LIQUEFACTION MITIGATION BY CHEMICAL GROUTING

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ABSTRACT

Liquefaction and lateral spreading hazards were identified for a waterfront school building expansion project in Newport Beach Harbor, California. Under the high design earthquake acceleration of 0.5 g and shallow water table depth, the top 0.3 m to 4.5 m of loose to medium dense silty sand (SM) and clean sand (SP) were considered potentially liquefiable. To mitigate liquefaction potential, the soil must be densified, drained, reinforced, or solidified. Considering the tight site condition inside the operational building and the shallow liquefiable soil depths, conventional ground improvement methods such as vibro stone columns, compaction grouting, and jet grouting were not considered as feasible solutions. Instead, a design-build chemical grouting program was implemented to form solidified grids under the building to take all static and seismic loads and to mitigate site liquefaction by shear reinforcement. Rather than traditional Portland cement grout, colloidal silica grout was used adjacent to the waterfront in order to conform to the stringent local environmental requirements in the harbor. To the authors' knowledge, this is the first time that environmentally friendly colloidal silica was applied in an industrial-scale grouting project and in an operational building with tight access. The PH natural colloidal silica grouted sands typically provide unconfined compressive strength values in a range of 20 kPa to 250 kPa, which is lower than soil-cement mixed soils. To compensate for the lower UCS values, a high replacement ratio of 53% was used to provide adequate shear reinforcement. This paper provides the site geotechnical investigation, liquefaction analysis, chemical grouting design, and QA/QC programs.

Keywords: liquefaction mitigation, chemical grouting, permeation grouting, colloidal silica

INTRODUCTION

Orange County Coast College, between the Newport Beach Harbor waterfront and the Pacific Coast Highway, is located in an active tourism zone, with high value real estate. In 2007, the college expansion plan included a two-story concrete classroom building, partially on top of an existing one-story garage building, and connected to an existing two-story office and classroom building. At the waterfront, the planned footings are only a few feet away from the seawall. The left side of the building is about 0.3 m from the neighboring building (see Figure 1).

A local geotechnical consultant performed the site investigation with three SPT borings on the college campus. Only one, SPT, B-1, was located within the north corner of the planned building footprint (see Figure 2). The geotechnical report indicated that the generalized soil profile consisted of 0.6 to 0.9 m of silty sand (SM) artificial fill which overlaid native soils. Groundwater was encountered at a minimum depth of 1.2 m. The native soil between depths of 0.9 to 4.6 m generally consisted of loose to medium dense clean sand (SP), with fine contents that ranged from 4.1% to 5.2%. Interbedded clean sand (SP) and thin layers of silty sand (SM) were found at portions of the site to a depth up to 3 m. The geotechnical report indicated that this sandy layer could liquefy during the site design earthquake. The SP layer between depths of 4.6 to 7.6 m was dense. Below the dense sand layer, there was very stiff to hard clay soil (CL) that extended to the bottom of the exploration (12.5 m). To confirm the site soil conditions, the specialty geotechnical contractor, performed seven additional dynamic cone penetrometer tests (DCP) to depths up to 8 m inside the planned building as labeled Pre-DCP-x in Figure 2.

Fig. 1. Waterfront view of the newly finished Orange County Coast College showing the proximity of the neighboring building

Liquefaction analyses were undertaken in general accordance with procedures outlined by Youd and Idriss NCEER (2001), and Martin et.al (1999). Given the earthquake parameters provided by the site geotechnical consultant, evaluations were conducted using date from SPT boring B-1 and seven DCPs based on following design assumptions:

- Design highest groundwater depth: 0.9 m
- Ground water table depth during SPT/DCP tests 0.9 m
- Design earthquake magnitude, Mw: 6.9
- Design peak ground acceleration (upper bond): 0.5 g

Liquefaction-induced settlement analyses were performed in general accordance with Tokimatsu and Seed (1987). The stated procedures were developed as a function of penetration resistance in terms of equivalent SPT N values, determined from correlations to the dynamic cone test results. A typical result of the calculations based on the DCP-Pre_1 is plotted in Figure 3. The liquefaction analysis was performed up to the full boring depth from the current ground surface based on SP117 by Martin et al, (1999). A 64-mm liquefaction-induced settlement was anticipated at the site under the design earthquake. The liquefiable sand would lose its bearing capacity under the building foundations and create high lateral pressure on the existing seawall.

