LIQUEFACTION MITIGATION BY SITE IMPROVEMENT--
A CASE STUDY

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SUMMARY

Strengthening soils can be as effective a way of mitigating seismic problems as strengthening a building. The potential for liquefaction at a California hospital site was preventing further expansion plans. The existing single story structure, constructed before hospital code changes in 1971, was founded on piles extending through the liquefiable deposits to underlying firm soils. The effects of the pile driving operations provided sufficient increase in liquefaction resistance to allow adding floors above the existing hospital. Due to environmental concerns, compaction grouting techniques were used to strengthen the soils for an adjoining addition which could not be founded on piles.

INTRODUCTION

Strengthening an existing building to mitigate the possible effects of a future earthquake is common practice. The analogous concept of strengthening the soils at a site should also be considered as a viable means of insuring engineering performance. If soil support capability is inadequate, common practice is to provide foundation support that is independent of the weak soils using piles or caissons or to relocate the construction site. Where seismic liquefaction is a potential hazard, strengthening the existing soil is an effective mitigation procedure because it can also prevent the occurrence of lateral displacements which piles and caissons cannot. This paper describes circumstances which necessitated strengthening soils to mitigate against liquefaction and the means by which it was achieved. The site studied involved a hospital and planned hospital expansions. Increasing soil resistance to seismic liquefaction was obtained in two ways, only one of which was considered in the initial design.

PROJECT SETTING

Following the 1971 San Fernando earthquake which caused the collapse of portions of the Veterans Hospital and extreme damage to the new Olive View Hospital, the State of California introduced new code requirements for the design and construction of hospitals. The seismic interpretation of this code, as administered by the Department of Mines and Geology for the Office of the State Architect, requires that in highly seismic areas the foundation and structures be designed for large ground motions of up to 0.5g

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in rock. The code also requires that if a structure is modified by more than 10 percent the entire structure must be made to satisfy the new requirements. The code is not retroactive. To the present time no probabilistic assessment of the likelihood of liquefaction has gained regulatory acceptance.

Before the more stringent code requirements came into effect, a single story hospital was constructed in South San Francisco on a pile foundation. The design included column studs extending through the roof to accommodate a possible addition. A foundation investigation had been completed by another geotechnical consultant before any site preparation was undertaken. The soil profile was found to consist of approximately 20 ft (6 m) of relatively clean medium fine sand overlying approximately 20 ft (6 m) of increasingly silty sand and silt with variable amounts of organic material. At a depth of 40 ft (12 m) the profile changed to alternating stiff clay and dense sand strata which produced refusal during standard penetration tests. Site preparation consisted of placing 10 ft (3 m) of engineered fill to reach grade. The structure was founded on 16 in (400mm) square prestressed concrete piles approximately 65 ft (20 m) in length which penetrate into the stronger soils. The use of piles was intended to minimize the effects of possible settlement of the silty layers and provide protection against liquefaction settlement. The pile layout and the location of soil borings are shown in Figure 1.

Construction of a second structure was begun prior to adoption of the new hospital code but was denied occupancy for patient care due to potential liquefaction problems and has instead been used as a medical office building for which the regulatory requirements are much less severe. As additional beds were still required for the hospital a complete review of the potential for liquefaction was considered necessary to provide answers to two related considerations. These concerned the feasibility of using the pre-existing extension plans if the liquefaction potential were low and what foundation preparation would be necessary for a separate addition at the same site if liquefaction were a cause of concern. The availability of alternative sites and the continuing and rising need to reduce increases in health care costs were such that the existing facility would continue to be used.

INDIRECT SOIL STRENGTHENING

As this liquefaction potential analysis did not consider either the effects of placing additional fill on the site or of driving displacement piles the authors were asked to review the original data and estimate what these effects might be. The results of this study could be of significant importance in the hospital's expansion plans if it could be shown that the combined effect of the fill and the pile driving had strengthened the soil sufficiently to prevent liquefaction even under the more severe design criteria. The effect of the fill and pile placements was evaluated using the relationship of Marcuson and Bieganousky (Ref. 1):

$$N = -9.4 + 3.0\ (OCR) + 0.23\ (\sigma') + 0.0046\ (D_r)^2 \qquad (1)$$

where N is the standard penetration value
OCR is the overconsolidation ratio
σ' is the effective vertical stress in psi and
D_r is the relative density in percent.

