(d/L) of stone columns, and the densification effect of stone column treatment could be estimated from the diameter and spacing parameter in a conservative manner.

Procedure of Liquefaction Evaluation and Quality Control

Fig. 3 shows a flowchart for quality control of stone column treated ground based on the CRR-V_s-e correlations mentioned above.

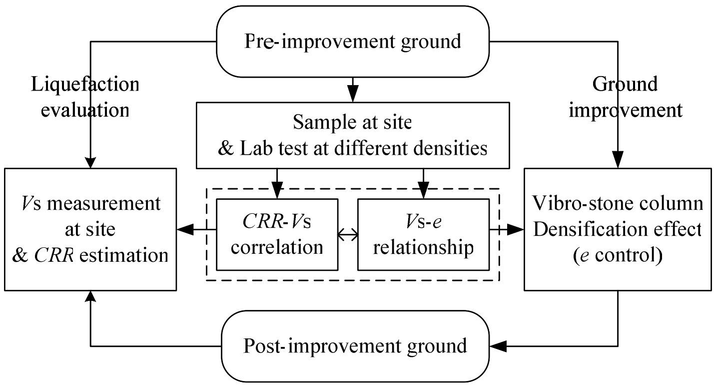

FIG. 3. Flowchart for evaluation of ground improvement by shear wave velocity.

For the site of interest, geophysical methods such as spectral analysis of surface waves (SASW) test is performed to obtain the shear wave velocity (V_{s0}) before treatment, and sample at site for laboratory tests. In laboratory, the CRR-V_s and V_s-e correlations are established via element test with shear wave velocity measurements (e.g., cyclic triaxial apparatus with bender elements) (Zhou et al., 2005). Then the liquefaction resistance before ground improvement is estimated by Eq. 1 with the aid of Table 1. If the soil deposits will liquefy at a given earthquake intensity, stone column treatment is adopted to densify the ground until the required CRR is reached, which means the critical shear wave velocity (V_{scr}) is reached. According to Eq. 2, this requirement in turn defines the limiting void ratio (e_{cr}) for stone column installation, that is

$$e_{cr} = \frac{V_{scr}}{V_{s0}} e_0 + \frac{A}{B}\left(\frac{V_{scr}}{V_{s0}} - 1\right) \tag{5}$$

If the average void ratio after improvement e_1 is larger than e_{cr}, then the ground is assumed not to liquefy and the quality is insured. Replacing e_1 in Eqs. 3 and 4 by e_{cr}, the minimum diameter-to-spacing ratio for square and triangular patterns are

$$\frac{d^2}{L^2} = \frac{4}{\pi}\frac{e_0 - e_{cr}}{1+e_0} \quad \text{and} \quad \frac{d^2}{L^2} = \frac{6}{\sqrt{3}\pi}\frac{e_0 - e_{cr}}{1+e_0} \tag{6}$$

respectively.

After improvement, SASW method is used to measure the shear wave velocity and check whether the required *CRR* is obtained or not.

CASE STUDY

Site Description and Stone Column Installation

The construction site is located in Hangzhou city. The ground water table is 1.5 m below the soil surface. In the depth range from 2 to 16 m the soil profile contains liquefiable silty sands which are underlain by 15-20 m depth of clayey soils. Before ground improvement, an SASW test, laboratory tests and detailed analysis were carried out based on Eqs. 1, 2 and 5, and the critical values concerned in design are listed in Table 2.

Table 2. Soil Profile and Main Indexes

Soil Layer	Soil Type	Depth (m)	*Gs*	e_0	e_{cr}	*n*
1	Reclaimed	0-1.5	2.68	0.85	0.70	0.56
2a	Silt	1.5-4.0	2.66	0.80	0.54	0.48
2b	Silty sand	4.0-8.0	2.66	0.68	0.52	0.51
2c	Silty sand	8.0-9.5	2.65	0.76	0.59	0.53
2d	Silty sand	9.5-12.4	2.65	0.63	0.50	0.53
2e	Silty sand	12.4-13.0	2.66	0.81	0.54	0.49
2f	Silt	13.0-15.6	2.67	0.90	0.68	0.49

The vibro-stone column technique was adopted to densify the upper part of the subsoil, increasing the liquefaction resistance and the bearing capacity. And stone column itself offers a drain path and helps the dissipation of excess pore pressure when earthquake occurs. The depth of improvement is 15 m. In the range of foundation, the stone columns with diameter of 0.8 m were installed at a center-to-center spacing of 1.8 m in a square pattern. While outside this range, stone columns were installed in a triangular pattern. The power rating of the vibratory probe is 30 kW (Chen et al., 1993).

