PB93-114791

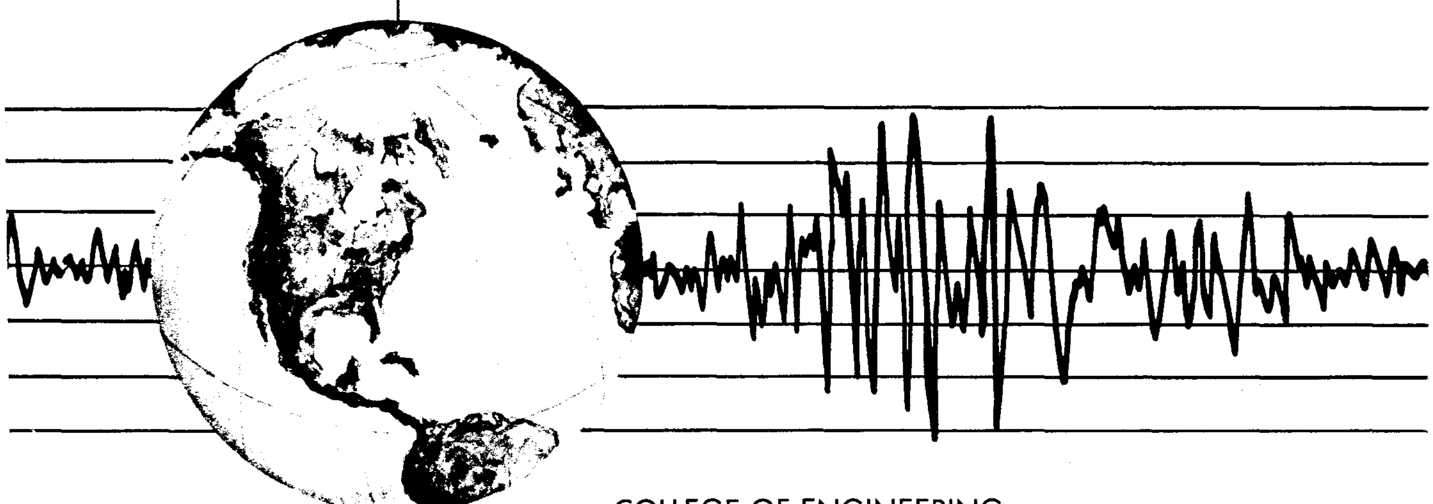
REPORT NO.
UCB/EERC-91/12
OCTOBER 1991

EARTHQUAKE ENGINEERING RESEARCH CENTER

PERFORMANCE OF IMPROVED GROUND DURING THE LOMA PRIETA EARTHQUAKE

by

JAMES K. MITCHELL
FREDERICK J. WENTZ, JR.



COLLEGE OF ENGINEERING

UNIVERSITY OF CALIFORNIA AT BERKELEY

For sale by the National Technical Information Service, U.S. Department of Commerce, Springfield, Virginia 22161

See back of report for up to date listing of EERC reports.

PERFORMANCE OF IMPROVED GROUND DURING THE LOMA PRIETA EARTHQUAKE

by

James K. Mitchell

and

Frederick J. Wentz, Jr.

Report No. UCB/EERC-91/12
Earthquake Engineering Research Center
College of Engineering
University of California at Berkeley

October 1991

ABSTRACT

Ground improvement by various means has been used increasingly in recent years to reduce the potential for liquefaction and lateral spreading of loose cohesionless to slightly cohesive soils. A number of deep densification techniques have been used, including vibrocompaction, vibro-replacement, deep dynamic compaction, penetration grouting, and compaction grouting. The Loma Prieta earthquake has provided one of the first opportunities to evaluate the behavior of treated ground that has actually been subjected to significant seismic shaking.

A comprehensive evaluation has been made of twelve sites where ground improvement had been used prior to the Loma Prieta earthquake. These sites included five on Treasure Island, two in Santa Cruz, and one each in Richmond, Emeryville, Bay Farm Island, Union City, and South San Francisco. The treatment methods that were used included vibro-replacement using stone columns, sand compaction piles, non-structural timber displacement piles, deep dynamic compaction, compaction grout, and chemical penetration grouting.

For each site available information was collected and analyzed with respect to the type of structure or facility, initial soil conditions, the level of ground improvement required, treatment methods considered and selected, construction procedures and problems, level of improvement achieved, shaking at the site during the Loma Prieta earthquake, and performance of the treated ground. At all but one of the sites the treated soil was a man-made fill. The soil at seven of these was a hydraulic sand fill. The required depths of treatment were up to about 30 ft in most cases, with treatment to a depth of 40 ft specified for one of them. The maximum peak ground accelerations experienced at the sites ranged from a low of 0.11g at Marina Bay in Richmond and at the Kaiser Hospital in South San Francisco, to a high of 0.45g at the two sites in Santa Cruz.

Without exception, there was little or no distress or damage due to ground shaking to either the improved ground or to the facilities and structures built upon it. In many cases, untreated ground adjacent to the improved ground cracked and/or settled, due primarily to liquefaction. In every case studied in which the ground accelerations were great enough that

liquefaction of the untreated ground would be predicted to occur, it did occur. Together these results support the conclusions that (1) the procedures used for prediction of liquefaction were reliable, and (2) ground improvement is effective for mitigation of liquefaction risk.

In assessing the results and their implications for the future, it is important to recall that the Loma Prieta was of only moderate intensity and of unusually short duration. On average, each of the sites, except for those near the epicenter, experienced maximum ground accelerations of only about 25 to 75 percent of the design earthquakes that were used. How these sites will perform in an earthquake of greater local intensity and duration is not known; however, almost certainly soil liquefaction and related effects will be reduced in comparison to what will happen to the untreated ground.

ACKNOWLEDGEMENTS

These studies were undertaken by Frederick J. Wentz, Jr. in partial fulfillment of the requirements for the degree of Master of Engineering. Partial support for Mr. Wentz was provided by income from the endowment of the Edward G. Cahill and John R. Cahill Chair in Civil Engineering, which is held by James K. Mitchell.

Many individuals and organizations kindly provided data and information from their files about the projects described in this report. In particular, the authors thank the following:

Steve Dickenson - University of California, Berkeley

Lelio Mejia and Larry Houps - Woodward-Clyde Consultants, Oakland, CA

Richard Short - Kaldveer Associates, Oakland, CA

Craig Shields - Treadwell & Rollo, Inc., San Francisco, CA

H. R. Al-Alusi - Pressure Grout Company, South San Francisco, CA

Henry Taylor - Harding Lawson Associates, San Francisco, CA

John Debecker and Vitauts Ozols - U.S. Navy Western Facilities Engineering Command, San Bruno, CA

Larry Karp - Orinda, CA

Mahmut Otus and John Egan - Geomatrix Consultants, San Francisco, CA

Robert Lopez and Francis Gularte - GKN Hayward-Baker, Ventura, CA

Publication of this report was made possible by a grant from the Earthquake Engineering Research Center, University of California, Berkeley.

TABLE OF CONTENTS

Preceding page blank

Page

Page

LIST OF FIGURES

Page

INTRODUCTION

As industrial and residential growth continues in and around the San Francisco Bay Area, building sites with adequate bearing capacity, acceptable settlement characteristics, and liquefaction resistance are becoming increasingly rare. Developers must now make use of land that was once considered poor or only marginally acceptable, and this is testing the ingenuity of geotechnical engineers to produce safe and economic structures and facilities.

In areas underlain by deep layers of fill and/or soft or loose soils, shallow foundations are usually impractical, and the usual practice has been until recently to use the costly alternative of carrying the applied loads down to competent soils, either by piles or deep excavation. Now, however, ground improvement techniques are increasingly being used to physically improve the engineering properties of the ground in-situ, thus allowing facilities to constructed on relatively simple and lightly reinforced shallow foundations.

In addition, during the past several years, various methods of soil stabilization or ground improvement have been used at several sites in and around the Bay Area specifically to mitigate the risk of damage to existing structures and other facilities due to liquefaction and other forms of ground distress caused by moderate to large earthquakes.

It was found that none of the sites where soil improvement had been used experienced damage or distress as a result of the Loma Prieta earthquake of October 17, 1989. While this provided evidence of the overall value of engineered fills and different ground treatment methods, it also provided a unique opportunity for a detailed study of the performance of improved soil sites that were subjected to various levels of ground shaking during the earthquake.

OBJECTIVE AND SCOPE

The purpose of this study was to evaluate the performance during the Loma Prieta earthquake of improved soil sites, and to develop an improved quantitative understanding of the relationship between treated soil properties and seismic performance. The information used in the study, mostly in the form of engineering reports, was collected primarily from several geotechnical engineering firms throughout the Bay Area, and the Naval Facilities Engineering Command.

Twelve sites involving five different methods of soil improvement were studied. The sites studied, the soil types treated, and the treatment methods used are listed in Table 1. There are five projects located on Treasure Island, two in Santa Cruz, and one each in Richmond, Emeryville, Bay Farm Island, Union City, and South San Francisco. The specific project locations are shown in Fig. 1.

For each case, as much information was collected and analyzed as was available concerning:

1. The type of structures or facilities.
2. The initial soil conditions.
3. The level of improvement required.
4. Treatment methods considered and selected.
5. Analytical studies performed, if any.
6. Construction methods and problems.
7. Field control and evaluation.
8. Performance of the site during the Loma Prieta earthquake.

The method of soil improvement is known in every case, as is the site performance during the Loma Prieta earthquake. Where possible, the behavior of treated ground areas was compared with adjacent areas where ground improvement was not used.

