Liquefaction resistance and earthquake-induced settlement of granular soil deposits stabilized by gravel drains

Résistance à la liquéfaction et tassement durant un séisme d'un dépôt de sol granulaire stabilisé au moyen de drain de gravier

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ABSTRACT: Installation of gravel drains in sand deposits mitigates liquefaction potential and reduces earthquake-induced settlement by (a) densifying the surrounding ground during drain installation, (b) providing drainage for earthquake-induced excess porewater pressures, and (c) introducing stiff elements into the deposit. Methods for settlement analysis are available for undrained or drained response of level granular ground subjected to seismic shaking. The drainage capacity is expressed in terms of earthquake-induced compressibility of the granular soil deposit and the permeability of the layer and that of the drain material. Simple charts are proposed for estimating the influence of partial drainage on both liquefaction resistance and earthquake-induced settlement of granular soil deposits improved by gravel drains.

RÉSUMÉ: L'installation de drains de gravier dans les dépôts de sable réduit le potentiel de liquéfaction et réduit les tassements associés à un séisme a) en densifiant les sols avoisinants lors de l'installation des drains, b) en permettant le drainage des surpressions interstitielles induites durant le séisme, et c) en introduisant des éléments plus rigides dans le dépôt. Des méthodes de calcul de tassement sont disponibles pour la réponse non drainée ou drainée des dépôts de sable horizontaux soumis à des secousses sismiques. La capacité de drainage est exprimée en termes de la compreeibilité du dépôt sous charge sismique et de la perméabilité du dépôt et du drain de gravier. Des abaques faciles à utiliser sont proposés pour estimer l'influence d'un drainage partiel sur la résistance à la liquéfaction et les tassements durant un séisme d'un dépôt granulaire améliorés par des drains de gravier.

1. INTRODUCTION

Gravel drains mitigate liquefaction potential and reduce earthquake-induced settlement by (a) densifying the surrounding soil, (b) providing drainage for shaking-induced excess porewater pressures, and (c) improving the seismic response by introducing stiff elements. The reinforcement effect may be slight and could be conservatively ignored (Baez and Martin 1992). Densification of the surrounding soil is reflected by in situ penetration test results and could be taken into consideration in the analysis. The main effect of gravel drains to be quantified is the drainage provided for shaking-induced excess porewater pressures.

Starting in 1980s, gravel drains have been used extensively in Japan to mitigate seismic risks. Sites improved by gravel drain installation have performed well in recent earthquakes (Sonu et al. 1993, Iai et al. 1994, Mitchell et al. 1995, Yasuda et al. 1996). Shaking table tests on gravel drains (Sasaki and Taniguchi 1982), and in situ vibration tests (Onoue et al. 1987) have verified the efficiency of well-designed and constructed gravel drain systems to mitigate seismic hazards. Gravel drain installation method may influence the drainage efficiency. Vibro-replacement methods may (Onoue et al. 1987) or may not (Iai et al. 1994) mix the drain material with in situ soil. The former lowers the permeability of the gravel drain and results in higher drain resistance.

Seed and Booker (1977) provided design charts for gravel drains in terms of maximum allowable shaking-induced porewater pressure ratio, ignoring well resistance. Onoue (1988) developed design charts similar to those proposed by Seed and Booker (1977), however, with correction factors for well resistance. Iai and Koizumi (1986) includes solutions for estimating seismic-induced porewater pressure. Pestana et al. (1997) have developed the finite element code FEQDrain for gravel drain analysis.

2. SETTLEMENT ANALYSIS

Methods are available for settlement analysis assuming drained or undrained response of granular deposits subjected to earthquake shaking. Methods for estimating earthquake-induced settlement due to drained response include Tokimatsu and Seed (1987), and Shahien (1998). These methods are based on the methodology of Silver and Seed (1969). The earthquake-induced vertical strain is expressed in terms of seismic shear stress ratio, $(N_1)_{60}$ or N_{60}, earthquake magnitude, and effective overburden pressure.

Methods for estimating earthquake-induced settlement for undrained response include Tokimatsu and Seed (1987), and Ishihara and Yoshimine (1992). Shahien (1998) estimates earthquake-induced secant coefficient of volume compressibility, $m_v = \Delta\varepsilon_v/\Delta\sigma'_v$, using both factor of safety against liquefaction F_1, and N_{60}. Field records were used to verify the proposed method.

Drainage during the shaking process creates a partially-drained behavior between the drained and undrained response. Such a behavior has not yet been quantified in a simple procedure. In the absence of a simple method for analyzing the partially-drained response, the drainage effect has been often ignored, assuming a completely undrained response. Such an assumption might be too conservative in evaluating the liquefaction potential of some deposits and in estimating earthquake-induced settlement.

