SITE RESPONSE OF IMPROVED GROUND IN THE COASTAL EASTERN UNITED STATES

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Abstract

The design for a major bridge to be located in seismically-active Charleston, South Carolina, included ground improvement beneath embankments. The original subgrade conditions generally consisted of loose, liquefiable sands, expected to significantly lose strength during the design earthquake and soft, compressible clays. To mitigate the risk of liquefaction-induced instability and to mitigate the risk of long term settlements in clays, ground improvement using vibro-replacement of the embankment subgrade was incorporated into the design plans. Using shear wave velocity data from actual sites in coastal South Carolina, a comparison was made of the predicted site and embankment responses for both original ground and improved ground. The results indicate that the vibro-replacement increases the spectral acceleration for embankments commonly constructed in coastal South Carolina. The influence of vibro-replacement is more dramatic in the characteristic clay profile as compared to the characteristic sand profile.

keywords: ground improvement, vibro-replacement, site response, seismic response

1. Introduction

With the exception of the 1811-1812 New Madrid sequence, the 1886 Charleston earthquake is the largest known event to have occurred in the central and eastern United States (CEUS). With a currently accepted moment magnitude (**M**) of 7.3 [1], the 1886 Charleston event was felt throughout the eastern U.S. and in such distant locations as Boston, Massachusetts; Chicago, Illinois; Milwaukee, Wisconsin; Cuba, and Burmuda [2,3,4]. Structural damage was widespread, extending as far as Alabama, Ohio, and West Virginia. A maximum Modified Mercalli (MM) intensity of X was experienced in the near-source region and liquefaction was extensive in the epicentral area [5,6,7]. Recent seismic study [8,9] confirms a repeat of the 1886 event will likely cause widespread liquefaction of susceptible sands and peak ground accelerations of up to 0.7g.

For design of the new cable-stayed Ravenel Bridge that will span the Cooper River between Charleston to Mount Pleasant (South Carolina), the design seismic hazard was dominated by a repeat of the 1886 earthquake. Due, in part, to the seismic design criteria, the bridge will cost an estimated $600M+. The main span of the bridge will be the longest for a cable-stayed structure in North

America at 471 m (1545 ft). The authors were responsible for the design geotechnical exploration and performed extensive seismic analysis of embankments including consideration of vibro-replacement to improve embankment subgrades. Vibro-replacement has the effect of both densifying loose, liquefiable sand and stiffening the soil profile to reduce static settlement. In addition to mitigating the risk of liquefaction and large static settlement, vibro-replacement also changes the dynamic response of the site and overlying embankments.

This paper examines the influence of vibro-replacement on the dynamic response of the Charleston subsurface conditions and overlying constructed embankments. Using actual field and laboratory data for local Charleston conditions, a comparison is made of the predicted site and embankment responses before and after vibro-replacement. One-dimensional site response analysis was performed using SHAKE91 [10]. Embankment response and deformation analyses were performed using both a well-established pseudo-static approach [11,12] and two-dimensional non-linear finite difference modeling [13].

2. Subsurface Characterization of Charleston, South Carolina

Charleston is located along the South Carolina coast in the continental United States. It lies within the Coastal Plain Physiographic province that is characterized by a sediment wedge overlying hard Paleozoic metamorphic rock or hard Pre-Cretaceous igneous and sedimentary rock. The Coastal Plain sedimentary wedge ranges in thickness from essentially zero at the Fall Line (i.e., the contact of the Coastal Plain province and the Piedmont province) to over 800 m (2500 ft) along the western South Carolina coast.

Local Charleston ground conditions most effecting embankment performance are loose fine sands susceptible to liquefaction and very soft clays. These soils typically range from 7 to 15 m (about 25 to 50 ft) in thickness. A variety of profiles exist including sites of uniform sand or clay and sites with extensively interbedded layers of sand, silt, and clay. A significant increase in stiffness and strength occurs in the underlying formations of the Cooper Group, which are characterized as calcareous clayey sands and sandy clays.

