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VIBRO REPLACEMENT FOR LIQUEFACTION HAZARD MITIGATION FOR OPERATIONAL STORAGE FACILITY IN CORONADO, CALIFORNIA, USA

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ABSTRACT

Vibro replacement stone columns were installed for soil improvement for the construction of a 20,000-square-foot operational storage facility in Coronado, CA. The soil improvement program was conducted to meet seismic and static performance criteria for spread footings founded on improved soil. CPT testing was conducted before and after stone column construction to verify the vibro replacement program. Comparison between pre- and post-construction CPTs showed remarkable increase in the tip resistance in loose sand layers. Accounting for densification and shear reinforcement, the anticipated post-improvement liquefaction-induced settlement was reduced significantly.

INTRODUCTION

The U.S. Naval Amphibious Base Coronado, located across the bay from San Diego, California, USA, is the largest of its type in the southwest United States, and covers over 57,000 acres. A new 40,000-square-foot operational storage facility was planned for construction on base grounds.

Located within the seismically active area of southern California, the site is subject to moderate to strong ground shaking from a local or more distant, large magnitude earthquake occurring during the expected life span of the structure. Such an earthquake may trigger liquefaction of the loose sandy soils existing at the site. Liquefaction will cause loss of bearing capacity of the shallow foundations and severe foundation settlement. Site liquefaction hazard mitigation by vibro replacement was recommended by the design and construction team as a cost effective solution to alleviate the site liquefaction hazard in accordance with the California Building Code (2007). With ground improvement the site could be classified as Site Class D. Otherwise, Site Class F would have applied, due to liquefaction potential.

SITE GEOTECHNICAL INVESTIGATION

The geotechnical report indicated that the generalized soil profile consists of undocumented fill to a maximum depth of approximately 16.5 feet (5.0 m) from the current ground surface. Fill is described as loose to medium dense, silty fine to coarse sand. Bay deposits below the fill exist to a maximum depth of 31.5 feet (9.6 m). Bay deposits are described as very loose to loose, silty fine to medium sand and soft clay. The Bay Point Formation was encountered at a depth of 31.5 feet (9.6 m), and is described as medium dense to very dense silty, fine to medium sand. The ground water table was encountered at 10 ft depth.

The geotechnical investigation report concluded that the site was underlain by potentially liquefiable soils and that the site would be feasible for development using spread footings after a soil improvement program was performed for liquefaction hazard mitigation and increased bearing capacity.

LIQUEFACTION AND SETTLEMENT ANALYSES

Liquefaction analyses were undertaken in general accordance with procedures outlined by Youd and Idriss (NCEER, 1997), and Martin and Lew (SCEC, 1999) with modifications for calculation of fines content in accordance with Baez, Martin, and Youd (2000). Pre-treatment liquefaction analyses were performed for five CPT locations, based on a design earthquake with properties listed in Table 1.

Table 1. Design Assumptions for Liquefaction Analyses

Design highest groundwater depth	10 feet (3.0 m)
Groundwater table depth during CPT tests	10 feet (3.0 m)
Design earthquake magnitude, Mw	7.0
Design Peak Ground Acceleration, PGA	0.33g

Liquefaction induced settlement analyses were conducted in general accordance with Tokimatsu and Seed, (1984). This procedure was developed based on the penetration resistance in terms of clean sand equivalent SPT blow counts. CPT tip resistance was converted to the SPT blow count by the method presented by Jefferies and Davis (1993), and then corrected to N_{corr} based on the sand fine content according to California SP-117. Results of the calculations based on the five CPTs are listed in Table 2. The thin layer correction was not used for liquefaction induced settlement analysis.

Table 2. Liquefaction Induced Settlement Calculations based on Five Pre-Treatment CPTs

CPT Test	Zone	Liquefaction Induced Settlement before Treatment (inch)
CPT1	1	4.1
CPT2	1	4.7
CPT3	1	4.1
CPT4	1	3.5
CPT5	1	4.7

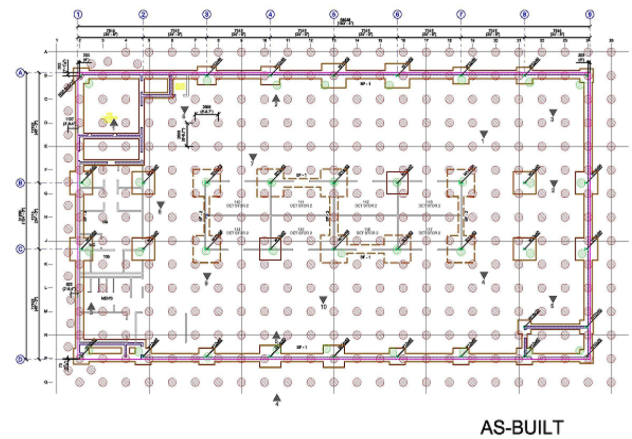
VIBRO REPLACEMENT GROUND IMPROVEMENT PROGRAM

The geotechnical contractor designed and constructed a vibro replacement program to mitigate the site liquefaction hazard. The dry, bottom feed method was used to install the stone columns. The feasibility and design was based on the following information obtained from the specifications which were written by the geotechnical engineer, listed in Table 3.

