Soil-structure interaction in earthquake-induced liquefaction

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ABSTRACT: A method is proposed here to estimate damage to a structure due to earthquake-induced liquefaction considering the effect of soil-structure interaction. The damage is estimated in terms of differential settlement. The structural rigidity can significantly reduce the extent of damage to a structure. The redistribution of vertical loads due to uneven settlement of the foundations due to earthquake shaking can be modeled effectively by considering the interaction between the structure and the soil. A finite element-based computer program is written specifically for this purpose. The methodology is described with the help of an example.

1 INTRODUCTION

After over two decades of research work, there is general agreement about the mechanism by which the onset of liquefaction occurs during and following an earthquake. This research includes field observations during and following earthquakes, experiments in the laboratory on saturated soil samples and models of foundations and earth structures, and theoretical studies. In some recent work, Haldar (1983), Haldar and Luettich (1985) and Haldar and Chern (1986) emphasized the consideration of damage during liquefaction and suggested it was necessary to go one step beyond the evaluation of liquefaction potential.

In a saturated sand deposit under constant volume conditions, the primary effect of the earthquake shaking is the generation of excess pore water pressure. The generation and dissipation characteristics of pore water pressure dictate the type and extent of damage the site will experience. The increase in pore water pressure will decrease the effective stress of the soil elements. The decrease in the effective stress will cause permanent settlement, also referred to as cumulative or residual strain, in the anisotropically consolidated soil elements (soil elements beneath a sloping surface or beneath an engineering facility) as the pore water pressure continues to be generated due to the earthquake shaking.

Any excess residual pore water pressure generated due to the earthquake shaking will eventually dissipate along some drainage route following the earthquake. The rate of dissipation will depend on the drainage characteristics of the soil, and may range from almost instantaneous to several minutes or hours. The final results of the shaking and application of earthquake shaking is reconsolidated settlement of the sand.

There are numerous possible damage scenarios in earthquake-induced liquefaction. The most common types of damage that can be expected following an earthquake are settlement, differential settlement and rotation or tilting of a structure at the site. Thus, it is quite logical to use these measurements as the damage criteria for earthquake-induced liquefaction. In this paper, differential settlement is used as the damage criterion.

In the past, most of the liquefaction-related research was for level ground or isotropic deposit conditions. However, when damage to a structure resting on the site is the primary concern, anisotropic deposit conditions must be considered. Due to the presence of the structure, the soil elements along potential failure plane surfaces are subjected to an appreciable amount of static shear stress. The behavior of such soil elements subjected to an earthquake is considerably different than that of the isotropic case.

Since the anisotropic soil elements strain progressively during cyclic loading, the earthquake-induced deformation of a soil element under a footing will include both the undrained residual deformation and the deformation due to the dissipation of the earthquake-induced excess pore pressure. The subject was discussed in great detail elsewhere by Haldar and Chern (1986). It will be discussed very briefly here for the sake of completeness. However, to estimate deformation in term of settlement or differential settlement during consolidation, the soil-structure interaction must be considered. The interaction between the structure and the soil through the redistribution of vertical loads due to uneven settlement of the foundations needs to be considered during the consolidation stage. This is the primary subject of this paper.

2 SETTLEMENT OF ANISOTROPIC SAND DEPOSIT

As mentioned earlier, the total settlement, S_T, of a deposit can be estimated by considering the residual settlement during an earthquake, S_d, and the consolidation settlement, S_c. Both S_d and S_c depend on the amount of excess pore water pressure developed during the earthquake shaking under anisotropic stress conditions. The pore pressure generated during N cycles of earthquake loading can be shown to be

$$u = u_f . \{\tfrac{1}{2} + \frac{1}{\Pi} \sin^{-1} [(\frac{N}{N_{50}})^{1/\alpha} - 1]\} \quad (1)$$

in which N_{50} = number of cycles to develop pore pressure equal to 50% of the limiting value of residual pore pressure that possibly can occur in a sample, u_f; and α is a parameter whose value depends on the consolidation stress ratio, K_c. This cannot be discuss here further due to lack of space.