Pile driving is believed to result in an increase in the horizontal soil stress and consequently a change of the at rest pressure coefficient K_o. Under normal depositional conditions this value could be expected to be 0.4 to 0.5. The effect of displacement piles would increase the horizontal stresses such that the pressure coefficient would rise to approximately 1.0. This was a tenuous assumption for this specific project as the piles are widely spaced. Using data after Peacock and Seed (Ref. 2) relating the horizontal pressure coefficient and overconsolidation ratio it was assumed that the changed conditions would be equivalent to an increase in the overconsolidation ratio from 1 to 4. The change in the effective vertical pressure caused by the fill could be directly computed. No change in D_r was assumed.

The mean values of the penetration resistance before construction and the estimated post-construction values are shown together with the target values required to resist liquefaction on Figure 2. To check the reliability of the predicted values subsequent borings were made. The results of these additional tests are summarized on Figure 3 where the mean curve of the actual test results is compared with the predicted mean curve. The comparison in the upper liquefiable sand layer is excellent. Regulatory approval has subsequently been given to construct the originally planned additional floors to the hospital.

DIRECT SOIL STRENGTHENING

Along with the floors being added to the hospital a further expansion into an area where fill had been placed during original construction was planned. Because the existing hospital would continue to function during construction, driving displacement piles, although recognized as a sure foundation solution, was not feasible. Removal and replacement of the liquefaction susceptible soils would require constructing deep retaining structures next to the existing building with an extensive dewatering system which was not economically feasible. Alternatively, compaction grouting methods were available which would provide the increased resistance to liquefaction more economically while creating minimum disturbance to the hospital operation. In compaction grouting, a thick grout mixture is pumped into the soil under high pressure forming a grout bulb which compacts the surrounding soil by displacement. The grout is not designed to fracture the soil or to flow freely. If fracturing does occur further injection of grout at that depth does not increase the degree of compaction but instead heaves the surrounding soils.

Test Program

To ascertain the effectiveness of the compaction grouting procedure and develop specification requirements such as grout quantities and the spacing between injection points, a pilot test grouting program was undertaken. Before grouting started both continuous standard penetration test (SPT) exploratory borings and cone penetration test probes were made. The results of these tests, summarized on Figure 4, provide the basic data against which the effectiveness of the compaction could be measured. The cone penetration results were converted to equivalent SPT values to allow joint presentation of the results of the test data and direct comparison with the empirically developed liquefaction criteria based on corrected SPT (N) values.

In the test program, grout was injected into the zone of liquefiable deposits in stages from the top downward. Grouting at a given level began after grout at the preceding level had set. The results of the test program showed a grout injection pattern with 8 ft (2.4 m) center to center spacing and an average volume of 2.7 cubic feet of grout per foot injected (0.25 m^3/m) at injection pressures between 400 and 450 psi (28 and 32 kg/cm^2) was most effective. The comparison of average corrected blowcounts for the test area pattern after grouting with the average blowcount values before grouting is shown on Figure 5. The grout was composed of silty sand, Portland cement, and water mixed to a stiff consistency with a slump limited to one to two inches.

Production Grouting

After the successful completion of the test program and approval of the procedures by the State Architect's Office a contract was let for the production grouting. Some changes in grouting procedures were accepted after the grouting contractor was unable to meet project schedules using the test program methods. In production grouting, soils were grouted in stages from the bottom upward. An upper level was grouted first from depths of 17 ft (5.2 m) to 7 ft (2.1 m) after which the remaining soils were grouted starting from the bottom of the liquefiable sands at a depth of 34 ft (10.4 m).

To prove the effectiveness of the production grouting an additional boring and cone penetration program was undertaken. The results of these together with data from the test area after grouting are shown on Figure 6. The scatter of data in this figure is much more extensive than the data in Figure 4 and reflects the variable nature of the soils over a wider portion of the site. The mean values before and after grouting are shown on Figure 7. Production grouting has been completed over most of the site. During this time several problems have been encountered, the more important of which are discussed below.

Discussion

Compaction grouting as applied to this project is possibly not a correct description. The volume of grout injected which has not resulted in directly measured heave is approximately 5 percent of the total volume of the soil in the grouted zone. This results in an estimated increase in relative density of about 20 percent which is not sufficient to account for the greatly increased resistance to liquefaction. The volume of grout injected is approximately 3 times the volume displaced by the piles in the original building when normalized to the same total soil volume. As both achieved approximately the same increase in the soil penetration resistance (N) the increase due to compaction grouting must be primarily the result of changed stress conditions in the ground rather than a direct increase in soil density.

