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Site Improvements with Stone Columns in Stratifield Silty Soils

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SYNOPSIS: A case history is presented where stone columns were used as a deep compaction method to increase the liquefaction resistance of stratified silty soils. Standard penetration test (SPT) and cone penetration test (CPT) resistances were used to evaluate the pre-treatment site conditions and post-treatment effects of deep compaction using stone columns. The results of the deep compaction are presented with predicted penetration resistances required to reduce the potential for liquefaction. Limitations of conventional liquefaction analysis in silty soils are discussed with regard to SPT-CPT correlations established for the site, cyclic simple shear tests performed on silts, and corrections to SPT penetration resistances for fines content.

INTRODUCTION

The use of in-situ testing techniques consisting of the standard penetration test (SPT) and the cone penetration test (CPT) has become routine practice to evaluate the liquefaction potential of soils. The SPT and CPT can typically be carried out as part of conventional soil investigations allowing for a large number of tests to be performed economically. Liquefaction evaluations are most often performed using Seed's method (Seed and Idriss 1983, Seed et al. 1983, Seed and De Alba 1986, Seed et al. 1985), which correlates SPT blow counts to the liquefaction resistance of sandy soils.

The correlations presented in Seed's method are based on a large data base of averaged SPT blow counts obtained from sites that did and did not liquefy during earthquakes. The evaluation of liquefaction resistance using SPT data is straight forward for sandy sites, but can be complex for sites with complicated soil stratigraphy and/or sites containing soils with high silt contents. A case history is presented where the evaluation of liquefaction potential for an elementary school site involved consideration of complex soil stratigraphy and maximum credible earthquake (MCE) ground accelerations.

BACKGROUND

The project consisted of an approximately 50,000-square-foot single-story school building, in Oxnard, California. The project was constructed during the period of 1990 to 1992. The building was constructed with a conventional spread footing foundation and slab-on-grade floor system.

The surficial sediments of the site have been mapped as alluvium consisting of deltaic deposits (Weber 1973). The alluvium has an estimated thickness of approximately 300 feet. Underlying the surficial sediments of the Oxnard Plain is a thick section of marine and non-marine sediments reaching a maximum thickness of approximately 20,000 feet (Dibblee 1988).

The design-level field investigation for the project generally consisted of a program of borings and CPT's to explore the soils within the upper approximately 40 feet. Three of the borings were drilled using mud rotary techniques, and one of the borings was drilled using a hollow stem auger. SPT's and modified California samples (penetration tests performed using a 3-inch diameter split spoon sampler) were taken at selected intervals in the borings. Four CPT profiles were performed using an electric cone penetrometer with a diameter of 1.72 inches. Cone penetration resistance (q_c) and sleeve resistance (f_s) were monitored continuously during penetration and were recorded using computer automated data acquisition systems. Field exploration generally revealed that the soils consisted of interbedded layers of sand, silt, clay, and gravel.

Groundwater conditions below the site consisted of various perched groundwater levels. The investigation for the site was performed during California's 5-year drought period that ended in March of 1992. Perched groundwater levels dropped from approximately 12 feet below the ground surface in June of 1988 to approximately 26 feet below the ground surface in January 1990. Based on regional data, an assumed high groundwater level of 9 feet below the ground surface was used for the liquefaction analysis.

LIQUEFACTION EVALUATION

Liquefaction analysis was performed using Seed's method (Seed and Idriss 1983, Seed et al. 1983, Seed et al. 1985, Seed and De Alba 1986). The method is based on empirical correlations between averaged SPT N-values obtained from sites that did and did not liquefy during earthquakes. Induced cyclic stress ratios occurring during earthquakes are calculated from peak ground accelerations estimated for the site using Equation 1.

$$\frac{\tau_{avg}}{\sigma'_{vo}} = 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_{vo}}{\sigma'_{vo}} \qquad [1]$$

The critical cyclic stress ratio needed to resist liquefaction is based on graphical correlations to the SPT N-value. Seed's correlations assume an average operating efficiency for the SPT of 60 percent. The values are then corrected for fines content, length of the drill string, and overburden stress. The corrected N-value is referred to by Seed as N'_{60}. Generally higher SPT resistance is needed to resist liquefaction with increasing earthquake magnitude, increasing ground acceleration, and decreasing fines content. Seed's correlation between the critical stress ratio and N'_{60} for a magnitude 7.5 earthquake is presented in Figure 1. The fines content is defined for the purposes of the analysis as the silt and clay sizes passing the No. 200 sieve.