TABLE 1: GROUND IMPROVEMENT PROJECTS

NAME	LOCATION	SOIL CONDITIONS	METHOD	YEAR OF TREATMENT
1. Medical/Dental Clinic	Treasure Island	Hydraulic Sand Fill	Stone Columns	1989
2. Office Building No. 450	Treasure Island	Hydraulic Sand Fill	Sand Compaction Piles	1967
3. Facilities 487 - 489	Treasure Island	Hydraulic Sand Fill	Vibrocompaction	1972
4. Approach Area, Pier 1	Treasure Island	Hydraulic Sand Fill	Stone Columns	1984
5. Building 453	Treasure Island	Hydraulic Sand Fill	Non-Structural Timber Piles	1969
6. Esplanade Extension East Shore, Marina Bay	Richmond	Silty, Sandy, and Gravelly Fill	Stone Columns	1986
7. East Bay Park Condominiums	Emeryville	Silty Sand Fill	Vibrocompaction	1981
8. Harbor Bay Business Park	Alameda	Hydraulic Sand Fill	Deep Dynamic Compaction	1985
9. Hanover Properties	Union City	Silty Sand Fill	Deep Dynamic Compaction	1988
10. Kaiser Hospital	South San Francisco	Hydraulic Sand Fill	Compaction Grout	1978
11. Riverside Avenue Bridge	Santa Cruz	Sands and Gravels	Chemical Grout	1986
12. Adult Detention Facility	Santa Cruz	Silty, Sandy Fill	Deep Dynamic Compaction	1978

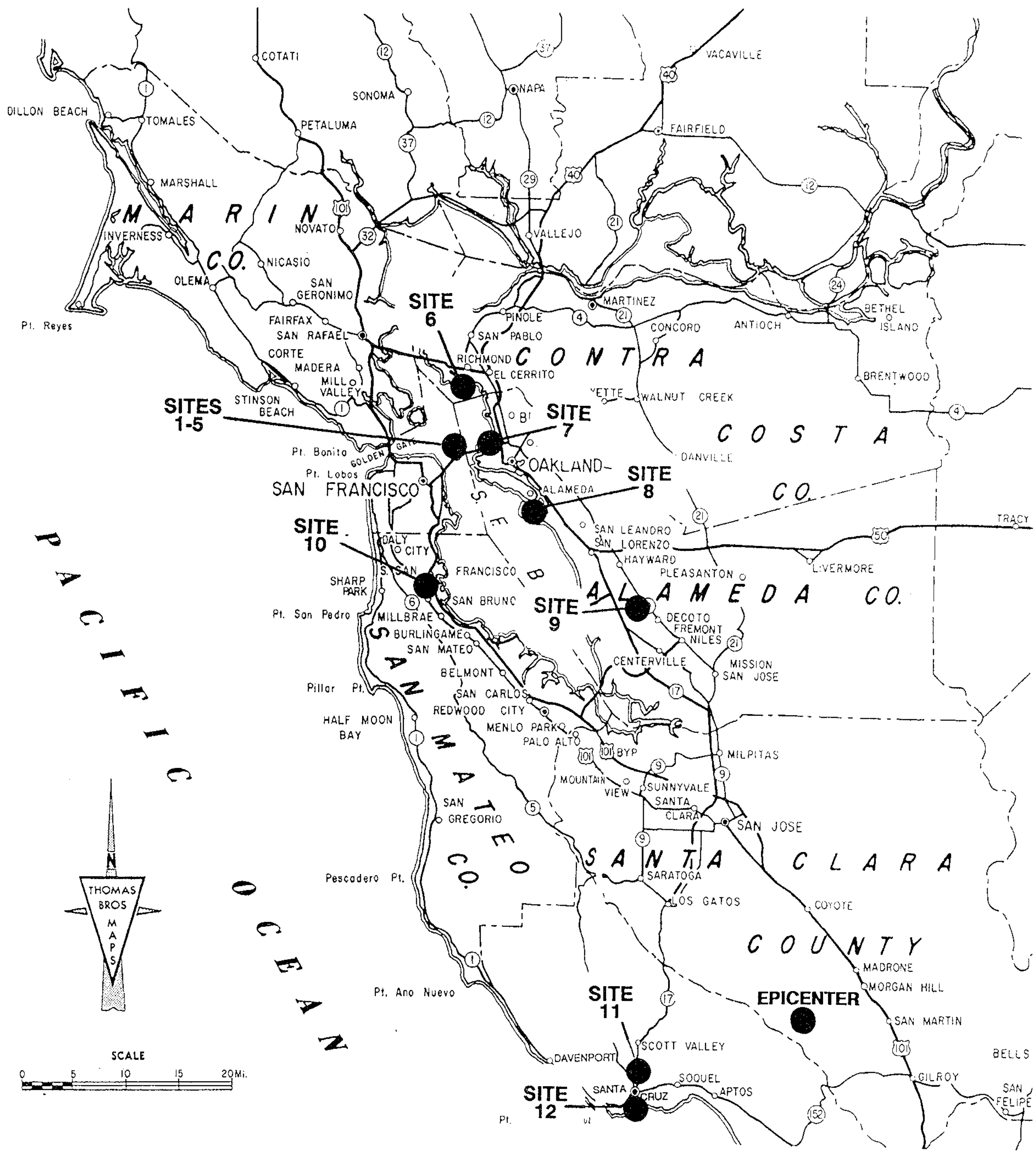


Fig. 1 Project Locations

The intensity and bracketed durations[1] of shaking for accelerations greater than 0.1g experienced at each site during the earthquake are listed in Table 2. These values were estimated based on information collected from the U.S. Geological Survey (USGS) and the California Division of Mines and Geology (CDMG). It may be seen that in all but one case the actual intensity of shaking during the Loma Prieta earthquake was substantially less than the design value. Furthermore, the total duration of shaking was less than 15 seconds; about half that to be expected for a magnitude 7 earthquake. The bracketed durations for accelerations greater than 0.1g were considerably less at all sites except those in Santa Cruz. Nevertheless, the ground motion was strong enough at most of the sites to cause liquefaction of untreated ground. For some of the sites, especially at Treasure Island, there is little specific information available concerning construction procedures, field testing, and evaluation of the final results. Many of the sites were treated during the late 1960's, and many project files are no longer available.

GROUND IMPROVEMENT METHODS

Brief descriptions of the different soil improvement methods that were used at the sites investigated are given below. More complete descriptions and details of the methods are given by Mitchell (1981), Welsh (1986), and Hausmann (1990).

1. **Vibro Stabilization Techniques** - These methods for deep compaction of cohesionless soils are characterized by the insertion of a cylindrical shaped probe into the ground followed by compaction by vibration during withdrawal. In a number of the methods a granular backfill is added so that a sand or gravel column is left behind within a volume of sand compacted by vibration. Sinking of the probe to the desired treatment depth is usually accomplished by vibration, often supplemented by water jets at the tip.

---

[1] The bracketed duration is the elapsed time period between the first and last waves with accelerations greater than some specified value.

TABLE 2: PEAK GROUND ACCELERATIONS - DESIGN EARTHQUAKE AND LOMA PRIETA EARTHQUAKE

NAME	LOCATION	DISTANCE FROM EPICENTER (MILES)	ACCELERATION (g's)		BRACKETED DURATION ($A \geq 0.10$ g) (SEC)
			ACTUAL	DESIGN	
Medical/Dental Clinic	Treasure Island	60	0.16	0.35	2.5
Office Building No. 450	Treasure Island	60	0.16	0.43*	2.5
Facilities 487 - 489	Treasure Island	60	0.16	0.43*	2.5
Approach Area Pier 1	Treasure Island	60	0.16	0.35	2.5
Building 453	Treasure Island	60	0.16	0.45*	2.5
Esplanade Extension East Shore, Marina Bay	Richmond	68	0.11	0.35	1.0
East Bay Park Condominiums	Emeryville	60	0.26	0.35	2.0
Harbor Bay Business Park	Bay Farm Island	49	0.25	0.35	4.0
Hanover Properties	Union City	39	0.16	N/A	3.0
Kaiser Hospital	South San Francisco	51	0.11	N/A	2.0
Riverside Avenue Bridge	Santa Cruz	10	0.45	N/A	15.0
Adult Detention Facility	Santa Cruz	10	0.45	0.40	15.0

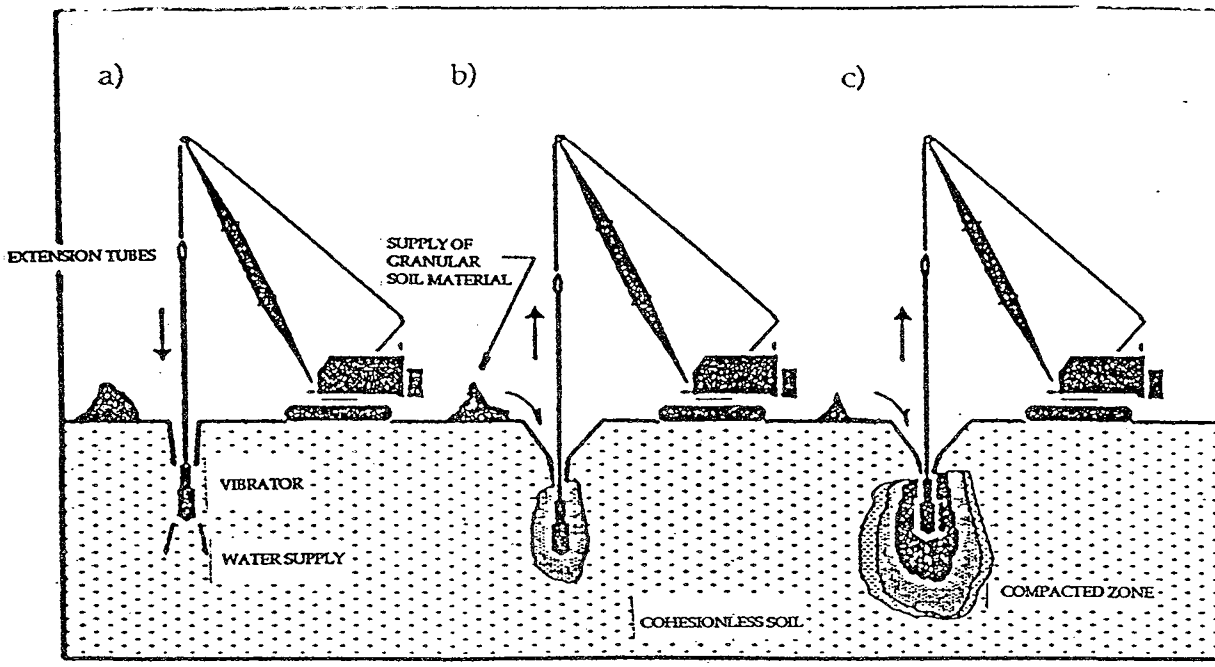
* These projects were completed in the late 1960's and the design earthquake used was the 1940 El Centro earthquake (scaled to a peak horizontal ground acceleration of 0.43 to 0.45 g).