Site Investigation Before and After Improvement

To monitor the quality of ground improvement, shear wave velocity was measured by SASW method before and after improvement. Cross-hole tests were also carried out to check the accuracy the SASW testing at this site. Standard penetration testing (SPT) was performed in parallel for comparison purposes. The field testing arrangements are shown in Fig. 4.

Fig.5 shows the SASW test results before and after ground improvement, and the parallel measurements in the depth from 2 to 7 m before improvement manifested the accuracy of SASW testing compared with the cross-hole testing. As shown in Fig. 5, sandy soils in this area could be effectively densified by vibro-stone columns at a spacing of 1.8 m, especially for relatively loose silty sands (e.g., layer 2c and 2e). Nevertheless, a slight drop of density was observed in a few parts of the subsoil after treatment (e.g., 2b), which is most probably due to the dilation of relatively dense sands caused by improper installation of vibro-stone columns, and will not affect the liquefaction resistance and bearing capacity of subsoil significantly.

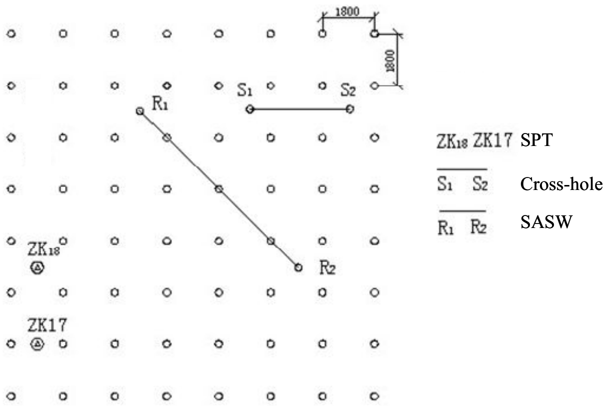

FIG. 4. Layout of seismic wave and SPT tests.

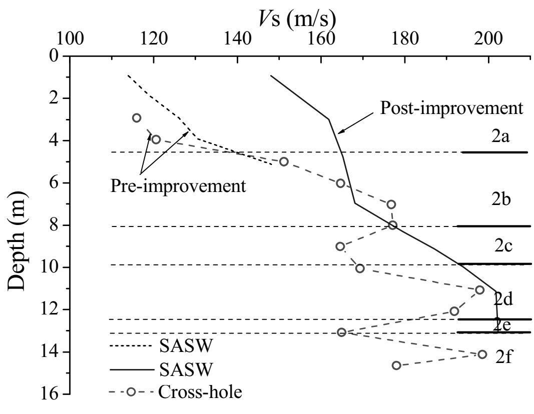

FIG. 5. SASW tests before and after ground improvement.

Liquefaction Evaluations

Fig. 6 shows the liquefaction resistance ratio determined by Eq. 1 before and after ground improvement, and the earthquake-induced *CSR* at peak ground acceleration a_{max} = 0.15g is also plotted. As shown in Fig. 6, soil layers 2c and 2e will liquefy before ground treatment while after treatment they don't. This evaluation result agrees well with that from SPT indexes (see Table 3).

