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Quantitative evaluation of stone column techniques for earthquake liquefaction mitigation

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ABSTRACT: An evaluation of current knowledge on the relative effectiveness of stone columns for the mitigation of soil liquefaction during earthquakes is presented. The limitations on the use of simplified analytical methods such as Seed and Booker's (1976) are noted. Reviewed experimental data indicates that the linear mechanisms of consolidation are valid only if the pore pressure ratio remains below 0.5. Pore pressures within the gravel drain have also been observed to vary, contrary to the assumption that they remain essentially unchanged. The evaluation concludes that the most appropriate and safe way to mitigate earthquake induced potential liquefaction remains in densifying the soil. Nevertheless, research to date suggests that additional benefits resulting from the drainage capacity of the stone column should be included in the design considerations provided certain limitations are accounted for.

1 INTRODUCTION

One of the most dramatic causes of damage to engineering structures during earthquakes has been the development of soil liquefaction beneath and around structures. The phenomenon is associated primarily, but not exclusively, with saturated cohesionless soils (U.S.NRC, 1985).

The concept of soil liquefaction has been understood for many years. When a saturated cohesionless deposit is subjected to strong shaking it tends to compact and decrease in volume. However, if drainage is prevented, volume changes cannot occur and stresses are partially transferred to the pore water. The stress transfer causes a rise in pore water pressure and a corresponding drop in effective stress. If the effective stress becomes zero, contact is lost between soil grains and a liquefied state develops.

The purpose of an effective soil improvement program is to mitigate the potential for liquefaction and minimize damaging settlement in the soil. Ways to achieve mitigation include: increase of soil density; increase of confining pressure; increase of material stiffness by intrusion of grout, chemicals or stone columns; control and/or prevention of pore pressure development. It is believed that the proper installation of stone columns by vibro-replacement mitigates the potential for liquefaction by increasing the density of the surrounding soil, providing drainage for the control of pore water pressures, and introducing a stiff element (stone column) which can potentially carry higher shear stress levels (Priebe, 1989).

The objective of this paper is to present and evaluate current knowledge which quantifies the relative effectiveness of the stone column system in the mitigation of soil liquefaction during earthquakes. Because the concept of soil densification by vibro techniques is fairly well understood, the paper primarily overviews theoretical and experimental models capable of predicting or simulating generation and dissipation of excess pore water pressures.

2 INSTALLATION OF STONE COLUMNS

Figure 1 shows a typical arrangement of stone columns. The three most common methods of stone column installation are, top feed (gravel fed around surface annulus), bottom feed (gravel fed from tip of vibrator), and auger-casing with internal gravel feeding system. The first two systems, commonly called vibro-replacement, involve the use of electrical vibrators employed to help advance the hole and densify the surrounding soil. The vibro-replacement process generally involves advancing the hole by means of water or air jets to a specified depth, feeding stone (gravel) into the hole, then beginning a series of lifting and lowering action of the vibrator (30 Hz) as the gravel is being added to densify the gravel and surrounding soil. Figure 2 shows the top-feed method of installation. Figure 3 shows the typical range of soils densifiable by vibro-replacement and range of soils with potential for liquefaction. The auger-casing system involves little or no significant densification. In general, the lack of