Two subsurface profiles from locations along the alignment of the proposed Ravenel Bridge are shown in Figures 1 and 2. The profiles also present cone penetration (CPT) data, including sleeve stress, tip stress, sleeve stress-tip stress ratio, pore pressure and shear-wave velocity. The profiles were selected because of the relative uniformity of the upper sediments encountered above the Cooper Group. The CPT sounding in Figure 1 encountered approximately 16.5 m (54 ft) of soft clay. The CPT sounding in Figure 2 encountered slightly less than 15 m (49 ft) of loose, fine sand. The thin sand layer in Figure 1 and thin clay layer in Figure 2 were ignored for the purposes of this paper. It is these two characteristic profiles that are used to evaluate the influence of vibro-replacement on site and embankment response analyses.

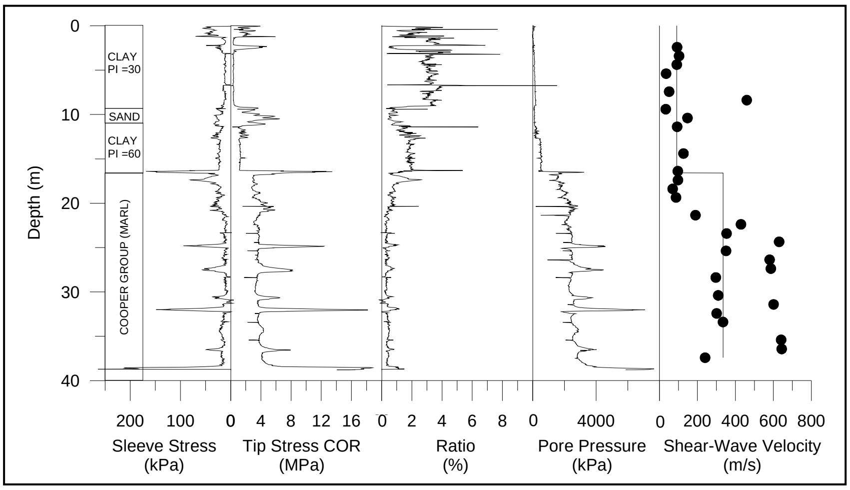

Figure 1. Characteristic Clay Profile

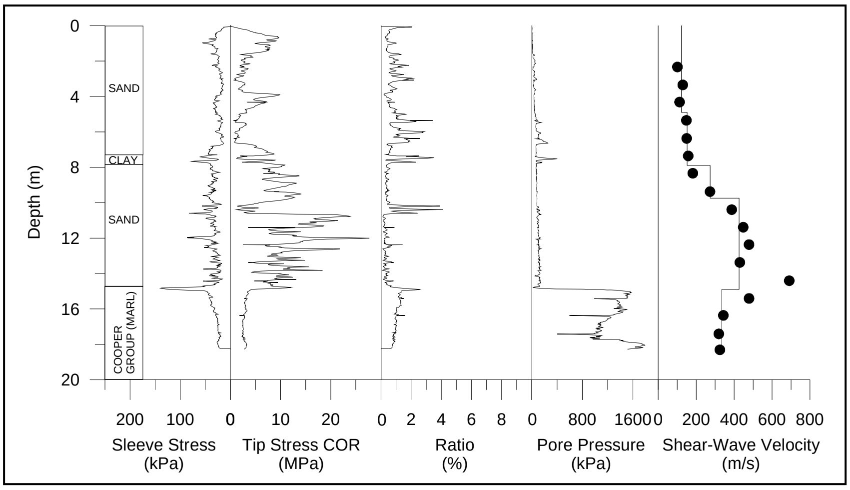

Figure 2. Characteristic Sand Profile

3. Ground Improvement Using Vibro-Replacement

The vibro-replacement process was developed in the late 1950's to improve loose sands and softer clays [14]. The construction state-of-practice consists of inserting a vibratory probe into the ground, either under its own weight or aided by water jetting [15]. Uniform clean stone backfill is then introduced in one of two ways depending on the project requirements. For the top feed method, the stone backfill is placed from the ground surface into the annulus between the probe and the sidewalls of the hole. For the bottom feed method, the stone backfill is placed from the tip of the probe by a feed tube. For both techniques, the stone backfill is compacted in lifts of 60 to 120 cm (2 to 4 ft). Repeated insertion of the vibratory probe into the stone backfill: (1) densifies the stone backfill; (2) forces stone radially into the surrounding soil to create a substantial reinforcing element; (3) increases lateral stresses, and; (4) densifies surrounding granular soil. Typical stone column diameter is between 76 to 107 cm (2.5 to 3.5 ft) and typical spacing is 210 to 270 cm (7 to 9 ft). The resulting area replacement ratio (AR) ranges from about 7 to 23 percent, where the area replacement ratio is defined as the area of the stone column divided by the tributary area per stone column. The spacing and size of the stone columns is typically selected based on the levels of densification and reinforcement required by the project.

