Abatement of soil liquefaction under existing structures

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ABSTRACT: Liquefaction mitigation under a functioning structure represents a challenge to the engineer and the geotechnical contractor and accentuates cost, time, and disturbance of the facility's use. Access is the most important aspect of such a project. Two case histories where soil liquefaction was required to be abated under existing structures are presented.

In Case One, a filter building was underlain by a 3.7 meter liquefiable sand layer. The approach taken to reach the target soil was to drill horizontally about 28 meters, and extrude compaction grout in 1.5-meter stages to densify the soil.

In Case Two, a liquefiable sand layer and a silty/clay fill layer were identified under an existing multi-story building. The building was supported on old wood piles. Two systems were selected for this project; permeation grouting for sands and lense grouting for silty/clay layer.

In conclusion, it was possible, using cost-effective methods, to mitigate the risk of liquefaction under existing structures without interrupting the use of the facilities.

1 INTRODUCTION

Improvement of liquefiable soils follows a number of methods and techniques that are well established in the industry. These available methods and techniques become very limited and restricted for locations where a structure is in place but underlain by liquefiable soils. In a situation where a structure must continue to function, the restrictions multiply. Liquefaction mitigation of a soil layer under a functioning facility represents a challenge to the engineer and the geotechnical constructor.

Almost every project for abatement is unique in its approach. Consultation and close coordination between the owner of the facility, the geotechnical engineer, and the specialty contractor are of tantamount importance for these projects.

This paper presents two case histories where grouting methods were successfully used under existing structures for the abatement of soil liquefaction in a cost-effective way without interfering with the operation of the facilities.

2 CASE ONE: COMPACTION GROUTING FOR SAND DENSIFICATION

Compaction grouting has been successfully used for sand densification for liquefaction mitigation (Mitchell & Wentz 1991). The process involves the controlled injection of a stiff sand/cement mixture to volumetrically displace the soils to increase its density. Grout stiffness is related to the slump value, ASTM C143-78. For a controllable grouting process, this value should be maintained to less than 5 centimeters (Al-Alusi 1994). The success of this process is hinged on having the ability to maintain the grout near the point of injection by volumetrically displacing the soils without fracturing them. Theoretically, compaction grouting loses its significance upon fracturing the soils. Ground surface monitoring for vertical and/or horizontal displacements (in cases involving slopes) is a must for every application.

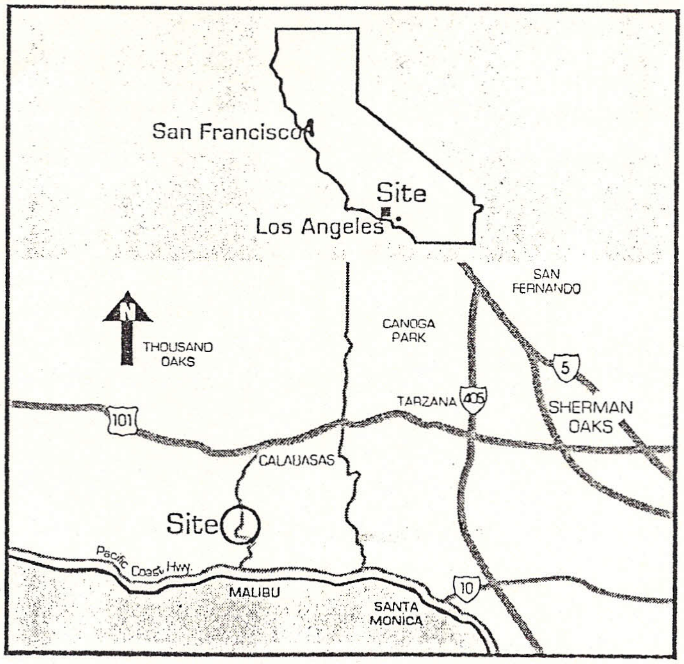

Fig. 1: Site Location

2.1 Site conditions and liquefaction potential

An exploration to ascertain the soil conditions for expanding a wastewater reclamation plant in Los Angeles, California (see Figure 1), identified a liquefiable soil layer approximately 3.7 meters thick. The expansion program included adding more filters to the existing filter building, which was 25 X 30 meters in plan. Although the new filters could be founded on a deep engineered fill, the liquefiable soils under the existing filters building needed to be approached differently.