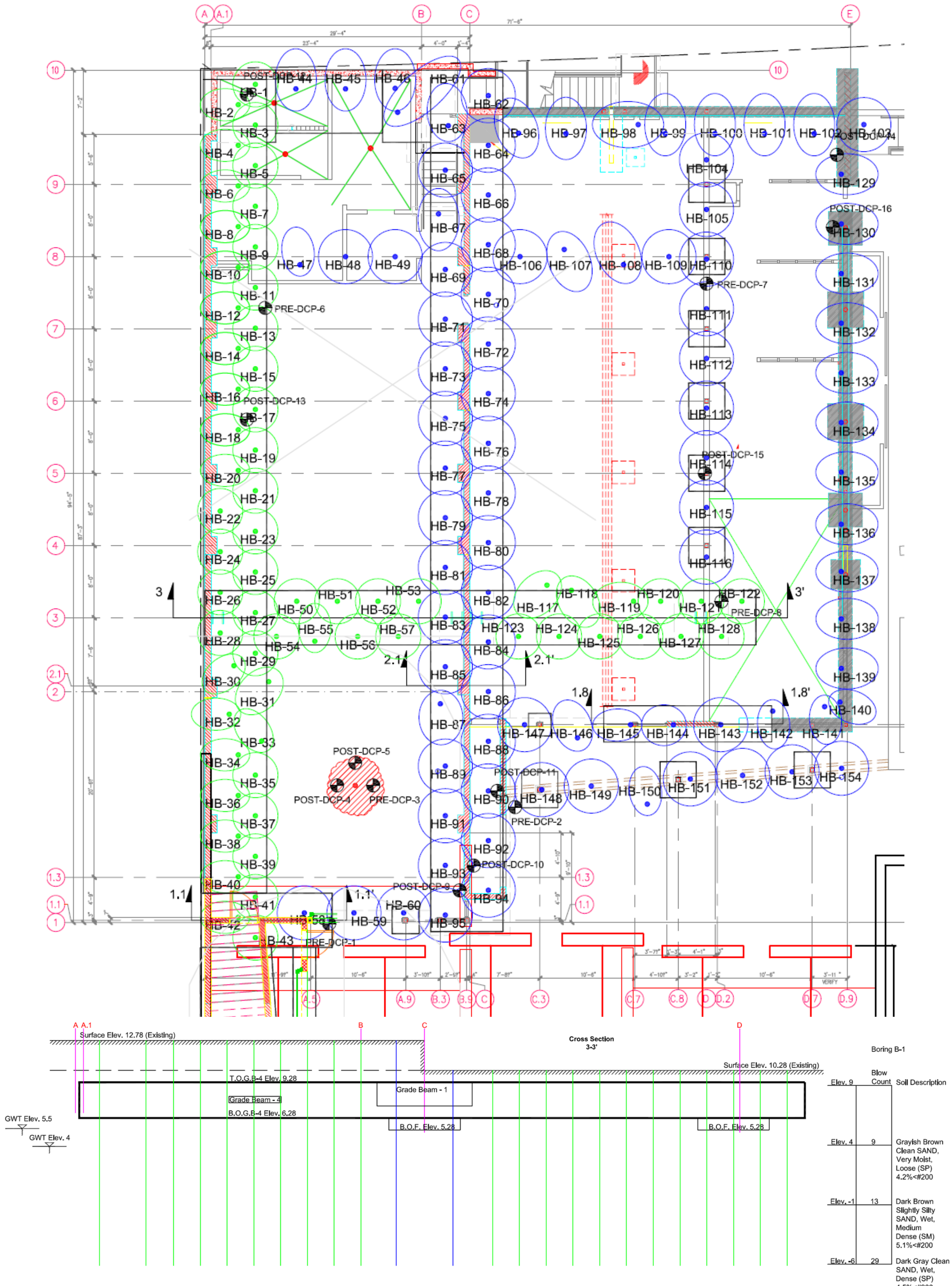

Fig. 2. Chemical Grouting Plan. The blue and green lines in the cross section represent SPGP locations.

