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STRUCTURAL AND SEISMIC ANALYSES OF WASTE FACILITY REINFORCED CONCRETE STORAGE VAULTS

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ABSTRACT

Facility 317 of Argonne National Laboratory consists of several reinforced concrete waste storage vaults designed and constructed in the late 1940's through the early 1960's. In this paper, structural analyses of these concrete vaults subjected to various natural hazards are described, emphasizing the northwest shallow vault. The natural phenomenon hazards considered include both earthquakes and tornados. Because these vaults are deeply embedded in the soil, the SASSI (System Analysis of Soil-Structure Interaction) code was utilized for the seismic calculations. The ultimate strength method was used to analyze the reinforced concrete structures. In all studies, moment and shear strengths at critical locations of the storage vaults were evaluated.

Results of the structural analyses show that almost all the waste storage vaults meet the code requirements according to ACI 349-85. These vaults also satisfy the performance goal such that confinement of hazardous materials is maintained and functioning of the facility is not interrupted.

1. INTRODUCTION

Facility 317 at the Argonne East Illinois site consists of several reinforced concrete vaults for the storage of radioactive waste material bins. These vaults, with different configurations, were designed and constructed in the late 1940's through the early 1960's. When the facility was designed, there were no special federal guidelines regarding the design of waste storage facilities. Also, 30 or 40 years ago there were no stringent requirements specifying how the facility should be designed to withstand various natural hazards like earthquakes, extreme winds, and tornadoes.

Recently, DOE developed general design criteria (DOE Order 6430.1A) and design and evaluation guidelines (UCRL-15910) for protection against natural phenomenon hazards at DOE sites throughout the United States [1,2]. The goal of these criteria and guidelines is to ensure that DOE facilities can withstand the effects of natural phenomena like earthquakes, extreme winds, tornadoes, and flooding. These guidelines provide procedures to evaluate, modify, or upgrade the existing facilities or to design new facilities that are secure against natural hazards.

On the basis of the usage category given in DOE Standard DOE-STD-1021-93 [3], Facility 317 is considered to be a moderate-hazard-usage facility (performance category = 3, hazard category = 2). The performance goal is to limit the facility damage so that confinement of hazardous materials is maintained and functioning of the facility is not interrupted.

Because the waste storage facility at Facility 317 is considered to be an essential structure, the structural integrity of each reinforced-concrete vault had to be evaluated. For simplification, this paper describes only the detailed structural analysis of the north shallow vault. Analytical results of other reinforced concrete vaults will be presented at the conference.

The vault analyzed here has a shallow depth and is located at the northwest side of Building 317. It was designed and constructed in 1949. This vault presently contains many storage bins of mixed waste. The vault is shielded by a 1-ft-thick reinforced concrete slab and gabled roof covers. The vault originally had a 5-ton Gantry crane for transferring the storage bins. In 1962 the 5-ton crane was relocated to the south-side trench and a 10-ton Gantry crane was subsequently installed for this north-side vault.

2. LOAD COMBINATION, MATERIAL PROPERTIES AND ANALYTICAL APPROACH

The required strength U is expressed in terms of combinations of factored loads, or related moments and forces. Factor loads are the loads specified in the ACI code multiplied by appropriate load factors. According to ACI 349-85 [4] and DOE Design and Evaluation Guidelines UCRL-15910 [2], the load combinations considered here are:

1. $U = 1.4D + 1.7L + 1.7H$

2. $U = D + H + L + E$

3. $U = D + H + W_t$

where U is the required strength; D is the dead load or related moment and force; L is the live load (i.e., crane load) or related moment and force; H is the lateral earth pressure or related moment and force; E is the design basis earthquake load or related moment and force; and W_t is the design basis tornado or related moment and force.

Note that the live loads are generated from either the moving crane during loading and unloading of waste storage bins or the service loads during construction. Since the crane load are much larger than the service load, we use the crane load for the live load calculation. We also assume that during the tornado the crane is not in operation. This assumption can be achieved through the weather forecast and tornado warning. Note, the snow load is not considered since it doesn't produce any overturning moment.

The material properties used in the analyses are:

(a) concrete: The compressive strength of concrete is 3000 psi.

(b) Re-bar: The yield stress is 40 ksi.

3. STRUCTURAL ANALYSIS OF NORTHWEST SHALLOW VAULT

The northwest shallow vault is about 92'-6" long and 15'-0" wide without any interior partitions. Since the vault is quite long, structural analysis was performed for a unit width (1.0

ft) of the transverse section (see Fig. 1) utilizing the plane strain approach. The concrete wall of the waste storage vault is modeled as a retaining wall cantilevered at the wall-floor junction. The frictional resistance force generated from the shield cover is appropriately accounted for in the analysis.

3.1 Load Calculations

3.1.1 Earth Pressure (H). Analogous to the action of a fluid, the unit pressure p at a distance h below the finish grade is [5]

$$p = C_a\, wh, \tag{1}$$

where C_a is the active pressure coefficient and w is the unit weight of the soil. Based on Rankine's theory, the coefficient C_a can be expressed as

$$C_a = \frac{1 - \sin\phi}{1 + \sin\phi}, \tag{2}$$

where ϕ is the angle of friction. Here, we assume ϕ of the backfill to be 30° [5]. Thus, the force P_H caused by active pressure on a wall of height h is

$$P_H = C_a\, w\, \frac{h^2}{2} = \frac{1}{2} \times \frac{1}{3} \times (0.120) \times (7.33)^2 = 1.074 \text{ kips.}$$

At the bottom, the moment caused by the earth pressure is

$$M_H = p \times h/3 = 1.074 \times (7.33)/3 = 2.62 \text{ ft-kips.}$$

At the location 8" above the wall-floor junction where the dowel terminates, the lateral force and moment are:

$$P'_H = \frac{1}{2}\,\frac{1}{3}\,(0.120)(6.66)^2 = 0.887 \text{ kips}$$

$$M'_H = 0.887 \times \frac{6.6}{3} = 1.95 \text{ ft-kips}$$

3.1.2 Live Load (L). Live loads consist of crane load and service load during the construction stage. Crane load is the dominant load which occurs during the time of loading and unloading the waste storage bins, and installation of concrete shield and gabled roof. For the north-side vault analyzed, a 10 ton Gentry crane is used for loading and unloading of the waste material bins. The minimum wheel distance is 12'-6"; the wheel reaction including 15% impact is 13.75 kips; the weight of the ASCE rail is 75 lb/ft.