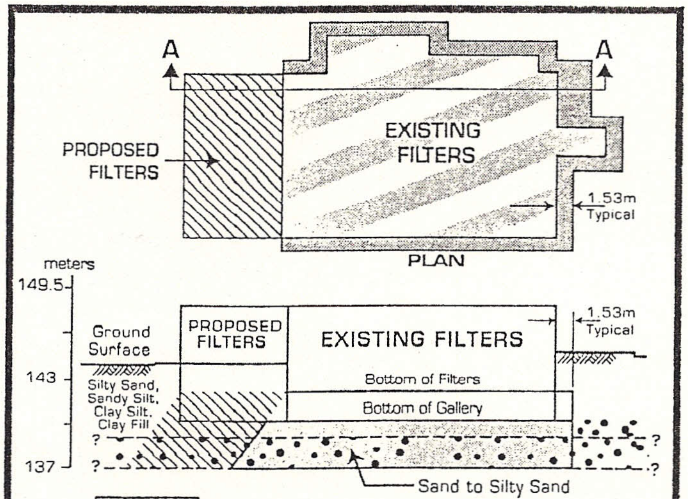

Fig. 2: Plan and Section of Areas Designated to be Densified using Compaction Grouting

Bordering the structure on three sides were other facilities in such close proximity that made it impossible to excavate for a horizontal drilling operation. The remaining side of the structure that was available for horizontal drilling was a short side. An excavation of about 10 meters was made with conventional soldier beams and lagging. The ground water level was dropped using a traditional pump and sump method.

The project site was located in a seismically active area of Southern California. The dominating fault, located approximately 4.8 kilometers miles from the site, was considered to be capable of generating a magnitude 7.5 earthquake. Probabilistic seismic risk analyses estimated the Peak Ground Acceleration at 0.35g and 0.6g for a design life of 100 years. The densification process became critical due to the building's sensitivity to settlement (Harding Lawson 1990).

Soil borings and Cone Penetrometer Test (CPT) soundings encountered a 2.5-meter layer of relatively loose sand. The geotechnical engineers' site seismicity evaluation and cyclic shear strength study indicated that this layer was potentially liquefiable when subjected to the ground accelerations of design-earthquakes, see section in Figure 2.

Seismically-induced settlements were found to be likely to occur in the loose to medium dense layer (3.7 meters thick). These settlements were calculated to be on the order of 5 to 13 centimeters using methods proposed by Tokimatsu and Seed (1987).

2.2 Approach, Drilling, and Grouting

The project specifications called for extending the densification process for a 1.5-meter strip around three sides of the building, see Figure 2. For this strip a single row of vertical injections, spaced at 1.5 meters, was used. The installation of these injections served to confine the grout under the building and to provide correlations between test results and the amount of the injected grout (grout take) under the building for quality assurance. The same termination criteria that were used for horizontal grouting, as discussed below, were used.

Two rows of horizontal injections were installed 1.8 meters apart, and centralized in the middle of the liquefiable zone, see Figure 3. The spacing between injections was about two meters. Each injection was extended horizontally for the full