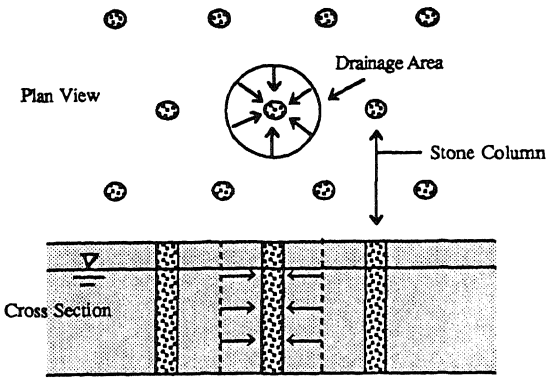

Figure 1. Stone Column Arrangement

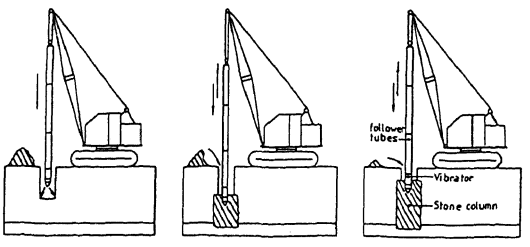

Figure 2. Vibro-Replacement Stone Column Installation

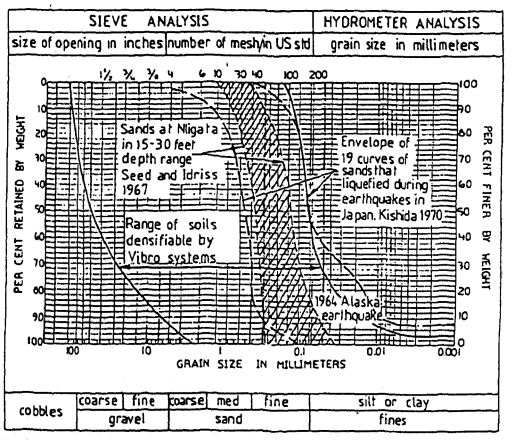

Figure 3. Range of Soils Treatable by Vibro-Replacement and Range of Liquefaction

densification implies relying on drainage capabilities only.

The process of controlled-induced liquefaction by vibro-replacement was studied in a recent full scale field experiment conducted by the authors involving the continuous electronic monitoring of pore water pressures and ground particle velocities. It was observed by the authors that densification occurs because the vibrations emitted trigger pore pressure generation until controlled soil liquefaction is achieved; then, dissipation of the excess pore pressures lead to closer packing of the soil system. Further densification occurs because the stone is displaced into the adjacent soil leading to a cavity expansion effect.

3 PARAMETERS INVOLVED IN DESIGN CONSIDERATIONS

Parameters affecting the stone column performance (earthquake induced pore pressures and settlements) include ground motion characteristics such as peak acceleration, frequency, duration, and the properties of the soil stone column system. These involve degree of densification, shear moduli, compressibility, permeability, and the geometry of the system. Current liquefaction mitigation design approaches in the U.S. consider an increase in soil density only; the ability of the stone column to act as a drain and the stiffness of the stone column are not usually accounted for and their effect are taken only as additional benefits. However, in Japan stone columns that are installed without densification are designed to act as pore pressure dissipation sinks in the event of an earthquake. The following is an overview of soil-stone column parameters affecting stone column response

3.1 Soil density

It is well understood that under cyclic loading, pore pressure generation in a dense soil occurs more slowly than in a loose sand and hence liquefaction potential will be less. For loose sands once the state of initial liquefaction is reached, large ground deformations may occur. However, in dense sands, in the event that peak pore pressure values become equal to the initial confining pressure, the larger shear strains mobilize significant dilation of the sand structure thereby maintaining significant residual stiffness and strength (U.S.NRC, 1985)

3.2 Coefficient of permeability

Saito et al. (1987) report a number of permeability tests for different gravel sizes as a function of the hydraulic gradient. Test results are shown in Figure 4. Regardless of gravel size, their tests indicate that as gradients increase the permeability coefficient decreases, but as the hydraulic gradient is increased above 0.2 the coefficient of permeability shows little change. Note that data is reported for maximum hydraulic gradients of 0.3 and hydraulic gradients at the point of liquefaction may be higher than this.

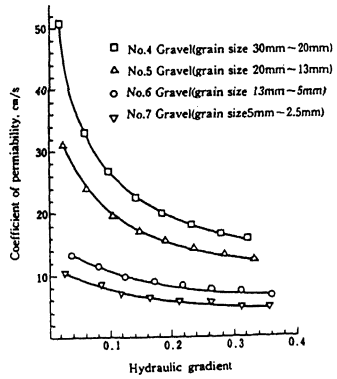

Figure 4. Relationship between coefficient of permeability and hydraulic gradient (after Saito et al., 1987)

In order to avoid significant generation of pore water pressures within the stone column, Seed and Booker (1976) specify that the permeability of the stone column should be at least two orders of magnitude larger than the surrounding soil. On the other hand, Yoshimi and Tokimatsu (1991) postulate that the probable field situation results in some excess pore pressures developing within the stone column as shown in Figure 5.