Heaving of the ground surface occurred during the grouting to a much greater extent than anticipated. In retrospect this may have been due to difficulties with the measuring devices used. When it was recognized that the heave from a single injection point could occur over a wide area a more stringent control of heave was implemented. Heave contours in inches are shown on Figure 8 for one of the earlier portions of the project where heave was excessive and for a later section where heave was under tighter control.

The grouting procedure in each section was to first inject grout in perimeter injection points and then continue toward the center. This resulted in greater heave in the center of each section. In some cases water and soil flowed from injection points that had yet to be grouted while grout was being injected nearby. This clearly showed the effectiveness of the compaction grouting and the sequence of grouting the perimeter points first.

CONCLUSIONS

This project demonstrated the effectiveness of soil strengthening using two procedures. These are:

1. strengthening produced by driving displacement piles; and

2. strengthening produced by compaction grouting.

Compaction grouting is an effective means of strengthening soils when conditions require the capability to resist both future settlements and liquefaction. The total cost of the grout compaction program was less than half the cost of other proposed means of site improvement. Because neither vibratory nor impact equipment were required, the compaction grouting procedures followed were also the least disruptive and permitted the hospital to continue functioning.

REFERENCES

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2. Peacock, W. H., and H.B. Seed (1968), "Sand Liquefaction Under Cyclic Loading Simple Shear Conditions," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 94, No. SM 3, May, pp. 689-708.

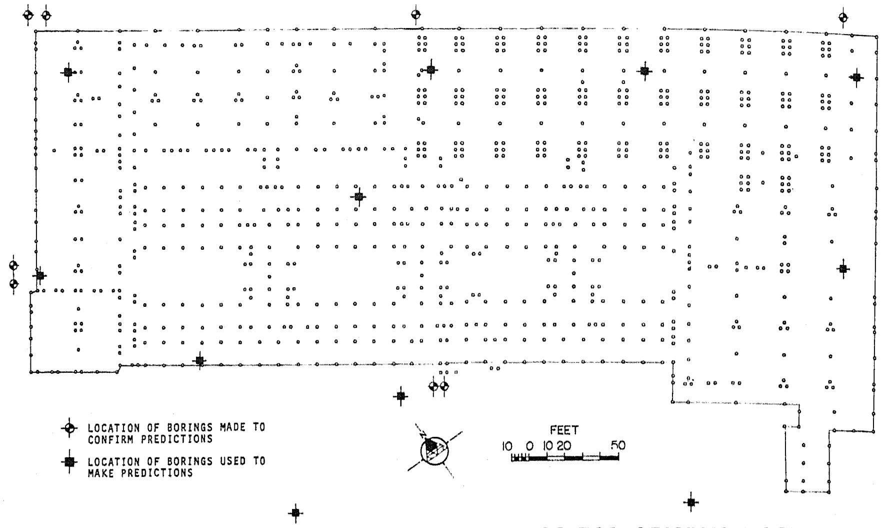


Figure 1: PILE LOCATION AND LAYOUT OF BORINGS FOR ORIGINAL HOSPITAL BUILDING.

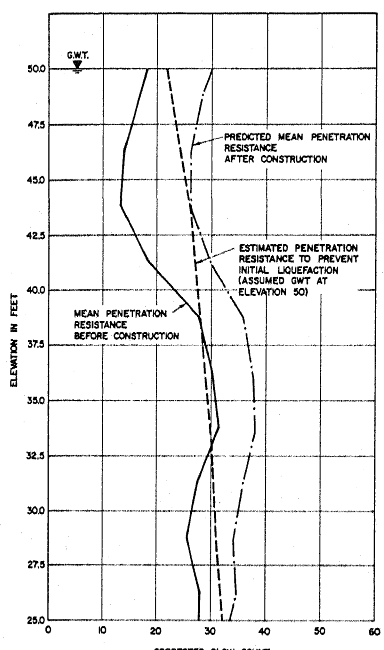


Figure 2: MEASURED MEAN BLOW COUNTS BEFORE CONSTRUCTION AND PREDICTED POST-CONSTRUCTION BLOW COUNTS FOR ORIGINAL HOSPITAL BUILDING.