2.1 Residual settlement during an earthquake

The change in vertical strain, $\Delta\varepsilon_1$, caused by the pore pressure increment, Δu, for anisotropic samples can be shown to be (Chang (1982)):

$$\frac{\Delta\varepsilon_1}{\Delta u} = \frac{R_f(\frac{\sigma'_d}{\sigma'_{ult} - \sigma'_d})^2 (\frac{2 \sin \phi'}{1 - \sin \phi'})}{K P_a (\sigma'_3/P_a)^n} + \frac{n(\frac{\sigma'_d}{\sigma'_{ult} - \sigma'_d}) (\frac{\sigma'_{ult}}{\sigma'_3})}{K P_a (\sigma'_3/P_a)^n} \quad (2)$$

in which $\sigma'_3 = \sigma'_{3c} - u$; P_a = atmospheric pressure; σ'_d = deviatic stress = $(K_c - 1)$ σ'_{3c}, which is assumed to be constant; ϕ' = friction angle; K, n, R_f = soil parameters that can be estimated from a set of static consolidated drained triaxial tests; and

$$\sigma'_{ult} = \frac{1}{R_f} [\sigma'_3 \frac{2 \sin \phi'}{1 - \sin\phi'}] \quad (3)$$

The pore water pressure build-up in anisotropically consolidated sand deposits can be estimated by equation (1). Consequently, the undrained residual strain between two consecutive loading cycles can also be obtained by using equation (2). After the vertical strain is accumulated to the Nth cycle using equation (2), the residual settlement, S_d, can be determined provided the thickness of the soil layer, h, is known, i.e.,

$$S_d = h . \varepsilon_1 \quad (4)$$

2.2 Consolidation settlement

Assuming that the sand layer is compressible, no lateral deformation is possible during the dissipation of excess pore water pressure, there is enough time for the pore pressure to dissipate, and during the pore pressure dissipation the volume compressibility m_v remains constant and equal to the maximum value reached during the pore-water pressure build-up, then the consolidation settlement of the layer can be obtained as

$$S_c = m_v h u \quad (5)$$

All the parameters were described earlier.

3 SOIL-STRUCTURE INTERACTION IN LIQUEFACTION

In the previous section, the loading system was idealized as a set of independent loads applied at the ground level, and structural continuity was ignored. If a particular foundation of a column of the structure is very heavily loaded, then the settlement underneath it is expected to be large. This will cause a redistribution of forces and part of the load will be transferred to less stressed support points, thus changing the settlement profile. In some cases, the rigidity of a structure will influence its settlement characteristics.

A methodology is proposed here for structures supported on shallow foundations.

The method considers only the consolidation settlement after the earthquake has ceased. Elastic structural behavior is implied in the analysis.

3.1 Coefficient of load transference

In a soil-structure interaction model, the redistribution of vertical loads is often described in terms of load transfer coefficients (Chamecki (1956)). These are structure-dependent parameters, and are elastic constants of the entire structure which can be easily calculated using structural theories for statically indeterminate structures. They can be represented as a square matrix T of order n for a structure supported on n points. A typical element of this matrix, T_{ij}, represents the value of the vertical reaction generated at the ith support point when the jth support settles a unit amount, while the other supports are prevented from displacing vertically. The concept is depicted in Figure 1 which shows a two-bay one-story frame supported on spread footings. The T matrix for this frame is a 3 x 3 matrix whose elements are the nine reactions illustrated in Figures 1a, b, c, and d. It is important to note that when a support moves, both tension and compression forces may be generated. Each row of the T matrix is equivalent to the vertical reaction influence line for settlement. Furthermore, the matrix is symmetrical (since $T_{ij} = T_{ji}$ from Maxwell's Theorem) and, from vertical equilibrium, the elements of each column add up to zero.

The original column reactions (obtained based on the assumption of unyielding supports) define the load vector {Q}. If {S} is a vector of size n calculated settlements, then the product [T] . {S} will give the values of the redistributed loads. The elements of the vector {Q} must be corrected by these values to obtain the true loading condition. The load corrections become increasingly important as the structure becomes stiffer. In an ideal flexible case, the T matrix is equal to the null matrix, i.e., [T] = [0], and no correction is necessary.

