho G)^{1/2} R^2 [\, \mathbf{3.00\, H} + 9.90\, \delta\, (\rho_s G_s / \rho G)^{1/2} (H - L/2)]$$

δ = Depth to radius ratio of vault
G = Shear modulus of foundation soil
G_s = Shear modulus of soil along sidewall
ρ = Mass density of foundation soil
ρ_s = Mass density of soil along sidewall
H = Height of vault CG above base

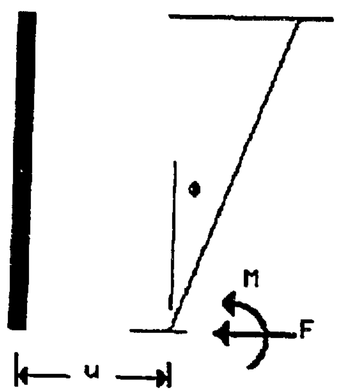


Fig.2 SSI FORCES

The **BOLD** terms involve base interaction effects; non bold involves sidewall interaction effects; and the *ITALIC* terms arise from shears along the cylinder. The shear terms are neglected in this analysis.

The portion of the SSI forces developed from the base interaction effects may be treated as concentrated forces but the portion resulting from sidewall interaction effects must be distributed over the surface of the wall. The spring/damper SSI model connects the structure to the free field. A distributed wall SSI model is required so that variable free field motions may be imposed at different elevations along the wall. The distributed parameters also allow for the computation of pressures acting on the wall. The parameters are distributed by assuming that the sidewall portion of the stiffness and damping coefficients defined above are uniformly distributed over the surface of the vault and act in the radial direction. The resultant pressures acting on the vault surface will be in the radial direction (i.e., normal pressures), will have a cosine variation around the circumference of the vault, and will vary over the depth of the vault. The distributed parameters will be:

$$k = K_{uusw} / \pi R L \qquad (2)$$

$$c = C_{uusw} / \pi R L$$

where the (sw) subscript indicates that the sidewall (non bold and non italic) portion of the above coefficients are to be used.

Two cases involving the pressures acting on the wall are considered. The first set of problems is linear so that no maximum or minimum restrictions are placed on the pressures. The second set of problems imposes restrictions so that the pressure is restricted based on estimates of the soil strength. The result is that the maximum allowable compressive (p_c) and tensile (p_t) pressures are:

$$p_c = \sigma_v \; [\tan^2 (45 + \theta /2) - 1/2]$$

$$p_t = \sigma_v \; [\tan^2 (45 - \theta /2) - 1/2] \qquad (3)$$

where,

σ_v = vertical overburden soil stress

θ = angle of internal friction for soil

The SSI model, discussed above, consists of a spring/damper system connecting the structure to the free field. The free field motion is the required input to the problem. A time history is used which fits the 5% damped Reg Guide 1.60 spectral shape anchored to either 0.2 or 0.5 G. Convolution studies are performed, using the CARES computer code [3], with the criteria motion applied either at the surface or at an outcrop at the same elevation as the base of the vault. Compatible free field motions are computed over the depth of the vault so that there is the input to the SSI model is different at each elevation in the vault. For the purposes of comparison, solutions are also obtained based on the assumption that the field motion is constant with depth and equal to the motion at the vault's base elevation. Solutions are obtained for soil columns over the depth of the vault having shear wave velocities of 375 fps, 750 fps, and 1500 fps. The soil damping is taken to be 10% of critical. The material below the vault is taken either as a continuation of the soil above the vault foundation or as a rock having a shear wave velocity of 8200 fps. The rock damping is taken to be 2% of critical.

NUMERICAL RESULTS

Solutions are obtained by numerically integrating the equations of motion for the rigid vault system for the cases discussed above. Response spectra (5%) at the top and bottom of the vault are calculated, and wall pressures are determined. The wall pressures are presented in two forms. First, the pressure distribution over the depth of the wall at the time when the resultant force is maximum is shown. Then, the peak pressure at each layer is presented. Of course, these peak pressures will not occur at the same time. The depth of the vault is divided into five layers for each of the solutions.

The following parameters are varied: control point location (surface or rock outcrop at elevation of base of vault), ZPA (0.2 G and 0.5 G), variable or uniform free field motion over the vault depth, soil shear wave velocity over the vault depth (375 fps, 750 fps, or 1500 fps), material shear wave velocity below the vault (same as upper soil or 8200 fps rock), and maximum wall pressure cutoff or not. The identification of the figure numbers for the solutions are given in Table 1.