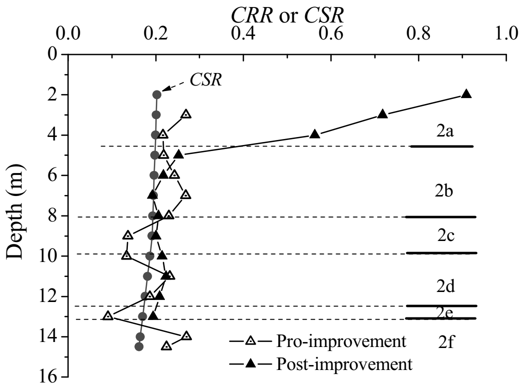

FIG. 6. *CRR* estimation based on shear wave velocities.

Table 3. Liquefaction Evaluation by SPT Indexes

Soil Layer	Critical value, N_{cr}	Pre-improvement		Post-improvement	
		SPT-N value	Liquefaction?	SPT-N value	Liquefaction?
Silty sand (2b)	11.6	19.9	No	24	No
Silty sand (2c)	13.9	4.7	**Yes**	24	No
Silty sand (2d)	15.7	16.8	No	30.2	No
Silty sand (2e)	18.0	12.0	**Yes**	20.1	No

CONCLUSIONS

In this paper, a procedure to evaluate the improvement level in liquefiable soils treated by stone columns was developed based on the liquefaction resistance-shear wave velocity-void ratio correlations of sandy soils. According to this procedure, the required level of ground improvement is supposed to be obtained once the target velocity is reached after improvement for a given earthquake magnitude, and this requirement will transfer to the void ratio control during stone column installation. Well defined *CRR*-V_s and V_s-*e* correlations are proposed, and specific expressions for void ratio control associated with stone column diameter and spacing are given for different patterns of installations.

A case study using vibro-stone column treatment is introduced, where field tests including seismic testing (SASW) and SPT were performed before and after ground improvement, and the effectiveness of stone columns for liquefaction mitigation was properly evaluated by shear wave velocity. The high consistency between V_s-based and SPT-N value-based evaluations indicates that shear wave velocity in conjunction with other soil parameter (e.g., void ratio) could be used to develop criteria for ground improvement needed to mitigate liquefaction.

ACKNOWLEDGMENTS

The work described in this paper was supported by the National Basic Research Program of China (973 Project) (No. 2007CB714203), the China Postdoctoral Science Foundation (No. 20080430219) and the Foundation for Seismological Researches, China Earthquake Administration (No. 200808022). These financial supports are gratefully acknowledged.

REFERENCES

Adalier, K. and Elgamal, A. (2004). "Mitigation of liquefaction and associated ground deformations by stone columns." *Eng. Geol.*, 72(3-4): 275-291.

Andrus, R.D. and Chung, R. (1995). "Ground improvement techniques for liquefaction remediation near existing lifelines." Report, Building and Fire Research Laboratory. National Institute of Standards and Technology, U.S. Department of Commerce.

Andrus, R. D., and Stokoe II, K. H. (2000). "Liquefaction resistance of soils from shear-wave velocity." *J. Geotechnical and Geoenvironmental Engrg.* 126(11): 1015-1025.

Andrus, R.D., Stokoe, K.H.II. and Juang, C.H. (2004). "Guide for shear wave-based liquefaction potential evaluation." *Earthq. Spectra*, EERI, 20(2): 285-308.

Baez, J.I. and Martin, G.R. (1995). "Permeability and shear wave velocity of vibro-replacement stone columns." *Soil Improvement for Earthquake Hazard Mitigation* (GSP 49), ASCE, New York: 66-81.

Baez, J.I. and Martin, G.R. (1992). "Quantitative evaluation of stone column techniques for earthquake liquefaction mitigation." *Proc. 10th World Conf. Earthquake Eng.*, Madrid, Spain, Vol. 3: 1477-1483.

Bryan, G.M. and Stoll, R.F. (1989). "The dynamic shear modulus of marine sediments." *J. Acoust. Soc. Am.*, 83(6): 2159-2164.

Cha, M. and Cho, G.C. (2007). "Shear strength estimation of sandy soils using shear wave velocity." *Geotech. Test. J.*, 30(6): 484-495.

Chen, Y.M., Ke, H. and Chen, R.P. (2005). "Correlation of shear wave velocity with liquefaction resistance based on laboratory tests." *Soil Dyn. Earthquake Eng.*, 25(6): 461-469.