New methods for earthquake-induced settlement analyses (Shahien 1998) are used to establish the drained and undrained limits in Fig. 1 which shows that the difference between settlements from drained and undrained response decrease as seismic shear stress ratio increases, approaching a settlement ratio of one as seismic shear stress ratio corresponds to $F_1 = 2$. This means that the need for partially-drained response analysis decreases as F_1 increases.

3. POREWATER PRESSURE GENERATION

Based on laboratory undrained cyclic test results (Lee and Albaisa 1974, De Alba et al. 1975), Seed et al. (1975) suggested an empirical equation for the relationship between porewater pressure ratio and equivalent number of uniform stress cycles caused by the earthquake, N_c, and number of uniform stress cycles required to cause liquefaction, N_1. The excess porewater pressure ratio can be also expressed in terms of t/t_1, where t_1 is the time required to reach liquefaction. The following linear relationship is used in the present formulation

$$\frac{u'}{\sigma'_{vc}} = \frac{t}{t_1} \tag{1}$$

which predicts excess porewater pressure ratios that are on the conservative side within the range of the observed data and close to the average response (Xu 1991).

The assumption of a linear relationship between log seismic shear stress ratio and log number of cycles to cause liquefaction with a slope β_1 leads to

$$F_1 = \frac{s_u(\text{yield})}{\tau(\text{seismic})} = \left[\frac{N_c}{N_1}\right]^{\beta_1} = \left[\frac{t}{t_1}\right]^{\beta_1} = \left[\frac{u'}{\sigma'_{vc}}\right]^{\beta_1} \tag{2}$$

The calculated values of β_1 fall within a narrow range between 0.1 to 0.3 with an average of 0.2 (De Alba et al. 1975, Ishihara 1985, Shahien 1998). Equation 2 can be rewritten as follows to estimate the excess porewater pressure ratio of a granular soil layer

$$\frac{u'}{\sigma'_{vo}} = \left[\frac{1}{F_1}\right]^5 \quad \text{for} \quad \frac{u'}{\sigma'_{vo}} \le 1.0 \tag{3}$$

4. POREWATER PRESSURE DISSIPATION

Lo (1991) proposed the following expression for average excess porewater pressure $\overline{u'}$ at any time t in a soil with vertical drains, subjected to instant excess porewater pressure $\overline{u'_o}$

$$\overline{u'} = \overline{u'_o}\zeta(t) \tag{4}$$

$$\zeta(t) = \exp\left[-\left[\frac{2c_h}{r_e^2\,[F(n,s)+2.5G]} + 4\frac{c_v}{H^2}\right]t\right] \tag{5}$$

Ignoring soil disturbance by drain installation, F(n,s) = F(n) and

$$F(n) = \frac{n^2}{n^2-1}\left(\ln n - \frac{3}{4}\right) + \frac{1}{n^2-1}\left[\frac{1}{4n^2}\right] \tag{6}$$

$$G = \frac{k_h}{k_w}\left[\frac{l_m}{2r_w}\right]^2 \tag{7}$$

where $n = r_e/r_w$, r_w = radius of the drain, r_e = radius of influence of drain ≈ 0.525 DS, DS = drain spacing, k_w = coefficient of permeability of the drain, and l_m = maximum drainage length of the drain.

For time-dependent porewater pressure generation for the duration t_d of earthquake, and using Eq. 1 at time $t = \tau$ (Mesri 1973), the increment of porewater pressure increase is

$$du'(\tau) = \sigma'_{vo}\frac{1}{t_1}d\tau \tag{8}$$

The value of t_d is estimated from duration of strong shaking (Seed et al. 1975).

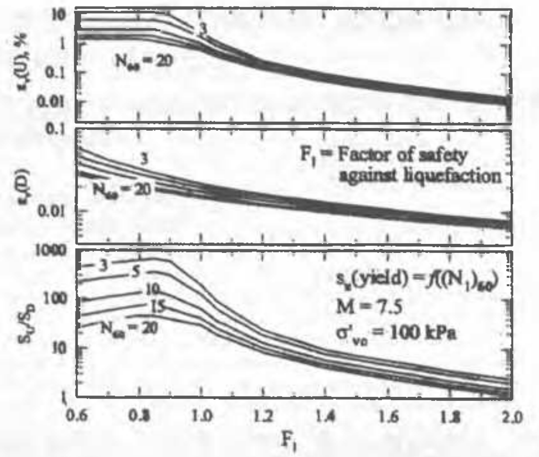

Fig. 1 Earthquake-induced vertical strain and settlement

5. PARTIALLY-DRAINED RESPONSE

The excess porewater pressure during an earthquake, accounting for drainage, is obtained by integrating Eq. 8. The average excess porewater pressure ratio in any layer during the earthquake shaking is