A wheel load or any load concentrated on a small area may be treated as a point load. The intensity of lateral pressure in this case varies not only with the depth but also with the horizontal distance from the load. The pressure is greatest along the vertical line ab closest to the load as shown in Fig. 2. Along this line ab, the unit horizontal pressure p may be computed by the following empirical equations [6,7]:

$$p_1 = 1.77 \frac{Q}{H^2} \frac{m^2 n^2}{(m^2+n^2)^3} \qquad (m > 0.4) \tag{3}$$

$$p_1 = 0.28 \frac{Q}{H^2} \frac{n^2}{(0.16+n^2)^3} \qquad (m < 0.4) \tag{4}$$

The unit horizontal pressure on any other points on both sides of ab is smaller than p_1 at the same depth, and may be calculated by the following equation

$$p_Q = p_1 \cos^2(1.1\psi) \tag{5}$$

The notations used in the equations above are self-explanatory in Fig. 2.

In general, the sharp pressure spikes generated from the impact during crane operation are smeared out by the soil. Thus eliminating the 15% impact load, the point load generated from the crane wheel is

$$Q = \frac{13.75 - 0.5 \times (10\times2\times0.15)}{2}$$

$$= 6.125 \text{ kips} + 0.075 \text{ (rail weight)} = 6.20 \text{ kips.}$$

Note the horizontal distance between the crane rail and vault wall is 2'-6", the embedment of the vault is 7'-4". Thus, Fig. 2 implies that

$$m = \frac{2.5}{7.33} = 0.34.$$

with m = 0.34 and Q = 6.20 kips, the pressure distribution along the vault wall can be calculated from Eq. (4), or

$$p_1 = 0.28 \frac{Q}{H^2} \frac{n^2}{(0.16+n^2)^3} \qquad (m < 0.4)$$

$$= 0.0323 \frac{n^2}{(0.16+n^2)^3} \tag{6}$$

in which n = z/H, z is the vertical distance at the particular location, and H is the height of the embedment.

Based on Eq. (6), pressure distribution along the concrete wall are calculated at every 1'-0" below the finish grade, and results of p_1 are given in Table 1.

Thus, following some arithmetic calculations the lateral force and moment at the wall-floor junction are:

$$P_L = \Sigma p_i A_i = \Sigma p_i \times 1 \times 1 = \Sigma p_i = 0.667 \text{ kips}$$

$$M_L = 2.85 \text{ ft-kips}$$

Table 1. Pressure Distribution Along Concrete Wall of North Shallow Vault

z (ft)	n	n^2	$(0.16+n^2)^3$	$\frac{n^2}{(0.16+n^2)^3}$	p_1 (ksf)
0	0	0	0	0	0
1.0	0.136	0.0186	0.0056	3.264	0.105
2.0	0.273	0.0744	0.0128	5.776	0.186
3.0	0.409	0.1675	0.0351	4.772	0.154
4.0	0.545	0.298	0.096	3.100	0.100
5.0	0.682	0.465	0.244	1.905	0.061
6.0	0.818	0.670	0.571	1.173	0.037
7.0	0.955	0.912	1.231	0.741	0.024
				Total =	0.667

At the location 8" above the wall-floor junction, the force and moment are:

$$P'_L = 0.667 - 0.024 = 0.643 \text{ kips}$$

$$M'_L = 2.41 \text{ ft-kips}$$

3.1.3 Seismic Load (E)

Annual probability of exceedance: 1×10^{-3}

Seismic Zone Factor: 0.12 g PGA [8]

Since a large portion of the structure is embedded in the soil, the soil-structure-interaction (SSI) effect becomes an important factor during earthquakes. Here, the seismic analysis is performed with a well-known computer program, SASSI (System of Analysis of Soil Structure Interaction) developed at the University of California, Berkeley [9].

The mathematical model of the waste storage vault is shown in Fig. 3. A two-dimensional plan strain model is used to investigate the structural response due to SSI. In the soil model, the first 27 ft of soil is brown and gray silty clay with some fine to coarse gravel. This region is modeled by nine layers of soil with different depths but the same soil properties. In addition to the soil layers with more defined properties, ten viscoelastic layers are used to simulate the halfspace. The thickness of the viscoelastic layers are calculated internally by the SASSI program.

In the structural model, the concrete wall and floor slabs are modeled by 4-nodes plate elements with three degrees of freedom in the 2-D plane, i.e., two translational plus one rotational degrees of freedom. The excavated soil region is represented by 2-D plane strain elements without the rotational degree of freedom. The input data pertaining to the concrete are:

$E = 6.9 \times 10^5$ ksf

$\nu = 0.278$

$w = 0.150$ kcf

The free field input horizontal spectrum are given in Fig. 4, with a zero period acceleration of 0.12 g. The corresponding 10-second acceleration history is shown in Fig. 5, which was obtained from the SIMQUE program of MIT. Based on the 10-second time history input, acceleration response obtained from the SASSI code calculation is given in Table 2.

Table 2. Horizontal Acceleration at Selected Nodes

Nodal No.	Max. Accel. (g)
2, 9, 16, 23	0.11
30, 37	0.12
44	0.13

The moment and force generated from the seismic excitation consists of two parts. The first part is due to structural acceleration. The second part is caused by the dynamic pressure exerted by the moving soil.

(a) Force and Moment Due to Structural Motion

The shear force is calculated using accelerations given in Table 2 and the actual design configuration (see Fig. 3). The shear force is

$$p = \Sigma\, m_i\, a_i = \Sigma\, \frac{W_i}{g}\, \hat{a}_i g$$

where $\hat{a}_i$ is the acceleration coefficient of element i. In case the wall element has different horizontal accelerations at its bottom and top nodes, average acceleration is then utilized.