Table 1

PARAMETER VARIATIONS FOR VAULT SOLUTIONS

Figure No	Control Point	ZPA (G)	Variation of FF Motion	Soil V_s (fps) <45'	Soil V_s (fps) >45'	Pressure Cutoff
3	Outcrop	0.2	No	750	8200	No
4	Outcrop	0.2	Yes	750	8200	No
5	Outcrop	0.2	Yes	750	8200	Yes
6	Outcrop	0.2	Yes	750	750	No
7	Surface	0.2	Yes	750	750	No
8	Outcrop	0.2	Yes	1500	8200	No
9	Outcrop	0.2	Yes	375	8200	No
10	Surface	0.5	Yes	750	8200	No
11	Outcrop	0.5	Yes	750	8200	No
12	Outcrop	0.5	Yes	750	8200	Yes
13	Surface	0.2	Yes	750	8200	No

CONCLUSIONS

The following conclusions are developed from the study:

1. The criteria motion applied at the hard layer of a non uniform site results in a larger seismic response than the case where the criteria motion is applied at the surface. This can be demonstrated by comparing the solutions shown on Figs. 4 and 13. The peak spectral acceleration at the top of the tank increases from about 0.8 G for the surface control point to 2.6 G for the rock outcrop control point. The wall pressures increase by about a factor of two between the two cases.

2. When the soil is uniform with depth, there is little difference in the results obtained with the surface or "rock" outcrop control point. As may be seen by comparing the results on Figs. 6 and 7 the surface control point results are slightly higher but close to the outcrop results.

3. The variation of free field ground motion with depth has a significant effect on the results for the non uniform sites. This can be seen by comparing the results shown on Figs. 3 and 4. The wall pressures obtained from the variable free field input are about 4 times those computed by assuming uniform free field motion.

4. Response spectra calculated by restricting the peak wall pressures are only slightly lower than those obtained without placing such limits on wall pressures. This is shown by comparing the spectra on Figs. 4 and 5 for the 0.2 G ZPA case and 11 and 12 for the 0.5 G case. Of course, the peak pressures are reduced when limits are placed on the wall pressures. This result suggests that the seismic response calculations can be done without considering the non linearity associated with the pressure cutoff, and the resulting wall pressures reduced to account for the limits.

5. The effect of soil shear wave velocities can be seen by comparing the results on Figs. 4, 8, and 9.
6. A comparison of the response of a vault founded on rock with one founded on soil can be made by examining the results shown on Figs. 7 and 13. The response spectra are almost identical but the wall pressures for the rock based vault are about double those for a soil based vault.

REFERENCES

1. "Seismic Analysis of Safety-Related Nuclear Systems," ASCE 4-86.

2. Beredugo, and Novak, "Coupled Horizontal and Rocking Vibrations of Embedded Footings," Canadian Geotechnical Journal, January 1972.

3. "Computer Analysis for the Rapid Evaluation of Structures - CARES," Brookhaven National Laboratory, NUREG/CR-5588, 1989.