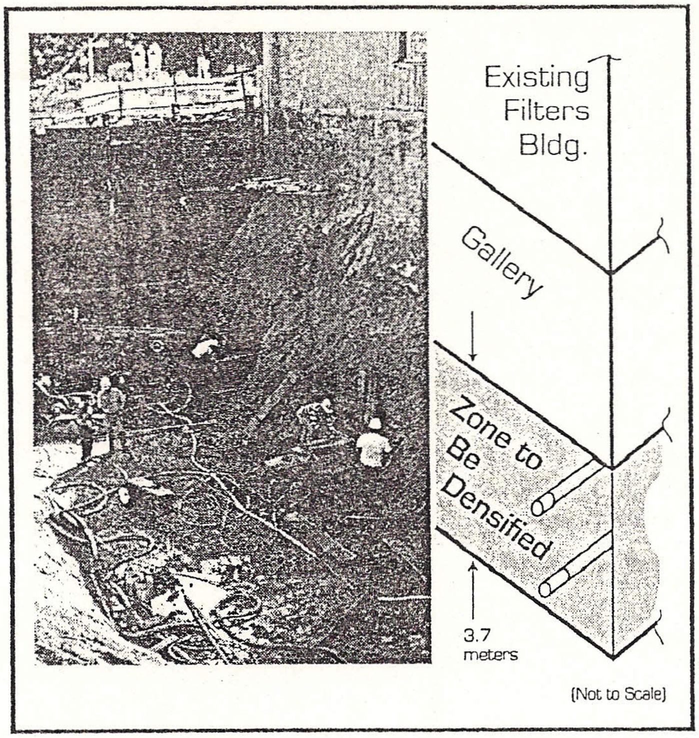

Fig. 3: On-Site Details During Compaction Grout Operation and Section through the area to be compacted

length of the existing filters, reaching a maximum of 26 meters. A partial drilling of 3.5 centimeter hole was first made with the aid of drilling foam. Upon completion of the hole a nominal five centimeter close-ended pipe was driven in.

Compaction grout extrusions started at the far end of the excavation side. A staging of 1.5 meters was used. The sequence of grouting was by the primary-secondary injection method, alternating injections between the two rows.

At each stage, grout was injected until one of the following criteria was met:

1. A maximum pressure of 5500 kPa is reached at the point.
2. Inception of ground or structural uplift.
3. A quantity of 400 liters is pumped at a given stage.

Throughout this operation the high pressure criterion controlled the process. Grout takes ranged between 11 and 37 liters. A total of 19 horizontal injections were completed.

Throughout the grouting operation, horizontal and vertical, a laser survey system was employed to monitor any uplift in the structure or ground surface. None was detected at any stage.

2.3 Test results and discussions

Standard Penetration Tests were utilized to evaluate the effectiveness of the grout densification process around the perimeter of the existing building. A comparison of test results conducted before and after the densification process is shown on Figure 4. Since the SPT results were erratic, a correlation between the SPT increase and the calculated density increase caused by the added grout mass in a given volume of soil was made using published correlations (Holeyman & Wallays 1984, Winterkorn and Fang 1991, Bowles 1982). These calculations revealed that the relative density of the soil was raised from about 45 to 70 percent, and that the SPT values were raised from a range of 10 to 20 blows to a range of 30 to 40 blows. These