The shear force and the overturning moment at the wall-floor junction, including the effect of the concrete shielding and gabled roof, are:

$P = 0.277$ kips

$M = \Sigma py = 1.586$ ft-kips

At the location 8" above the wall-floor junction the force and moment are:

$P' \cong 0.266$ kips

$M' = 1.40$ ft-kips

(b) Force and Moment Due to Dynamic Earth Pressure

During an earthquake the lateral pressure against a retaining structure may be temporarily increased due to the vibration of the ground. The increase is a result of inertia force which is difficult to evaluate. For design of retaining walls with moderate height (up to 20 ft), the increase is generally less than 10% of the normal design pressure [6]. In the case of high retaining walls, however, combined pressure may be determined approximately by the trial wedge method [6].

The total soil force during a seismic event consists of a static (active) component, p_A, and a dynamic one Δp_{AE} [10]:

$$p_{AE} = p_A + \Delta p_{AE} \tag{7}$$

One method commonly used is the Mononobe-Okabe solution [10] as shown in Fig. 6. In this approach the total active soil force during the seismic event is

$$p_{AE} = \frac{1}{2} K_{AE}\, \gamma h^2 \tag{9}$$

where

$$K_{AE} = \frac{\frac{g - (a_v)}{g} \cos^2(\phi - \varphi - \beta)}{\cos\varphi \cos^2\beta \cos(\delta + \beta + \varphi)\left[1 + \left\{\frac{\sin(\phi + \delta)\sin(\phi - \varphi - L)}{\cos(\delta + \beta + \varphi)\cos(L - \beta)}\right\}^{1/2}\right]^2},$$

$$\varphi = \tan^{-1} \frac{a_h}{g - a_h}.$$

With $a_h = 0.11$ g, $a_v = 0.667 \times 0.11$ g $= 0.073$ g

$$\varphi = \tan^{-1} \frac{0.11}{1 - 0.11} = \tan^{-1}(0.123) = 7°$$

Using $\phi = 30°$, $\varphi = 7°$, $\delta = 15°$, $\beta = L = 0$, we have

$$\frac{g - a_v}{g} \cos^2(\phi - \varphi - \beta) = 0.927\ (0.9205)^2$$

$$\cos\varphi \cos^2\beta \cos(\delta + \beta + \varphi) = 0.9925 \times 0.9271$$

$$1 + \left\{\frac{\sin(\phi + \delta)\sin(\phi - \varphi - L)}{\cos(\delta + \beta + \varphi)\cos(L - \beta)}\right\}^{1/2} = 1.5453$$

Thus, the pressure coefficient K_{AE} is

$$K_{AE} = \frac{0.927(0.9205)^2}{0.9925 \times 0.9271(1.5453)^2} = 0.357$$

Based on the Mononabe-Okabe method, the total active earth force is

$$P_{AE} = \frac{1}{2}(0.357)\gamma h^2 = \frac{1}{2}(0.357)(0.120)(7.33)^2$$

$$= 1.1508 \text{ kips}$$

The static earth force is

$$P_A = \frac{1}{2} C_a \gamma h^2 = \frac{1}{2}\frac{1}{3}(0.120)(7.33)^2 = 1.073 \text{ kips}$$

The dynamic soil force Δp_{AE} has a value of

$$\Delta p_{AE} = 1.1508 - 1.073 = 0.084 \text{ kips}$$

The ratio of dynamic to static earth force is

$$\gamma = \Delta p_{AE}/p_A = 0.084/1.073 \cong 0.08 \text{ or } 8\%$$

Thus, the dynamic force is about 8% of the static earth force or

$$\Delta p_{AE} = p_D = 0.08\ p_A.$$

As can be seen from the calculations, the dynamic earth force during seismic excitations is less than 10%. This is quite close to the estimate given in Ref. [6] for walls with moderate height. Here, in the analysis of the north shallow wall, we assume that the dynamic earth force is about 11% of the static force which corresponds to the average horizontal acceleration below the finishing grade. A slightly conservative value of 0.11 is used to cover the uncertainties of angle of wall friction and backfill soil properties.

The shear force and moment caused by the dynamic earth pressure are:

$$P = C_a\, w\, \frac{h^2}{2}\,(0.11)$$

$$= 1.074 \times 0.11 = 0.118 \text{ kips}$$

$$M = 0.118 \times \frac{2h}{3}$$

$$= 4.886 \times 0.118 = 0.576 \text{ ft-kips}$$

At the location where the 1/2" ϕ dowel reinforcements terminate, the force and moment are:

$$P' = 0.887 \times 0.11 = 0.097 \text{ kips}$$

$$M' = 0.097 \times \frac{2 \times 6.67}{3} = 0.430 \text{ ft-kips}$$

At the wall-floor junction, the total shear force and moment due to the structural motion and dynamic earth pressure are:

$$P_E = 0.277 + 0.118 = 0.395 \text{ kips}$$

$$M_E = 1.586 + 0.576 = 2.162 \text{ ft-kips}$$

At the location of dowel termination, the force and moment are:

$$P'_E = 0.266 + 0.097 = 0.363 \text{ kips}$$

$$M'_E = 1.40 + 0.43 = 1.83 \text{ ft-kips}$$

3.1.4 Tornado Wind (W_t). The velocity pressure q_z at height z is calculated from the formula [11]:

$$q_z = 0.00256\, K_z(IV)^2, \tag{10}$$

where K_z is the velocity pressure exposure coefficient, V is the basic wind speed, and I is the importance factor. Thus the velocity pressure q_z is

$$q_z = 0.00256\,(0.8)(1.00 \times 142)^2 = 41.29$$

The design pressure is

$$p = q_z\, G_n\, C_p \tag{11}$$

$$= 41.29\,(1.32)(0.8) = 43.61 \text{ psf.}$$

Thus, the force and moment at the wall-floor junction are:

$$P_{wt} = p \cdot A = 43.61 \times 5.0 \times 1 = 0.218 \text{ kips}$$

$$M_{wt} = P_{wt} \cdot h = 0.218 \times (7.33+2.5) = 2.143 \text{ ft-kips}$$

At the location 8" above the wall-floor junction the lateral force and moment are:

$$P'_{wt} = 0.218 \text{ kips}$$

$$M'_{wt} = 0.218 \times (9.83-0.67) = 2.00 \text{ ft-kips}$$

3.2 Design Strength

3.2.1 Moment and Force

a) Wall-Floor Junction

In the structural design, 1/2" ϕ (No. 4) @ 12" o.c. reinforcement and 1/2" ϕ dowel are used at each face and each way. The tension force T, based on f_y = 40 ksi and the ultimate strength method [12], is

$$T = A_s f_y = 2 \times 0.20 \text{ in.} \times 40 \text{ ksi} = 16 \text{ kips.}$$

Assume a is the depth of equivalent rectangular stress block with concrete stress of 0.85 f'_c, the compression stress is

$c = 0.85\ f'_c\ (a)\ (12)$

Utilizing the force equilibrium condition and $f'_c = 3{,}000$ psi, the following relationship holds

$C = T,$

and

$$a = \frac{16}{0.85\ f'_c\ (12)} = 0.52 \text{ in.}$$

The moment arm γ for the resisting moment is

$\gamma = 10.5 - 0.25 - 0.5 \times 0.52 = 10.00''$

Thus, the maximum nominal moment corresponding to the design reinforcement is

$M_N = 16 \times 10.00/12 = 13.32$ ft-kips.