Vibrating Probes - The Terra Probe method, developed in the U.S.A., uses a Foster Vibro-driver pile hammer on top of a 2.5 ft diameter open tubular probe (pipe pile) that is 10 to 16.5 ft longer than the desired penetration depth. Other types of vibrating probes have recently been developed, including the "Vibro-Wing" and the "Tri-Star or Y-Probe" (Wightman, 1991).

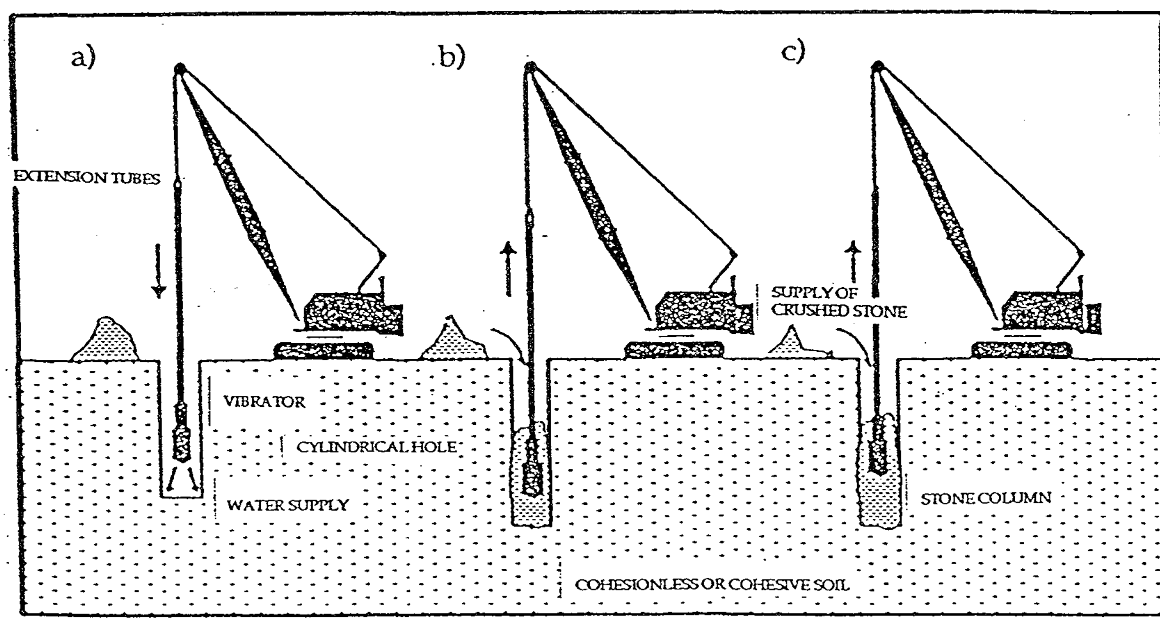
Vibro-Compaction (Vibroflotation) - This technique achieves good results in clean granular soils with less than about 15-20% fines. The action of the vibrator, usually accompanied by water jetting, reduces the inter-granular forces between the soil particles allowing them to move into a more compact configuration. After a certain time, the optimum configuration has been reached and the vibrator is raised a short distance and the procedure repeated. This increase in density is accompanied by a reduction in volume, which is compensated by backfilling the annulus around the vibrator with sand as it is withdrawn. In stratified soils where layers of soft cohesive material are present, the resulting column of compacted sand functions as compression and shear reinforcement. A graphic representation of this method is shown in Fig. 2.

Vibro-Replacement (Stone Columns) - Vibro-Replacement is used in soils with a higher fines content (>15-20%) than can be densified by vibroflotation, or even in clay soils where the strata do not respond satisfactorily to vibrations.

Most stone column installations are made using the vibro-replacement method in a manner similar to vibrocompaction, as shown in Fig. 2. A probe is penetrated to the desired depth by vibration and jetting. Gravel backfill is dumped into the hole in increments of 1.5 to 2.5 ft and compacted by the vibrating probe which simultaneously displaces the material radially into the soil. The diameter of the resulting column can be estimated from the rock consumption. It will usually be in the range of 2 to 3.5 ft. In the dry process, which is being increasingly used, a bottom feed system is



Vibrocompaction



Vibro-Replacement

Fig. 2 Vibroflotation and Vibro-Replacement: Equipment and Processes employed in which the gravel or crushed rock backfill is discharged at the bottom of the hole through a pipe attached to the side of the vibroflot.

Although clays and silts do not respond to vibration, the stone columns confine the soil thus increasing the bearing capacity and reducing settlement. In addition, in fine-grained liquefiable soils, besides stiffening the matrix, the stone column also acts as a vertical drain. Therefore, not only does this technique increase the relative density of the layers susceptible to liquefaction, but it also allows rapid dissipation of excess pore water pressures induced by earthquake loading. Also, the increased stiffness and shear resistance provided by the columns themselves create additional reinforcement of the soil mass.

Dynamic Deep Compaction - This method involves repeated dropping of 10 to 40-ton weights onto the ground surface from heights of up to 120 feet in a grid pattern. An illustration of a typical grid pattern and representative equipment is shown in Fig. 3. Shockwaves created by the weight hitting the ground densify the soil by rearranging the particles into a more compact arrangement. This method is capable of achieving substantial improvement in the engineering properties of a wide range of soils including loose sands, mining spoils, collapsible soils, and construction rubble.

Layers or large obstructions which could inhibit the penetration of vibrators will not affect the Dynamic Compaction method. This technique is typically limited to a maximum depth of improvement of about 40 feet. The major limitations of this method are the possible effects on nearby facilities of the vibrations, flying debris, and noise. This method is best employed in large open areas.

2. **Compaction Piles** - Compaction piles densify the soil by displacement, as well as providing compression and shear reinforcement in soft soils. Two different types were used for projects covered by this report, sand compaction piles, and non-structural timber displacement piles.

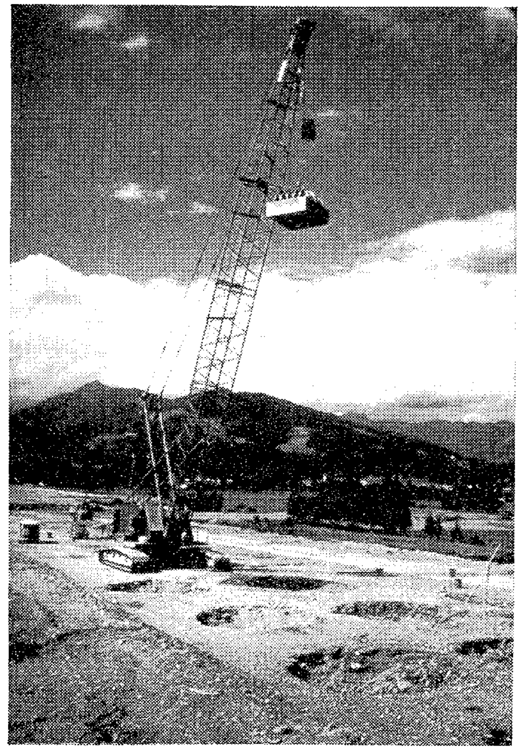


Fig. 3 Illustration of Dynamic Deep Compaction

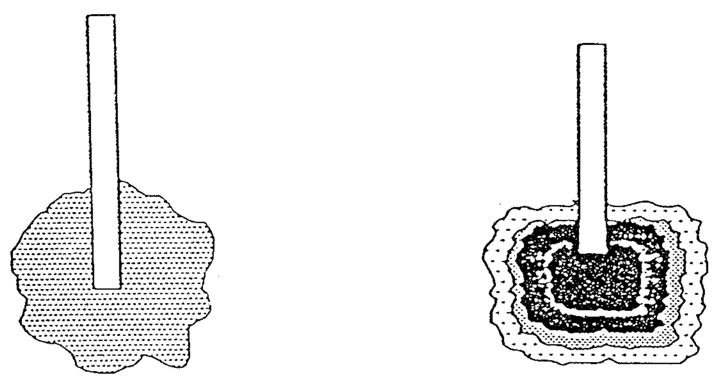


Fig. 4 Types of Grouting

Sand Compaction Piles - In this method, a casing pipe is driven to the desired depth using a mandrel and then filled with sand. The pipe is withdrawn part way while compressed air is blown down inside the casing to hold the sand in place, and then the pipe is redriven down to compact the sand pile and enlarge its diameter. The process is repeated until the pipe reaches the ground surface.

Non-Structural Timber Piles - With this method, non-structural timber piles are driven with a follower to a depth equal to the length of the pile. The soil surrounding the piles is compacted by displacement. The piles do not serve any structural function but remain in place permanently. This method is rarely used today.

3. **Grouting** - Grouting is the injection of materials into voids in the soil, generally through boreholes and under pressure. A major advantage of grouting is that it can be used in small, difficult-to-access areas. Two grouting techniques were used in projects studied in this report, compaction grouting, and chemical grouting.

Compaction Grouting - For sites where Dynamic Deep Compaction and vibratory techniques may be impractical, particularly those with deep layers of loose liquefiable soils, compaction grouting can be used to displace and densify the soil. Typically, a very stiff (1-2 inch slump) soil-cement-water mixture is injected into the soil, forming grout bulbs which displace and densify the surrounding ground, without penetrating the soil pores (see Figure 4). With slightly more fluid grout, thick fissures rather than bulbs may form; this is sometimes referred to as "squeeze grouting" (Hausmann, 1990).

Chemical Grouting - This technique for liquefaction potential mitigation is used mainly to stabilize foundation soils under existing structures. With this method, pressure injecting low-viscosity chemical grouts into granular soil pores forms a strong sandstone-like material (see Fig. 4). The cohesion added by the chemical grout provides increased bearing capacity as well as reducing liquefaction potential. This method has proven less disruptive and more economical than underpinning in many cases (Hausmann, 1990).