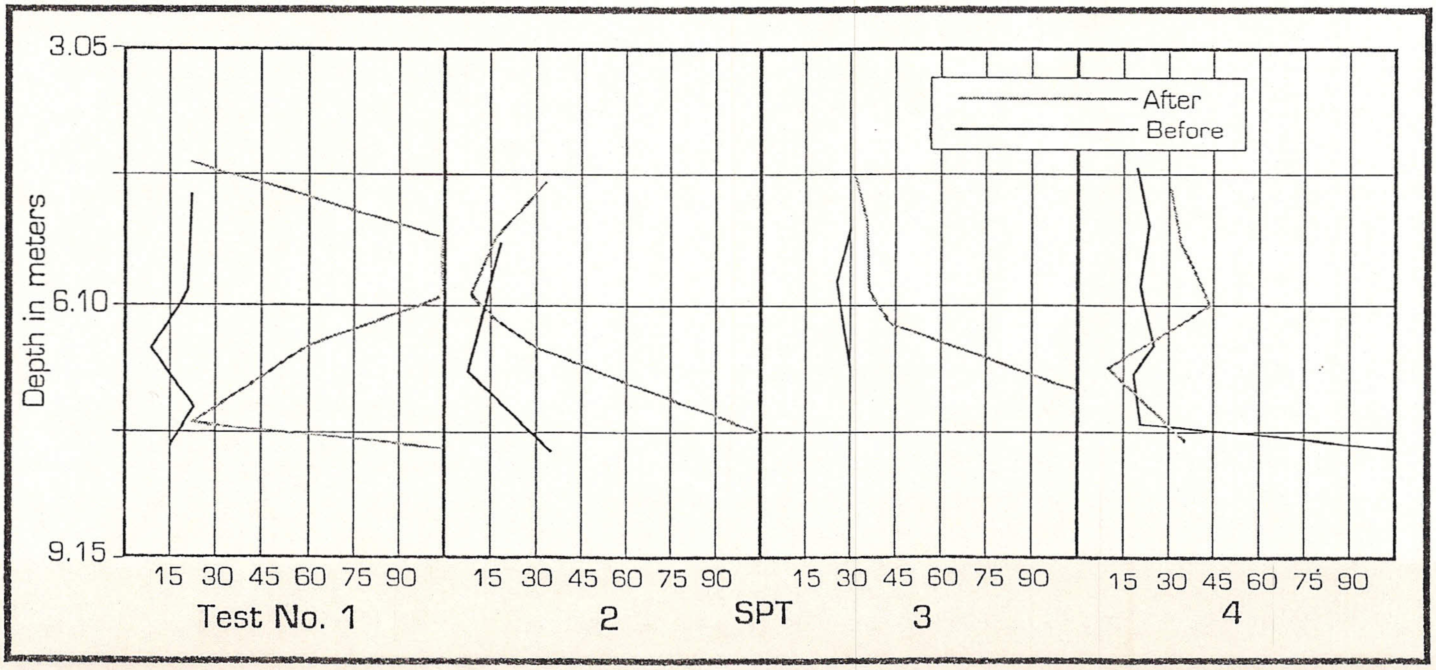

Fig. 4: Standard Penetration Test Results for After and Before Compaction Grouting

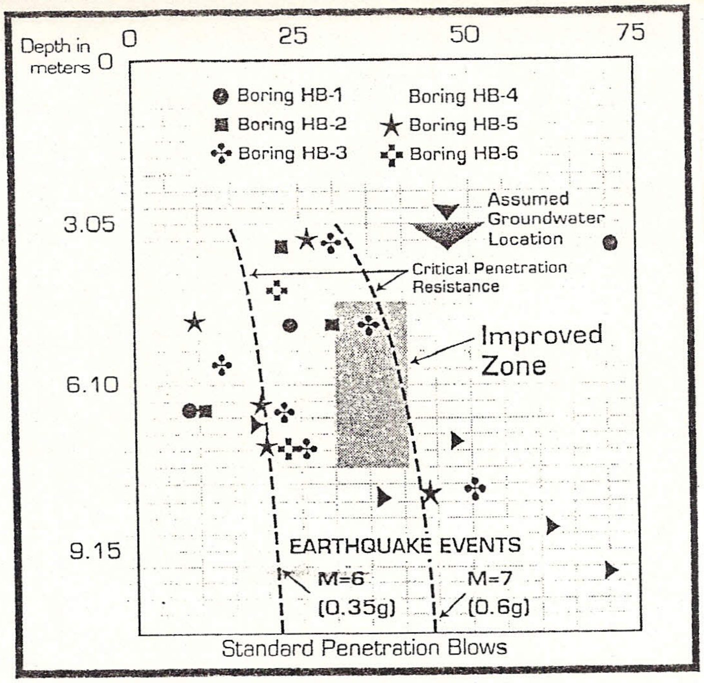

Fig. 5: Soil Improvement Relative to Earthquake Events

results indicated that the targeted soils were improved to well above the critical penetration values required for a magnitude 6 earthquake event, and at or near that of a magnitude 7 earthquake event, see Figure 5.

It was not possible to perform blow count tests or any sounding tests, such as static cone penetrations, under the structure. Although other sounding tests, such as ultrasound and cross-hole geophysical tests, were available, it was concluded that such methods would not reveal sufficient useful results, especially in zones where known gravels existed. The effectiveness of the grouting program in this zone