$$\frac{\overline{u'}}{\sigma'_{vo}} = \frac{t/t_1}{A}\,[1 - \exp(-A)] \tag{9}$$

$$A = \frac{2T_h}{[F(n)+2.5G]} + 4T_v\,,$$

$$T_h = \frac{k_h t}{m_v\gamma_w r_e^2}\,,\ \text{and}\ T_v = \frac{k_v t}{m_v\gamma_w L^2}$$

The average excess porewater pressure ratio at $t = t_d$ is

$$\frac{\overline{u'_m}}{\sigma'_{vo}} = \frac{t_d/t_1}{A_d}\left[1 - \exp(-A_d)\right] \tag{10}$$

$$A_d = \frac{2T_{dh}}{[F(n) + 2.5G]} + 4T_{dv}\,,$$

$$T_{dh} = \frac{k_h t_d}{m_v\gamma_w r_w^2 n^2}\,,\quad \text{and}\quad T_{dv} = \frac{k_v t_d}{m_v\gamma_w L^2}$$

u'_m/σ'_{vo} is the maximum porewater pressure ratio developed during earthquake for partially drained condition. The average excess porewater pressure ratio at any time $t > t_d$ is

$$\frac{\overline{u'}}{\sigma'_{vo}} = \frac{\overline{u'_m}}{\sigma'_{vo}}\,\zeta\zeta_d \tag{11}$$

$$\zeta = \exp(-A)\,,\ \text{and}\ \zeta_d = \exp(-A_d)$$

The excess porewater pressure reduction ratio reflecting partial drainage is

$$\mu_u = \frac{[\overline{u'_m}]_{PD}}{[\overline{u'_m}]_U} = \frac{\left[1 - \exp(-A_d)\right]}{A_d} \tag{12}$$

and the increase in factor of safety against liquefaction, using Eqs. 3 and 12 is

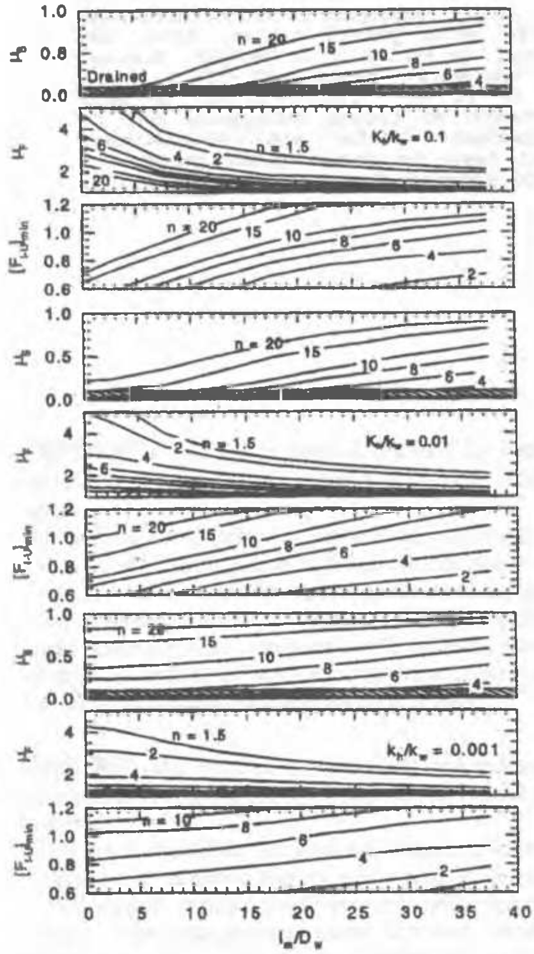

Fig. 2 Charts for $[F_{1-U}]_{min}$, μ_F , and μ_S

$$\mu_F = \frac{[s_u(\text{yield})]_{PD}}{[s_u(\text{yield})]_U} = \left[\frac{1-\exp(-A_d)}{A_d}\right]^{-\frac{1}{5}} \tag{13}$$

Shahien (1998) estimates earthquake-induced settlement of a sand sublayer of thickness L according to

$$S = m_v \, \Delta\sigma'_v \, L \tag{14}$$

where m_v = secant earthquake-induced coefficient of volume compressibility, and $\Delta\sigma'_v$ = increase in effective vertical stress following shaking, may take values corresponding to fully or partially undrained conditions. Both S_{PD} and S_U are calculated by Eq. 14 using the values of m_v and $\Delta\sigma'_v$ for the partially drained and undrained conditions, respectively. An empirical chart has been developed for m_v in terms of F_1 and N_{60} (Shahien 1998). The settlement reduction factor for partially drained condition is obtained using Eqs. 3 and 14

$$\mu_S = \frac{S_{PD}}{S_U} = \frac{m_{v-PD}}{m_{v-U}}\left[\frac{F_{1-U}}{F_{1-PD}}\right]^5 \tag{15}$$

Because m_v is dependent on F_1, an iterative procedure is used to reach convergence for a set of F_1, m_v, and S.