Fig 3 RG 1.60 (0.20g) @ ROCK OUTCROP
Vs (SOIL) = 750 ft/sec
UNIFORM INPUT OVER DEPTH
NO PRESSURE CUTOFF

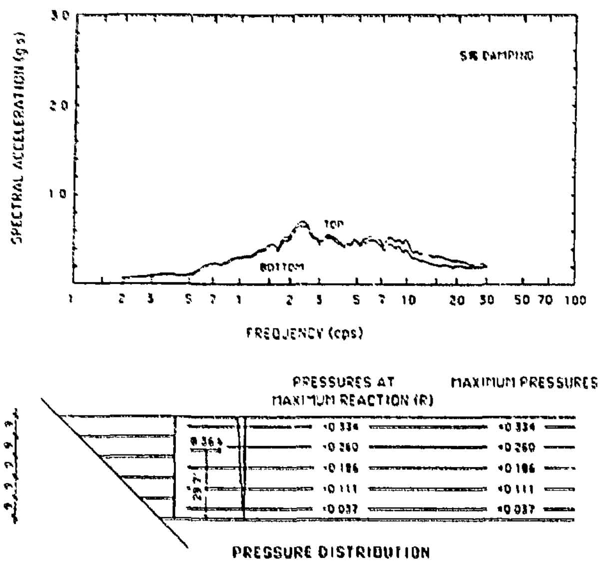


Fig 4 RG 1.60 (0.20g) @ ROCK OUTCROP
Vs (SOIL) = 750 ft/sec
VARIABLE INPUT OVER DEPTH
NO PRESSURE CUTOFF

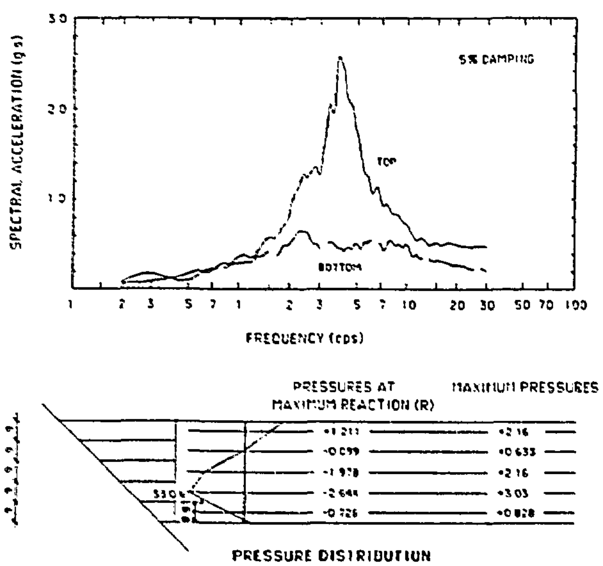


Fig 5 RG 1.60 (0.20g) @ ROCK OUTCROP
Vs (SOIL) = 750 ft/sec
VARIABLE INPUT OVER DEPTH
STANDARD PRESSURE CUTOFF

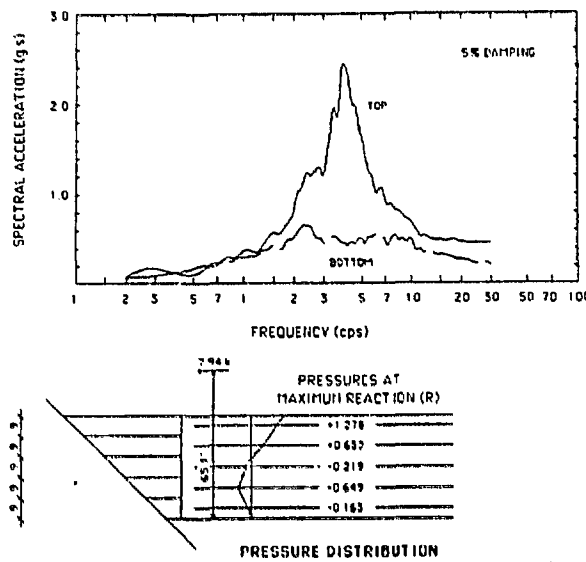


Fig 6 RG 1.60 (0.20g) @ SOIL OUTCROP
Vs (SOIL) = 750 ft/sec
VARIABLE INPUT OVER DEPTH
NO PRESSURE CUTOFF

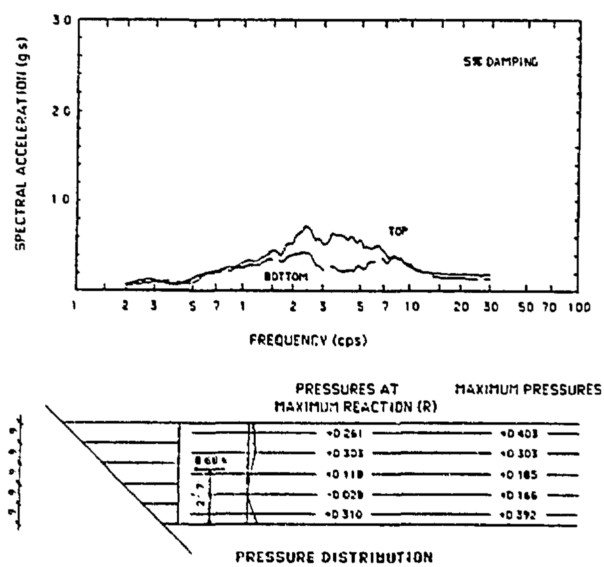