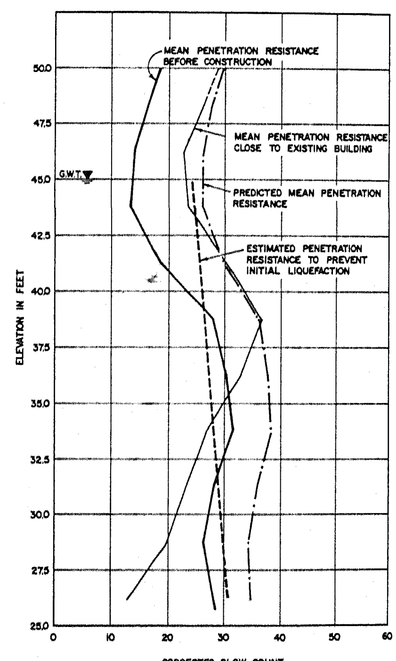


Figure 3: MEASURED MEAN BLOW COUNTS AFTER CONSTRUCTION ADDED TO Figure 2 DATA.

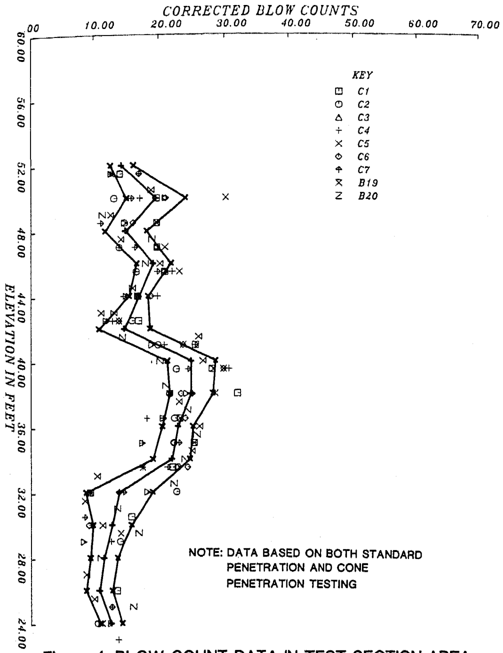


Figure 4: BLOW COUNT DATA IN TEST SECTION AREA BEFORE GROUTING. MEAN CURVE AND STANDARD DEVIATION CURVES SHOWN

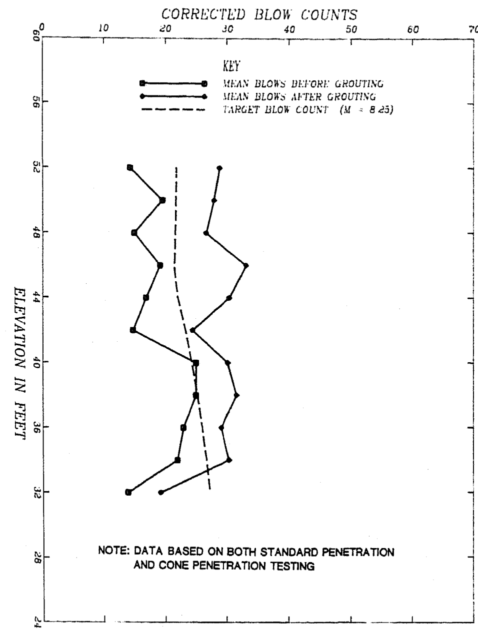


Figure 5: STANDARD PENETRATION BLOW COUNTS BEFORE AND AFTER THE GROUTING TEST PROGRAM

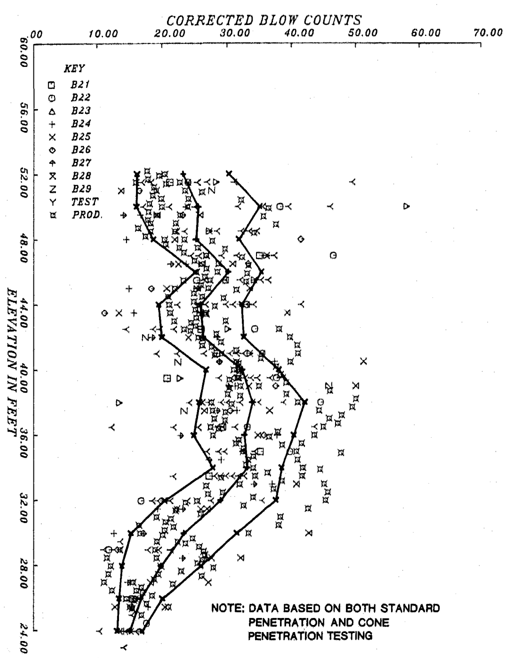


Figure 6: REPRESENTATIVE BLOW COUNT DATA AFTER GROUTING