Table 3. Geotechnical Information for Design Basis

Avg Liquefaction Induced Settlement	≤ 1.5 inch (38.1 mm)
Max Liquefaction Induced Settlement	≤ 2.0 inch (50.8 mm)
Design Groundwater Level	+4 ft (+1.2 m) MSL
Design Earthquake Magnitude, Mw	7.0
Design Peak Ground Surface Acceleration	0.33g
Allowable soil bearing capacity below load bearing walls	2500 psf (120 kpa)

Based on past experience and the characteristics of the site soil profile, the specialty contractor determined that primary stone columns spaced 8.7 feet, (average) center-to-center in a square pattern would achieve the intended performance criteria under the building (Figure 1).



Notes:

1. ○ Indicates 9.144 m (30 ft) deep stone column.
2. ○ Indicates 6.096 m (20 ft) deep stone column.
   Both Columns are .914 m (36") in Diameter.
3. Columns on geotechnical contractor's Grid 1 are moved to minimize potential of damage to existing utilities and to avoid concrete thrust blocks.
4. ▲ Indicates approximate Pre-CPT locations.
5. ▼ Indicates approximate Post-CPT locations.

Fig 1. Vibro replacement stone column layout.

A performance based criteria was established to design the vibro replacement program to meet a deformation criterion that would satisfy the structural requirements of the building, as listed in Table 3. The site liquefaction induced settlement, calculated from the post improvement CPTs, is a weighted average and reflects the real liquefaction risk level. This method considers the thickness of the liquefiable soil layers, relative density, fines content, site design peak ground surface acceleration, and CRR/CSR ratio. It reflects the real soil behavior under earthquakes; that loose sandy soils tend to loose volume under cyclic shear.

Other vibro replacement design criteria have been considered in the past and are used occasionally, including options to perform the vibro replacement to a certain relative density, to a minimum tip resistance measured by post-treatment CPTs, or to a minimum factor of safety against liquefaction. These criteria are indirectly related to the foundation performance. Utilizing a liquefaction induced settlement criteria has proved to be effective in reducing the risk of liquefaction induced settlement, while being cost effective.

To accomplish the liquefaction hazard mitigation, the soil must be densified, drained, reinforced, or replaced in part or completely, according to SP-117. The degree of soil densification resulting from the installation of vibro replacement stone columns is a function of many factors, including: soil type, silt and clay content, uniformity of soil gradation, plasticity of the soils, initial penetration resistance, energy input, backfill material, and area replacement ratios (a ratio between the area of the stone column cross section to its tributary area).

The area replacement ratio (ARR = Surface Area of Stone Column / Tributary Area) was 9.5% for 3.0 ft diameter stone columns in the planned grid.

The authors calculated the approximate fines content from the pre-improvement CPT data, based on a correlation provided by Robertson and Wride (1998), and verified with sieve test results from specimens obtained from a boring near CPT-5. The approximate fines contents in the sand layers were typically below 10%. The soil behavior type index value, Ic, is below 1.8, which suggests the presence of relatively clean sands. Based on past experience, the authors estimated that under the design replacement ratio of 9.5% in the relatively clean loose sands, the post-treatment CPT tip resistance would significantly increase. The target CPT tip resistance, q_{c1N}, was approximately 200 tsf (19.1 MPa), well above the NCEER liquefaction criteria.

The potential for lateral spreading and flow sliding is low. The site is approximately 38 m (125 feet) from the shoreline and it is relatively level.

IMPROVEMENT FACTOR DUE TO REINFORCEMENT EFFECT OF STONE COLUMNS

The major benefit of vibro replacement stone column improvement is the densification of sands. The details of the densification effects are presented in the next section, including post construction testing and analyses. In addition, the presence of the stone columns provides a stiffening effect in silty sands and sandy silts, where the vibro densification effects are limited, as evidenced in centrifuge testing presented in Adalier et al. (2003). Adalier et al. measured the dynamic settlement with a stone column installed in liquefiable loose silt. FLAC finite difference analyses indicated a settlement reinforcing factor of 1.63 in both sands and silts for areas treated with an approximate area replacement ratio of 10%. In the FLAC analysis, the residual strength of the liquefiable silt surrounding the stone column was used to evaluate the stone columns' vertical and radial deformations during the seismic events. The modulus ratio between the stone column and the surrounding sands was conservatively assumed as 3.0.