The State of California Title 24, modified from the Uniform Building Code, requires that the liquefaction analysis for the site be based on the estimated maximum credible earthquake (MCE). The MCE is the maximum earthquake that could impact the site under the known tectonic framework. Under Title 24, school buildings within California must be able to withstand the MCE without collapse. Liquefaction evaluation of foundation support soils that could potentially result in collapse of a structure must be evaluated using the design MCE.

Based on seismic analysis, the MCE for the site was estimated to be a magnitude 7.5 earthquake on the Oak Ridge Fault. The Oak Ridge Fault is mapped approximately 2.2 miles north of the site. Based on attenuation relationships by Cambell (1987), Sadigh et al. (1987) and Joyner and Boore (1982), the M7.5 earthquake was estimated to be capable of generating a peak ground acceleration at the site of approximately 0.65g. Equation 1 reduces the peak ground acceleration (a_{max}) to the estimated repeatable high ground acceleration (rhga) using a factor of 0.65 to convert the a_{max} to rhga.

The liquefaction analysis for the site identified potentially liquefiable soil conditions within the upper approximately 20 feet. Soils encountered below 20 feet were generally found to consist of hard clays and dense sands having a low potential for liquefaction. For the purpose of analysis, the soils in the upper 20 feet were divided into the four soil layers presented in Table I. The upper 20 feet of soil generally ranges

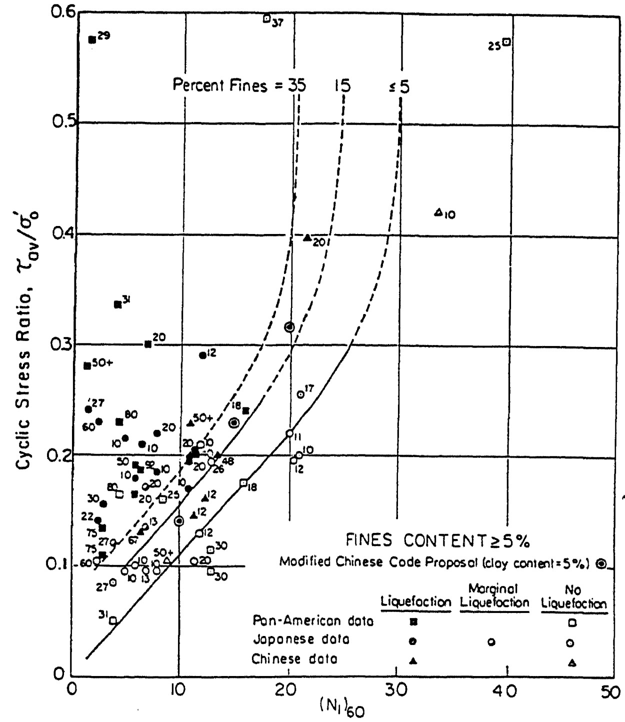

Figure 1. Relationships between stress ratio and N-values for silty sands for M = 7-1/2 earthquakes (after Seed and Alba 1986).

TABLE I. Soil Layers to Depth of 20 feet

Soil Layer	Typical Depths (feet)	Soil Types (USCS)	Fines (%)	Avg. N'_{60} (bpf)
Upper Alluvium	0 to 9	ML SM	28 to 65	12.2
Sandy Alluvium	9 to 13	SP SM	10 to 45	21.8
Silty Alluvium	13 to 17	ML	66 to 91	11.3[1]
Clayey Alluvium	17 to 20	CL-ML	77 to 89	11.3[1]

[1]N'_{60} not differentiated between silty and clayey alluvium.

from silty to relatively clean sands, and from non-plastic silts to silty clay.

Based on the results of the analysis, deep compaction was recommended as a method that could be used to reduce the potential for liquefaction in the sandy and silty alluvium layers identified in Table I. A performance specification was prepared for the project that would: 1) permit the contractor to select the method of deep compaction; and 2) require the contractor to provide a factor of safety of at