Fig 7 RG 1 60 (0 20g) @ SOIL SURFACE
Vs (SOIL) = 750 ft/sec
VARIABLE INPUT OVER DEPTH
NO PRESSURE CUTOFF

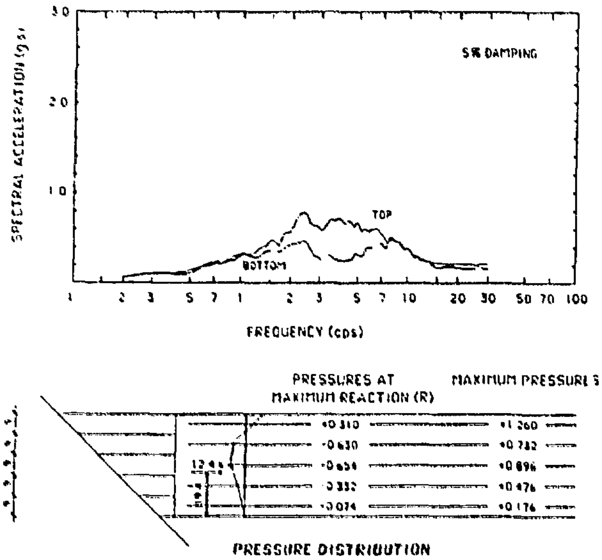


Fig 9 RG 1 60 (0 20g) @ ROCK OUTCROP
Vs (SOIL) = 375 ft/sec
VARIABLE INPUT OVER DEPTH
NO PRESSURE CUTOFF

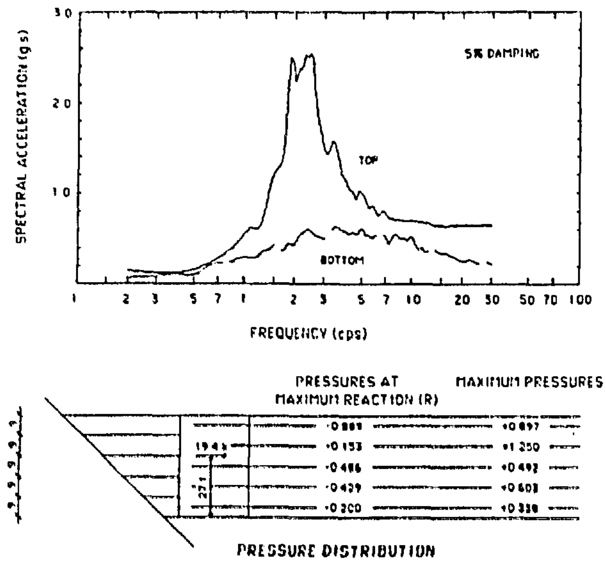


Fig 8 RG 1 60 (0 20g) @ ROCK OUTCROP
Vs (SOIL) = 1500 ft/sec
VARIABLE INPUT OVER DEPTH
NO PRESSURE CUTOFF

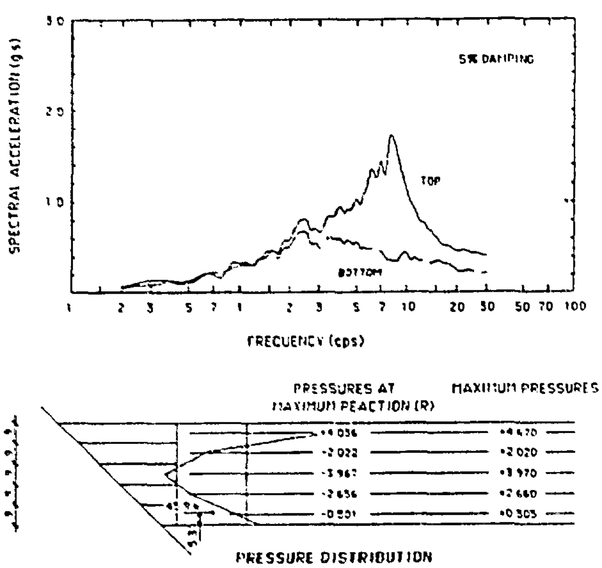


Fig 10 RG 1 60 (0.50g) @ SOIL SURFACE
Vs (SOIL) = 750 ft/sec
VARIABLE INPUT OVER DEPTH
NO PRESSURE CUTOFF
TANK ON BEDROCK