Neglecting the effect of axial force, the nominal shear strength provided by the concrete wall is

$$V_N = 2\sqrt{f'_c}\ b\,d$$

$$= 2\sqrt{3{,}000}\ (10.25)(12) = 13.47 \text{ kips.}$$

Thus the design strengths of moment and shear are:

$\phi\ M_N = 0.90 \times 13.32 = 11.98$ ft-kips

$\phi\ V_N = 0.85 \times 13.47 = 11.45$ kips.

b) <u>Dowel Cut-off Location</u>

At the dowel cut-off location, the tension (T) and compression force (C) are:

$T = A_s\ f_y = 0.20 \times 40 = 8$ ksi,

$C = 0.85\ f'_c\ (a)(12).$

Based on the force equilibrium condition C = T, the depth of the equivalent rectangular stress block, a, is

$a = 8/(0.85{\times}3{\times}12) = 0.26$ in.

The maximum design moment and later force are:

$$M'_N = 8 \times (10.5\text{-}0.25\text{-}0.13)/12 = 6.75 \text{ ft-kips}$$

$$V'_N = 13.47 \text{ kips}$$

The design strength of moment and force are:

$$\phi\ M'_N = 0.90 \times 6.75 = 6.10 \text{ ft-kips}$$

$$\phi\ V'_N = 11.45 \text{ kips}$$

3.2.2 Resistant Load of Concrete Shield. Since the concrete shield is supported on the ledges of the concrete walls, the friction force will reduce the effects of other loads, such as the earth pressure, crane-induced lateral pressure, and seismic response of the structure.

The static frictional coefficient is about 0.4-0.8, the dynamic frictional coefficient is about 25% less and practically has a minimum value of 0.3.

For a load combination involving the static loads, the static friction force is

$$F = \mu N = 0.4 \times (0.150)(0.5\times 11.46)$$

$$= 0.344 \text{ kips}$$

Thus, at the wall-floor junction the static resisting moment and force, after multiplying by a factor of 0.9 [4], are:

$$\phi M_R = 0.9\ (0.344\times 6.33) = 1.96 \text{ ft-kips}$$

$$\phi V_R = 0.9 \times 0.344 = 0.31 \text{ kips}$$

At the dowel cut-off location, the resisting moment and force are:

$$\phi M'_R = 0.9 \times (0.344\times 5.66) = 1.75 \text{ ft-kips}$$

$$\phi V'_R = 0.31 \text{ kips}$$

For a load combination involving the seismic load, the dynamic frictional force should be used which has the value:

$$F = \mu N = 0.3 \times (0.150)(0.5\times 11.46)$$

$$= 0.2578 \text{ kips}$$

At the wall-floor junction the resisting moment and force, after multiplying by a factor of 0.9, are:

$$\phi\, M_R = 0.9\ (0.257 \times 6.33) = 1.468 \text{ ft-kips}$$

$$\phi\, V_R = 0.9 \times 0.2578 = 0.23 \text{ kips.}$$

At the dowel cut-off location, the resisting moment and force are:

$$\phi\, M'_R = 0.9 \times (0.2578 \times 5.66) = 1.31 \text{ ft-kips}$$

$$\phi\, V'_R = 0.23 \text{ kips.}$$

3.3 Load Combination and Comparison of Results. To facilitate the comparison of the required strength and design strength, Table 4 lists the comparison of required moments and shear forces at the wall-floor junction. In this table M_u and V_u denote the required moment and shear force, $\phi_m M_N$ and $\phi_v V_N$ denote the design strength of moment and shear force with ϕ_m and ϕ_v equal to 0.90 and 0.85, respectively.

Table 4. Comparison of Moment and Shear Force at the Wall-Floor Junction

Load Combination U	Required Strength		Design Strength	
	M_u ft-kips	V_u kips	$\phi_m M_N$ ft-kips	$\phi_v V_N$ kips
1. 1.4D + 1.7H + 1.7L	9.69	2.96	11.98	11.45
2. D + H + L + E	7.91	2.13	11.98	11.45
3. D + H + W_t	5.04	1.29	11.98	11.45

From Table 4 we can see that the first load combination (1.4D+1.7H+1.7L) involving dead load (D), earth pressure (H), crane-induced live load (L) is the controlling case. Required strengths for load combination cases 2 and 3 involving design basis earthquake and tornado are small.

From this table we can see that the design strengths of both moment and shear force are larger than the corresponding value of the required strength, hence the design is adequate at the wall-floor junction. It should be mentioned here that the design strength of shear force is about 11.45 kips, way larger than the maximum required strength of 2.96 kips.

To compare the result at 8" above the wall-floor junction Table 5 lists all the values of the required and nominal strengths, as well as the design strength for bending moments plus the resisting moment strength due to the frictional force generated from the concrete shield. The total strength for resisting the overturning moment obtained from the load combination is given in the last column of this table. Note that the static resisting strength is used in the load

Table 5. Comparison of Moment Strength at 8" Above the Wall-Floor Junction

Load Combination U	Required Strength M_u ft-kip	Nominal Strength M_N ft-kips	Design Strength ϕM_N ft-kips	Resisting Strength $0.9 M_R$ ft-kips	Total Strength* M_T ft-kips
1. 1.4D + 1.7H + 1.7L	7.80	6.75	6.10	1.75	7.85
2. D + H + L + E	6.47	6.75	6.10	1.31	7.41
3. D + H + W_t	4.23	6.75	6.10	1.75	7.85

*Total strength $M_T = \phi M_N + 0.9 M_R$.

combination cases 1 and 3, whereas the dynamic resisting strength is utilized for the load combination case 2.