The choice between the various systems usually depends on the prevailing soil profile, environmental conditions, and cost. The presence of near surface ground water or soft fine-grained soils, and proximity to existing structures and utilities place constraints on what methods can be used. Figure 5 illustrates in general terms, the types of ground improvement that are useful relative to different soil particle size ranges.

CASE HISTORIES

TREASURE ISLAND - GENERAL

Treasure Island is a man-made island in San Francisco Bay consisting of up to 50 feet of hydraulically placed sand fill over the natural bay deposits. A perimeter dike surrounds the island. A map of the island showing the relative locations of the five projects where ground improvement has been used is presented in Fig. 6. The effects of strong ground shaking during the Loma Prieta earthquake were evident by the presence of liquefaction sand boils across most of the island. The rock motion record from Yerba Buena Island just to the south is compared with the soil record at Treasure Island in Fig. 7. The much stronger ground shaking at Treasure Island was a direct result of the amplification of relatively modest rock motions by the deep soil layer beneath the island.

The liquefaction of subsurface soils was evidenced by the presence of sand boils in numerous locations. Settlement, both areal and localized, was observed in many locations. In the area of the perimeter dike, ground cracking caused by bayward lateral spreading was readily apparent. In addition, there were several cases of buckled pavements, broken utility lines, and distressed buildings across the island. In the case of structures or facilities built upon soil improved areas, however, without exception, there was little or no observed distress or damage.

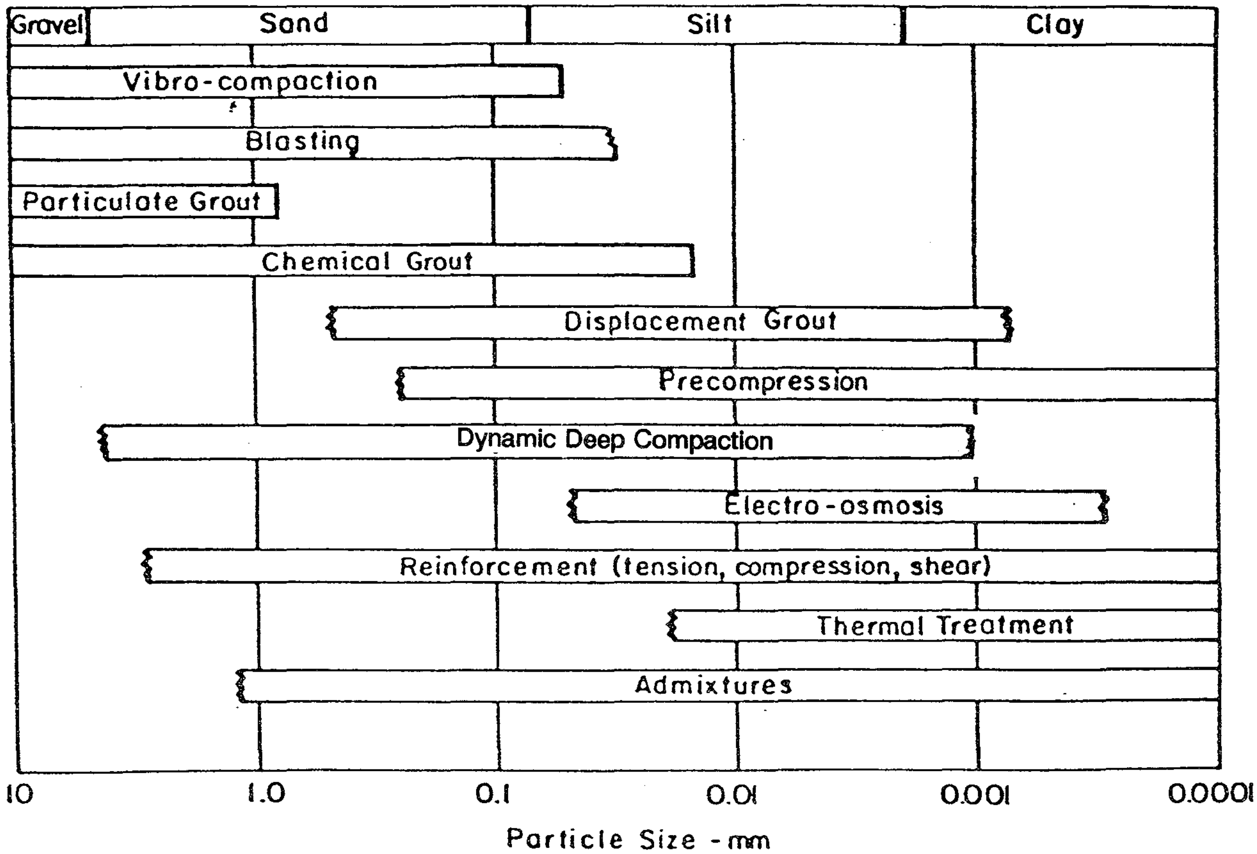


Fig. 5 Applicable Grain Size Ranges for Different Stabilization Methods (Mitchell, 1981)