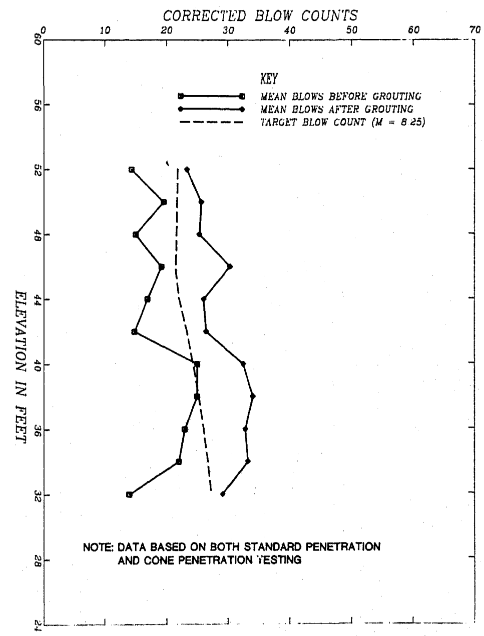
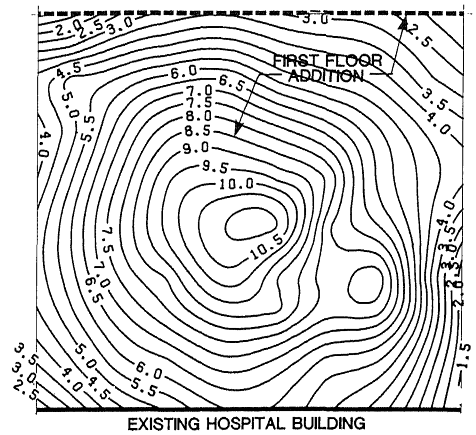


Figure 7: STANDARD PENETRATION BLOW COUNTS BEFORE AND AFTER PRODUCTION GROUTING



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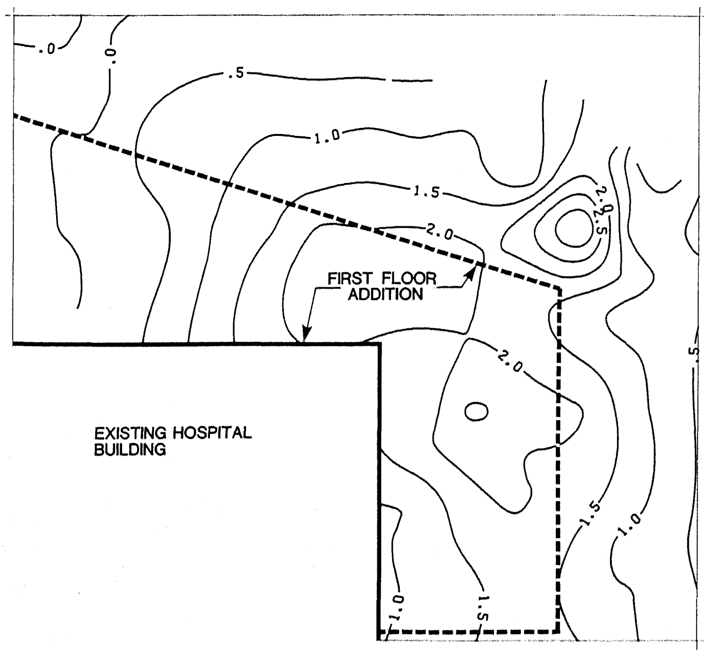

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Figure 8: TOTAL SURFACE HEAVE DURING GROUTING. CONTOURS SHOW HEAVE IN INCHES. 8a SHOWS AREA WITH POOR HEAVE CONTROL DURING GROUTING. 8b SHOWS AREA GROUTED LATER WITH CAREFUL HEAVE CONTROL.