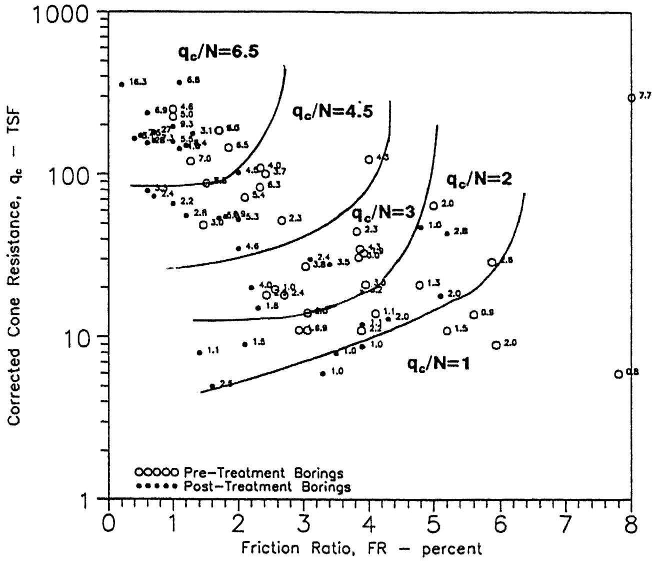

Figure 2. Relationships between FR and q_c to N for site.

least 1.3 against liquefaction. Based on the specified minimum factor of safety, the contractor would be required to provide a minimum SPT N'_{60} of at least 20 to 24 blows per foot in the sandy alluvium and silty alluvium layers.

DEEP COMPACTION WITH STONE COLUMNS

The contractor chose to install stone columns by the vibro-replacement method at 9-foot centers in a triangular grid spacing to provide the specified deep compaction. As a result of the lower groundwater conditions encountered at the time of the construction, the contractor pre-drilled the stone column locations using a 2-foot diameter helical auger. Gravel consisting of 1-1/2-inch river run and 3/4-inch crushed stone was then placed in the pre-drilled holes in 2 to 3-foot lifts. Each lift was compacted using a Model 480S vibrator. During compaction the vibrator typically reached frequencies of between 250 and 300 amps. 851 stone columns were installed at the site with an average depth of approximately 20 feet.

Deep compaction was monitored using a combination of CPT's and SPT's performed at approximately the center of the grid between selected columns. For monitoring deep compaction, CPT penetration resistances (q_c) were converted to SPT N-values using the on-site correlation presented in Figure 2. Figure 2 shows isocrones fitted to ratios of q_c/N values for adjacent CPT and SPT profiles. Pre-treatment and post-treatment borings were used to develop the correlation.

CPT results were obtained during initial deep compaction. Overall, the results of the CPT's indicated that improvement was being provided

TABLE II. Summary of SPT Results

Soil Layer	N' Before (bpf)	N' After (bpf)	Change (%)
Upper Alluvium	12.2	24.3	+99
Sandy Alluvium	21.8	32.5	+49
Silty and Clayey Alluvium	11.8	11.3	~0

TABLE III. Summary of CPT Results

Soil Layer	q'_c Before (tsf)	q'_c After (tsf)	Change (%)
Upper Alluvium	110	141	+28
Sandy Alluvium	153	213	+39
Silty Alluvium	25	48	+88
Clayey Alluvium	7.1	12	+69

to both the sandy and silty alluvium, although the improvement did not meet the penetration resistance requirements of the project specifications in the silty alluvium. Based on the results, the contractor prepared stone column test sections with spacings of 8 feet and 4.5 feet on center. CPT's were performed within the center of the test sections, and at various distances from the edge of selected stone columns. In general, the CPT's indicated that no significant improvement was being provided in the silty alluvium as a result of the closer stone column spacing. It was therefore recommended that the contractor complete the deep compaction with the original 9-foot spacing. A summary of SPT and CPT results prior to and after the deep compaction is summarized in Tables II and III. Post-treatment and pre-treatment CPT profiles showing the relative increase in q_c are presented in Figure 3.

The contractor had completed the work ahead of schedule, and an approximately one-month period was provided by the owner's representative to evaluate the deep compaction and collect soil samples for cyclic simple shear testing. Two Shelby tube samples obtained from the silty alluvium layer were tested at a cyclic stress