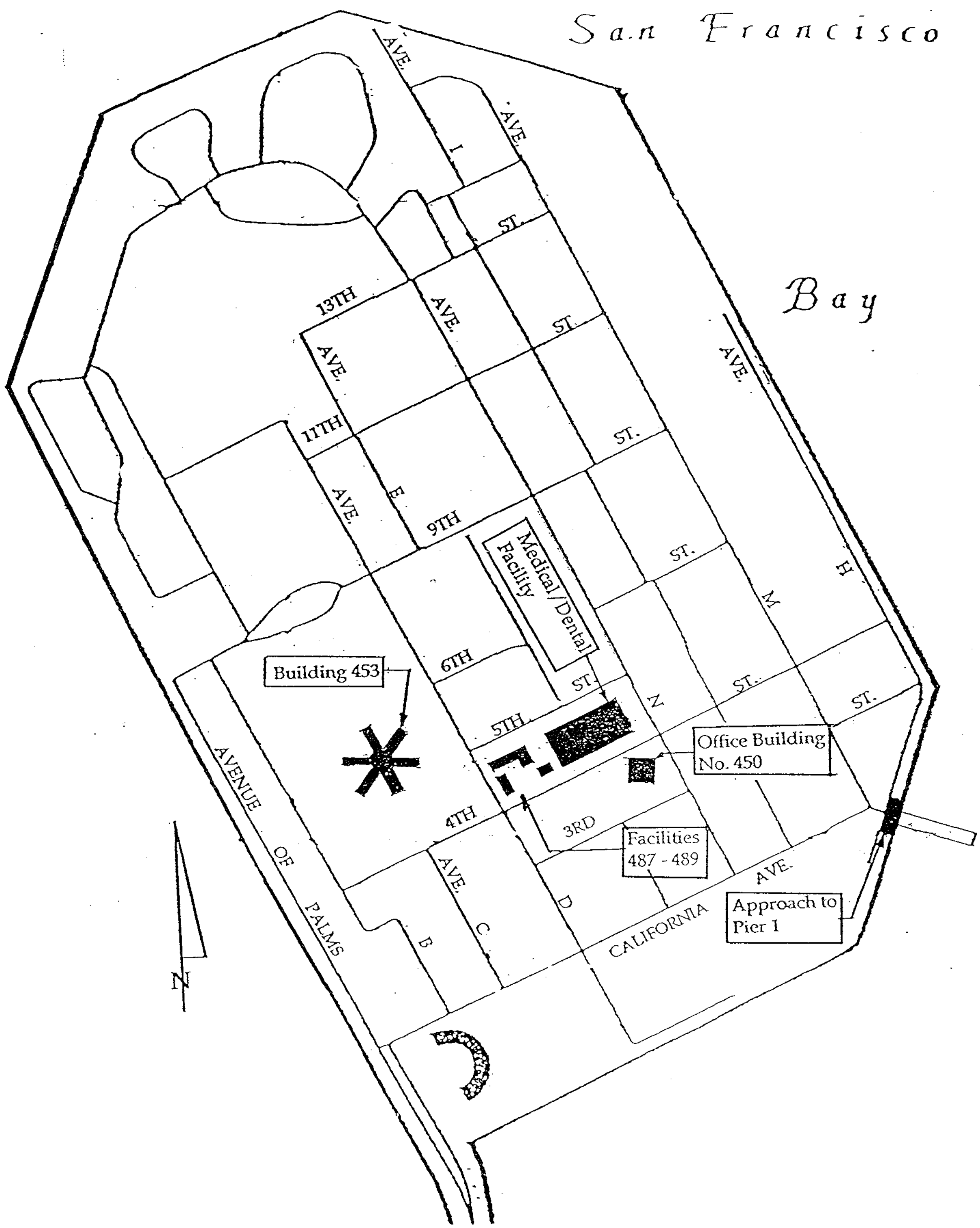


Fig. 6 Project Locations on Treasure Island

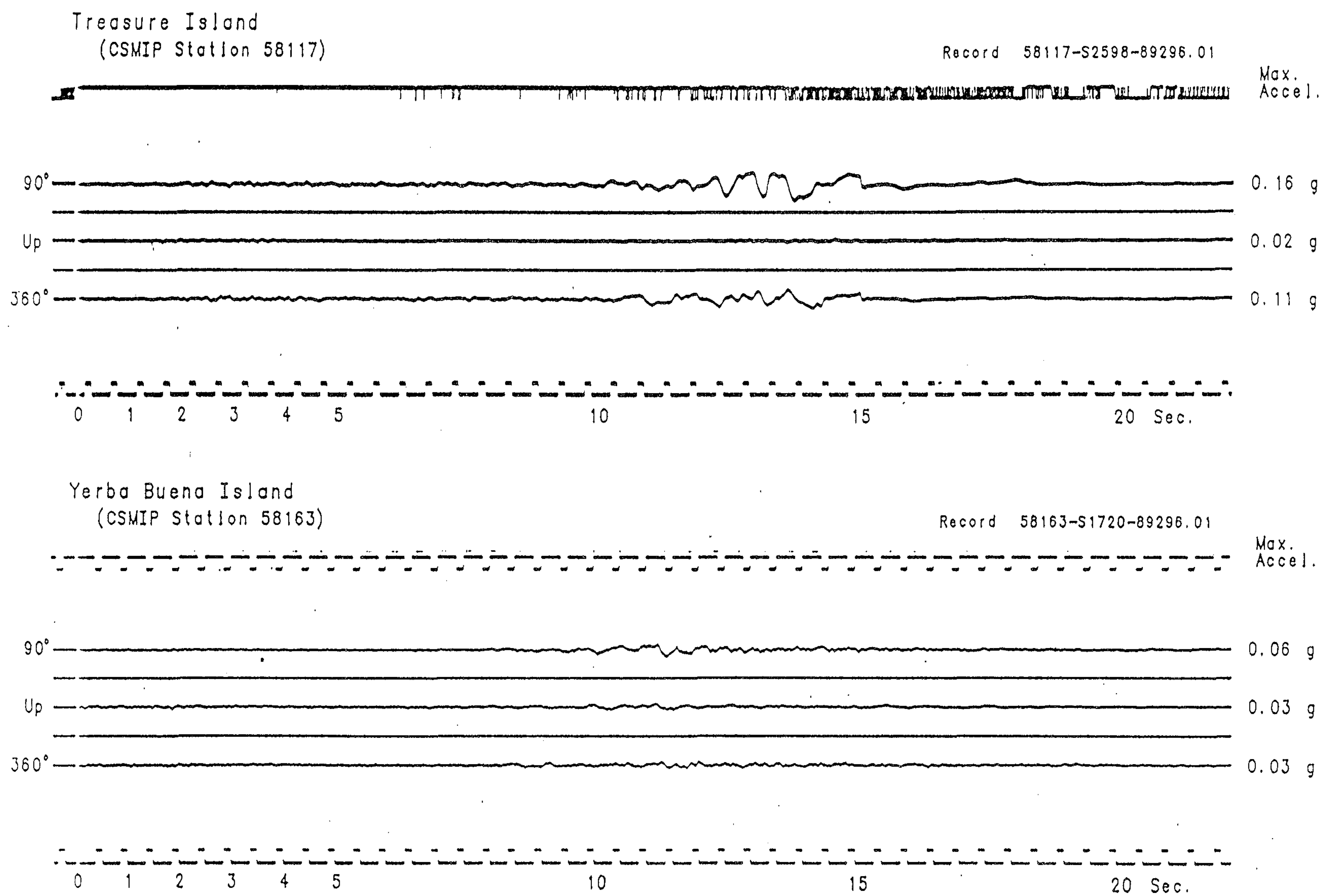


Fig. 7 Strong Motion Records from Treasure Island and Yerba Buena Island during the Loma Prieta Earthquake

MEDICAL/DENTAL BUILDING, TREASURE ISLAND

Project Description

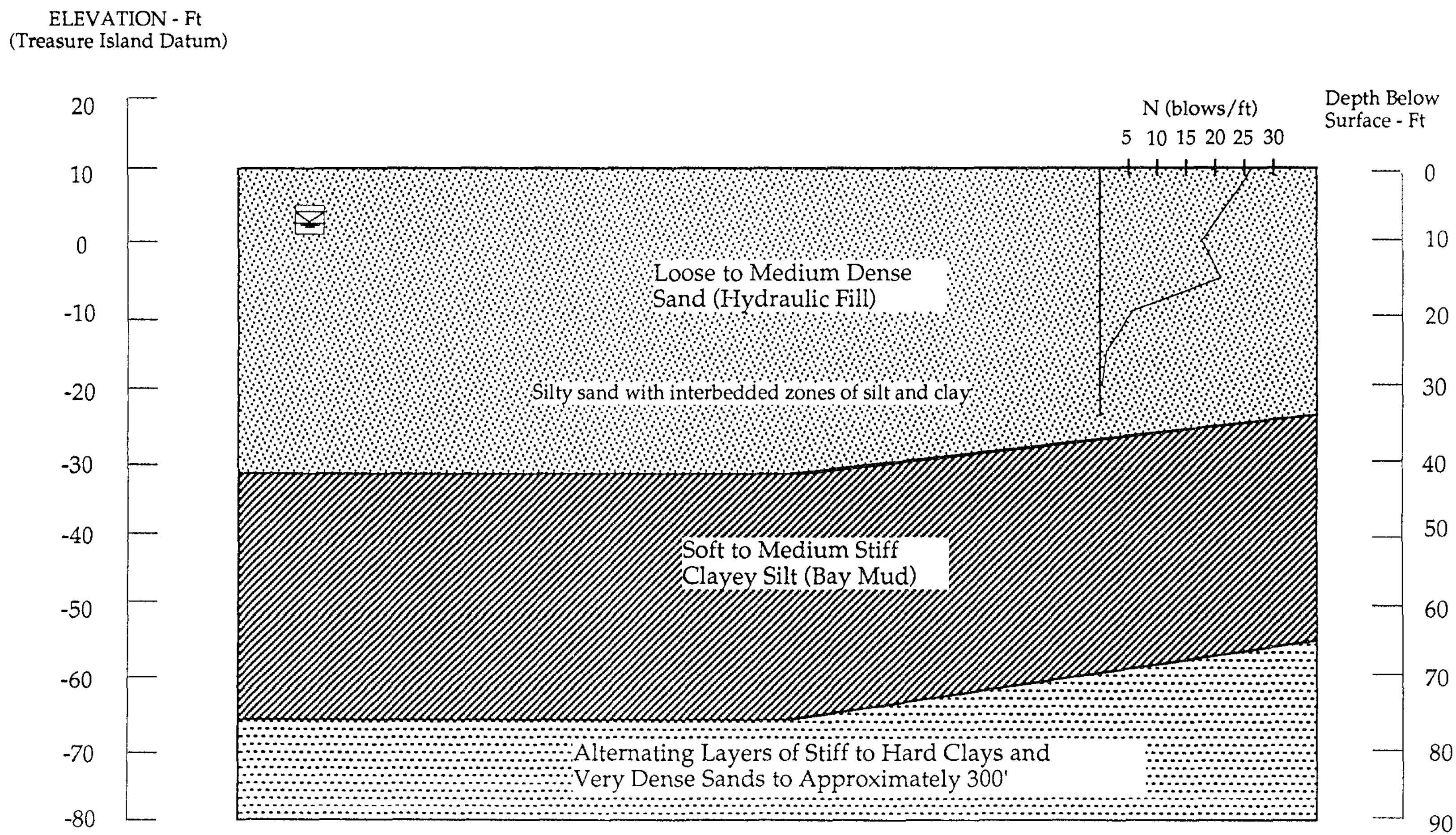
The medical/dental clinic, located as shown in Fig. 6, was under construction at the time of the Loma Prieta earthquake, with about 40% of the building footings cast and two 22-foot deep elevator shafts excavated. The building consists of a two-story, steel-frame structure with a total floor area of approximately 55,000 square feet. The first floor is slab-on-grade with a finished floor elevation approximately 1-1/2 feet above the initial grade. The dead plus live column loads are up to 230 kips; typically, the columns are spaced 30 feet apart.

Initial Conditions

The site is essentially level. Near the western boundary is a maintained landscaped berm with small trees, shrubs, and grass. The ground surface is approximately 13 feet above Mean Lower Low Water (MLLW).

A subsurface section along the east-west centerline of the project site, illustrating the soil conditions encountered in the test borings is shown on Figure 8. The upper 31 to 43 feet of soil is composed of loose to medium dense hydraulically placed sand fill. The sand is generally fine- to medium-grained and contains less than 10 percent of material finer than the #200 sieve. Occasional thin layers of soft compressible silt (dredged Bay Mud) were encountered throughout the fill.

The sand fill is underlain by a layer of soft to medium stiff clayey silt (Bay Mud) approximately 30 feet thick interspersed with thin sand lenses. The Bay Mud is underlain by alternating layers of dense to very dense sands and stiff to hard clays to the depths penetrated. The groundwater level was approximately 7 feet below the ground surface (i.e., approximately +6 feet MLLW) across the site at the time of drilling.



SCALE: 1" = 30' Horizontal
1" = 20' Vertical

Fig. 8 Subsurface Profile at the Medical/Dental Clinic, Treasure Island

Liquefaction Potential - The liquefaction potential of the hydraulically placed sand fill was evaluated at three levels of ground shaking using the empirical method of Seed et al. (1983). The results, based on the assumption of the water table at a depth of 7 ft., are summarized below:

Earthquake Magnitude	Peak Ground Acceleration (g)	Depth of Liquefiable Zone (feet)
6-1/4	0.15	17 - 43
6-3/4	0.25	17 - 43
7-1/2	0.35	7 - 43

Foundation Alternatives

Based on the results of the geotechnical investigation, it was concluded that the proposed structure could be supported either on spread footings or piles. However, if spread footings were to be used, then the sand fill would require densification to reduce the liquefaction potential. It was determined that the foundation alternative would require 12-inch square, prestressed, precast concrete piles with a 125-ton structural capacity. The piles would have to be driven approximately 130 feet below the ground surface to gain their full structural capacity.

It was judged that using a pile supported foundation would not be as economical as densifying the site and supporting the structure on spread footings. In addition, the movement of the subsiding ground surface relative to the stationary, pile-supported building during liquefaction would require that the utility connections be specially designed thus increasing costs.

Improvement Goals and Methods

Specified Level of Improvement. - The geotechnical engineer specified that at a minimum, the upper layer of sand fill should be densified to a minimum relative density of 75 percent beneath the building and to a distance of 20 feet beyond the building perimeter in order to prevent liquefaction. The densification achieved was measured using the Cone Penetration Test (CPT) and the Standard Penetration Test (SPT).

It was noted that normally, sand densification methods are not effective in the upper few feet of the soil layer because of the lack of confining pressure at the surface. Thus, it was specified that the actual thickness of sand not densified to the required density as determined by CPT and SPT be excavated and backfilled and compacted to a minimum relative compaction of 95 percent using surface compactors.

Improvement Method. - Several deep densification methods believed to be appropriate for the soil conditions at the site, including vibro-replacement, Terraprobe densification, dynamic consolidation, conventional compaction piles, and sand compaction piles, were investigated. The final recommendation of the geotechnical engineer was that either the vibro-replacement or Terraprobe technique be used. The vibro-replacement technique, using gravel backfill, was ultimately chosen, as this method would produce the most densification in the shortest amount of time. Deep dynamic compaction was also considered, but the possibility of disturbance to neighboring structures was considered too great a risk.