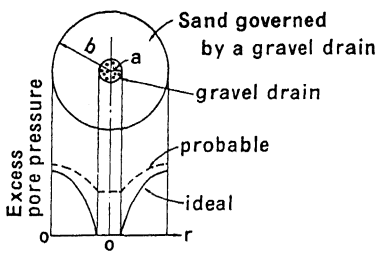

Figure 5. Schematic curves showing distribution of excess pore pressures in and around a gravel drain (after Yoshimi and Tokimatsu, 1991)

3.3 Coefficient of volume compressibility

Figure 6 shows results of an experiment performed by Saito et al. (1987) indicating the relationship between the coefficient of volume compressibility and the pore pressure ratio (excess pore pressure to effective stress). It is striking to notice how this coefficient remains almost constant up to a pore pressure ratio of approximately 0.6, but as the pore pressure ratio increases beyond 0.6, the compressibility increases dramatically. Notice also, the nonlinearity of the relationship. Other authors such as Tokimatsu (1979), and Iai (1988), have reported that significant settlements begin when the pore pressure ratio rises above one half to 0.6. Recent field experiments conducted by the authors revealed that when pore pressure ratios did not exceed 0.7 there was little noticeable volume change (no improvement in densification).

One of the purposes of the stone columns is to reduce the rate of increase in pore water pressures and to increase the rate of dissipation in pore water pressures. If the stone columns can be designed so that maximum pore pressure ratios are maintained below 0.5, the use of a constant coefficient of volume compressibility would be appropriate for dissipation analyses and the potential for settlement would also be reduced.

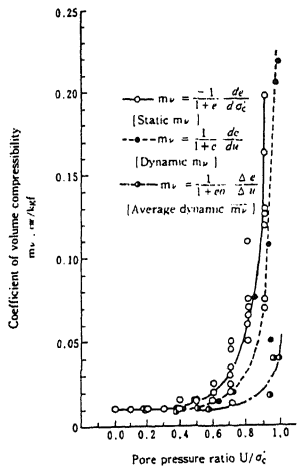

Figure 6. Relationship between coefficient of compressibility and pore pressure ratio (after Saito et al., 1987)

3.4 Selection of gravel material

As previously mentioned, there is a likelihood that hydraulic gradients may exceed critical gradients (greater than one). This may create a problem of soil erosion which could lead to the propagation of cavities within the soil structure and potentially undesirable volume change. In retrospect, it is unlikely that due to the duration of the strong motion much soil material could be carried into the stone column. Nevertheless, it may be fairly important that the gravel drain material be selected so it allows passage of water to relief the pore pressures and prevents piping.

The U.S Bureau of Reclamation (1974), has set standards for filters used in road and embankment construction. Gradation requirements based on particle size passing No 15 & 50 sieves in a grain size analysis are combined to obtain a suitable filter that prevents pore pressure build-up and erosion.

Saito et al. (1987) report a similar principle but suggests a different equation for the selection of the stone column material. Based on experimental data they propose a formula based on the grain size distribution of the stone column and the surrounding soil to ensure

maximum permeability and prevent erosion of the soil.

$$20D_{S15} < D_{G15} < 9D_{S85} \quad (1)$$

where D_{S15} is diameter (mm) of soil passing 15%, D_{G15} is the diameter of gravel (stone) passing 15%; and D_{S85} is the diameter of soil passing 85% in a grain size analysis test.

4 RESPONSE OF STONE COLUMNS-SOIL SYSTEM TO STRONG GROUND MOTION

In a recent study Mitchell and Wentz (1991) evaluated the performance of 12 ground improved sites that were subjected to the 1989 Loma Prieta earthquake. They found that there was no distress or damage due to ground shaking to either the improved ground or to the structures and facilities built upon it. To better understand prediction of pore pressure generation performance, theoretical models have been tested against experimental models on large shaking tables.