Again, Table 5 shows that the first load combination (1.4D+1.7H+1.7L) is the controlling case and has the largest values of moment and force. It also indicates that at 8" above the wall-floor junction where the dowel terminates, the requirement moments obtained from the first load combination exceeds both nominal and the design moment strengths (1/2" ϕ @ 12" o.c.) of 6.75 and 6.1 ft-kips, respectively. Including the resisting moment due to frictional force of the concrete shield, the total moment strength would be 7.85 ft-kips, which is greater than the required strengths.

One reason for underestimating the tension reinforcement is believed to be due to the change of crane capacity in 1962. As mentioned in the introductory section, in the original 1949 design a 5-ton crane was used. However, this 5-ton crane was relocated to the south side runway in 1962, and a 10-ton crane was subsequently employed for the north side vaults. Thus, if a 5-ton crane was utilized, the crane-induced live load would be reduced considerably and the required moment strength of the first load combination is about 5.90 ft-kips which is less than the design strength of 6.10 ft-kips.

The design at its present stage is adequate. In the future care should be taken during removal of the waste storage bins. Frictional force should be maintained during the crane operation. One way to accomplish this is to remove the concrete shield and waste storage bins from one section first, put the concrete shield back, and then proceed to the next section.

4. RESULTS OF OTHER REINFORCED CONCRETE VAULTS

In addition to the northwest shallow vault, analyses of other reinforced-concrete vaults were also performed. Results indicate that the structural designs of the two south shallow vaults and the sodium disposal vault all meet the performance goal and code requirements according to ACI Code 349-85 [4].

The cast iron pipe storage vault located at the northeast of Facility 317 is being dismantled and hence no analysis of it was performed. The deep vault located at the north side of the waste facility has a depth of 21 ft. This vault consists of six cells separated by permanent reinforced-concrete partitions for storage of bins of solidified waste. Results of the analysis reveal that the structural capacity of the interior cells is adequate, but the strength of the two exterior cells is slightly insufficient.

Because Facility 317 is quite old and portions of the deep vault are outdated by today's standards, new storage buildings are being designed and constructed. At the completion of these new projects, all wastes currently stored in Facility 317 will be removed and stored in the new buildings. The facility will be decontaminated and restored for general use. Therefore, Facility 317 has a limited remaining lifetime.

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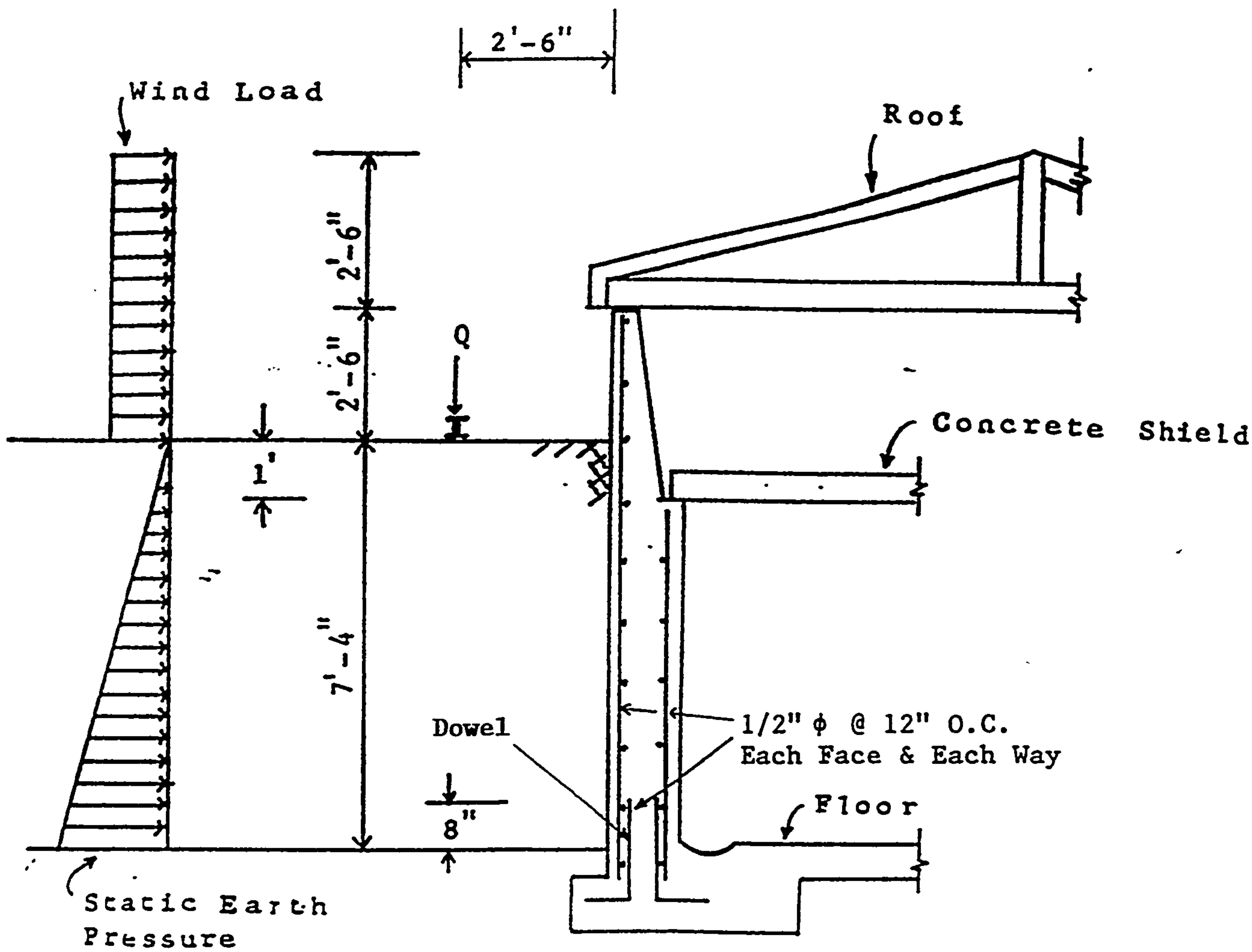
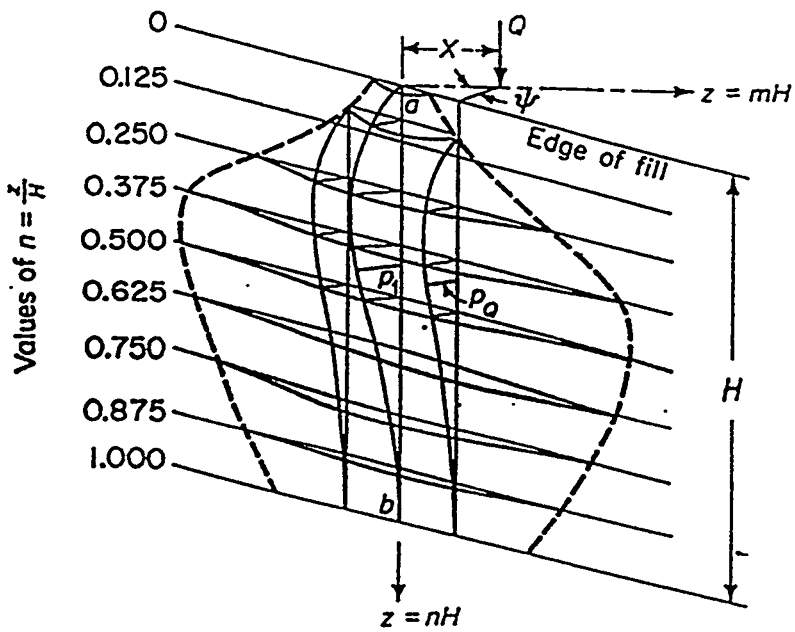
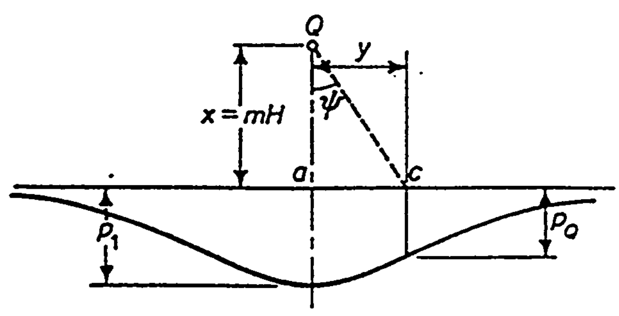