4. Vibro-Replacement in Seismic Areas

Englehardt and Golding [16] performed field testing to quantify the effectiveness of vibro-replacement to improve the seismic response of soil, specifically by increasing liquefaction resistance, shear strength and stiffness. Numerous researchers have reported that vibro-replacement has been effective in preventing the onset of liquefaction and reducing the damaging effects of severe ground shaking [17,18]. Some researchers have suggested that stone columns constructed by vibro-replacement will decrease both shear strains and excess pore pressures generated during cyclic loading [18,19].

The increase in ground stiffness and resulting influence on site seismic response is recognized by Mitchell et al. [20]. Although numerous approaches [21,22,23,24,25,26,27,28] have been developed to estimate the static behavior of improved ground (i.e., soil with stone columns), significantly less attention has been given to characterizing its dynamic behavior. In recognition of the non-linear behavior of soils, an important distinction is made between strains (and corresponding stiffnesses) during a seismic event and higher strains induced by typical static loads. Thus, the approach presented herein uses exploration techniques for determining low strain soil moduli and explicitly considers non-linear soil behavior in evaluating the response of embankments overlying ground improved by vibro-replacement.

5. Site and Embankment Response Analyses

The site and embankment response analyses were performed for the two characteristic subsurface profiles (shown in Figures 1 and 2) and for ground (hypothetically) improved by vibro-replacement. The area replacement ratios considered were 10% and 20%. The site response analysis was performed using the one-dimensional equivalent non-linear program SHAKE91. The embankment response and deformation analyses were performed using the pseudo-static approach presented by Makdisi and Seed and the two-dimensional non-linear finite difference program FLAC. The input ground motions for the site and embankment response analyses were developed by Walt Silva, and represent the acceleration time history within the Cooper Group for an earthquake with a 2% probability of exceedance in 50 years. Deformations associated with liquefaction-induced instability were not considered in this paper. If liquefaction had been considered, a different site response and/or greater embankment deformations would likely have been determined for the characteristic sand (unimproved) profile.

5.1 Site response using SHAKE91

One-dimensional site response was performed for the characteristic subsurface profiles graphically described in Figures 1 and 2. The non-linear soil behavior is modeled using modulus reduction and damping curves, which are used by SHAKE91 to compute the variation in stiffness and damping at different strain levels. Curves determined from testing of Charleston soils (clay, sand, and Cooper Group) are shown in Figure 3. The unit weight of the original ground was estimated from laboratory test data. Acceleration response spectra (for 5% damping) for the input motion (within the Cooper Group) and at the ground surface for both the sand and clay characteristic profiles are graphically shown in Figure 4.