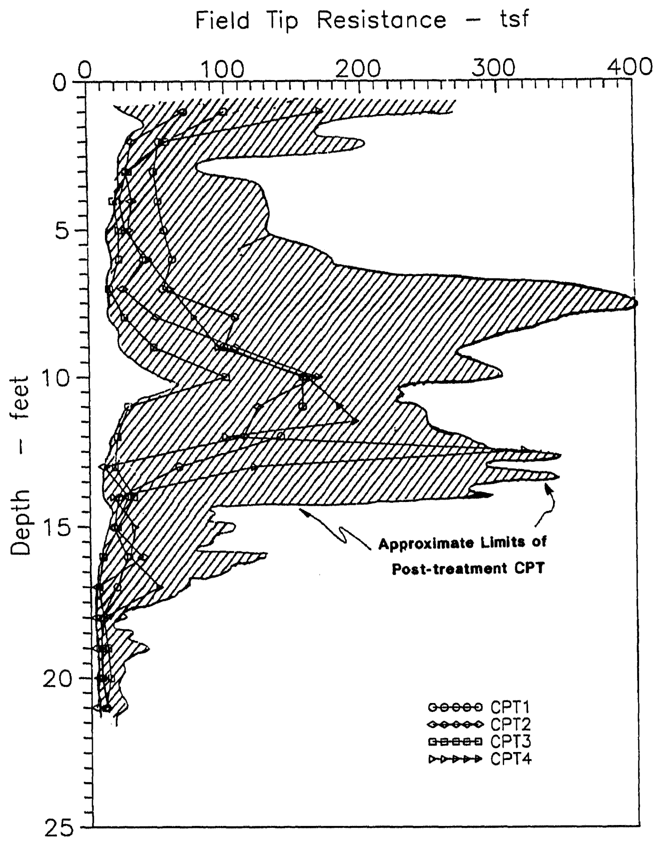

Figure 3. Relative improvement in q_c as a Result of Deep Compaction.

ratio of approximately 0.5, equivalent to the estimated induced cyclic stress ratio for the MCE. Under a cyclic simple shear stress ratio of 0.5, one sample exhibited porewater pressures exceeding the confining stress at 10 cycles of loading and the second sample exhibited porewater pressures exceeded the confining stress at 70 cycles of loading. The sample that liquefied after 10 cycles of loading maintained nearly elastic cyclic shear strains to approximately 50 cycles of loading. Based on Seed (1983), approximately 15 cycles of loading can be expected for a magnitude 7.5 earthquake. It was therefore concluded that the post-treated silty alluvium was not susceptable to liquefaction under the design MCE.

EVALUATION OF POST-TREATMENT LIQUEFACTION POTENTIAL

Evaluation of the liquefaction potential following deep compaction with stone columns considered: 1) redistribution of overburden stress through load transfer to stone columns; 2) increased relative density as a result of deep compaction; 3) confinement of potentially liquefiable soil layers by denser soils having a low potential for liquefaction; 4) reduction in cyclic porewater pressures as a result of drainage through stone columns; and 5) reinforcement, or increased rigidity, of the

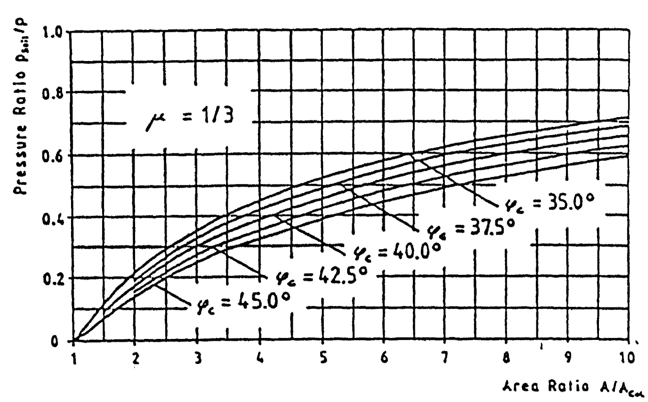

Figure 4. Reduction of total overburden pressure of soils treated by vibro placement (after Priebe 1989).

compacted soils as a result of the stone columns placement.

Transfer of overburden stress to stone columns has been considered in settlement analysis for sites treated with vibro-replacment (Preibe 1978). Considering the increase in stiffness at stone column locations, Preibe was able to model stone columns as discrete, elastic elements capable of reducing settlements by transferring overburden stresses to the columns through soil arching phenomena. Preibe (1989) has expanded the theory of transfer of overburden stresses to stone columns for considering liquefaction resistance of deeply compacted soils. The transfer of overburden stress to the stone columns is related to the area ratio, A/Acol (see Figure 4). For A/Acol up to 10, at least 70 percent of the overburden stress can be expected to be transferred to stone columns as a result of vibro-replacement.

As N'_{60} is corrected for overburden stress, the transfer in overburden stress to stone columns must be considered in evaluating post-treatment penetration resistances. Seed's correction for overburden stress (tsf) is approximated by Equation 2.

$$C_n = (1/\sigma'_{vo})^{1/2} \qquad [2]$$

The correction factor increases with lower overburden stress. This is especially true at shallow depths where relatively small increases in overburden stress can result in relatively large differences in C_n. An approximately 20 percent increase in N' was obtained by considering Preibe's reduction in overburden stress. Corrected penetration resistance from SPT and CPT are presented in Figure 5 and Figure 6, respectively.