Chen, Y.M, Wu, S.M. and Bian, S.Z. (1993). "Inspection and analysis of the anti-liquefaction improvement effect of vibroflotation stone column." *J. Zhejiang Univ.*,

27(1): 54-60. (in Chinese)
Hardin, B.O. and Drnevich, V.P. (1972). “Shear modulus and damping in soils: Design equations and curves.” *J. Soil Mech. and Found. Div.*, ASCE, 98(7): 667-692.
Kayen, R., Seed, R.B., Moss, R.E., Cetin, O., Tanaka, Y., and Tokimatsu, K. (2004). “Global shear wave velocity database for probabilistic assessment of the initiation of seismic-soil liquefaction.” *Proc., 11th Int. Conf. On Soil Dynamics and Earthquake Engrg.*, 2, Berkeley, CA: 506-512.
Mitchell, J.K., Baxter, C.D.P and Munson, T.C. (1995). “Performance of improved ground during earthquakes.” *Soil Improvement for Earthquake Hazard Mitigation* (GSP 49), ASCE, San Diego, CA: 1-36.
Shenthan, T. Nashed, R., Thevanayagam, S. and Martin, G.R. (2004). “Liquefaction mitigation in silty soils using composite stone columns and dynamic compaction.” *Earthquake Eng. & Eng. Vib.*, 3(1): 39-50.
Zhou, Y.G. and Chen, Y.M. (2007). “Laboratory investigation on assessing liquefaction resistance of sandy soils by shear wave velocity.” *J. Geotechnical and Geoenvironmental Engrg.* 133(8): 959-972.
Zhou, Y.G. and Chen, Y.M. (2005). “Influence of seismic cyclic loading history on small strain shear modulus of saturated sands.” *Soil Dyn. Earthquake Eng.*, 25(5): 341-353.

Effectiveness of Stone Columns for Liquefaction Mitigation of Silty Sands With and Without Wick Drains

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ABSTRACT: Pre-fabricated vertical (wick) drains have been used in conjunction with stone columns to improve treatment effectiveness in sands with high fines content. However, no comparison testing has been performed with and without drains. In this study, side by side comparisons demonstrated improved performance of stone columns with drains relative to that without drains. Despite the presence of wick drains, improvement effectiveness still decreased as the fines content increased. The average increase in SPT $(N_1)_{60}$ was 114% in a zone with an average of 31% fines, but decreased to about 70% in a zone with an average fines content of 43%. Treatment effectiveness was often minimal in layers containing 15% or more clay sized particles. Significant increases in SPT values were observed with time after treatment in the test areas with drains, but little improvement with time was observed in areas without drains. Increased penetration resistance was similar to that observed at other stone column projects where similar soils were encountered and wick drains were also used.

INTRODUCTION

Stone column treatment has become a very common method for mitigating liquefaction hazards. Although this approach has proven effective in densifying clean sands, the effectiveness typically decreases substantially as the fines content increases above 20% (Mitchell, 1981). Higher fines content tends to decrease the soil permeability and strengthen the soil structure, both of which reduce compaction efficiency. To improve the efficiency of stone column treatment in sands with high fines content, pre-fabricated vertical drains (wick drains) have been employed along with stone columns at relatively high replacement ratios (≈ 25%) (Rollins et al. 2006, Leurhing, 2002). While reasonable improvement has been achieved, there is some question whether this is a result of improved drainage provided by the drains or simply the high replacement ratio. Unfortunately, at sites where wick drains have been employed, comparison tests have not been performed without drains to determine how much of the improvement was associated with the drains. As a result, some uncertainty about the efficacy of the method remains. To evaluate the influence of wick drains on stone column effectiveness in soils with high fines contents, tests were conducted with and without wick drains. This paper describes the soil conditions, the treatment approach, and the improvement achieved by stone column treatment with and without drains.