For the frame shown in Figure 1a and assuming it is underlain by a single compressible sublayer, the soil-structure equations shown in matrix form are:

$$\begin{bmatrix} f_{11} & 0 & 0 \\ 0 & f_{22} & 0 \\ 0 & 0 & f_{33} \end{bmatrix} \begin{bmatrix} \alpha_{11} & \alpha_{12} & \alpha_{13} \\ \alpha_{21} & \alpha_{22} & \alpha_{23} \\ \alpha_{31} & \alpha_{32} & \alpha_{33} \end{bmatrix} \begin{bmatrix} Q_1 \\ Q_1 \\ Q_1 \end{bmatrix}$$

$$+ \begin{bmatrix} T_{11} & T_{12} & T_{13} \\ T_{11} & T_{12} & T_{13} \\ T_{11} & T_{12} & T_{13} \end{bmatrix} \begin{bmatrix} s_1 \\ s_2 \\ s_3 \end{bmatrix} = \begin{bmatrix} s_1 \\ s_2 \\ s_3 \end{bmatrix} \quad (6)$$

in which f_{ij} = soil flexibility at mid-depth in the layer under support j; α_{ij} = stress coefficient under support i due to column load j; and s_j = settlement under support j.

For the settlement beneath column 1, i.e., j = 1, equation (6) can be expressed as:

$$\begin{aligned} f_{11}[(Q_1 &+ T_{11}s_1 + T_{12}s_2 + T_{13}s_3)\ \alpha_{11} \\ &+ (Q_2 + T_{21}s_1 + T_{22}s_2 + T_{23}s_3)\ \alpha_{22} \\ &+ (Q_3 + T_{31}s_1 + T_{32}s_2 + T_{33}s_3)\ \alpha_{33}] \\ &= s_1 \end{aligned} \quad (7)$$