The ground improvement program addressed both the liquefaction settlement as well as the static bearing capacity and settlement requirements. Additional static reinforcement was provided by the secondary columns installed under foundation footprints (Figure 1). A column length of 20 feet, or vibrator refusal depth, was adequate for the secondary columns. The static settlement of the stone column reinforced footings was evaluated according to the method presented by Sehn and Blackburn (2008).

CONSTRUCTION

The dry, bottom feed method involves the use of a purpose-built depth vibrator, electrical conduit follower tubes, an air pressure chamber, a skip bucket feeder, and gravel tremie pipe to install the stone columns to desired depths, as shown in Figure 2. The vibrator penetrates the ground under its own weight. When the column design depth is reached, stone is introduced through the tremie pipe in lifts, and for each lift the stone column and the surrounding soils are compacted by re-penetration of the high-energy vibrator. The column is complete when ground surface is reached by the vibrator tip.

A total of 375, 36-inch-diameter (91.44 cm) primary stone columns were installed to a depth of 30 feet below working grade for liquefaction hazard mitigation, and a total of 35, 36-inch-diameter (91.44 cm) secondary stone columns were installed to a depth of 20 feet below planned footings for increased bearing capacity.

Fig 2. Installation of a Vibro replacement stone column at the storage facility at Naval Amphibious Base in Coronado, CA.

During installation, parameters such as amperage, stone quantity, duration, and depth were monitored and logged to ensure consistent column construction.

After construction of the stone columns the top 24 inches (61 cm) of soil was removed and re-compacted to at least 95% relative density, in accordance with ASTM D1557.

POST CONSTRUCTION TESTING AND ANALYSES

Ten post-treatment CPT tests were performed at the interstice of the stone column grid. This location, being farthest from the installed stone columns, provides a conservative estimate of soil liquefaction.

Figures 3 and 4 show typical pre-CPT and post-CPT data interpretation and liquefaction analyses, and demonstrate the vibro stone column densification effects.

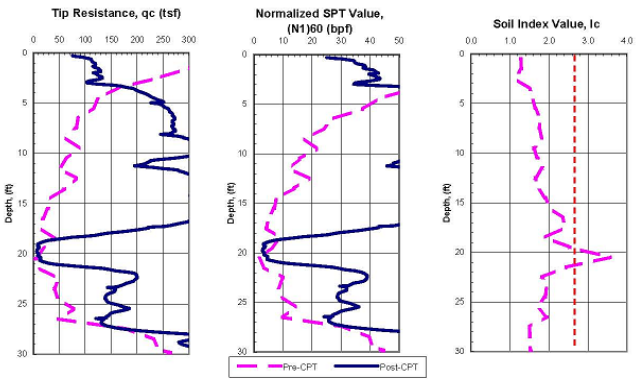


Fig 3. Tip resistance versus depth; Normalized SPT Value versus depth; and soil behavior type index versus depth.

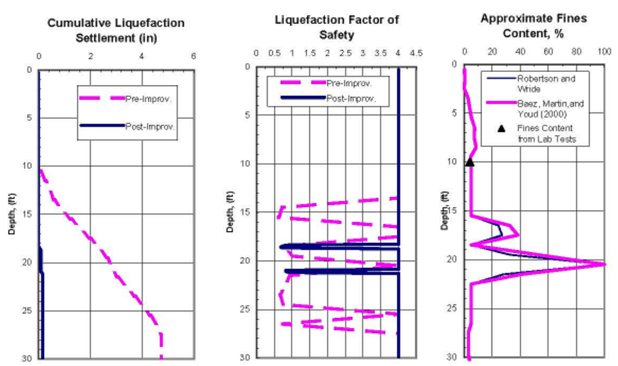


Fig 4. Cumulative liquefaction settlement versus depth; liquefaction factor of safety versus depth; and approximate fines content versus depth.

The authors found that the soil fines content has significant impact to the vibro densification effectiveness. According to Robertson and Wride (1998), the soil fines content can be correlated directly to the soil behavior type index Ic. Figure 5 compares the normalized CPT tip resistance before and after the ground improvement as a function of Ic. The distances between the red circles and the blue diamonds represent the vibro densification effects. In the clay layer, where Ic > 2.6 at the depth between 19 ft to 21 ft (5.8 to 6.4 m), the CPT tip resistance did not increase.

