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Analysis of gravel drains in liquefiable soils

Analyse de tuyau de gravier en sols liquéfiables

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ABSTRACT: liquefaction of saturated cohesionless soils during earthquakes can cause substantial damage to structures founded in these soils. Remedial measures such as gravel drains are a popular and economical means of preventing liquefaction in saturated cohesionless soils by providing short drainage paths, which prevent significant rise in pore pressures. For evaluating the effectivness of gravel drains as a remedial measure against liquefaction, one dimensional finite element models incorporating simultaneous generation and dissipation of pore pressure for both vertical and radial drainage under earthquake loading have been developed. The models have been used to evaluate the influence of drain spacing and method of installation on the pore pressures developed and potential for ground damage for a site during earthquake.

RESUME: Liquefation de sable sature peut causer dommage significatif a structures. Tayau du gravier peut utiliser pour prevenir liquefation de sable sature. Un modele FEM est creer pour analyser tayau du gravier. Le modele est utiliser pour evaluer l'enpacement et le methode d'installation de tayau du gravier sur le pore-pression produire pendant liquefation.

1 INTRODUCTION

Loose saturated sands can liquefy during earthquakes, leading to large settlements and possible catastropic failure. Conventional methods of analysing liquefaction are based on either laboratory or field test data to evaluate the cyclic resistance under undrained conditions (Seed et al 1985). The generation and dissipation of pore-pressures during the earthquake is usually not considered. This can sometimes result in conservative design. Remedial measures against liquefaction include both densification and/or improving the drainage characteristics of the natural soil. Gravel drains are an popular and economical means of preventing liquefaction in saturated cohesionless soils during earthquakes, by providing short drainage paths which enable rapid dissipation of excess pore pressures (Seed and Booker 1977).

In this paper finite element models for one dimensional vertical and radial generation and dissipation of pore-pressures during liquefaction are developed. The models have been implemented in two finite element programs. The programs have been tested and found to be accurate. The programs have been used to evaluate the effectiveness of gravel drains as a remedial measure against liquefaction. The important factors in a gravel drain remedial scheme are the drain diameter, drain spacing, grain size distribution of gravel and method of installation. Gravel drains can be installed by boring or driving. The process of driving will densify the natural soil, increasing its resistance to liquefaction. However the soil permeability will also be reduced, preventing rapid dissipation of pore pressures. Details of the finite element models, soil parameters and influence of drain spacing and installation for a number of soil conditions at a site are presented.

2 FINITE ELEMENT MODELS

Finite element models for one dimensional generation and dissipation of pore pressures in sands during liquefaction are developed for vertical and radial flow.

2.1 Vertical flow

The basic equation for one dimensional generation and dissipation of pore-pressure can be written as (Seed et al 1976);

$$\delta/\delta z((k_z/g_w)\,\delta u/\delta z) = m_v(\delta u/\delta t - q) \qquad (1)$$

where u is the excess pore pressure, k_z is the vertical permeability, g_w is the unit weight of water, m_v is the coefficient of volume compressibility and $q=\delta u_g/\delta t$ is the rate of generation of pore-pressure caused by the earthquake and u_g is the pore-pressure generated. The finite element formulation of the above equation leads to;

$$KU + C\delta U/\delta t = Q \qquad (2)$$

The terms of the stiffness matrix can be evaluated used conventional procedures. Time integration of the above equation leads to;

$$(C+K\beta dt)U_1 = (C-K(1-\beta)dt)U_0+\beta dTQ_1+(1-\beta)dTQ_0 \qquad (3)$$

Where β is the time integration constant and β >= 0.5 leads to an unconditionally stable solution. The rate of generation of pore-pressure under undrained conditions can be estimated from the procedures of Seed et al (1976).

$$r_u = 1/2 + \arcsin(2r_N^{1/\alpha}-1)/\pi \qquad (4)$$

Where r_u is the pore-pressure ratio, $r_N=N/N_l$ is the cycle ratio, N is the current cycle number, N_l is the number of cycles to liquefaction and α is a constant.