Construction Procedures and Problems

Trial Densification Tests. - In order to find the maximum probe spacing for achieving the required relative density, a series of tests were performed using probe spacings of 8, 9, and 10 feet. SPTs and CPTs were performed prior to and after densificaiton to evaluate the results of the densification. Because there were zones of silt and clay in the sand fill, the specifications did not require improvement in zones where the CPT friction ratio was greater than 2 percent.

Problems - A review of the SPT and CPT results prior to and after densification showed that:

1. The upper 10 feet of the sand was already dense prior to treatment, and was not densified further by vibroreplacement. The lower layer (10 to 22 ft deep) was loose to medium dense prior to densification, and the treatment increased the density. The closer the probe spacing, the higher the cone resistances after treatment. With a probe spacing of 8 feet, the specified cone resistances were achieved in the entire layer except in the lower few feet.

2. Silty sand fill interbedded with zones of silt and clay underlay the upper fill layer to a depth of about 40 feet. Adequate densification of this layer was not achieved. The cone penetration tests indicated that the cone resistances of the silty sand were practically unchanged by treatment; however, the SPT blow counts increased from 2 to 5 prior to densification, to 3 to 19 after densification. The post-densification blow counts were lower than the specified values, however.

Although the trial densifications with the chosen spacings did not densify the soil enough to meet the specifications, the contractor proceeded with production densification using a10-foot probe spacing penetrating to a depth of approximately 22 feet below the existing ground surface.

Final Results of Improvement

The increase in density of the fill was measured using CPT. It was also estimated in the upper 22 ft using a volume calculation method. The cone resistances measured in the upper 22 feet of the fill indicated that there were still sand fill layers between 10 and 22 ft where the specified penetration resistance was not achieved. Using the volume calculation method, however, the computed average final relative density of the sand fill between 10 and 22 feet was estimated to range between 77 and 80 percent. Ultimately, it was decided that the densification of the upper 22 feet of fill did meet the required specifications.

There was no attempt made to densify the soil between 22 and 40 feet as required by the specifications. Because of this, it is believed that there are layers of sandy fill between these depths that still have potential for liquefaction in a large earthquake which could cause additional settlement of the structure. It was estimated that the additional total settlement caused by liquefaction would be from 1 to 3 inches and that the differential settlement between adjacent columns would be 1/2 inch. The risks and benefits of not densifying the lower sand layer were evaluated, and it was decided not to perform further densification.

Performance of the Project During the Loma Prieta Earthquake

Based on ground acceleration readings taken from a CDMG strong motion instrument located on Treasure Island, the project site was subjected to a peak ground acceleration of approximately 0.16g within a bracketed duration of about 2.5 seconds during the Loma Prieta earthquake. At the time of the earthquake, approximately 40 percent of the building footings had already been cast. There was no cracking visible in the footings. It was observed, however, that the bottom 8 feet of the two 22-foot deep elevator shafts that were drilled prior to the earthquake were filled with sand. The engineers were also informed that sand flowed to the ground surface through one of the elevator shafts during the earthquake. From these observations, it was concluded that liquefaction had occurred in the lower, untreated sand fill between a depth of 22 and 40 feet. In the area outside of the building footprint, which was not densified, sand boils and ground cracking was observed. The differential settlement of the footings was measured in November, 1989 and was reported to be a maximum of 0.073 ft over a distance of 180 feet. The total settlement of the site could not be determined because the benchmark had settled during the earthquake.
It appears that no liquefaction occurred in the upper 22 feet of sand fill that had been densified by stone columns. Thus, this project provides direct evidence of the effectiveness of the columns for mitigation of liquefaction potential.

OFFICE BUILDING NO. 450, TREASURE ISLAND

Project Description

Office Building No. 450, located as shown in Fig. 6, was constructed in 1967. The project actually consists of two buildings, each three stories high and of steel-frame construction with concrete walls and floors. The larger building is 160 by 160 feet in plan and has a central court measuring 30 by 60 feet. The smaller building is 54 by 124 feet in plan and is located approximately 100 feet northwest of the first building. Typical dead plus live column loads are 250 to 300 kips. The finish floor elevation is approximately 2-1/2 feet above the initial grade. This project was the subject of a study by Basore and Boitano (1968) to evaluate and compare the effectiveness of both sand compaction piles and vibroflotation in densifying a deep sand fill.

Initial Conditions

The building site is essentially level. The ground surface is approximately 11 feet above Mean Lower Low Water (MLLW). A subsurface section along the north-south centerline of the project, illustrating the soil profile is shown on Figure 9. The upper 30 feet of soil are composed of loose to medium dense hydraulically placed sand fill overlying 8 feet of medium dense sand. The sand is generally fine- to medium-grained and contains less than 10 percent of material finer than the #200 sieve. Some coarse-grained sand was encountered in the upper 10 feet of the fill. Occasional thin layers of soft compressible silt (dredged Bay Mud) were encountered throughout the fill.

The sand fill is underlain by a layer of soft to medium stiff grey silty clay (Bay Mud) approximately 20 feet thick. The Bay Mud is underlain by alternating layers of very dense sands and stiff to very stiff clays to the depths penetrated. The groundwater level was approximately 6 feet below the ground surface (i.e., approximately +5 feet MLLW) across the site at the time of drilling.

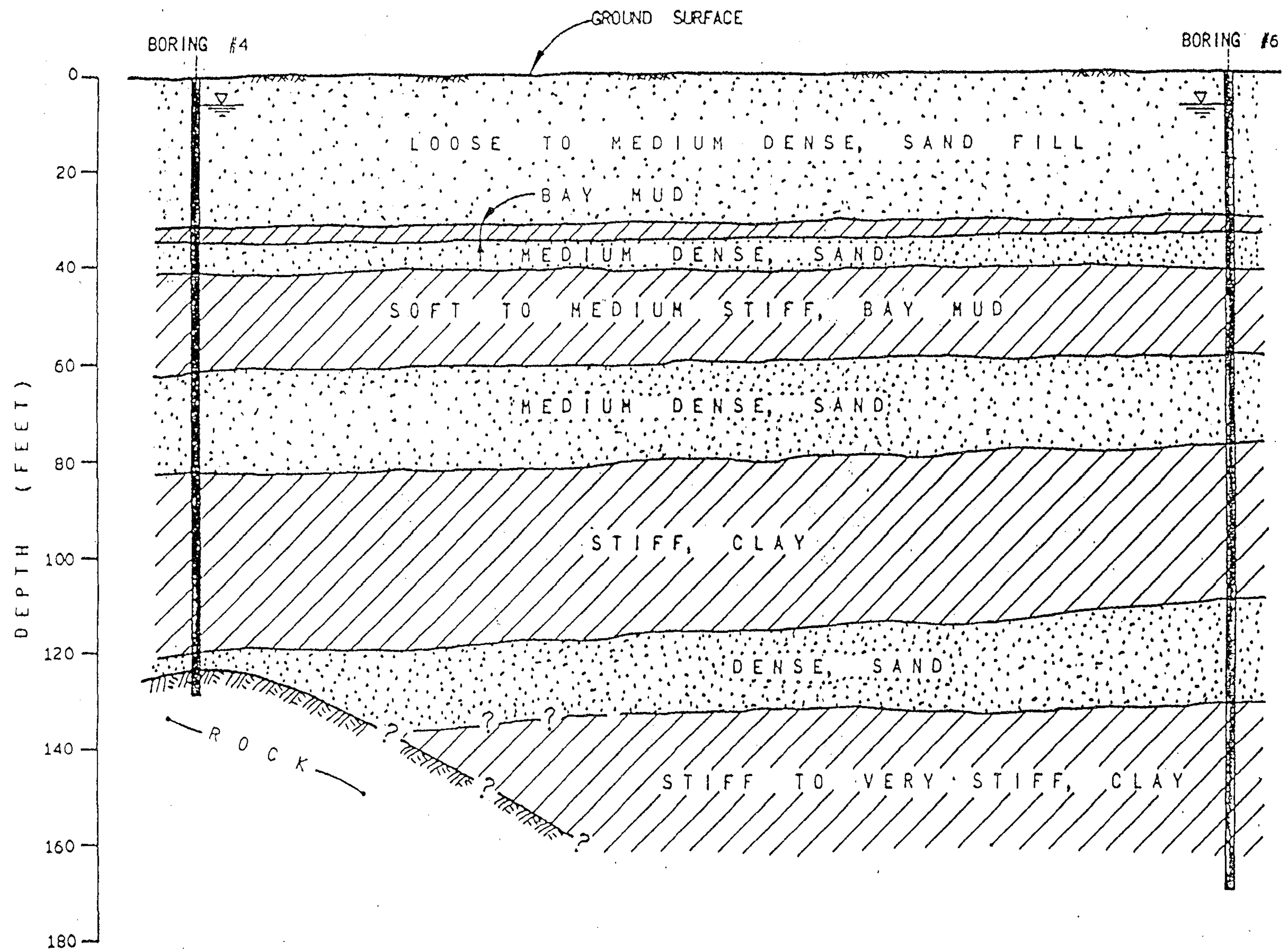


Fig. 9 Subsurface Profile at Office Building No. 450 (Woodward-Clyde-Sherard and Assoc., 1966)

Liquefaction Potential. - The liquefaction potential of the saturated sand fill was analyzed using the method developed by Seed and Idriss (1971). The results of the analysis showed that the fill would liquefy under the expected peak ground accelerations of 0.30 - 0.40g site during a large earthquake.

Foundation Alternatives

Consideration was given to supporting the building on either spread footings bearing on densified fill or on driven piles extending through the fill and underlying compressible clays into bearing soils encountered below a depth of 113 feet. The use of piles was ruled out for three reasons: 1) The extreme length of the piles and associated high cost; 2) the fact that piles under one corner of the building would be endbearing on bedrock while the remaining piles would be friction piles; and 3) the lateral stability of the piles would be insufficient should the sand fill liquefy. It was decided therefore to support the buildings on spread footing foundations bearing on densified fill.