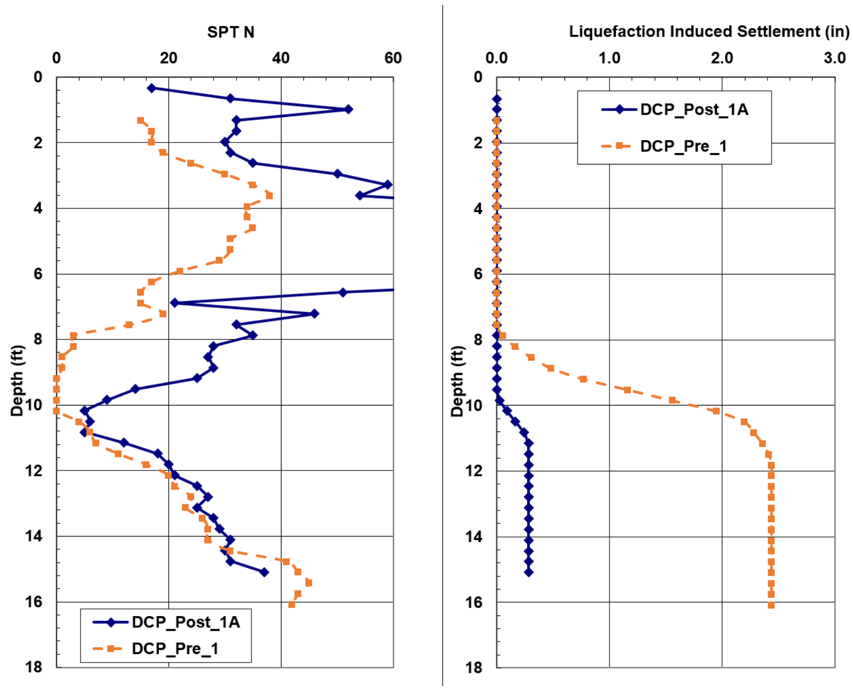

Fig. 3. Liquefaction analysis for pre- and post-improvement DCP values.

SELECTION OF GROUND IMPROVEMENT METHODS

Due to the site liquefaction issues, ground improvement was recommended to mitigate the liquefaction potential to an acceptable level of performance. This would allow the proposed building to be supported on spread footings, some of which are located inside the existing garage. The following ground improvement methods were evaluated to mitigate the site hazards.

Vibro Replacement. The use of vibro replacement (also known as stone columns) provides mitigation of potential liquefaction-induced settlement by densification and drainage, reduction of settlement due to static loading, and improvement of the bearing capacity by densification and reinforcement of the soils (Martin et al 1999). Vibro stone column construction is accomplished by downhole vibratory methods. A vibratory probe penetrates the ground, induced by rotating eccentric weights mounted on a shaft driven by a motor housed within the casing. Stone backfill is introduced in controlled lifts, either from the surface down the annulus created by penetration of the probe (top-feed) or through feeder tubes directed to the tip of the probe (bottom-feed). Although the vibro-stone column is cost effective and a popular treatment method, it could not be applied at this site due to the potential vibration damage to the existing building and neighboring building. The owner wanted to use the building's classrooms during the ground improvement operation.

Deep Soil Mixing (DSM). This method involves introducing a cement-based slurry into the soil and mixing it, using multiple or single axis augers, to create a stable soil-cement mass (or soilcrete) having unconfined compressive strengths ranging from 0.07 to 3.45 MPa (10 to 500 psi) depending on the soil type and cement content (Filz and Templeton 2011). Generally, for DSM the improved soils zone would be continuous along the footing areas. The soil mix columns have two benefits: direct replacement of potentially liquefiable soils and reduction of seismic shear strains in the adjacent soil due to the stiffness of the soil cement grids. However, the tight site access to the soil mixing rig prevented the contractor from applying the soil mixing treatment.