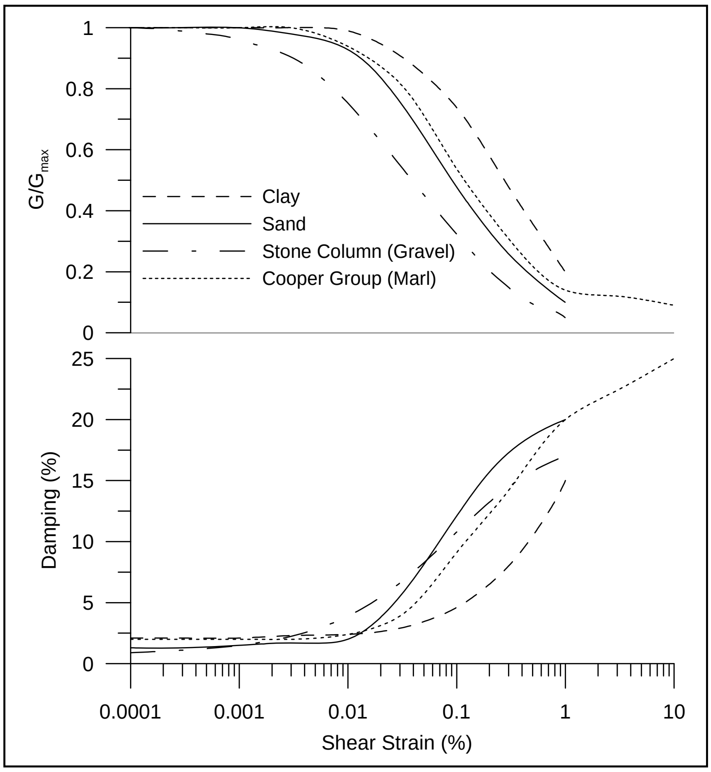

Figure 3. Modulus Reduction and Damping Curves

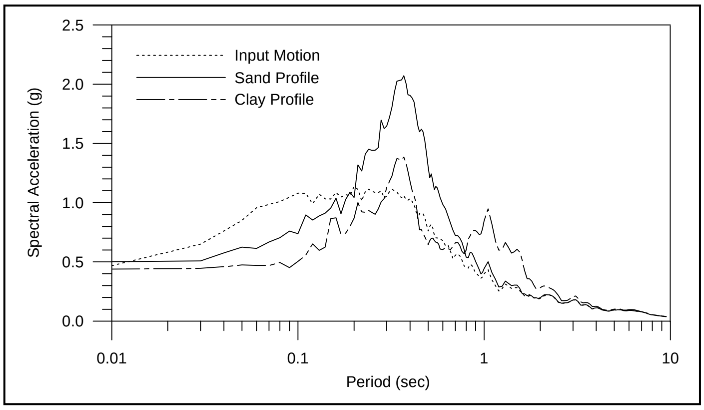

Figure 4. Acceleration Response Curves for Unimproved Soil

To perform one-dimensional site response for improved ground, it was necessary to estimate the initial shear modulus after improvement by vibro-replacement. Baez [19] developed theoretical equations to compute the distribution of shear stress between stone columns (constructed by vibro-replacement) and the surrounding soil. The primary assumption is that the shear strain is the same in both the stone column and surrounding soil. Baez's equations may be extended to compute the distribution of the shear stress and the effective initial shear modulus as follows:

$$V_{\text{improved ground}} = V_{\text{stone column}} + V_{\text{soil}} \quad (1)$$

where V is shear force. By rewriting the shear forces (τ) in terms of shear stresses and areas (A),

$$\tau_{\text{improved ground}} * A_{\text{tot}} = \tau_{\text{stone column}} * A_{\text{stone column}} + \tau_{\text{soil}} * A_{\text{soil}} \quad (2)$$

By making substitutions using $\tau = G\gamma$ (where G is shear modulus and γ is shear strain) and the area replacement ratio (AR), the initial shear modulus of the improved ground may be written as:

$$G_{\text{improved ground}} = G_{\text{stone column}} * AR + G_{\text{soil}} * (1\text{-}AR) \quad (3)$$

The initial shear moduli for the original ground were computed from the shear-wave velocity data for the two characteristic profiles and the stone column initial shear moduli was computed from published shear-wave velocity data [25]. The unit weight of the improved ground was computed in a similar manner to the initial shear modulus. As a simplification to the analysis presented in this paper, identical modulus reduction and damping curves were used for both the characteristic sand/clay profiles and the improved ground. It is recognized that gravels have been shown to be non-linear at smaller strain levels than the curves used in this paper for either the sand or clay [29]. Figures 5 and 6 graphically present the acceleration response spectra for improved ground with area replacement ratios of 10% and 20% along with those of the characteristic profiles for comparison.