Improvement Goals and Methods

Specified Level of Improvement - The geotechnical engineer specified that at a minimum, the sand fill be densified to a minimum relative density of 75% to a depth of 30 feet beneath the building footings, and 65% to a depth of 30 feet beneath the floor areas and to a distance of 10 feet beyond each building perimeter.

Improvement Method - Both vibroflotation and sand compaction piles were considered as methods of densifying the fill, and an extensive field testing program was performed to determine which of the two methods would be more effective.

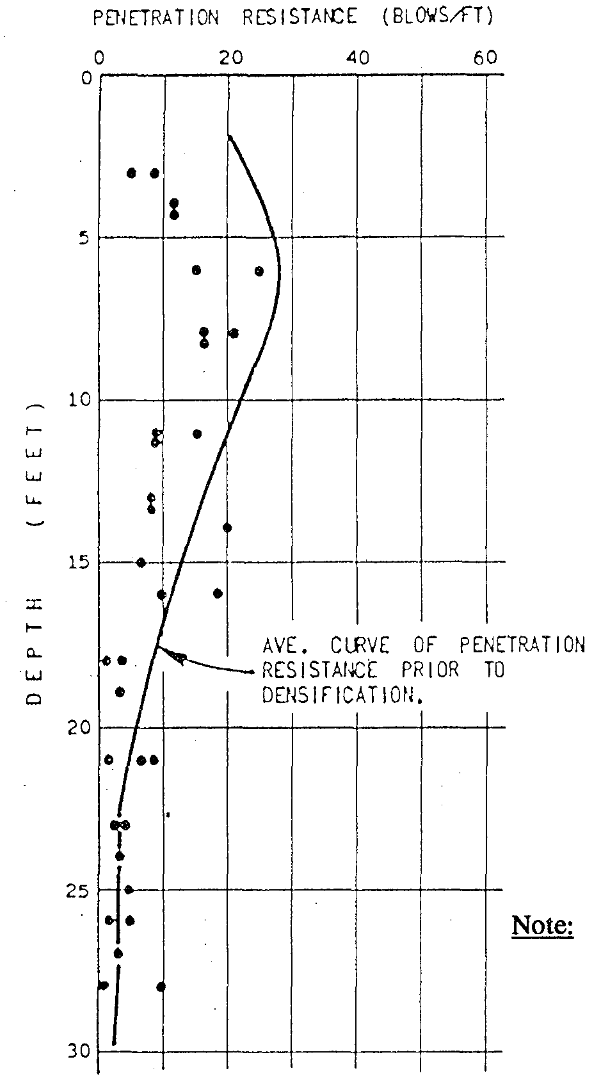
Construction Procedures and Problems

Testing Program for Compaction Piles. - A small test area was selected at the project site for determination of the required spacing between compaction piles. Approximately 80 compaction piles were driven in the test area. Sixty-five piles were spaced on 6 foot centers, 7 piles on 7 foot centers, and 8 piles on 8 foot centers, in a triangular pattern. The mandrel used was a 14-inch diameter steel casing fitted with a loose steel bottom plate. A cable was attached to the bottom plate and the mandrel in order to retrieve the plate when the mandrel was withdrawn. The mandrel was driven to the required depth and backfilled with coarse sand. The top end of the mandrel was closed and a minimum air pressure of 100 psi was applied to the column of sand as the mandrel was withdrawn.

After the 80 piles were driven, SPTs were taken at the centroid of 3 pile groups with the 6 foot spacing and at the centroid of a group with a 7 foot spacing. The SPT results are shown on Figure 10. The average curve showing the variation of penetration resistance with depth prior to densification is superimposed on the data to indicate the effect of densification on the standard blow count. The test results show no increase in penetration resistance resulting from compaction piles spaced on 6 or 7 foot centers.

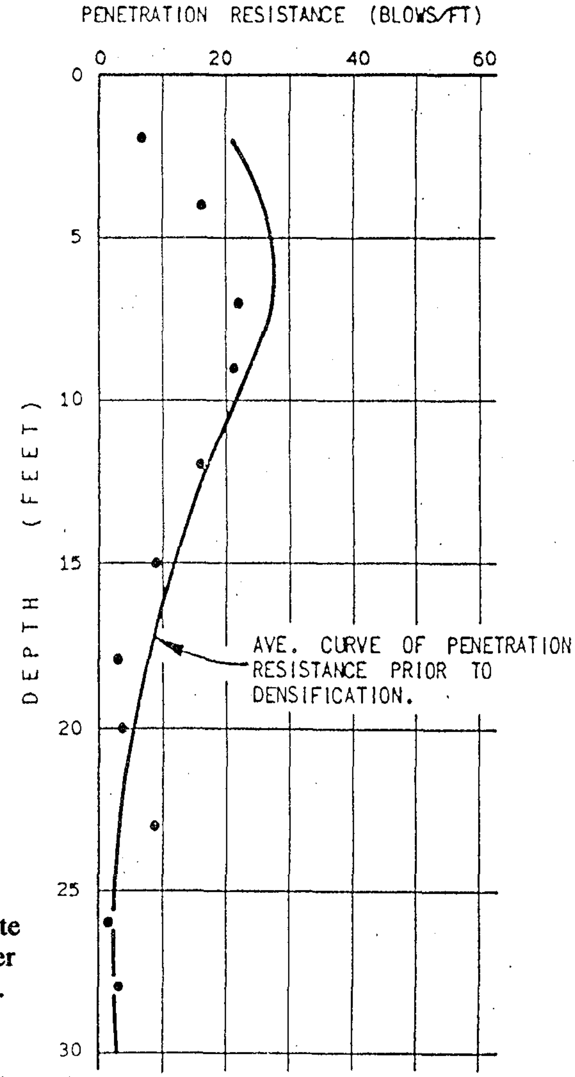
Because the compaction piles spaced on 6 foot centers did not consistently produce a minimum relative density of 75 percent in the sand fill, it was decided to drive intermediate compaction piles between the existing piles, thereby reducing the center to center spacing between the piles from 6 to 8 feet to 3 to 4 feet, respectively. The results of the SPTs taken after driving the intermediate piles are shown on Figure 11. Again, the penetration resistance prior to densification is superimposed on the data for comparison, and it may be seen that a significant increase in penetration resistance was obtained.

Testing Program for Vibrocompaction - In view of the relatively high cost of sand compaction piles, a test program using vibrocompaction was performed to investigate the effectiveness and cost of this method of densification.



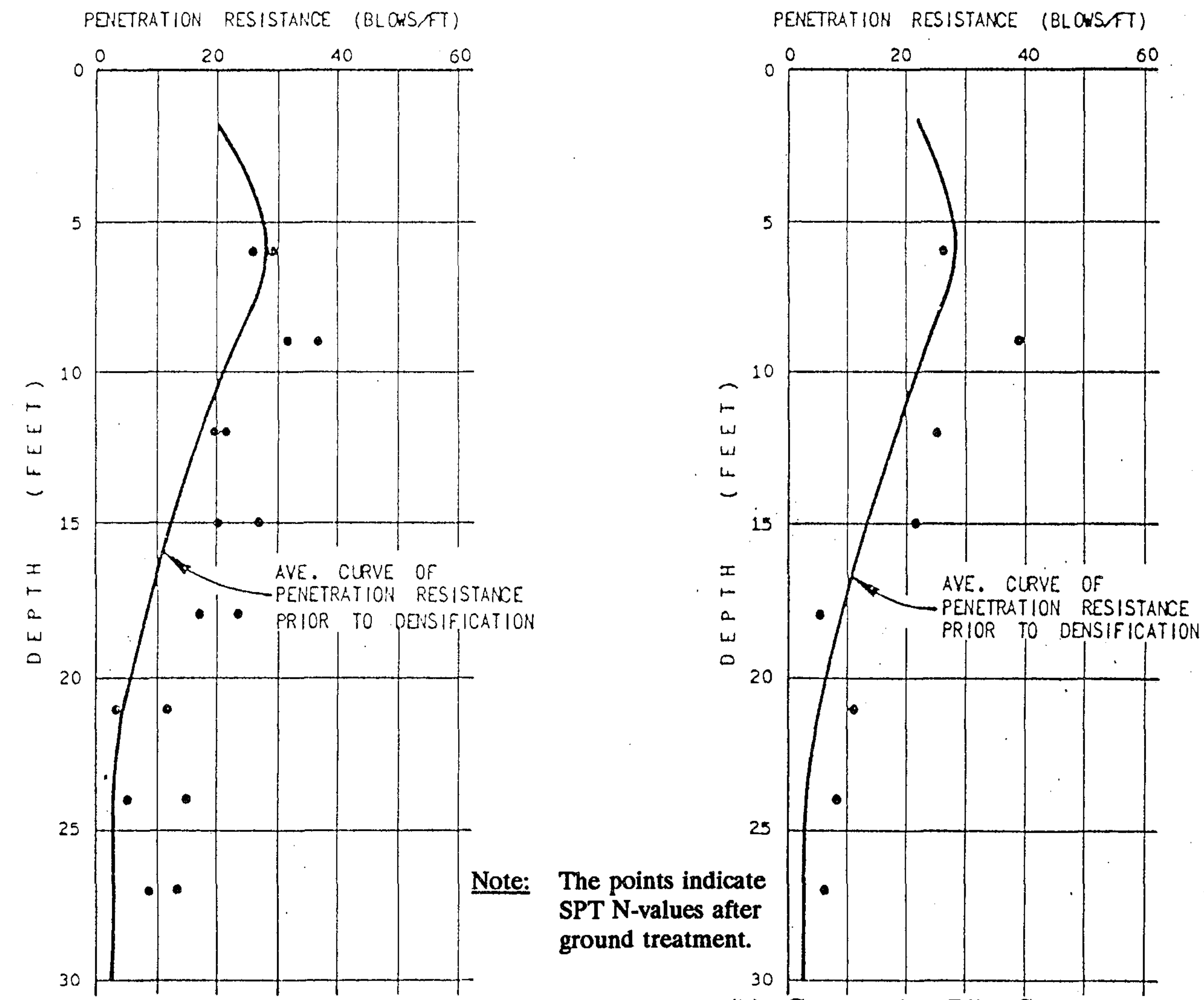
(a) Compaction Piles Spaced 6 ft on Centers

<u>Note:</u> **The points indicate SPT N-values after ground treatment.**



(b) Compaction Piles Spaced 7 ft on Centers

Fig. 10 Standard Penetration Resistance Before and After Densification by Compaction Piles Spaced 6 and 7 Feet on Centers. Office Building No. 450, Treasure Island (Basore and Boitano, 1968)



(a) Compaction Piles Spaced 3 ft on Centers

(b) Compaction Piles Spaced 4 ft on Centers

Fig. 11 Standard Penetration Resistance Before and After Densification by Compaction Piles Spaced 3 and 4 Feet on Centers. Office Building No. 450, Treasure Island (Basore and Boitano, 1968)

The test section consisted of 16 compaction points arranged in groups with spacings of 4, 5, 6, 7, and 8 feet between compaction points as shown on Fig. 12. Prior to compaction, SPTs were performed at the centroid of a group of three compaction points for each spacing. The compaction operation consisted of jetting the vibroflot through the fill to a depth of 30 feet and raising it in 1 foot intervals at a rate of 1 foot per minute. Approximately 6 to 7 cubic yards of sand was added at each compaction point.