Figures 5 and 6 do not reflect corrections in penetration resistance for fines content greater than 35 percent. Seed's method is generally limited to sandy soils. The method establishes guidelines for evaluation if cohesive soils are susceptible to liquefaction based on plasticity and colloid content. Seed's method does not provide for evaluating

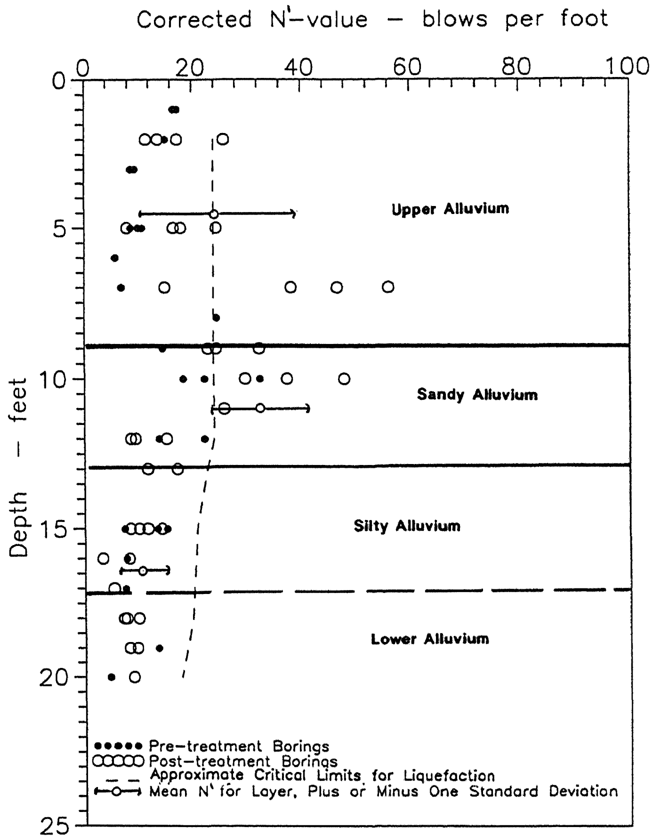

Figure 5. Average increase in N' as a Result of Deep Compaction.

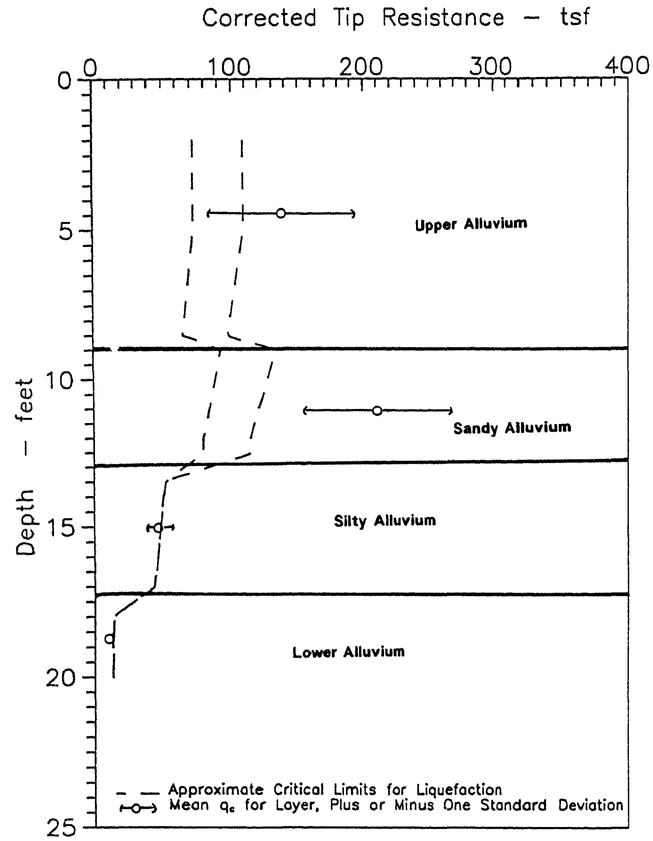

Figure 6. Average increase in q_c as a Result of Deep Compaction.

the liquefaction potential of fine grained soils that have been identified as being susceptible to liquefaction.