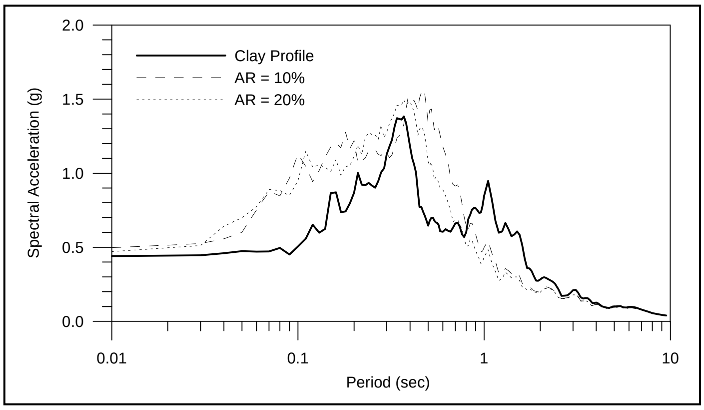

Figure 5. Acceleration Response Curves for Clay

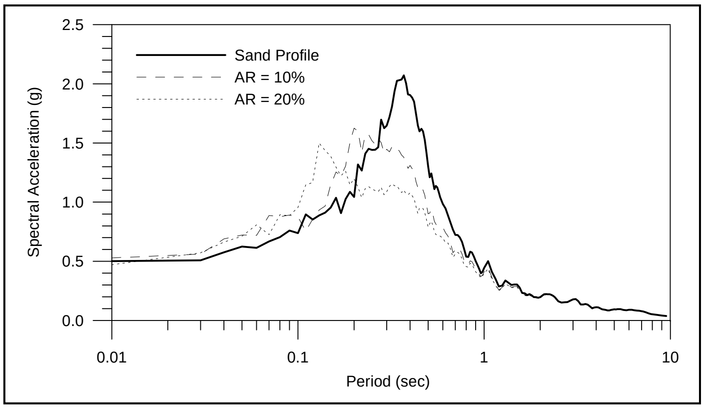

Figure 6. Acceleration Response Curves for Sand

5.2 Embankment response

The results of the site response analysis were used to compute the permanent horizontal displacement using the Makdisi and Seed (MS) pseudo-static approach for embankment heights of 3 m, 6.1 m, 9.1 m, and 12.2 m (10 ft, 20 ft, 30 ft, and 40 ft, respectively). Side slopes with an orientation of 2 (horizontal) to 1 (vertical) were selected for this analysis because of their frequent use on SCDOT highway projects. The preference for this slope orientation results from: (1) the expectation that steeper slopes may experience greater surficial instability during construction and over the long term; (2) maintenance of steeper slope orientations becomes more difficult, (3) flatter slopes require greater right-of-way. The horizontal displacements computed by the MS approach are graphically presented in Figure 7. The properties of the embankment fill were selected to be representative of borrow materials typically available in the Charleston, South Carolina region. In summary, the properties are (1) an internal friction angle of 32 degrees, (2) zero cohesion, (3) a unit weight of 18.9 kN/m^3 (120 pcf), (4) modulus reduction and damping curves for sand presented in Figure 3.