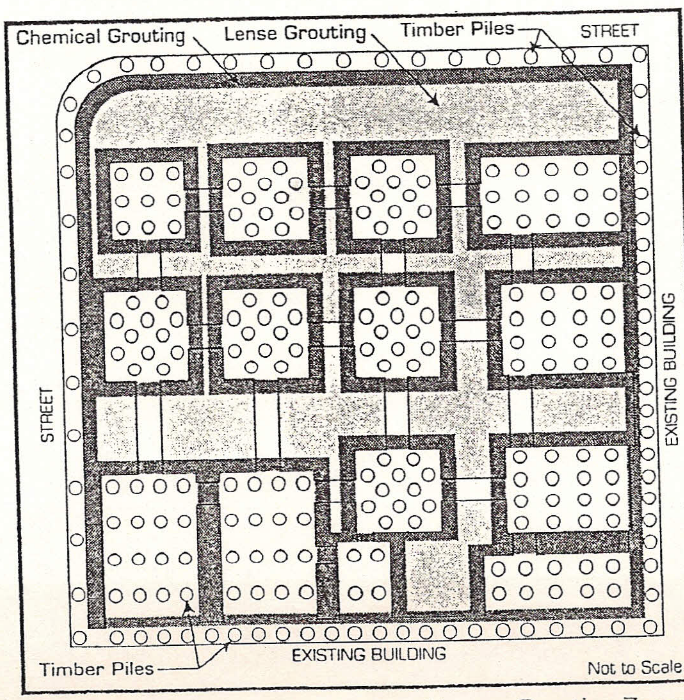

Fig. 6: Plan showing Chemical and Lense Grouting Zones

was evaluated by comparing the amount of grout injected in various stages, which was calculated to be about three percent of the total volume of soil, to the theoretical density improvement.

3 CASE TWO: CHEMICAL AND LENSE GROUTING AROUND EXISTING PILES

Liquefiable soils, due to their particle size distribution and resulting high permeability, are usually amenable to chemical/permeation grouting. Chemical grouting, simply put, is a pure permeation grouting, which utilizes two or more material components whose chemical reaction results in a hardened matrix within the soil mass. In saturated soil masses, the grout is expected to displace water from the soil pores. In partially saturated soils it displaces air and water. Confinement and control of the grout are two key elements of a successful chemical grouting job. They can be achieved by working in a designed pattern, with or without a prescribed gel time (i.e., time required after mixing to start hardening).

Lense grouting is a soil fracturing technique where a cement slurry grout is injected at an initial high pressure of 700 to 3000 kPa, then reduced until a predetermined amount of grout is injected. In man-made fills and alluvial deposits, near horizontal fractures are achieved using engineered tips at the bottom of the injection pipe to facilitate fracture initiation (Al-Alusi 1994).

3.1 Building and Foundation Conditions

A seismic upgrade program was to be implemented for an eight-story concrete and masonry structure measuring 25 x 25 meters at the basement level. The building was located on a corner in the downtown area of San Francisco, California. It was erected in 1907 in the area that had experienced ground failure during the April 18, 1906 earthquake. During the Loma Prieta earthquake of October 17, 1989, the building sustained structural damage. The foundation consisted of pile groups, as shown on Figure 6. Timber piles, 46 centimeters in diameter and 10+ meters long, are believed to have supported this structure. The pile caps and floor were of reinforced concrete construction. Below the basement bottom, the soils were predominantly sandy silt with clay and some rubble fill (layer 1), for a distance of 2.5 meters, then a loose gravelly

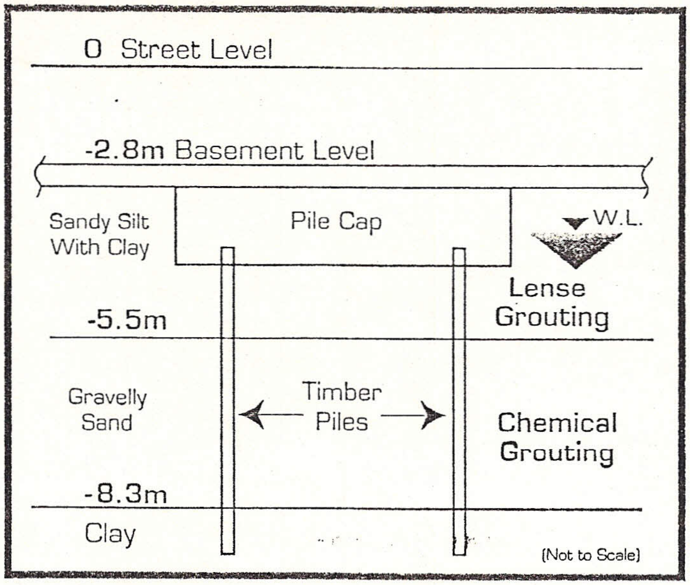

Fig. 7: Typical Section

sand layer which extended another 2.8 meters (layer 2), where a clay layer started, see Figure 7. The blow count (standard penetration test) was between 1 and 6 for layer 1, and between 2 and 13 for layer 2. Based on the very low blow count and high ground water level at which was at about the basement level, it was determined that liquefaction was most likely to occur in these sand formations during an earthquake comparable to the design event of magnitude 7.