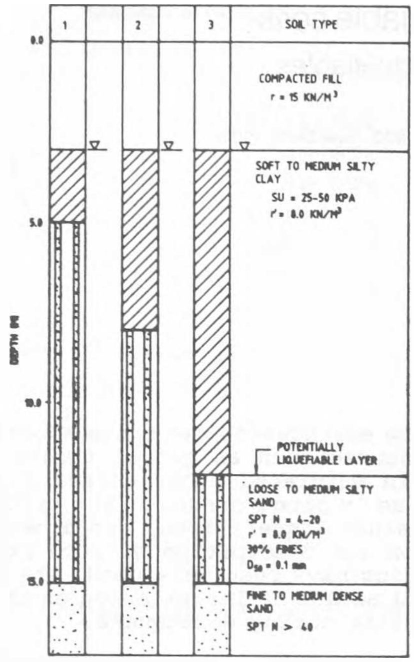

Fig.1. Soil profiles used for analysis

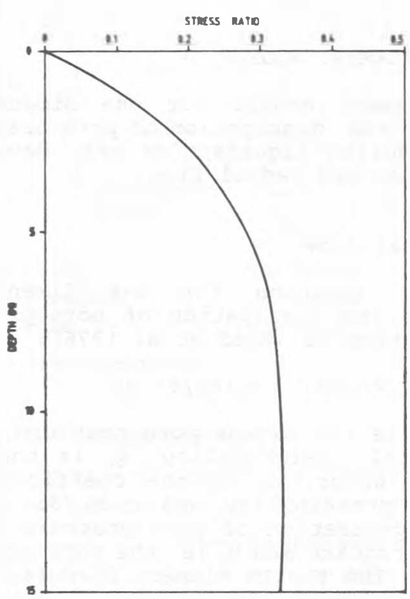

Fig.2. Stress ratio with depth

2.2 Radial flow

The basic equation for radial flow for generation and dissipation of pore-pressure during liquefaction can be written as (Seed and Booker 1977);

$$(1/r)\,\delta/\delta r(r(k_r/g_w)\,\delta u/\delta z) = m_v(\delta u/\delta t - q) \qquad (5)$$

Where k_r is the radial permeability and r the radial distance. The finite element equations can be derived as before, using conventional procedures.

The rate of pore-pressure generation can be numerically evaluated from the equation 3. The equation 3 is nonlinear and is solved incrementally by updating parameters every time step.

3 DETERMINATION OF PARAMETERS

Procedures for determination of the various parameters is discussed.

3.1 Earthquake data

The earthquake magnitude M and the peak ground accleration a_{max}/g are required. The time history of the earthquake can be converted to an equivalent number of cycles N_{eq} and an equivalent duration Teq (Seed et al 1985);

$$\tau/\sigma_0' = 0.65(a_{max}/g)(\sigma_0/\sigma_0')\,r_d \qquad (6)$$

$$r_d = (1.0-0.015z) \qquad (7)$$

Where τ is the earthquake induced shear stress, σ_0 is the total stress, σ_0' is the effective stress and z is the depth.

3.2 Cyclic strength

Where laboratory test data is available, the relationship between the stress ratio(τ/σ_0') and number of cycles to liquefaction N_l can be developed. Alternatively a number of emprical correlations are available (Ishihara 1985);

$$(\tau/\sigma_0)_{20} = 0.0676(N_1)^{1/2}+0.225\log_{10}(0.35/D_{50}) \qquad (8)$$

for $0.04\text{mm} < D_{50} < 0.60$ mm

$$(\tau/\sigma_0)_{20} = 0.0676(N_1)^{1/2}-0.05 \qquad (9)$$

for $0.60\text{mm} < D_{50} < 1.5$mm

$$(\tau/\sigma_0)_{20} = 0.0676(N_1)^{1/2}+0.0035c \qquad (10)$$

$$N_1 = C_N N_{spt} \qquad (11)$$

$$C_N = 1.7/(\sigma_v' + 0.7) \qquad (12)$$

Where $(\tau/\sigma_0')_{20}$ is the stress ratio required to cause liquefaction in 20 cycles, σ_v' is the vertical effective stress in kgf/cm^2, N_{spt} is the blow count and c is the percent of fines passing seive size of 0.074mm.