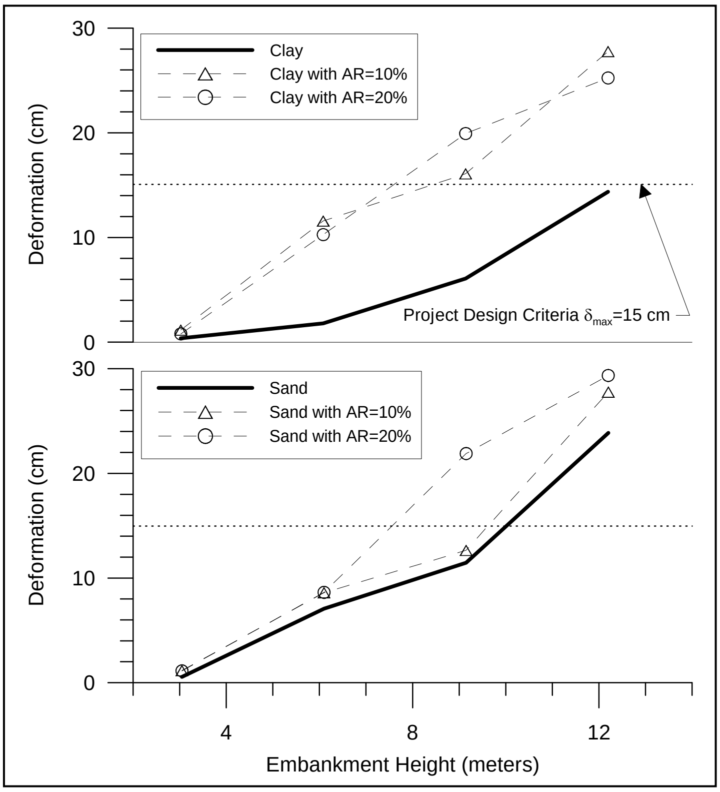

Figure 7. Horizontal Embankment Deformation

Comparative two-dimensional non-linear analysis was performed using FLAC for the same slope orientation (i.e., 2H:1V) and for the same range of slope heights as computed by the MS approach. Non-linear soil behavior in the embankment was implicitly considered in a simple manner by the use of a Mohr-Coulomb failure criteria, where plastic strain accumulates during yield. Review of the deformed FLAC models that result from the application of the earthquake time history confirms that the embankment slopes tend to flatten and the slope crest becomes less distinct. Unlike the MS approach which provides essentially one value of horizontal displacement for a distinct failure surface, the FLAC analysis provides the deformation throughout the embankment. For purposes of this paper, the maximum horizontal deformation near the toe of the slope predicted by FLAC is considered comparable to the MS approach results. The results of the FLAC analysis and the MS approach are compared in Figure 8, which shows embankment deformations for improved ground normalized to the embankment deformations for the characteristic Charleston profiles. The embankment heights in the FLAC analysis were limited to 9.1 m (30 ft) and below because higher embankments experienced large deformations (and numerical instability) at shallow depths along the slope.

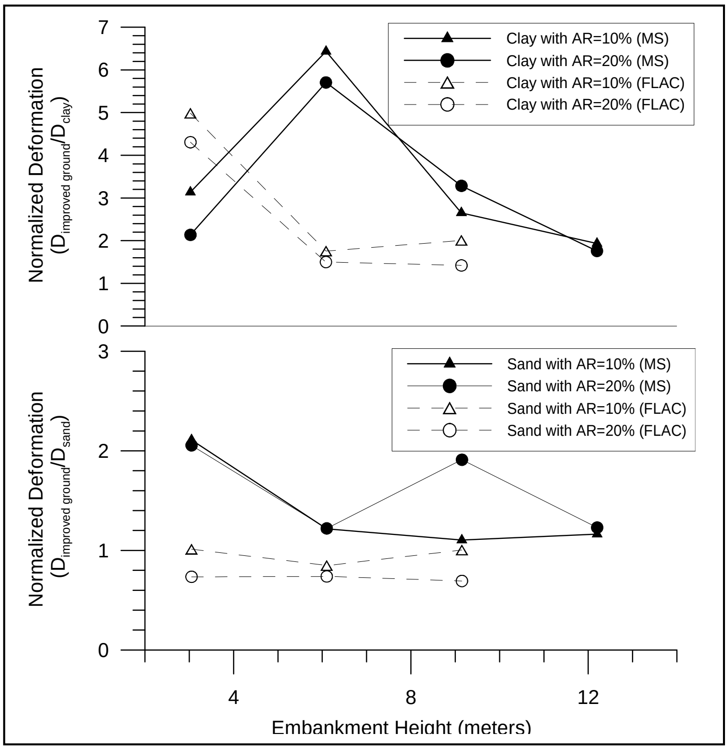

Figure 8. Normalized Horizontal Embankment Deformation

6. Discussion

The results of the site response analysis predict that the ground improved by vibro-replacement will respond as a stiffer profile than the original ground (i.e., the characteristic Charleston profiles). The acceleration response spectra for the improved ground exhibits greater spectral accelerations at shorter periods, and the peak spectral acceleration is also shifted to shorter periods (or higher frequencies) for the sand profiles. The peak spectral acceleration of greater than 2g occurs at a period of about 0.4 sec in the characteristic (unimproved) sand profile. For the improved sand profiles with AR values of 10% and 20%, the peak spectral acceleration shifts to a period ranging from approximately 0.1 to 0.2 sec. The results for the clay profiles do not exhibit such a clearly defined shift in the peak spectral acceleration as for the sand profiles; however, spectral accelerations are significantly greater for periods between 0.02 to 0.7 sec. This period range is particularly important because it is similar to the range of the natural periods for the embankments.