For non-plastic silts, by extrapolation of Seed's critical blow count needed to resist liquefaction at high cyclic stress ratios versus fines content, it can be estimated that an N'_{60} value of approximately 12 blows per foot would be needed to resist liquefaction in silt (greater than 75 percent material passing the No. 200 sieve). These results are generally consistent with the simple shear test data for silt samples obtained at the site, which indicated that the silty alluvium has a low potential for liquefaction under the MCE. As simple shear test data was limited for the site, and the correlation between N'_{60} and fines content for silts could not be documented from the literature, the fines correction was not considered in the post-treatment evaluation of liquefaction potential.

Confinement of potentially liquefiable soils and reduced cyclic mobility of the soil was considered in evaluating deep compaction of the site. Based on the penetration test data presented in Figures 5 and 6, it was concluded that the silty alluvium could potentially liquefy under MCE cyclic shear stresses. Seed et al. (1983) indicates that although medium dense soils (N'_{60} greater than 15 blows per foot) may liquefy at high cyclic stress ratios, settlement or deformation of such soils would likely be limited. Seed et al. (1983) and Seed (1987) also indicate that medium dense soils would likely not exhibit boiling, and that in the event the soils did liquefy they would likely retain relatively high residual strength. Based on these considerations, it was concluded that the consequences of liquefaction in the silty alluvium would be relatively minor as a result of the site improvements.

Drainage through stone columns was not considered to reduce the potential for liquefaction at the site. Drainage through vertical columns has been described by Seed and Booker (1977). Generally, to reduce the potential for liquefaction, the drains must be able to relieve excess porewater pressures in the soil as they develop during ground shaking. As the soils at the site contained relatively high silt contents, it was generally concluded that porewater would not dissipate to the stone columns during the shaking duration expected for a Magnitude 7.5 event.

Although not quantified for the site, it was generally concluded that there was improvement to the site as a result of the increased rigidity of the treated soils. Similar to the transfer in overburden stress allowed by Preibe (1989), there is also likely to be a transfer of lateral stresses resulting from earthquakes to the stone columns. In general, the increase in cyclic stress is dependent on acceleration of the soil mass, represented by the total overburden stress (Lopez and Hayden 1992). By transferring lateral shear stresses to the more rigid stone columns, the net increase in cyclic shear stresses to the soils between the columns is reduced. The increased rigidity allows for the treated soils to act more as a unit, and therefore helps to reduce cyclic shear stresses during earthquake loading. It was therefore concluded that where penetration resistances may indicate borderline liquefiable soils, that the actual shear stresses used to estimate the factor of safety were likely conservative.

CONCLUSIONS

1. Based on the results of pre-treatment and post-treatment SPT and CPT, the deep compaction was able to provide a factor of safety of at least 1.3 against liquefaction in the sandy alluvium based on Seed's method of liquefaction analysis.

2. Deep compaction was unable to provide sufficient increase in density within the silty alluvium to resist liquefaction. Cyclic simple shear tests performed on samples obtained following deep compaction indicate that the post-treated silty alluvium had a low potential for liquefaction at the design MCE cyclic stress ratios.

3. Deep compaction with stone columns likely provides benefits to resisting liquefaction potential besides increased density. The liquefaction resistance of soils treated by stone columns should consider the following factors:

- Reduction in overburden stress as a result of load transfer to stone columns when evaluating site improvements through penetration testing;
- Reduction in cyclic shear stresses between stone columns as result of increased rigidity in soils treated with vibro-replacement techniques; and
- Consequences of liquefaction with regard to the cyclic mobility of relatively dense soils.

4. Based on comparison of field penetration resistances to cyclic simple shear test results, higher fines corrections than those presently indicated by Seed's method should be used when evaluating the liquefaction potential of silty soils. The data generally indicate that values of N' greater than approximately 12 blows per foot may be sufficient to resist liquefaction in silts. The data obtained during this investigation are not sufficient to provide a basis for estimating fines corrections for silty soils.

5. Additional research is needed to evaluate the reduction in cyclic shear stresses in soils treated by vibro-replacement. Providing a basis for reducing cyclic shear stresses would prove to be a valuable tool in liquefaction analysis of soils that do not readily compact with stone columns (silts and clays).

6. Where improvement of fine grained soil deposits is required to reduce the potential for liquefaction, construction monitoring should incorporate a program of laboratory cyclic shear testing to supplement field penetration tests. Laboratory tests are particularly important for sites where little increase in penetration resistance is likely to occur during vibro-replacement, such as in fine grained soil deposits.

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