3.3 Permeability

The permeability of sands may be estimated as;

$$k = 85\text{-}130(D_{20})^2 \qquad (13)$$

3.4 Coefficient of volume compressibility

The coefficient of volume compressibility varies with the pore-pressure ratio as (Seed et al 1976);

$$m_v/m_{v0} = \mathrm{EXP}(Ax^B)/(1+Ax^B+0.50A^2x^{2B}) \qquad (14)$$

Where x is the pore-pressure ratio, $A=5(1.5-D_r)$, $B=3/(2^{2D_r})$, D_r is the relative density and m_{v0} is the initial coefficient of compressibility ($21\text{-}42\times10^{-6}$ m^2/KN).

4 RESULTS

The models developed in the previous sections have been used for some preliminary studies

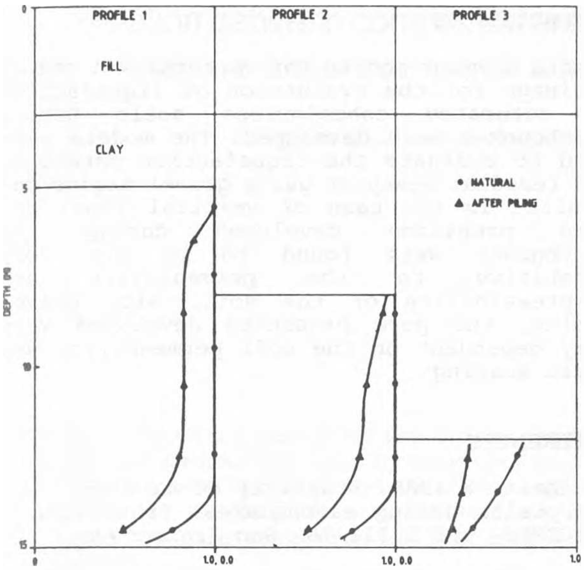

Fig.3. Pore pressure ratio at end of earthquake

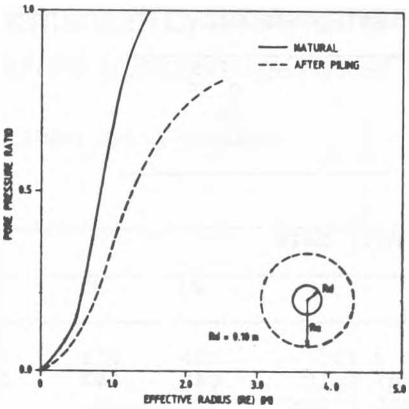

Fig.4. Influence of radial drainage on pore pressure ratio for profiles 1 & 2

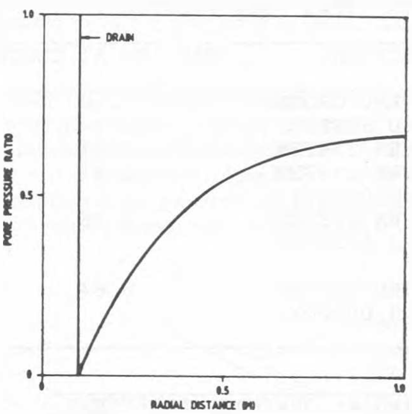

Fig.5. Pore pressure ratio for radial drainage

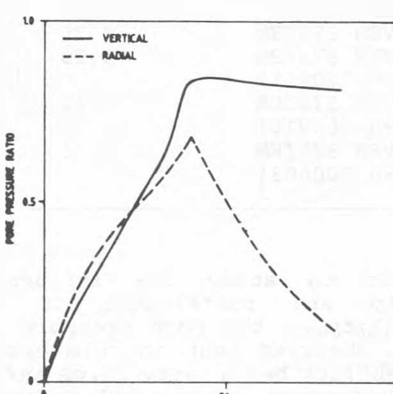

Fig.6. Development on pore pressure during earthquake for profile 2

for liquefaction and remedial measures evaluation for a site. Figure 1 shows three typical soil profiles considered in the analysis. In the soil profiles, the top 3.0 m is filled up soil, underlain by soft to medium clay of thickness varying between 2.0 to 9.0 m. This is followed by loose to medium silty sand with SPT N values varying between 4-20. The silty sand layer is underlain by dense sands. The ground water table is at the surface. Table 1 tabulates the earthquake data and figure 2 illustrates the earthquake induced stress ratio in the ground.