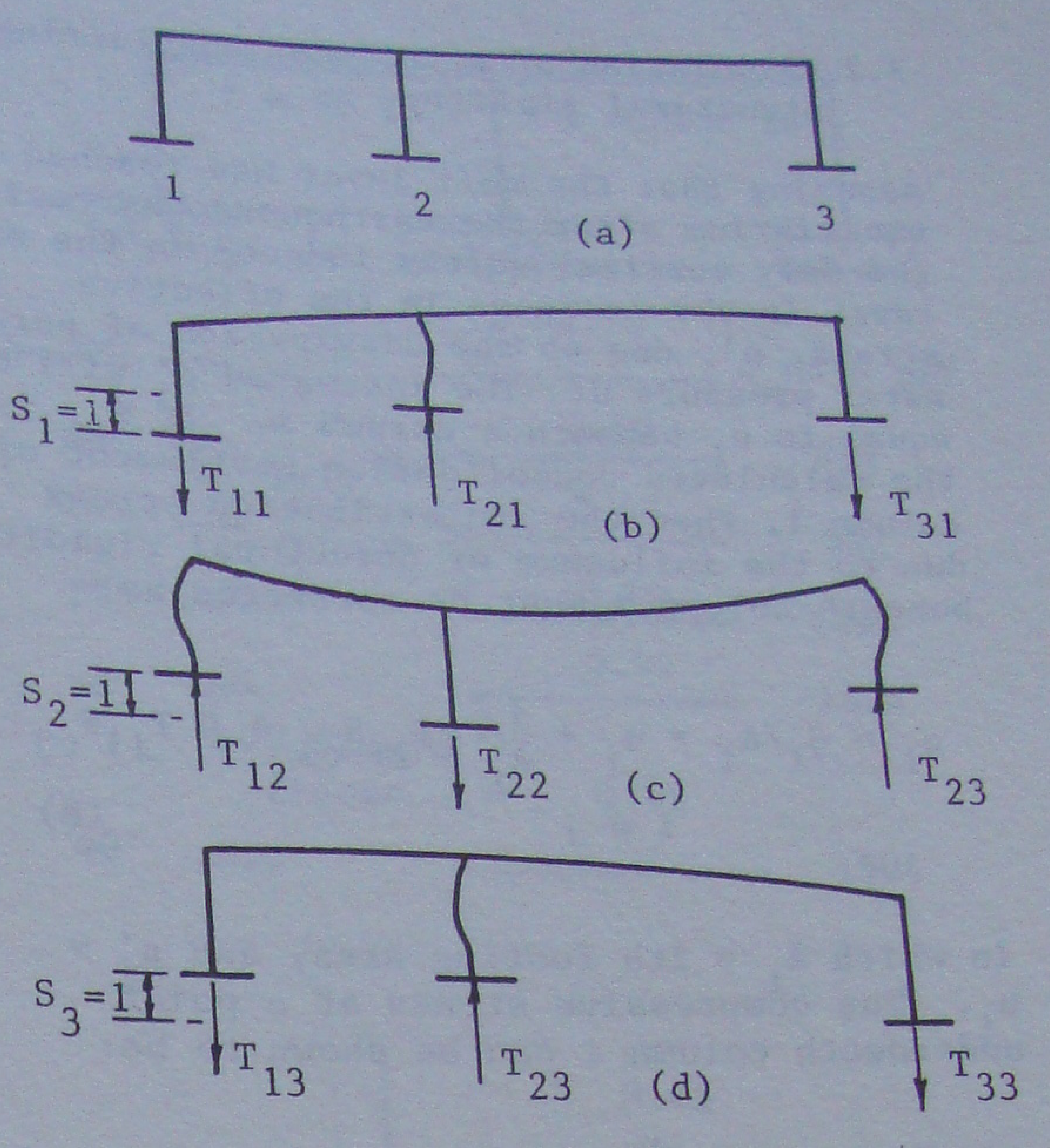

Figure 1. Load transfer coefficients for a two-bay one-story frame

3.2 Calculation of settlement considering structural rigidity

Assuming that the soil layer has reached equilibrium after the earthquake has ceased, the only vertical stress induced in the soil layer is the increase in the effective stress, p', due to the dissipation of pore-water pressure u. The increased p_i' will be equal to u_i beneath a column i. If S_{ci} is the calculated consolidation settlement of column i, then the redistribution stress due to the influence of structural rigidity beneath column i must be corrected as:

$$q_i = Q_i/A_i = u_i + \frac{1}{A_i}[T_{ii}S_{ci} + \Sigma\, T_{ij}S_{cj}], \quad i \neq j \qquad (8)$$

in which A_i = ith footing area, and $p_i' = u_i$. The compressive stress at a point underneath column i can be shown to be:

$$\sigma_i = \alpha_{ii}q_i + \sum_{j=1}^{n} \alpha_{ij}q_j, \quad i \neq j \qquad (9)$$

The consolidated settlement beneath column i will be:

$$S_{ci} = m_{vi}\, h_i\, [\alpha_{ii}q_i + \sum_{j=1}^{n} \alpha_{ij}q_j] \qquad (10)$$

For a single compressible layer and a three-column structure, equation (10) can be expressed as:

$$\begin{bmatrix} m_{v1}h_1 & 0 & 0 \\ 0 & m_{v2}h_2 & 0 \\ 0 & 0 & m_{v3}h_3 \end{bmatrix} \begin{bmatrix} \alpha_{11} & \alpha_{12} & \alpha_{13} \\ \alpha_{21} & \alpha_{22} & \alpha_{23} \\ \alpha_{31} & \alpha_{32} & \alpha_{33} \end{bmatrix} \begin{bmatrix} p_1' \\ p_2' \\ p_3' \end{bmatrix}$$

$$+ \begin{bmatrix} T_{11}/A_1 & T_{12}/A_2 & T_{13}/A_3 \\ T_{21}/A_1 & T_{22}/A_2 & T_{13}/A_3 \\ T_{31}/A_1 & T_{32}/A_2 & T_{33}/A_3 \end{bmatrix} \begin{bmatrix} s_1 \\ s_2 \\ s_3 \end{bmatrix}$$

$$= \begin{bmatrix} s_1 \\ s_2 \\ s_3 \end{bmatrix} \qquad (11)$$

in which $m_{v1}h_1 = m_{v2}h_2 = m_{v3}h_3$. If a linear and constant load-settlement relationship during consolidation is employed (assuming m_{vi} is constant), the influence of the structural rigidity can be formulated in closed form and can be solved in one step. For a non-linear stress-strain relationship during consolidation, m_{vi} is a function of the stress, and an iterative procedure is required.