3.2 Restrictions and criteria of treatment methods

Compaction grouting could eliminate the potential for liquefaction of the soils below the basement floor slab. However, because of the presence of the timber piles and the likelihood that some of them were in a partially deteriorated condition, coupled with the high pressures inherently associated with compaction grouting, this method was deemed to be unacceptable even though it would have been the most cost effective.

In order not to affect the timber piles by high grouting pressures, a chemical system was selected that would solidify the sands of layer 2 with a low strength grout to render the material non-liquefiable. A sodium silicate based grout was used with a gel time of ten minutes and an ultimate unconfined compressive strength of the grouted sand of 190 kPa was selected and installed. Because layer 1 was not susceptible to chemical grouting, this method was limited to layer 2 only. The exact location of the piles within each group (cap) was unknown. Few attempts were made to define the pile locations, which revealed that the actual locations were not as shown on the available plans. Consequently, the design of the chemical grout system was made to confine each group in an isolated cell, then inject more grout within the confined cell to refusal, see figure 6. Refusal is defined as a grout pressure of 1300 kPa or a predetermined amount of grout based on the theoretical volume of voids within the cell using a porosity of 0.35.

The procedure followed was that of a closed end pipe vertically jetted into the ground using water. At the proper depth, grout was injected in stages of 30 centimeters in the vertical direction for the full 2.8 meter depth which was required (between elevation -5.5 and elevation -8.3). By following a primary and secondary injection pattern, a wall of chemically grouted soils was installed around each group of piles. see figure 6. Voids, caused by the difference in rigidity between slabs, soils, pile caps, and the generation of settlements of the underlying mud of the San Francisco Bay, were suspected to be in this area. Before injecting the chemical grout, a probing program was adopted to look for voids immediately below both the basement floor slab and the pile caps. Encountered voids were filled up and the soils were tightened.

After completing the chemical injections, a lensing program for layer 1 was initiated. Lense grout injections were spaced on a 1.5 x 1.5-meter grid, covering the space between the pile caps. These injections were extended vertically between the bottom of the basement floor slab and the top of layer 1, elevation -2.8 to elevation -5.5. A vertical staging of 30 centimeters was used. In each stage 28 liters of 12-sack cement slurry was injected (one sack = 42.7 kilograms). The initiation pressure was between 700 and 3000 kPa, then dropped to 200 to 450 kPa. There were few instances where these pressures were not achieved until several tens of liters of grout were emplaced, indicating the presence of a void.

3.3 Results and Discussions

Laboratory prepared samples of site sand with chemical grout, indicated an unconfined compressive strength well above the required 190 kPa at 28 days, see Table 1. Grout takes and pressures for each injection were checked to assure the proper installation of each stage. During the primary chemical injections, the feedback pressure at the point ranged between 70 and 200 kPa. The secondary injections ranged between 70 and 500 kPa, indicating grout presence within the nearby soils.

Table 1. Chemical Grout Strength

Sample	Age, Days	Unconfined Compressive Strength, kPa
1	27	276
2	28	262
3	28	283
4	30	310

For the lense grouting program, no testing of the grouted soils can be practically made. Quality assurance was achieved by monitoring the grout amounts and pressures at the point of injection.

4 CONCLUSIONS

The risk of potential liquefaction can be eliminated or at least reduced in certain cases using cost-effective methods without interrupting the functions of the facility. The increased awareness of owners, engineers, and public agencies to the soil liquefaction potential beneath their projects usually makes them search and explore available methods for an acceptable solution.

Geotechnical grouting techniques have been available for quite some time (chemical grouting for about 150 years, compaction grouting for about 30 years, and lense grouting for about 15 years), but their adaptation and use for the abatement of soil liquefaction has been limited to the last 10 to 15 years. The use of compaction grouting in horizontally driven casings proved to be a workable solution. At least as far as the author is aware, such an approach has never been tried before to the extent used in Case 1 above.

In Case 2, the use of a combination of grouting methods other than compaction grouting was dictated by the presence and condition of the timber piles and the soil conditions. The goals of both projects were successfully achieved to the point where the subject soils were made non-liquefiable under design-earthquake events.

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