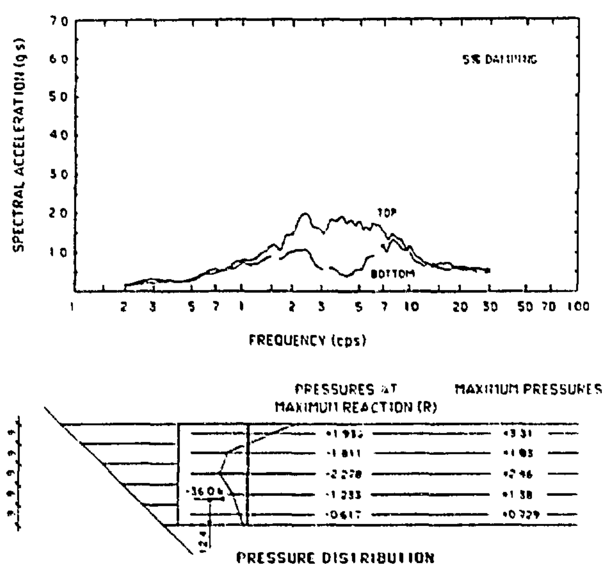


Fig 11 RG 1.60 (0.50g) @ ROCK OUTCROP
Vs (SOIL) = 750 ft/sec
VARIABLE INPUT OVER DEPTH
NO PRESSURE CUTOFF

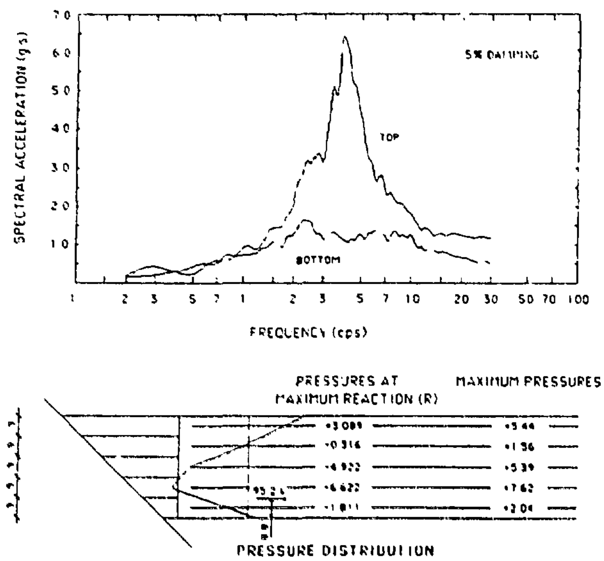


Fig 13 RG 1.60 (0.20g) @ SOIL SURFACE
Vs (SOIL) = 750 ft/sec
VARIABLE INPUT OVER DEPTH
NO PRESSURE CUTOFF
TANK ON BEDROCK

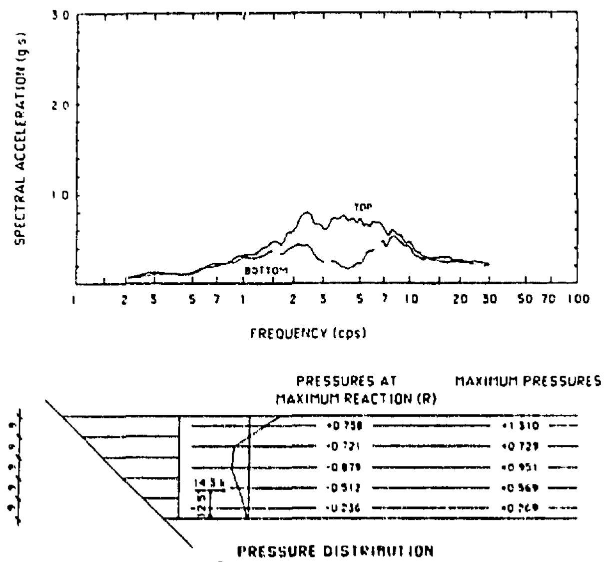


Fig 12 RG 1.60 (0.50g) @ ROCK OUTCROP
Vs (SOIL) = 750 ft/sec
VARIABLE INPUT OVER DEPTH
STANDARD PRESSURE CUTOFF