Fig. 1. Analytical Model of the Northwest Shallow Vault





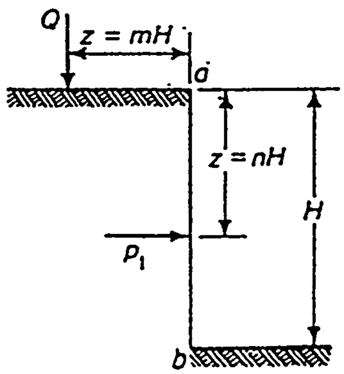


Fig. 2. Lateral Pressure Due to Point Load

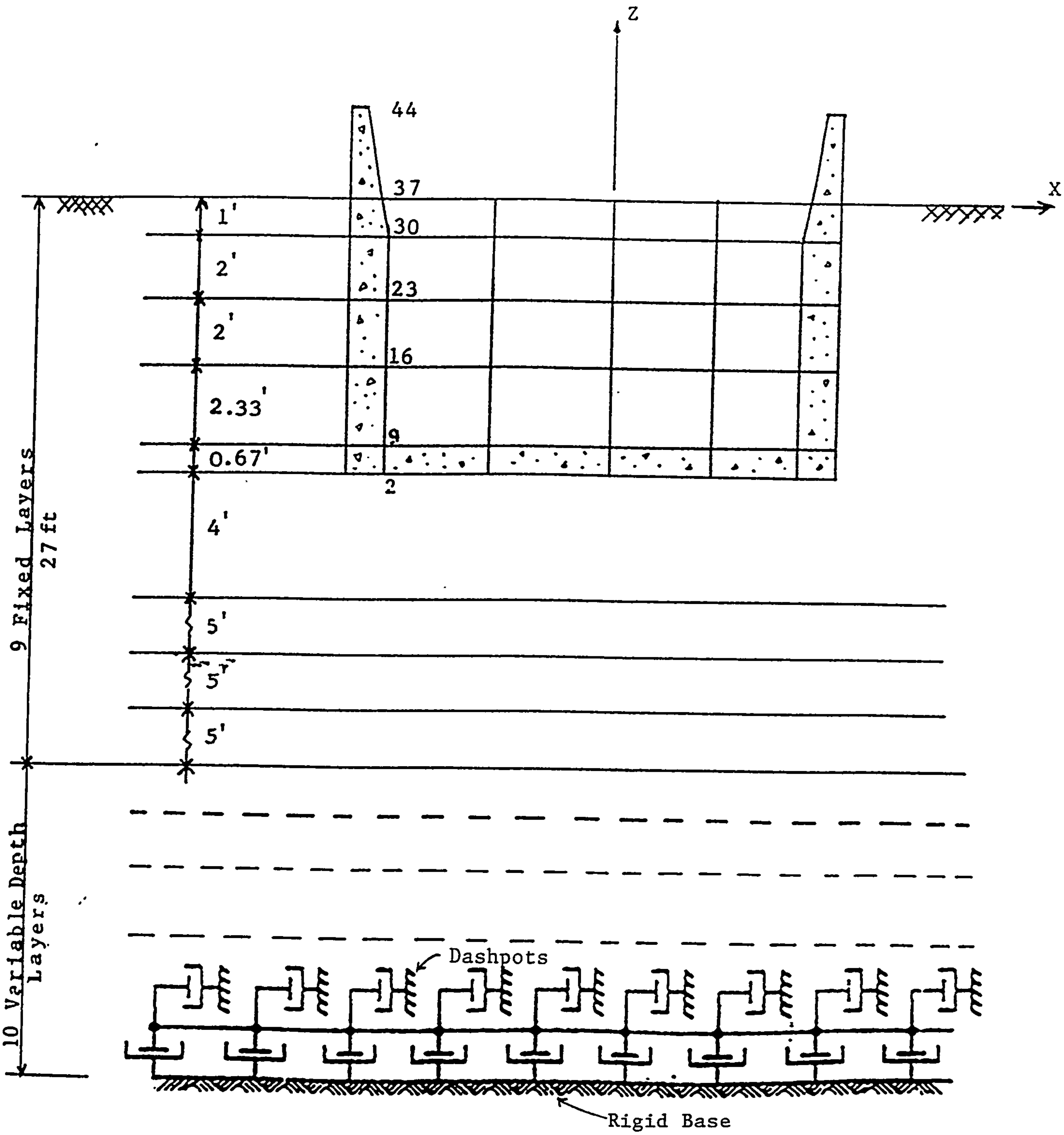


Fig. 3. Mathematical Model of the Northwest Shallow Vault Used in the SASSI Code Seismic Soil-Structure-Interaction Analysis