A series of simple charts have been developed for designing vertical gravel drains to mitigate liquefaction potential and reduce settlement of sand layers. Ground condition is defined in terms of N_{60}, k_v, and k_h/k_v. The gravel drain is specified in terms of its permeability, k_w, diameter, D_w, maximum drainage length, l_m, and spacing DS = $0.95nD_w$ for a triangular pattern.

A parametric analysis was performed using $k_h/k_v = 1$, k_v in the range of 10^{-3} to 10^{-5} m/s and N_{60} in the range of 2 to 20. The relationship between μ_F or μ_S with F_1 is practically independent of N_{60} , and the gravel drain is effective in mitigating liquefaction and decreasing settlement when F_{1-U} is higher than a threshold value of $[F_{1-U}]_{min}$.

The values of μ_F, μ_S, and $[F_{1-U}]_{min}$ are shown in Fig. 2 for three values of k_h/k_w , and a range of values of n and l_m/D_w ; the hatched zones correspond to fully drained behavior. For a given gravel drain permeability (e.g., $k_w = 10^{-2}$ m/s) as k_h and therefore k_h/k_v increase well resistance as determined by l_m becomes an important factor.

In order to use Fig. 2 to design a vertical gravel drain system to mitigate liquefaction and decrease settlement, one must first compute F_{1-U} Then l_m/D_w is obtained by selecting an l_m corresponding to a fully or partially penetrating gravel drain of an available diameter D_m and k_w . Using the value of l_m/D_w together with k_h/k_w, a value of n is selected in Fig. 2 for any $[F_{1-U}]_{min}$ greater than F_{1-U}. The values of μ_F and μ_S corresponding to l_m/D_w , n and k_h/k_w, are read from the chart. The design process may be repeated for different values of l_m, D_w, k_w and n to arrive at the desired values of μ_F and μ_S .

6. FIELD CASE HISTORY

Sonu et al. (1993) have reported the response of a reclaimed land at the port of Kushiro in Hokkaido, Japan, to the 7.8 magnitude earthquake of January 15, 1992. The reclaimed soil was characterized as coarse sand with a median diameter in the range of 0.2 to 0.3 mm. The ground water level was 1.0 to 1.7 m below the ground surface and N values were in the range of 5 to 8 (Iai et al. 1995). Non-displacement type, 400 mm diameter gravel drains were installed at DS of 5m in part of the site (Sonu et al. 1993, Iai et al. 1994). The epicenter of the earthquake was 15 km offshore of the port and 107 km beneath the ocean, recording a ground acceleration of 0.48g within the port.

Liquefaction damage was widespread throughout the site; however, none of the facilities protected with gravel drains suffered damage from liquefaction with ground surface settlements that were less than 40 mm.

The proposed methodology for earthquake-induced settlement analyses estimates ground surface S_U in the range 50 to 80 mm and S_D in the range 5 to 10 mm. F_{1-U} is in the range of 0.6 to 0.8. The charts in Fig. 2 suggest F_{1-PU} in the range 1.1 to 1.5, and estimate surface settlement equal to S_D. These are in excellent agreement with field observations.

7. CONCLUSIONS

Gravel drains can be designed and installed in sand deposits to mitigate liquefaction and decrease settlement by dissipating earthquake-induced porewater pressures. After computing F_{1-U} and noting l_m and available D_w, an n is selected for any $[F_{1-U}]_{min}$ greater than F_{1-U} , and the corresponding values of μ_F and μ_S are read from the charts for the appropriate value of k_h/k_w . Different values of D_w, n, and even k_w may be considered to arrive at the desired values of μ_F and μ_S. For example, for a sand layer with l_m = 6 m, $k_h = 10^{-4}$m/s, F_{1-U} = 0.8 and S_U = 50 mm, gravel drains with $k_w = 10^{-2}$m/s, D_w = 0.4 m, and DS = 2.4 m would reduce settlement to S_D = 2.5 mm. An alternative DS = 4.0 m would result in S_{PD} = 10 mm.

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