If the assumed compressible sand deposit is divided into k sublayers, the general form of equation (10) may be formulated in matrix form as:

$$(\sum_{i=1}^{k} m_{vi}\, h_i\, \alpha_i)\, (\underset{\sim}{p}' + t\, \underset{\sim}{s}) = \underset{\sim}{s} \qquad (12)$$

in which $\underset{\sim}{x}$ = the vector with n elements if n columns are considered; and $t = T_{ij}/A_i$.

$$\text{Let} \quad (\sum_{i=1}^{k} m_{vi}\, h_i\, \alpha_i) = \omega \qquad (13)$$

Then equation (12) becomes

$$\omega\, (\underset{\sim}{p}' + t\, \underset{\sim}{s}) = \underset{\sim}{s} \qquad (14)$$

and s can be solved by rewriting equation (14), i.e.,

$$\underset{\sim}{s} = (I - \omega\, t)^{-1}\, \omega\, \underset{\sim}{p}' \qquad (15)$$

in which I is an n x n unit matrix. Evaluating equation (15) at the mean values of all the parameters involved, the mean consolidation settlement considering the soil-structure interaction can be estimated.

3.3 Model for the evaluation of structural damage

The methodology described here for the prediction of pore pressure-induced settlement of structures is applied to estimate the structural damage in this section.

The structural damage can be estimated in terms of the induced maximum differential settlement, δ_{max}, measured from the deformed shape of the foundation after the uniform settlement and the tilt components have been removed (Grant et al, (1972)). Figure 2 illustrates this definition for a three-footing structure