Jet Grouting. This method can replace potentially liquefiable soils with cylinders of hardened soils, or soilcrete, by injecting a cement slurry at depth and eroding the surrounding soils (USACE 1999). Soilcrete columns of more than 4.6 m in diameter can be achieved in loose to medium dense sands. Due to the lack of harmful vibrations, limited space required, and the ability to maneuver safely around buried utilities, the use of this method would be ideal. For these reasons, this method has been used in the past to underpin and rehabilitate existing structures. Jet grouting uses high-pressure fluid to cut the soils, mix in the cement slurry, and lift the soil cuttings to the surface. The soilcrete column can be interconnected with adjacent columns to create soilcrete grids. However, the cost is generally greater for jet grouting in comparison to other forms of soil improvement, especially because of the jet grouting waste disposal.

Compaction Grouting. This involves the injection, under high pressure, of a low-slump, mortar-like grout to compact and displace the adjacent soils. Grout material components can include sand, silt, clay, cement, ground slag, flyash, water, and other admixtures. The strength of the grout is intended only to be greater than existing strengthened soil conditions. The grout does not penetrate soil pores but instead displaces the subsurface soils by forming a homogeneous grout bulb near the grout pipe tip. Compaction grouting can effectively densify loose sands for liquefaction mitigation (USACE 1999). Compaction grouting can be performed within the crowded space with a small drill rig. This technique was adopted as the supplement ground improvement method in those portions of the site where the chemical grouting could not adequately treat silty sands at depth and were least 10 m away from the seawall for the lateral spreading mitigation.

Permeation Grouting is the injection of a fluid grout into voids of granular soil to produce a solidified mass to carry increased load and reduce water flow. Grouts with enough small particle size can flow through voids in sands, gravels, and coarser open materials under low pressure without fracturing the soil matrix (USACE 1999). Cement-based grout, including ultrafine cement, cannot permeate through the fine sand and silty sand at the site.

Colloidal silica was chosen as the primary ground improvement method for the following reasons:

- Accommodate the site access and maintaining classroom operation
- Minimize disturbance to the existing and neighboring buildings
- Permeate through fine sands and silty sands
- Minimize lateral pressure applied to the seawall and ground heave
- Nontoxic and pH neutral to marine life and gels in the presence of brine solutions (i.e. salt water).
- Provide flexibility for utility line layouts below or around footings
- Form solidified weak-sandstone-like grids under the building to mitigate site liquefaction by the shear reinforcement

The ground improvement solution consisted of chemical grouted panels with a cellular geometry with some intermediate independent columns for floor slab support. This cellular panel layout directly supports the foundation static and seismic loads. The stiffer grouted grids will take more shear stress and reduce shear strains in the unimproved soil within the cells, therefore preventing liquefaction (O'Rourke and Goh 1997, Nguyen et al., 2013).

Gallagher, Conlee, and Rollins (2007) reported that colloidal silica is an aqueous dispersion of silica nanoparticles that can be made to gel by changing the ionic strength and pH of the dispersion. Colloidal silica can form a permanent gel that binds soil particles and fills the pore space. Gallagher, et al. (2007) found that colloidal silica grouted soils can significantly increase the deformation resistance of loose sand to cyclic loading and prevent collapse of the soil structure. In Richmond, B. C., Gallagher, et al. (2007) conducted a field-scale demonstration test program of chemical grouting by colloidal silica, and confirmed the liquefaction mitigation effectiveness using blast induced vibrations. This is the first field test in the world for soil liquefaction mitigation with colloidal silica. Conlee, Gallagher, Boulanger, and Kamai (2012) reported the results of two centrifuge tests that were conducted to evaluate the effectiveness of colloidal silica for liquefaction mitigation.

Colloidal silica grouting was considered to be the best liquefaction mitigation method for the waterfront section of the site because it would have minimal environmental impact to the marine life and reactivity with salt water. Colloidal silica gel has been used for flocculation treatment in beverage industries and used in skin certain care products. In the zone 10 m away from the waterfront, conventional sodium silicate was used as the chemical grout because of its higher in-situ unconfined compressive strength.

CHEMICAL GROUTING DESIGN

Based on the site geotechnical investigation, substantial portions of the stratigraphy are amenable to permeation with colloidal silica and sodium silicate chemical grouts. In late 2007, the chemical grouting program was designed beneath the new and existing building footprint (see Figure 2). Borings B-1 and the geologic cross section 3-3' show a typical soil profile in the grouted area. The owner's consultants, California Coastal Commission, and California Division of the State Architect approved the chemical grouting program.