4.1 Experimental models

The work by Sasaki and Taniguchi (1982), and Iai (1988) using large shaking table tests on stone columns/walls are summarized and discussed below.

Sasaki and Taniguchi (1982), used saturated sand in a rectangular box mounted on a shaking table to study the effects of pore pressure generation and dissipation in and around gravel drains. The size of the shaking table was 12m x 12m and the size of the models studied was 12 m length, 3 m in depth, and 2 m in width. Cyclic laboratory tests and finite element models were also used to predict the same generation and dissipation of pore pressures during the cyclic loadings.

The gravel drains utilized were of wall type, instead of pile type. Also, two types of gravel were used. Figure 7 shows the models studied and a cross section of the gravel drain. The input motion at the table involved an acceleration loading of 200 gal (0.2 g), and a sinusoidal input of 60 seconds at 5 Hz. To monitor the bed of sand and gravel, 17 accelerometers and 35 pore pressure meters were installed.The average relative density of six tests was 50%.

Tests showed that soil located more than 50 cm from the center of the drain did not benefit from the presence of the drain as pore water pressures were the same as those for soils 2 and 3 meters away. Whereas none of the cases studied indicated that gravel drains prevented liquefaction outside a radius of 50 cm (column diameter was 40 cm), a significant difference was observed following the end of excitation where the drains allowed a faster dissipation of pore pressures. This finding indicates the limited area influenced by the gravel column and, as such, careful consideration of this aspect must be taken into account to properly design the gravel column system. Sasaki and Taniguchi believed that the reason for this effect was the high frequency of the excitation which overcame the drainage properties of the system.

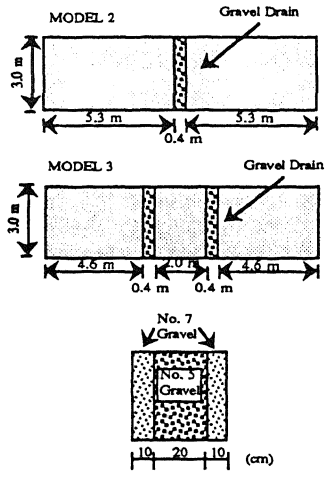

Figure 7. Models used in shaking table tests and cross section of gravel drain (after Sasaki and Taniguchi, 1982)

Iai (1988), evaluated a bed of sand and stone column inside a 2 m diameter container made up of 64 aluminum rings stacked up to a height of 2 meters and mounted on a shaking table (Figure 8). The tests were conducted under sinusoidal as well as earthquake input motions. Test results were analyzed using a model of consolidation theory with an additional term for the development and dissipation of pore pressures. The average relative density of the soil was 33%.

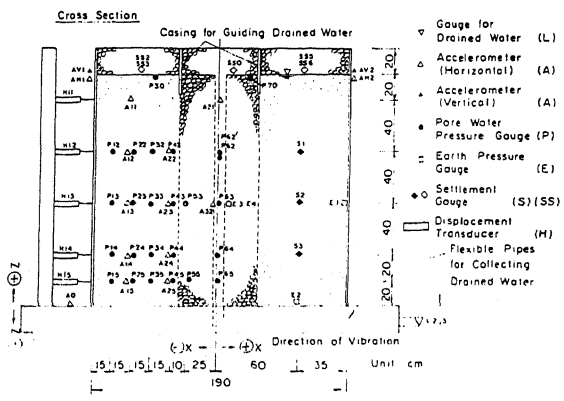

Figure 8. Model and instrument location used in circular shaking table tests (after Iai, 1988)

It was found that shaking induced excess pore water

pressures at the sand deposit dissipate according to linear consolidation theory if the maximum excess pore water pressures attained during the shaking were less than about half that of the initial vertical effective stress. Once the maximum excess pore pressure exceeded this limit, the rate of dissipation became slower as the maximum excess pore pressure became greater. The effect was even more apparent once the maximum excess pore pressure reached the initial vertical effective stress; here, the rate of dissipation became drastically low reflecting high coefficients of compressibility at low confining stress.