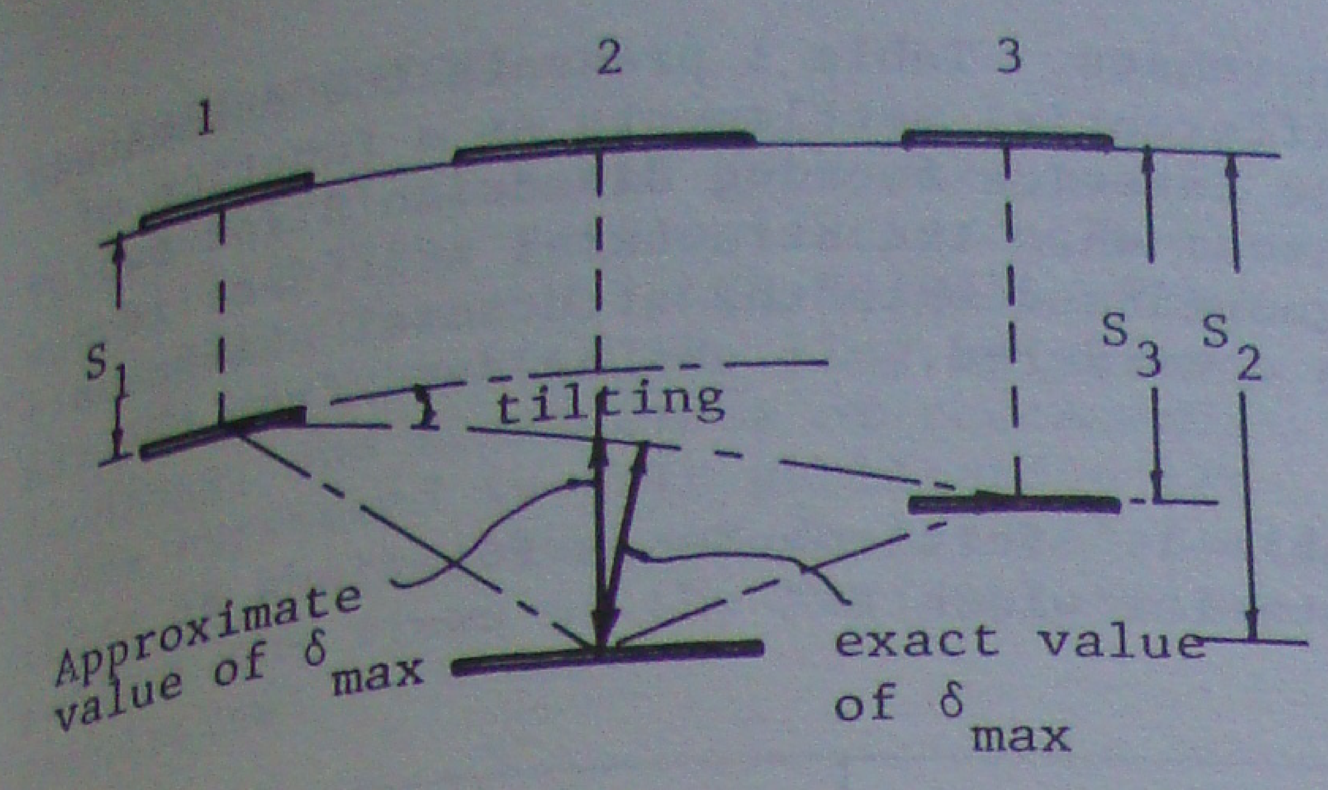

Figure 2. Definition of the maximum differential settlement, δ_{max}

where the middle support is assumed to settle more than the exterior supports. In a symmetric case, the value of the tilt is zero and therefore δ_{max} is simply computed as the difference between the total settlements for the central and the exterior support points. In this case, δ_{max} can be approximately defined for a three-footing structure as illustrated in Figure 2, or can be expressed as:

$$\delta_{max} = S_2 - \frac{1}{2}(S_1 + S_3) \qquad (16)$$

where S_2, S_1 and S_3 are the total pore pressure-induced settlements of the interior and the exterior supports, respectively.

The methodology proposed here is quite involved. A finite element-based computer program was written specifically for this purpose. However, it will not be described here. An example is given below to show the application of the proposed method.

4 EXAMPLE

A structure with three separated footings as shown in Figure 3 is considered here. The structure under consideration is a symmetrically supported two-bay, one-story frame building, with symmetrical loads. The foundation consists of a group of isolated footings which are designed for an allowable bearing pressure of 2000 ksf. The structure is resting on the ground surface of a hypothetical site. The corresponding footing dimensions are 8 ft x 8 ft for the exterior columns and B ft x B ft for the interior column. The site