Gallagher and Mitchell (2002) reported that the unconfined compressive strength of loose Monterey No.0/30 sand treated with 5% by weight colloidal silica ranged from about 20 to 55 kPa, while the same sand treated with 20% by weight ranged from 200 to 250 kPa. Based on these results, they concluded that liquefiable sands treated with 5% by weight colloidal silica should provide adequate liquefaction resistance.

Based on experience, the authors determined that the chemical grouting solidified about 53% of the liquefiable sand volume (see Figure 2). The chemicals permeated into the soil formed 1.52 m (5ft, in green) and 1.83 m (6ft, in blue) equivalent columns. The oval circles in Figure 3 represent locations where the grout holes were installed at an angle to avoid obstructions from the existing building structure. The grouted columns connected and formed cells. The design grouted soil could have an unconfined compressive strength of 172 kPa based on laboratory tests. Its shear modulus was 16.0 MPa, interpolated from Young's modulus measured during the unconfined compressive test. The grouted soil has approximately three times the estimated shear modulus of the surrounding silty sand (4.8 MPa), based on correlations from SPT blow count. Per Baez and Martin (1993), the soil shear stress reduction factor was calculated to be 0.45, a reduction factor in calculating the post-treatment sand liquefaction. Using the reduced shear stress in the soil confined by the chemically grouted cells, a revised SPT-based liquefaction analysis of B-1 was conducted and the anticipated liquefaction-induced settlement was near zero.

Since the project completions, Nguyen et al., (2013) investigated the shear strain compatibility of the soilcrete cell and soils inside the cell under seismic loads and pointed out that the Baez and Martin (1993) approach may over-predict the soilcrete shear reinforcement effect. Based on the Nguyen et al. approach, the soil shear stress reduction factor is 0.52 and the post-treatment anticipated liquefaction-induced settlement was still near zero.

CHEMICAL GROUTING SEQUENCE/PROGRAM

The approximate porosity of the medium dense silty sand was 25%, which required a grout factor of approximately 270 liters per cubic meter of treated soil. To achieve 1.83 m diameter grout columns (as shown in blue circles in Figure 2), the design injection dosage was 700 l/m.

The chemicals were injected through 380 mm diameter sleeve port grouting pipes (SPGP) with 0.6 m port spacing. Each 1.2 m of SPGP contained two sleeve ports and required 856 l of chemical grout. Typically, 60% of the planned injection volume was injected through the primary port. After the primary grout reached final set, the remaining 40%, or 342 liters, was injected through the adjacent secondary. This methodology resulted in more uniform and interlocking geometries.

The geotechnical contractor used around 50% colloidal silica with 50% of salt-based brine blended through an automated blending and pumping station. The sodium silicate chemical grout was also blended in an automated station with liquid sodium silicate and ester compound. The percentages of each component were verified by the required gel time and temperature. Based on the geotechnical contractor's experience in similar soils, the grout criteria in each port was established as:

- Design volumes are achieved
- Sustained grouting pressures of 10.3 kPa per 0.3 m of overburden with a corresponding flow rate of less than 1.89 l/min. (0.5 gpm) are present at primary locations and 13.8 kPa per 0.3 m at a flow rate of less than 1.89 l/min. at secondary locations
- Surface or utility heave exceeds 3.2 mm
- Any seawall movement, measured by crack gauges and total station surveying
- Grout communication to the surface or utility.

To avoid the grout fracturing into the soil, the grout flow rate (less than 3 l/min) and grout pressure were set at very low (less than 300 kPa) values.

DRILLING AND GROUTING OPERATIONS

In late March 2008, the ground improvement work began. A hydraulic track drill was used to install the 13.8 cm diameter SPGP. After the low strength annulus grout used to secure the SPGPs had set (typically 12 to 24 hours), chemical grouting commenced.

The chemical grout was supplied to the grout process monitors (GPM) through two separate chemical lines. Just prior to passing through the GPM, all grout components are blended at a mixing tee and check valves. The GPM consists of digital electromagnetic flow meters and fluid isolated pressure gauges in a manifold arrangement of grout and water, bypass, and sampling valves.