Iai also observed that settlements were significant even with the drains installed. As the experiments were conducted in relatively loose sand, this implies that a lack of densification of the soil between gravel columns is responsible for volume changes of the soil system.

4.2 Analytical models

The direct application of a theoretical method (Seed and Booker, 1976) and the results of an evaluation on stone column performance by a finite element method (Millea, 1990) to predict drainage properties and stone column response to strong ground motion are briefly presented and discussed here.

4.2.1 Seed and Booker method (1976)

This theory assumes that the permeability of the stone column is infinite so that no excess pore pressures are developed at this boundary. Linear consolidation theory is used to evaluate dissipation effects for purely radial drainage. For these conditions, the equation of pore pressure development as influenced by a number of shear stress cycles over a period of time, is given as:

$$\frac{K_H}{\gamma_w m_{v3}}\left(\frac{\partial^2 u}{\partial r^2}+\frac{1}{r}\frac{\partial u}{\partial r}\right)=\frac{\partial u}{\partial t}-\frac{\partial u_g}{\partial N}\frac{\partial N}{\partial t} \qquad (2)$$

where u is the excess hydrostatic pore water pressure, k_H is the horizontal coefficient of permeability, m_{v3} is the coefficient of volume compressibility, r is the radius, t is time, and N is the number of cycles contributing to the pore pressure increase. The equation is further handled, for practical purposes, in terms of number of cycles to liquefaction and number of equivalent cycles for a given earthquake magnitude. Note that equation (2) uses a constant for the coefficient of volume compressibility m_v. As discussed before, a constant m_v may be appropriate for situations where excess pore pressures ratios do not exceed 0.5. Thus, excess pore pressure ratio results above this number should be viewed with caution. Densification of the soil is taken into account by the specification of soil liquefaction resistance and by relating the coefficient of volume compressibility (elasticity assumed) to a corresponding penetration resistance value such as SPT (Standard Penetration Test) or CPT (Cone Penetration Test).

Seed and Booker, utilized a finite element code, LARF (Liquefaction Analysis for Radial Flow), to solve equation (2) and summarize the results in practical graph format. Figure 9 shows an example illustrating the use of the drainage design criteria.

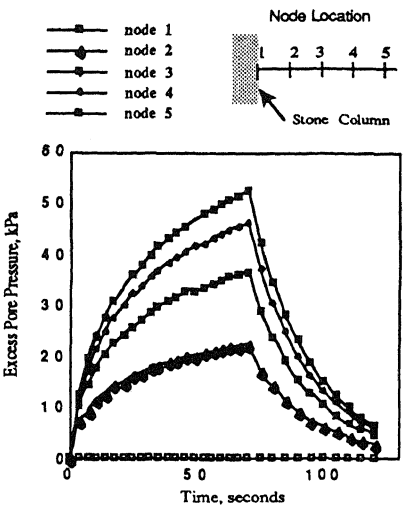

Figure 9. Use of drainage design criteria

Sasaki and Taniguchi (1982) implemented a modified version of Seed and Booker's model by adjusting it for 2 dimensional flow and incorporating it in the finite element code SADAP. Comparison with measured excess pore pressure values in the shaking table experiment described before revealed that Seed's model predicted an accumulation of pore pressure occurring more rapidly (see Figure 10). The researchers attributed the difference of results, in part, to a lack of full saturation in the sand tested.

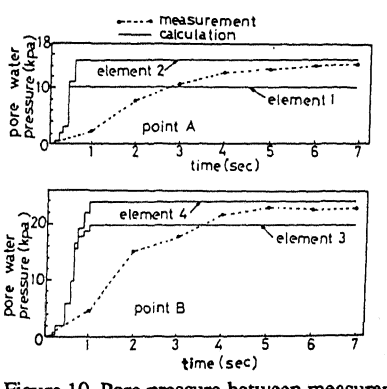

Figure 10. Pore pressure between measurement and Seed model (after Sasaki and Taniguchi, 1982)

4.2.2 Millea (1990)

This method is based on constitutive models developed by Prevost (1985) and implemented in the finite element program DYNAFLOW (NCEL, 1988). Features of the model include the ability to analyze transient phenomena in fluid saturated porous media by means of a two-phase system consisting of a solid and a fluid phase. The solid skeleton may be linear, or nonlinear and hysteretic. Large deformations may also be included. The model may be used to predict the generation of excess pore water pressures in saturated sand deposits.