On the basis of the horizontal deformations computed using the well-established Makdisi and Seed approach, it is concluded that embankments that are approximately 4 m (13.1 ft) or greater in height and overlie a clay profile will experience significantly greater lateral deformation, if the subgrade is improved by vibro-replacement. Comparison of the analysis results to the design criteria for the Ravenel Bridge indicates that vibro-replacement for the clay profile could reduce the maximum design embankment from 12 m (40 ft) to 7.5 m (25 ft). Though less dramatic, an slight increase in horizontal deformation is also predicted for the characteristic sand profile.

The FLAC results also support the conclusion that vibro-replacement performed in the characteristic clay profile will increase the horizontal deformations in overlying embankments. As shown in Figure 8, FLAC computes approximately 1.5 to 5 times greater horizontal displacement for the improved ground when compared to the characteristic clay profile. FLAC computes essentially the same horizontal deformation for the improved sand profile (i.e., after vibro-replacement) as for the characteristic sand profile.

7. Conclusions

Ground improvement by vibro-replacement, which densifies loose sands and reinforces softer soil profiles, changes the dynamic responses of the site and overlying embankments. Response analyses for two characteristic Charleston profiles and for (hypothetically) improved ground indicate that the vibro-replacement increases the spectral acceleration for embankments commonly constructed in coastal South Carolina. The influence of vibro-replacement is more dramatic in the characteristic clay profile as compared to the characteristic sand profile. The results of the analysis presented in this paper are based on ground motions and subsurface information that are representative of the conditions near the proposed Ravenel Bridge in Charleston, South Carolina. And while the conclusions presented in this paper are based on accepted engineering procedures, it is recognized that the results of such procedures may differ for different ground motions and/or subsurface conditions.

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References

[1] Johnston, A.C. (1996). “Seismic moment assessment of earthquakes in stable continental regions, III, New Madrid 1811-1812, Charleston 1886 and Lisbon 1755, *Geophysical Journal International*, 126, 314-344.

[2] Dutton, C.E. (1889). *The Charleston earthquake of August 31, 1886, in U.S. Geological Survey Ninth Annual Report 1887-1888*, 203-528.

[3] Bollinger, G.A. (1977). “Reinterpretation of the intensity data for the 1886 Charleston, South Carolina, earthquake, in Rankin, D.W. (ed.), Studies Related to the Charleston, South Carolina, Earthquake of 1886; A Preliminary Report, *U.S. Geological Survey Professional Paper 1028*, 17-32.

[4] Stover, C.W., and Coffman, J.L. (1993). Seismicity of the United States, 1568-1989 (Revised), *U.S. Geological Survey Professional Paper 1527*, 415 p.

[5] Obermeier, S.F., Gohn, G.S., Weems, R.E., Gelinas, R.L., and Rubin, M. (1985). “Geologic evidence for recurrent moderate to large earthquakes near Charleston, South Carolina”, *Science*, 227, 408-411.

[6] Amick, D., and Gelinas, R., (1991). “The search for evidence of large prehistoric earthquakes along the Atlantic seaboard”, *Science*, 251, 655-658.

[7] Talwani, P., Amick, D.C., Shaeffer, W.T. (1999). “Paleoliquefaction studies in the South Carolina coastal plain.”, *Final Report for National Research Council Contract No. NRC-04-93-067, NUREG/CR-6619*, 109 pp.

[8] Silva, W., Wong, I., Siegel, T.C., Gregor, N., Wright, D., Shapiro, R., Lee, R., and Talwani, P. (2002). “A comprehensive seismic vulnerability and loss evaluation of the state of South Carolina

using HAZUS: Part II Ground Motion Hazard, *Paper accepted for publication at 7th National Conference on Earthquake Engineering*.

[9] Siegel, T.C., Camp, W.M. III, Silva, W., Wong, I., and Wright, D. (2002). "A comprehensive seismic vulnerability and loss evaluation of the state of South Carolina using HAZUS: Part III Liquefaction Hazard, *Paper accepted for publication at 7th National Conference on Earthquake Engineering*.

[10] Idriss, I.M., and Sun, J.I. (1991). *User's manual for SHAKE91: A computer program for conducting equivalent linear seismic response analyses of horizontally layered soil deposits*. Center for Geotechnical Modeling, University of California, Berkeley.