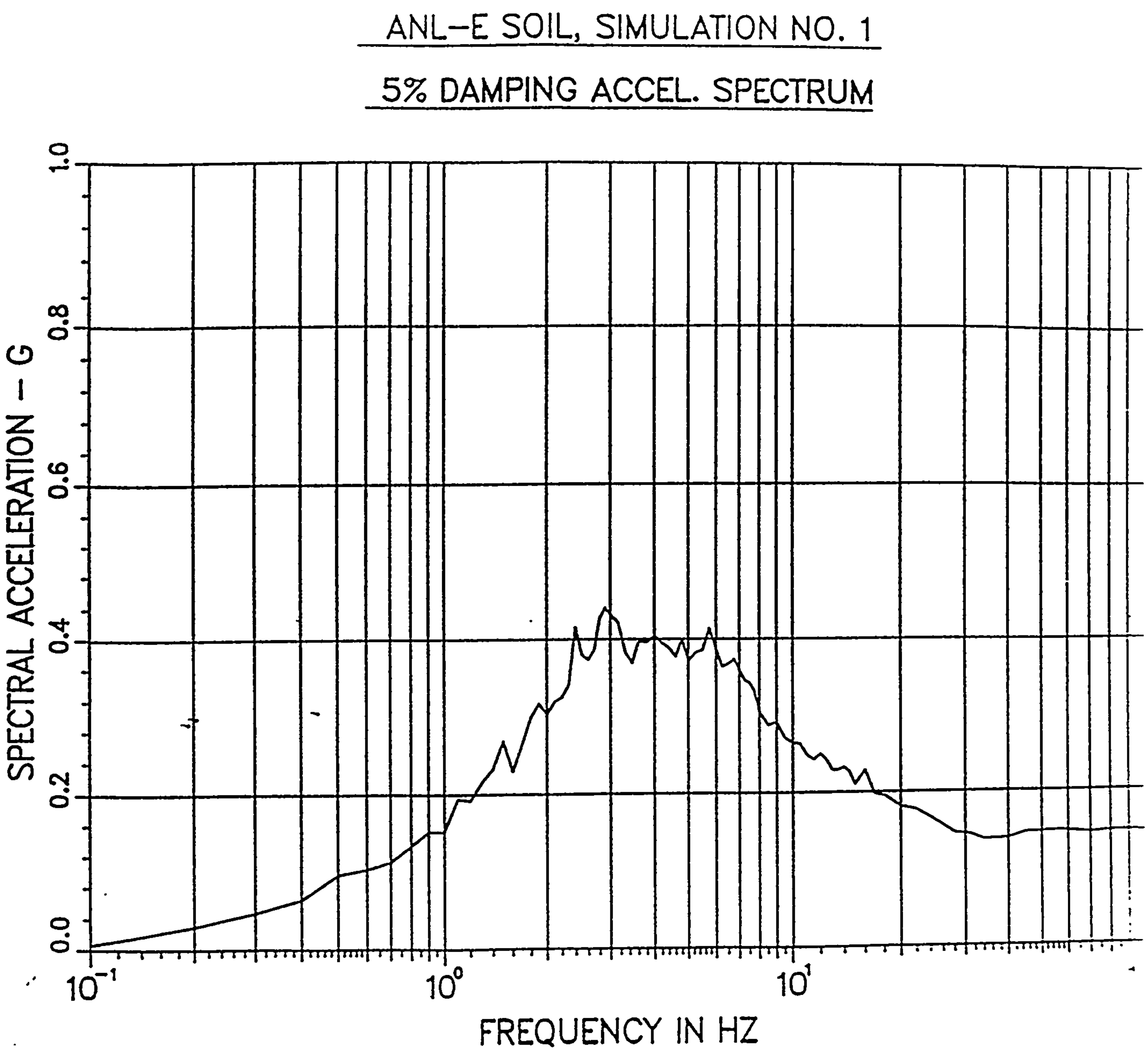


Fig. 4. Free Field Response Spectrum Used for the Seismic Analysis

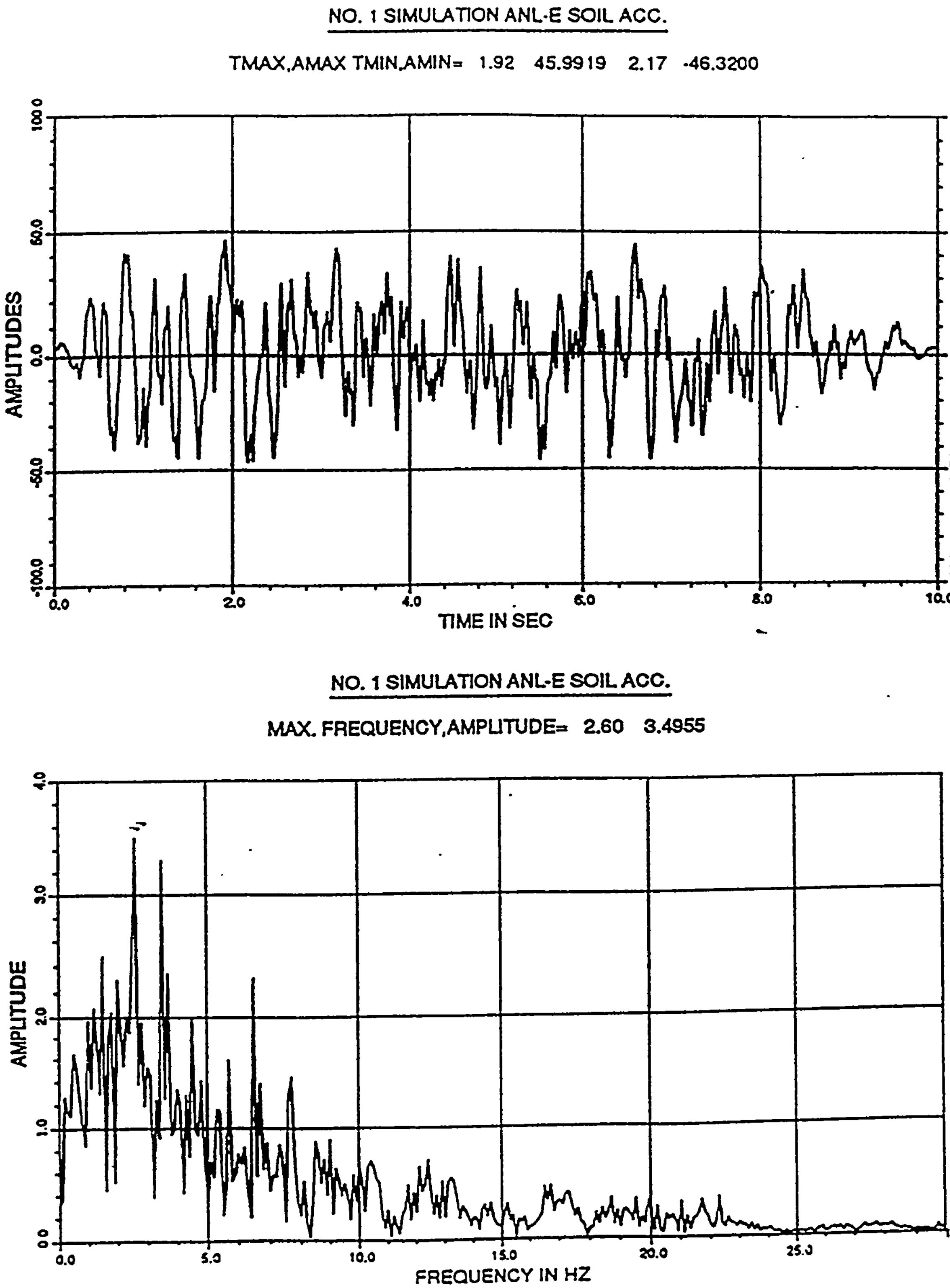
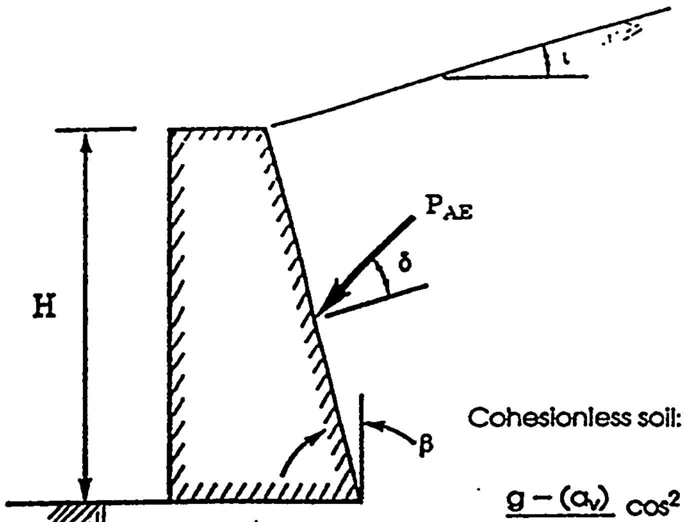
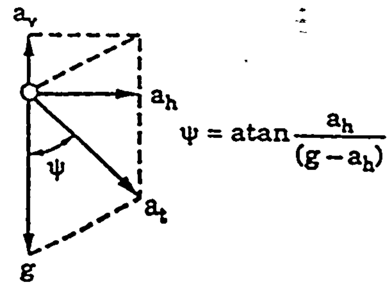


Fig. 5. Input Acceleration Time History Used for the SASSI Code Analysis

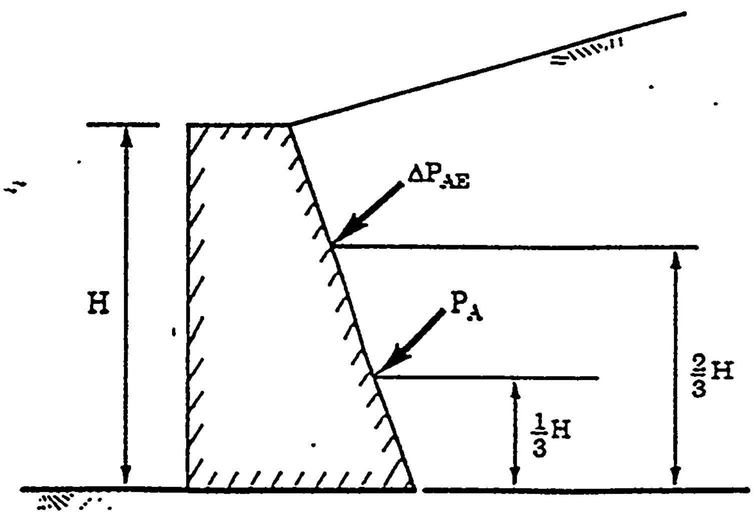




Cohesionless soil: γ_s = Unit weight.
ϕ = Angle of internal friction.

$$K_{AE} = \frac{\dfrac{g-(a_v)}{g}\cos^2(\phi-\psi-\beta)}{\cos\psi\cos^2\beta\cos(\delta+\beta+\psi)\left[1+\left\{\dfrac{\sin(\phi+\delta)\sin(\phi-\psi-\iota)}{\cos(\delta+\beta+\psi)\cos(\iota-\beta)}\right\}^{1/2}\right]^2}$$

$$P_{AE} = \tfrac{1}{2}\gamma_s H^2 K_{AE}$$



P_A = Static active force
P_{AE} = Seismic active force
ΔP_{AE} = $P_{AE} - P_A$

Fig. 6. Mononobe-Okabe Solution of Earth Pressure During Earthquakes