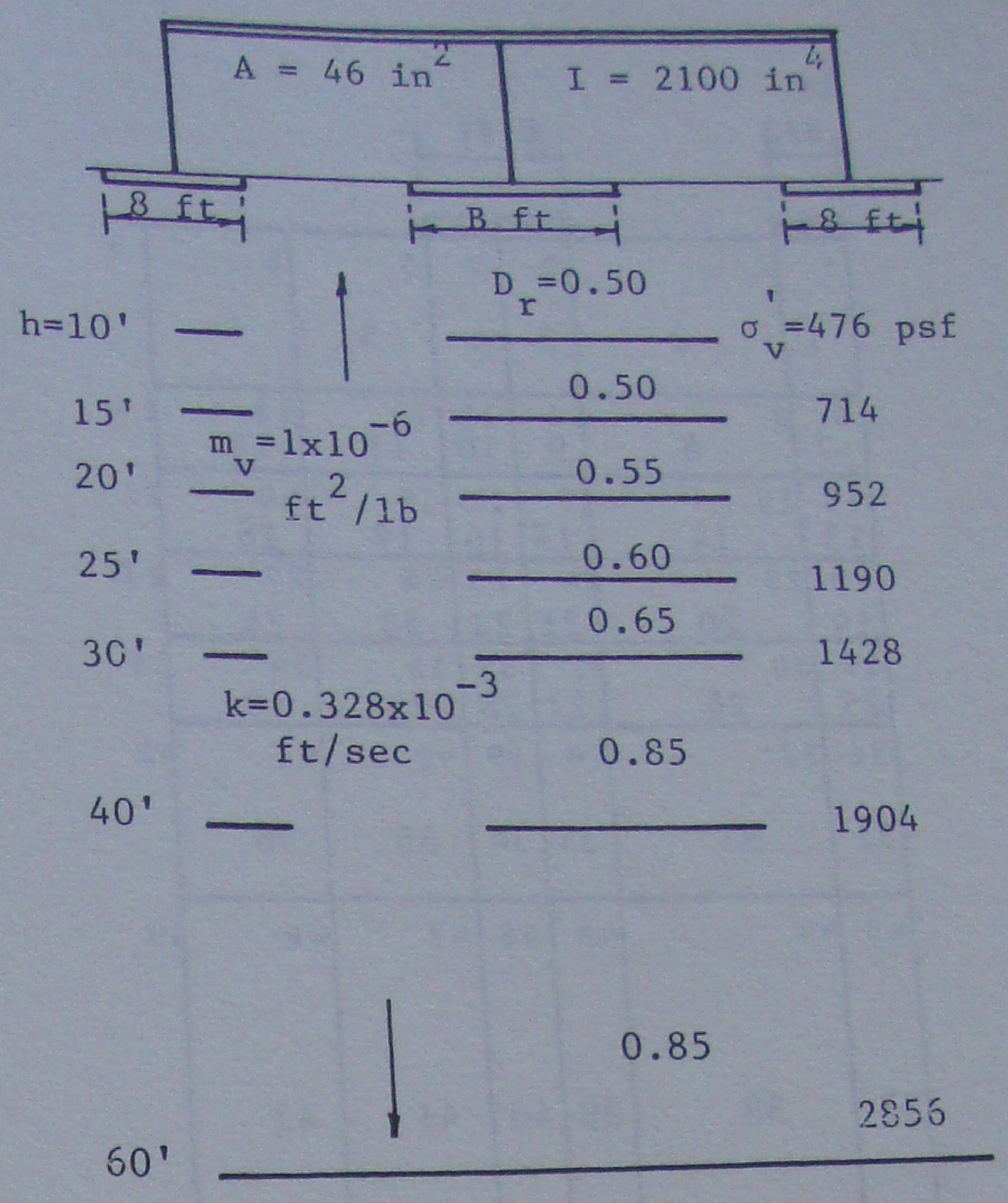

Figure 3. Spread footings foundation and soil strata

is assumed to have sand similar to Oosterchelde sand.

The site consists of seven sublayers. All the sublayers are assumed to be homogeneous. The initial vertical effective stresses can be calculated from static analysis and are shown in Figure 3. Notice that the vertical stress at the ground surface has been set equal to 0.001 psf to avoid difficulties in calculationg the pore pressure ratio at the surface.

The site is subjected to an earthquake of magnitude 7.5 for a duration of 30 seconds and an estimated acceleration of 0.20 g at the ground surface. The time history of the shear stresses at different depths can be found from a site response analysis. By using the simplified method suggested by Seed and Idriss (1971), the average dynamic shear stresses in the sand elements beneath a footing are estimated.

A finite element mesh consisting of forty-two quadrilateral elements are used to obtain a numerical solution for the site as shown in Figure 4. The problem is one of plane shear, and the permeability is assumed to vary as the excess pore pressure increases. Initially, each increases. Table 1 presents the estimated differential settlements as a function of the interior footing dimension B for two cases: when the structural stiffness is ignored and when the structural stiffness is considered. It can be seen from the