The investigatory program involved using DYNAFLOW as a tool to determine if the stone column mitigation technique reduces the risk of liquefaction. DYNAFLOW was calibrated by means of a centrifuge test of a brass footing on a saturated sand deposit (Lambe, 1981). Leighton-Buzzard 120/200 sand was used as the model soil for the investigation.

Two models were considered: first, a comparison between a homogeneous mass and a mass with a stone column; second, a comparison of a footing on a homogeneous soil mass and a footing on a soil mass with four stone columns as shown in Figure 11.

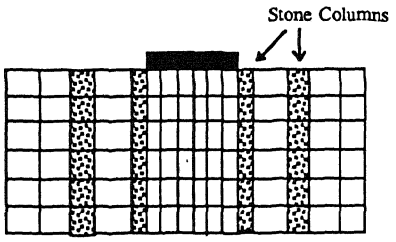

Figure 11. Footing on soil with stone columns

The parametric studies involved varying the stone column permeability, stone column depth, and load-time application time steps. Horizontal and vertical effective stresses, shear stresses, and pore pressures were recorded as well as nodal displacements, velocities, and accelerations.

Results for the soil without the footing indicated that elements inside the stone column experienced a rise in effective stress with negative pore pressure being developed (this was also observed by Sasaki and Taniguchi, 1982). The addition of the stone columns successfully re-distributed effective stresses to 1 diameter away from the stone column only. All other elements remained unchanged.

Results of the soil with the footing on the saturated bed sand indicated that the influence of the stone column was to affect elements two stone column diameters away. The analysis during shaking indicates a re-distribution of the load towards the stone columns. There is a slight generation of pore pressures within the stone columns in the early stages of dynamic loading but it is reduced later on and even becomes negative.

Millea concludes that the stone columns provided a successful reduction of excess pore pressure under the brass footing. Furthermore, the existence of load shifting to the stone columns during the cyclic motion was observed.

5 SUMMARY AND CONCLUSIONS

This paper presents and evaluates current knowledge which quantifies the relative effectiveness of stone column in the mitigation of soil liquefaction during earthquakes. Also, an overview of installation techniques is briefly discussed. The vibro-replacement technique relies on densification between stone columns with the drain and the inclusion of the stiffer stone column acting as additional benefits, while the auger-casing technique relies on the drainage capabilities of the stone column alone.

The importance of soil densification around the stone columns is emphasized and the diminished effectiveness from lack of densification is demonstrated. Pore pressures will tend to rise quicker in a loose soil condition and thus the control of pore pressures relies entirely in the ability of the stone column to act as a drain. Some researchers fear that quick and strong earthquakes may exceed the stone column drainage capabilities thereby rendering it ineffective. However, if soil densification is included these potential problems are greatly minimized.

The limitations on the use of simplified analytical methods such as Seed and Booker's (1976) are noted. Experimental data by Iai (1988) demonstrates that the linear mechanisms of consolidation are valid only if the pore pressure ratio remains below 0.5. Other researchers (Saito et al.,1987) have demonstrated the high degree of non-linearity for coefficients of compressibility and permeability if pore pressure ratios are in excess of 0.5. The experimental work by Sasaki and Taniguchi (1982), demonstrates that pore pressures within the gravel drain do vary, contrary to the assumption that they remain essentially unchanged. In spite of these limitations, the Seed and Booker model is a useful model of drainage capabilities of a real stone column-soil system if allowable maximum pore pressure ratios are maintained below 0.5. In addition, this threshold seems desirable in order to limit potential damaging settlements in the soil system.

It is clear that the most appropriate and safe way to mitigate earthquake induced potential liquefaction remains in densifying the soil. Nevertheless, research to date suggests that additional benefits resulting from the drainage capacity of the stone column should be included in the design considerations provided certain limitations are accounted for.

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