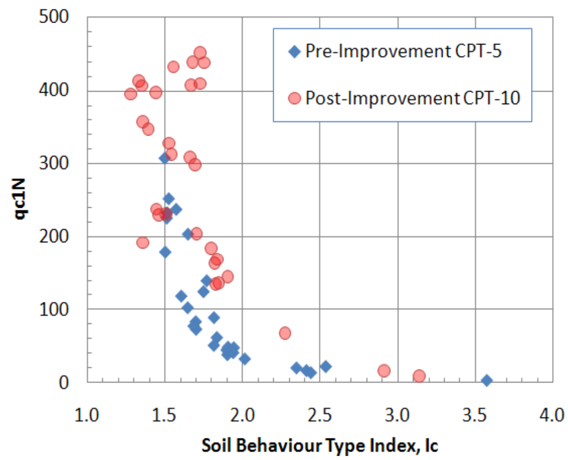


Fig 5. Comparison of the normalized tip resistance before and after improvement as a function of soil behavior type index.

The vibro densification effect can be expressed as the normalized CPT tip resistance improvement ratio, as shown in Figure 6. The average improvement ratio approached 3, and agreed well with the authors' design prediction. When Ic is higher than 2.3 or the pre improvement q_{c1N} is higher than 200 tsf (19.1 MPa), the vibro improvement ratio will approach 1.

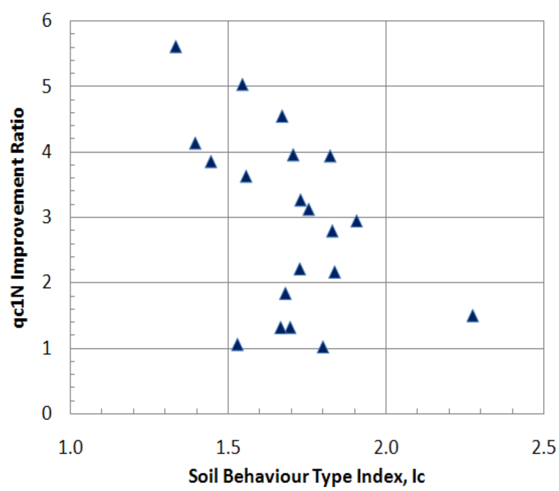


Fig 6.CPT Tip resistance improvement ratio as a function of Ic in sandy soils.

It is evident that the post CPT tip resistance is very sensitive to the soil behavior type index, Ic, or the soil types and fine content. The vibro densification shares the same mechanism of soil liquefaction as loose sandy soil (reduced volume under cyclic shear. Therefore, verification of vibro stone column treatment can use the same analyses procedures established by Robertson and Wride (1998). The post treatment CPT tip resistance relates to the soil type and fines content, as well as vibro densification efforts. The large variation in Q_{c1N} improvement ratio, as shown in Figure 6, was caused by varying Ic values and CPT data points near sand-clay layer interfaces. Therefore, using minimum Q_{c1N} value as the quality control criteria could be quite difficult.

Following NCEER 97 procedures, the liquefaction factor of safety was calculated as the ratio of CRR and CSR. Soils with an Ic value higher than 2.6 were considered as non-liquefiable. In the liquefaction analysis, thin layer corrections to the CPT tip resistances were not used.

In general, liquefaction induced settlement was calculated according to Tokimatsu and Seed, 1984 procedures, with a CPT tip resistance to SPT blow count conversion. The liquefaction induced settlement calculated from the post improvement CPT-10 was only 0.16 inch (4 mm), compared with the pre-improvement CPT-5 of 4.74 inches (12 cm) at the same location; a significant improvement. CPT-10 was a typical result, and similar improvement was observed in all post-treatment CPT tests.

Taking into account the stone column reinforcement effect, and utilizing the improvement factor of 1.63, the liquefaction induced settlement of the ground can be further reduced to approximately 0.1 inch (2.5 mm) (1.60/1.63).

As shown in Figures 3 and 4, the apparent liquefiable zones are the sand-clay layer interfaces, due to the CPT resolution and layering effects. At 15 ft (4.5 m) below footings and covered with very dense sand and stone column matrix, these thin layers have little impact to the footing bearing capacity. The ground lateral cyclic displacement during an earthquake is usually a few inches, only a small fraction of the 3–ft (0.9 m) diameter stone columns, and unlikely to cause stone column shear or bending failure. Although there are many publications related the behaviors of vibro stone column treated sites, there are no reports of stone column failure during a real earthquake.

CONCLUSION

The case history presented in this paper described the successful application of vibro replacement stone columns to effectively mitigate soil liquefaction hazards. This mitigation method is fully verifiable through comparison of in-situ testing after treatment. Correlation between soil fines content, or soil behavior index (Ic) indicated that the densification effectiveness is significantly affected by soil fines content.

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