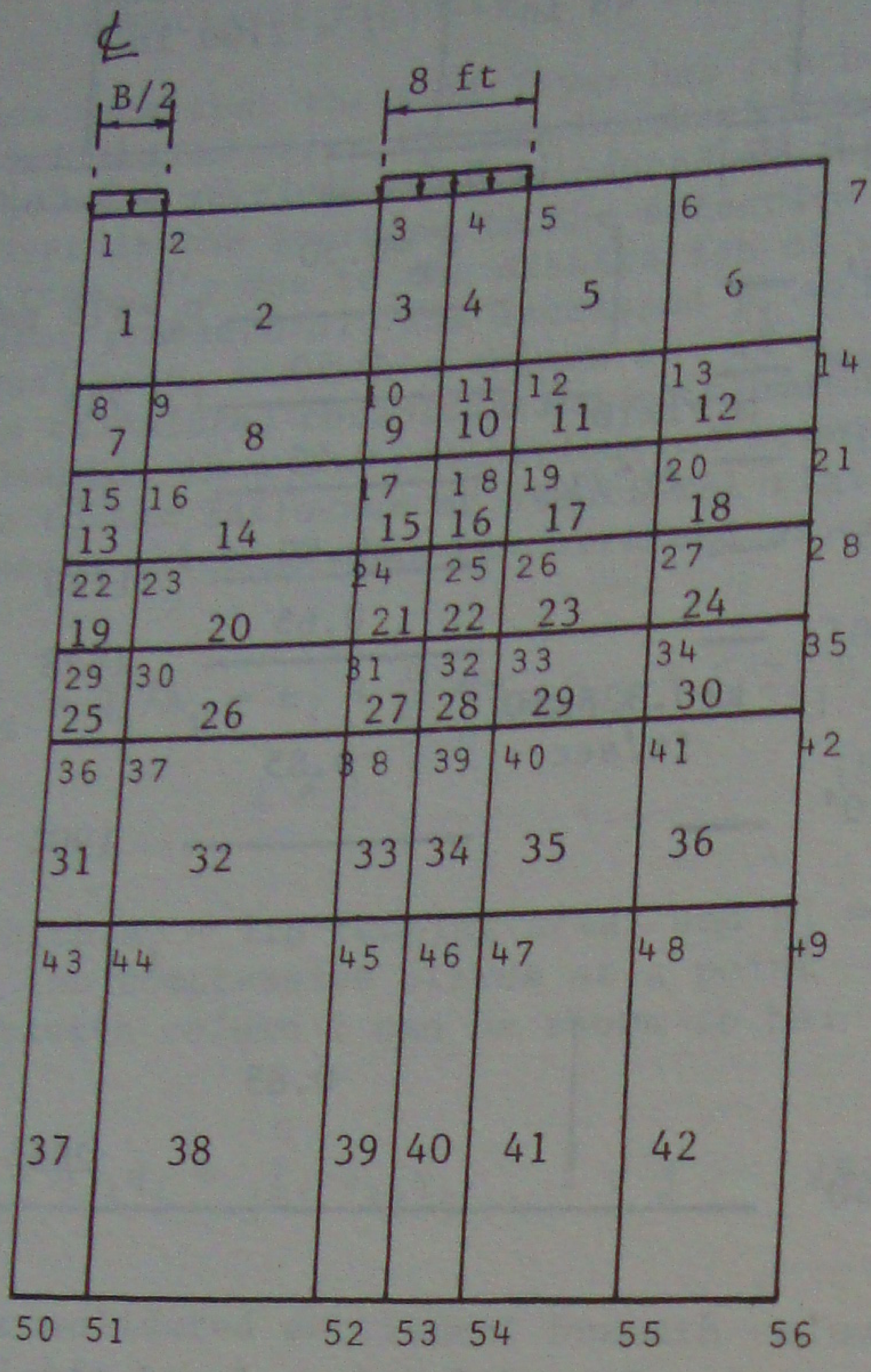

Figure 4. Mesh used in the finite element computer program

Table 1. Settlements as functions of interior width B

Interior Footing Width B (ft)		8	10	12	14
δ_{max} (in)	Structural Stiffness Ignored	0.725	1.015	1.248	1.626
	Structural Stiffness Considered	0.651	0.902	1.101	1.500

table that when soil-structure interaction is accounted for during the consolidation stage after the earthquake has ceased, the differential settlement decreases by a considerable amount.

element is considered to have compressibility of 1.0 x 10^{-6} ft^2/lb and vertical permeability of 3.28 x 10^{-4} ft/sec. The relative densities for each layer are shown in Figure 3. The base and the two sides are assumed to be impermeable. The sand deposits at the edge of the soil body are assumed to be far away from the structure, and are considered to be under isotropically consolidated conditions. For the earthquake under consideration, the number of equivalent cycles is assumed to be 30.

The total settlement of a footing is estimated by adding the undrained residual settlement and the consolidation settlement. Then, the differential settlement of the foundation is evaluated. Since the pore pressure beneath the interior footing decreases as B increases, the settlement of the interior footing decreases as B increases. Also, since the pore pressure beneath the exterior footing increases as B increases, the settlement of the exterior footing increases as B increases. Consequently, the differential settlement increases as the interior footing width B

5 CONCLUSIONS

A method is proposed here to estimate damage to a structure due to earthquake-induced liquefaction considering the effect of the soil-structure interaction. The damage is estimated in terms of diffential settlement.

For the foundation with three spread footings considered here, the excess pore pressure developed below the center of the interior footing is less than that below the outer footings. As the interior footing width increases, both the pore-water pressure build-up and the associated settlement beneath the outer footings increases significantly. As a result, the differential settlement increases to a certain degree. The structural rigidity can reduce the differential settlement and the consequent probability of structural damage. For a realistic assessment of structural damage due to earthquake-induced liquefaction, the structural rigidity should be considered.

6 ACKNOWLEDGEMENT

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