The models are used to evaluate the liquefaction potential of the site. For each of the three profiles, pore pressure generation during the earthquake is analysed for (a) vertical flow and (b) for radial flow with gravel drains for both bored and driven systems. In case of the bored system, it is assumed that boring causes no change in the soil properties. For the driven system, a improvement of 1.5 times of the SPT N values due to densification has been considered. Table 2 tabulates the soil parameters used in the analy sis.

Figure 3 illustrates the pore-pressure ratio for the three profiles at the end of the earthquake obtained for vertical flow only. It can be observed that both profiles 1 and 2 liquefy. If a 1.5 times improvement in blow counts is considered, the maximum pore-pressure ratio for profile 2 in the center of the layer reduces to 0.84. In profile 3, partial liquefaction takes place, with the maximum pore-pressure ratio reducing from 0.70 to 0.40 because of the densification. In the above analysis, no change in the soil permeability was considered due to driving. The process of driving is likely to reduce soil permeability and compressibility, in addition to increasing soil density. A number of parametric studies were carried out to study the influence of permeability and coeffcient of volume compressibility on the soil response. Table 3 tabulates the summary of the parametric study. It may be observed, that large changes in both the permesbility and coefficient of volume compressibility have limited influence on the maximum pore-pressure ratio in the center of the liquefiable soil layer for profile 2.

Figure 4 illustrates the influence of installing a 0.20m diameter drain at various spacing for both bored and driven systems. It can be observed that a drain spacing of 2.0 m (R_e=1.0m) for bored system and a spacing of 4.0m (R_e=2.0m) for a driven system significantly reduce the pore pressures developed in the ground. In the above analysis, no change in soil permeability and compressibility was considered. Table 4 tabulates the results of parametric studies

TABLE 1. EARTHQUAKE DATA

MAGNITUDE	7.5
a_{MAX}/g	0.35
N_{eq}	20
T_{eq}	40

TABLE 2: SOIL DATA

PROPERTY	P1	P2	P3
K (M/SEC) x 10^{-3}	.015	.015	.015
mvo (M2/kn) x 10^{-3}	.042	.042	.042
SPT NATURAL GROUND	8	8	15

TABLE 3. VERTICAL FLOW PROFILE 2

NO.	CONDITION	MAX. Ru AT CENTER
1.	NATURAL GROUND/ BORED SYSTEM	1.0
2.	DRIVEN SYSTEM	0.84
3.	DRIVEN SYSTEM (mV=0.000021)	0.81
4.	DRIVEN SYSTEM (kz=0.000075)	0.75
5.	DRIVEN SYSTEM (kz=0.000003)	0.84

TABLE 4. RADIAL DRAINAGE PROFILE 2

NO.	CONDITION	MAX. Ru
1.	NATURAL GROUND/ BORED SYSTEM	0.67
2.	DRIVEN SYSTEM	0.39
3.	DRIVEN SYSTEM (mV=0.000021)	0.23
4.	DRIVEN SYSTEM (kz=0.000075)	0.11
5.	DRIVEN SYSTEM (kz=0.000003)	0.75

carried out to study the influence of permeability and coefficient of volume compressibility on the pore pressure ratio. It can be observed that in this case, the soil permeability has a very large influence on the pore-presure ratio, and a five fold reduction in permeability can offset any gain in strength due to the process of driving. A 100% change in compressibility on the other hand does not cause much change in the pore pressures developed during the earthquake.

Figure 5 illustrates a typical pore pressure distribution around the drains. Figure 6 illustrates a typical development of pore-pressure during the earthquake for profile 2 for both vertical flow and radial flow. For the case of radial flow, lower pore-pressures are developed during the earthquake and faster dissipation takes place after the earthquake. The final pore pressure ratio will be the a combination of the the vertical and radial pore pressure ratios. For practical purposes the combined pore pressure ratio can be taken as the product of the two pore pressure ratios.

5 CONCLUSIONS

Finite element models for vertical and radial drainage for the evaluation of liquefaction of saturated cohesionless soils during earthquakes were developed. The models were used to evaluate the liquefaction potential and remedial measures using gravel drains for a site. In the case of vertical flow, the pore pressures developed during the earthquake were found to be not very senstitive to the permeability and compressibility of the soil. With gravel drains, the pore pressures developed were very dependent on the soil permeability and drain spacing.

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