GEOTECHNICAL AND SEISMIC CONDITIONS

The site was located at the 24th street overpass on Interstate 15 in Ogden, Utah. This site is located within a few kilometers of the Wasatch fault which is capable of producing a M7.4 earthquake. The peak ground acceleration with a 2% probability of being exceeded in 50 years (~2500 yr recurrence interval) was estimated to be 0.57 g. In the upper 12 m of the soil profile, the soil is composed of loose to medium dense silty sands and sandy silts susceptible to liquefaction and mitigation was required. Pre-treatment test borings were initially made at each of four test areas with nearly continuous sampling to define the soil conditions. The soil profiles were quite similar across the site. A generalized soil profile at the site is provided in Figure 1 along with profiles of the fines content and clay content. To a depth of 4 m the silty sand has an average fines content of 26% and a clay content of 6%. Below this depth the average fines and clay contents increased, along with the variability in these values, owing to the interbedded nature of the soil at depth. Between 6 and 12 m below the ground

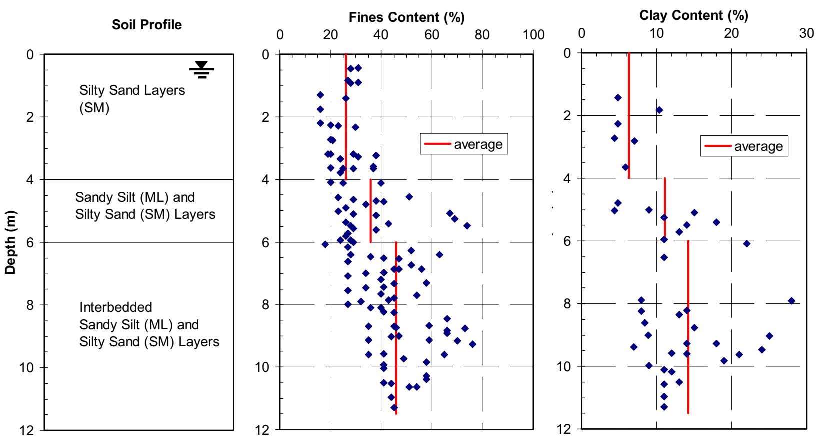

FIG. 1. Idealized soil profile along with fines content and clay content profiles.