QUALITY CONTROL AND ASSURANCE

Daily field quality control consisted of calculating the grout mix proportions and obtaining gel samples every 60 minutes, or as conditions changed determined by grouting technicians.

It was planned in the original design that the grouted soil strength was to be measured by its unconfined compressive strength of no less than 172 KPa (25 psi) based on the lab tests or reached the equipment SPT $N_{1,60}$ no less than 30. Sixteen DCPs were performed after the grouting work (see Figure 3) as Post-DCP-x.

The grouting program resulted in a limited increase in dynamic cone penetrometer (DCP) blow count values. However, this was partially expected, as Gallagher, Conlee, and Rollins (2007) found that post-improvement SPT, CPT tests, and shear wave velocity profiles did not appear to successfully capture the improvement provided by colloidal silica grouting. Furthermore, their field test in Richmond, B. C., still observed reduced settlement during controlled sequential blasting despite limited change in exploration results. A typical comparison of the pre- and the post-DCPs is shown in Figure 3.

In addition to post-treatment DCP testing, the geotechnical contractor also drilled verification borings and sprayed phenolphthalein on the boring cuttings to verify the chemicals reached the design diameters. The tests and borings were conducted at least every 46.5 m^2 of the grouted area. Coring was attempted in the colloidal silica treatment zone; however, intact samples were irrecoverable.

Fig. 4. Test section of chemical grouted sands.

To further investigate the effectiveness of the chemical grout dispersion, HBI excavated a test pit, 2.1 m below the ground surface (Fig. 4). During excavation, pocket penetrometer readings were taken every 300 mm of excavation depth and the observed UCS of the colloidal silica grouted soils were all greater than

172 kPa. Three large cube samples were retrieved and sent to a geotechnical lab. Only one specimen was successfully trimmed and obtained UCS strength of 255 kPa.

A few DCPs at the intersection of the planned grout boundaries (the intersections of the 1.8 m diameter circles in the design) showed low values in a 0.30 to 0.61 m thick silty sand layer at the depth around 3.05 to 3.66 m although the chemicals reached the design permeation diameter detected by phenolphthalein color reaction. A compaction grouting program was added to further densify this silty sand layer. The use of compaction grouting induces radial displacement and stiffening of the surrounding soil at the point of injection (tip of grout casing).

In the zones away from the waterfront, conventional sodium silicate based with grout was used for chemical grouting, which yielded strength over four times stronger than the colloidal silica grouted sands. One core hole was successfully drilled in the strong soilcrete and obtained near full core recovery. Both the core samples and cube samples of the sodium silicate grouted zones achieved UCS higher than 1.03 MPa .

The building structural footings were directly cast in the chemical grouted soils (see Fig. 5). The cohesion of the chemical grouted soil prevented caving in the footing excavation in the originally flowable sands.

Fig. 5. Spreading footings were cast in the chemical grouted sands.

MONITORING

Surface monitoring was accomplished by utilizing rotating horizontal lasers that have an accuracy of +/- 1.6 mm. Surface monitoring locations were within the immediate vicinity of the injections near the top of the sleeve port grout pipe. The seawall was monitored by the crack gauges attached to the joints of the seawall concrete panels and were regularly monitored with optical surveying. The 154 grout holes were drilled and grouted with 466 m^3 of chemicals without heaving the building nor moving the seawall.

SUMMARY

Chemical grouting with colloidal silica was used to mitigate the liquefaction and lateral spreading hazards for a waterfront school building expansion project in Newport Beach Harbor, California in 2008. This is the first time that colloidal silica was applied in an industrial scale grouting project and in an operational building with tight access.

The chemical permeation grouting program was performed adjacent to the seawall with near neutral PH value colloidal silica and salt-based brine, safe for marine life, and satisfied the local stringent environmental standards. Grout-stabilized soil grids were formed under the building, mitigated site liquefaction, and supported the building static and seismic loads.

Although the project experienced difficulties in verifying the weak strength of the colloidal silica grouted soil by coring and DCP testing, block sampling and pocket penetrometer testing was adopted to verify the material strength.

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