[11] Makdisi, F.I., and Seed, H.B. (1978). "Simplified procedure for estimating dam and embankment earthquake-induced deformations." *Journal of Geotechnical Engineering Division*, ASCE, 104 (GT7), 849-867.

[12] Makdisi, F.I., and Seed, H.B. (1979). "Simplified procedure for evaluating embankment response." *Journal of Geotechnical Engineering Division*, ASCE, 105 (GT12), 1427-1434.

[13] Itasca (2000). *FLAC Version 4.0 Users Manual*. Itasca Consulting Group, Inc., Minneapolis, Minnesota.

[14] Rathgeb, E., and Kutzner, C. (1975). "Some applications of the vibro-replacement process." *Geotechnique*, 25 (1), 45-50.

[15] Shaefer, V.R., Abramson, L.W., Drumheller, J.C., Hussin, J.D., and Sharp, K.D. (eds.) (1997). *Ground Improvement; Ground Reinforcement; Ground Treatment*, ASCE, Geotechnical Special Publication No. 69, 616 pp.

[16] Engelhardt, K., and Golding, H.C. (1975). "Field testing to evaluate stone column performance in a seismic area." *Geotechnique*, 25 (1), 61-69.

[17] Mitchell, J.K., Baxter, C.D.P., and Munson, T.C. (1995). "Performance of improved ground during earthquakes." *Soil Improvement for Earthquake Hazard Mitigation*, ASCE, Geotechnical Special Publication No. 49, 1-36.

[18] Martin, J.R. III, Mitchell, J.K., Olgun, C.G., Durgunoglu, H.T., and Emren, C. (2001). "Performance of improved ground during the 1999 Turkey earthquake." *Foundations and Ground Improvement*, ASCE, Geotechnical Special Publication No. 113, 565-579.

[19] Baez, J.I. (1995). "A design model for the reduction of soil liquefaction by vibro-stone columns." *Ph.D. Dissertation*, University of Southern California, Los Angeles, CA, 207 pp.

[20] Mitchell, J.K., Cooke, H.G., Schaeffer, J.A. (1998). "Design considerations in ground improvement for seismic risk mitigation." *Geotechnical Earthquake Engineering and Soil Dynamics III*, University of Washington, Seattle.

[21] Greenwood, D.A. (1975). "Vibroflotation: Rationale for design and practice, methods of treatment of unstable ground." *Ed FG Bell*, Newness-Buttersworth, London, 189-209.

[22] Goughnour, R.R., and Bayuk, A.A. (1979). "A field study of long-term settlement of loads supported by stone columns in soft ground." *Proceedings*, International Conference on Soil Reinforcement: Reinforced Earth and Other Techniques, Vol. 1, Paris, 279-286.

[23] Barksdale, R.D., and Bachus, R.C., (1983). "Design and construction with stone columns*." Final Report to FHWA*, SCEGIT-83-104, Georgia Institute of Technology, Atlanta, GA.

[24] Mitchell, J.K., and Huber, T.R. (1985). "Performance of a stone column foundation." *Journal of Geotechnical Engineering*, ASCE, 111(2), 205-223.

[25] Martinez, R.E., Buggy, F.J., and Gupton, C.P. (2001). "Case history of ground improvement for construction and hydrotesting of a large LNG tank in Puerto Rico*." Foundations and Ground Improvement*, ASCE, Geotechnical Special Publication, 580-594.

[26] Priebe, J.H. (1993). "Design criteria for ground improvement by stone columns." *Proceedings*, Fourth National Conference – Ground Improvement, Lahore, Pakistan.

[27] Stewart, D., and Fahey, M. (1994). "An investigation of the reinforcing effect of stone columns in soft clay." *Vertical and Horizontal Deformations of Foundations and Embankments*, ASCE, Geotechnical Special Publication No. 40, 513-524.

[28] Ashmawry, A.K., Rokicki, R., and Plaskett, M.E. (2000). "Predicted and actual settlement of soils improved by vibroreplacement." *Advances in Grouting and Ground Modification*, ASCE, Geotechnical Special Publication No. 104, 281-295.

[29] Rollins, K.M., Evans, M.D., Diehl, N.B., and Daily, W.D. III. (1995). "Shear modulus and damping relationships for gravels*." Journal of Geotechnical and Geoenvironmental Engineering*, 124(5), 396-405.