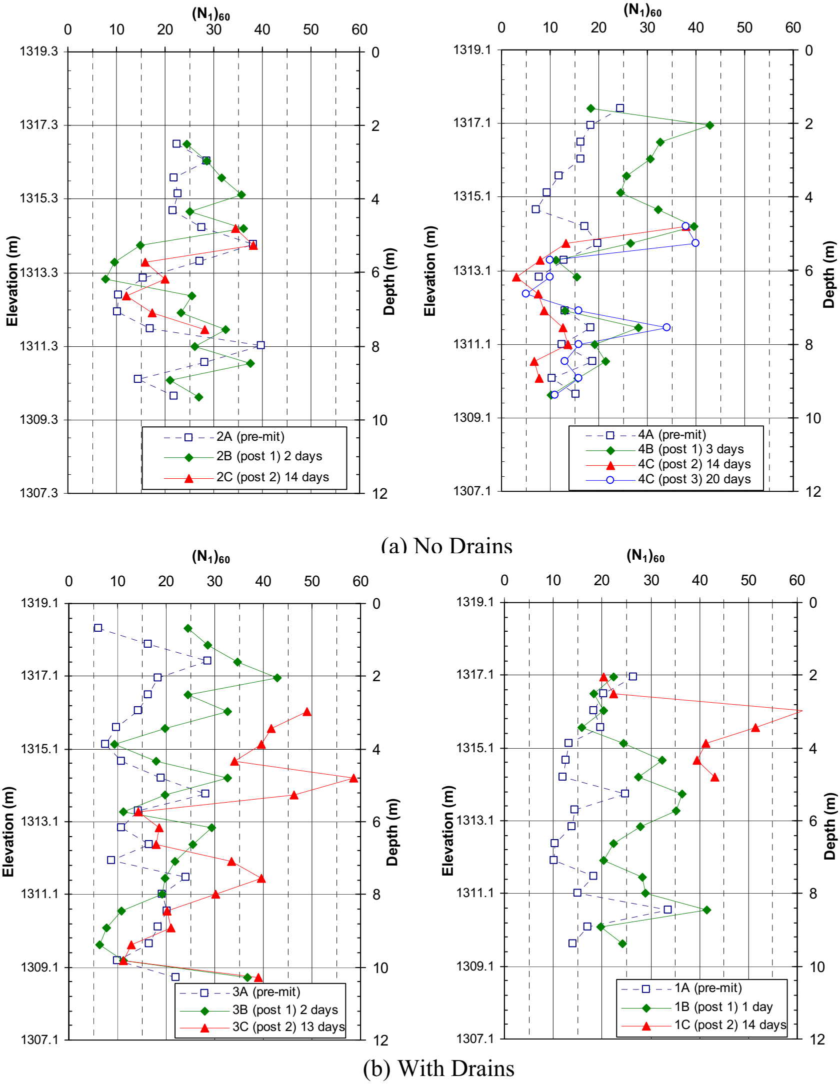

FIG. 2. Results from SPT testing in (a) two test areas without wick drains and (b) two test areas with wick drains during stone column treatment.

surface the average fines content is 45% with an average clay content of 14%. These variations in soil properties would be expected to produce significant variations in the success of the stone column treatment. Pre-treatment blow count profiles in the four test areas are plotted in Figure 2.

STONE COLUMN TREATMENT LAYOUT

To prevent liquefaction, the average $(N_1)_{60}$ value was specified as 23 and the minimum value was set at 18. Stone columns were installed to a depth of 12 m to increase the SPT blow counts to the desired levels. The stone columns, with a diameter 1.07 m, were spaced at 2 m on centers in a triangular pattern to produce an area replacement ratio of 26%. During phase 1 of the project, four test areas were treated as shown in Figure 3. Prior to stone column installation, wick drains were installed at the midpoint between each column as shown in Figure 3 in two of the four test areas while drains were not used in the other two areas. Treatment was performed using the dry, bottom-feed approach. The columns were installed with a Keller System S23120 unit (380 Volts, 1775 rpm). During installation, the maximum amperage ranged from 150 to 300 amps and the time for installation was typically about 30 minutes. In phase 2 of the project drains were used with stone columns in all areas.

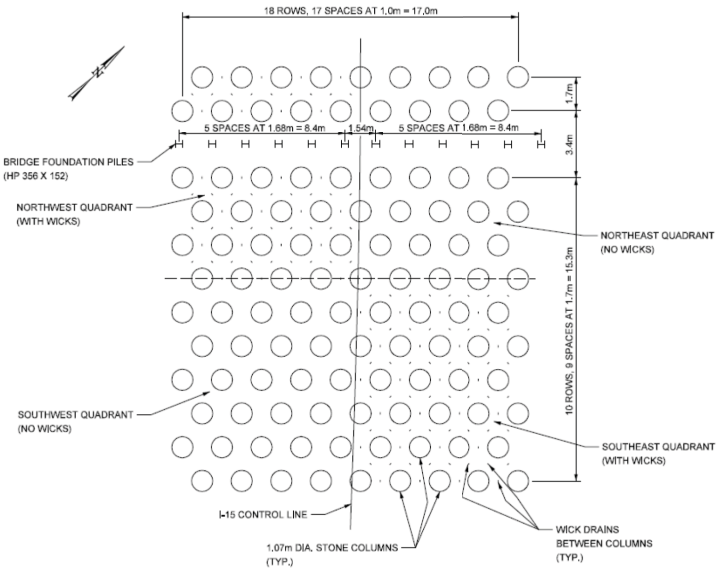

FIG 3. Layout of stone columns and wick drains.

POST-TREATMENT TESTING

Post-treatment test holes were drilled at each of the four tests areas within one to three days after stone column treatment. Once again sampling was essentially continuous to identify potentially soft zones in the profile and their thickness. Plots of the post-treatment SPT blows counts as a function of depth are provided in Figure 2 along with the pre-treatment profiles. Test holes in the areas without drains are shown