At the completion of the compaction operations, SPTs were taken at the same locations as the initial tests. The results of the tests, indicating a greater increase in penetration resistance with decreasing spacing between compaction points, are shown on Figs. 13 and 14. The general relationship between penetration resistance and distance between compaction points for depths of 10 and 20 feet is shown on Fig. 15.

Comparison of Compaction Piles and Vibrocompaction

For a given spacing, vibrocompaction produced a much denser fill than the compaction piles, although both methods were considered effective. However, vibrocompaction cost about a $1.10 less per cubic yard of fill densified than sand compaction piles.

Ultimately, however, the owner, giving consideration to the time schedule for the project, the funds available, and existing contractual arrangements, decided to densify the building area with sand compaction piles spaced four feet on centers beneath the footings and five feet on centers beneath the floor slabs.

Final Results of Improvement

The increase in density of the fill was measured using SPT and by calculating the relative density of samples of the densified fill recovered from 15 borings located throughout the building area. The results of the control testing indicated that the density of the fill was somewhat variable. The average relative density measured at 13 of the 15 borings exceeded the specified minimum requirements, and it was concluded that while the average overall

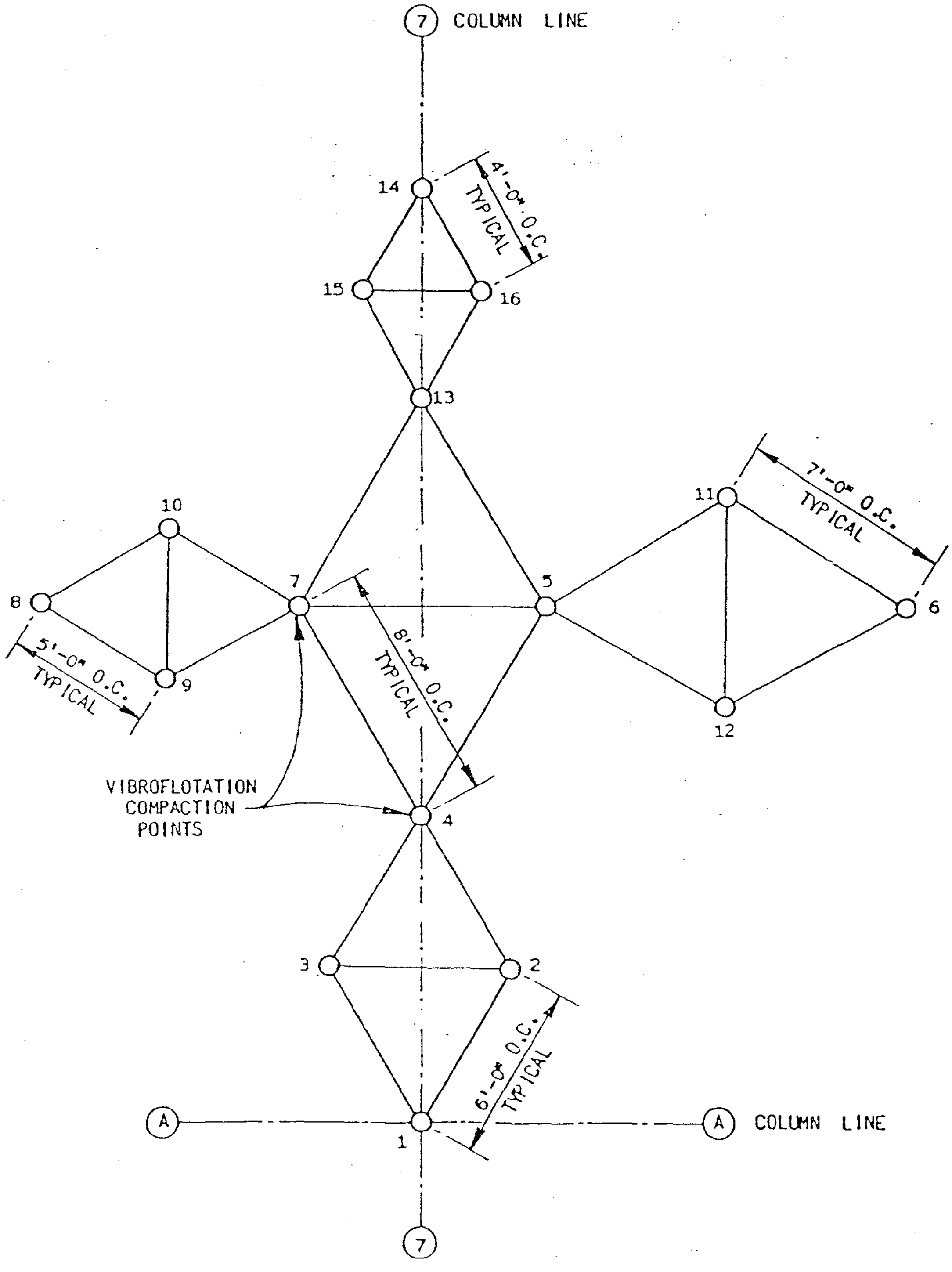


Fig. 12 Test Pattern of Vibroflotation Probe Spacings. Office Building No. 450, Treasure Island (Basore and Boitano, 1968)

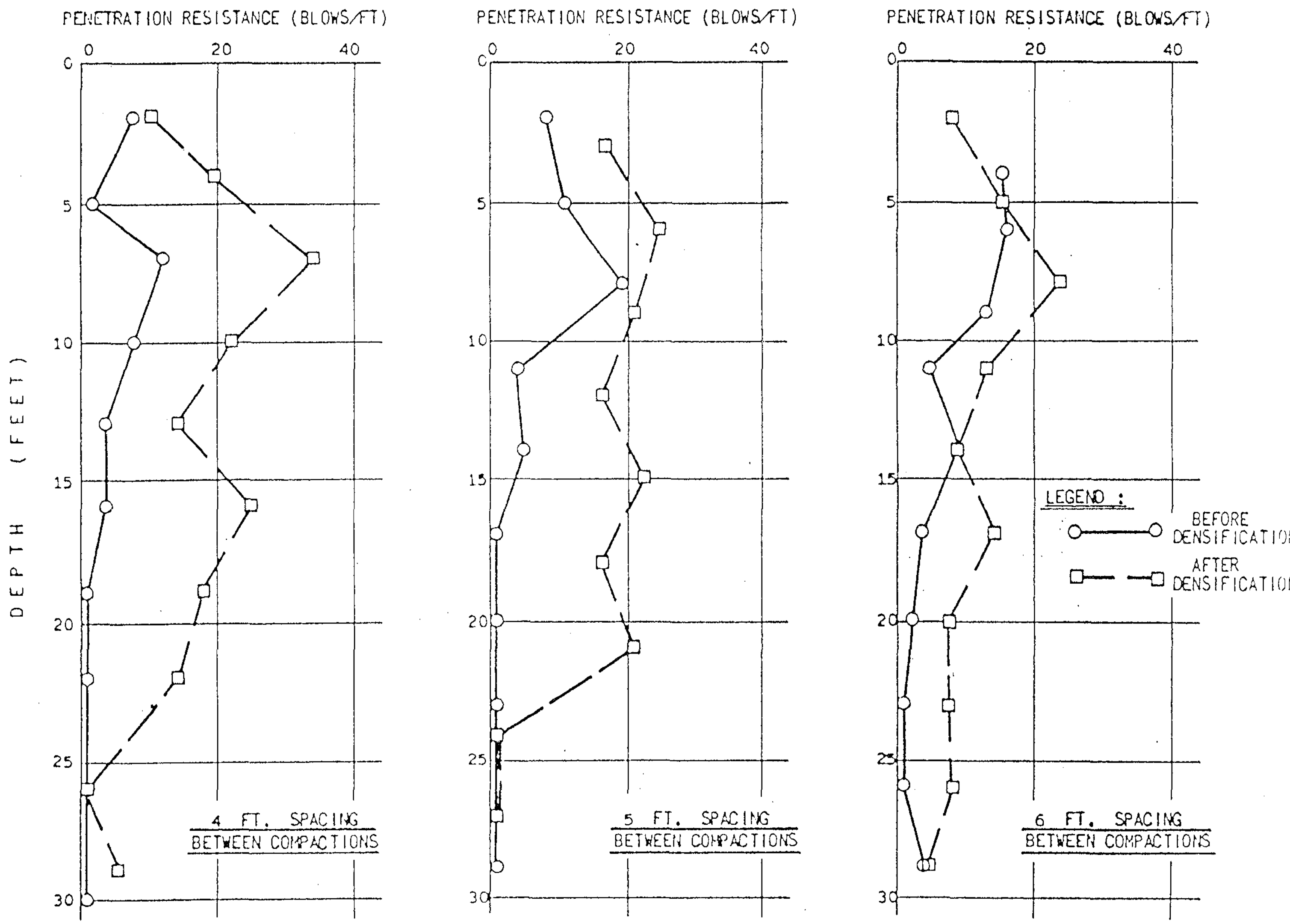


Fig. 13 Standard Penetration Resistance Before and After Densification by Vibroflotation. Office Building No. 450, Treasure Island (Basore and Boitano, 1968)

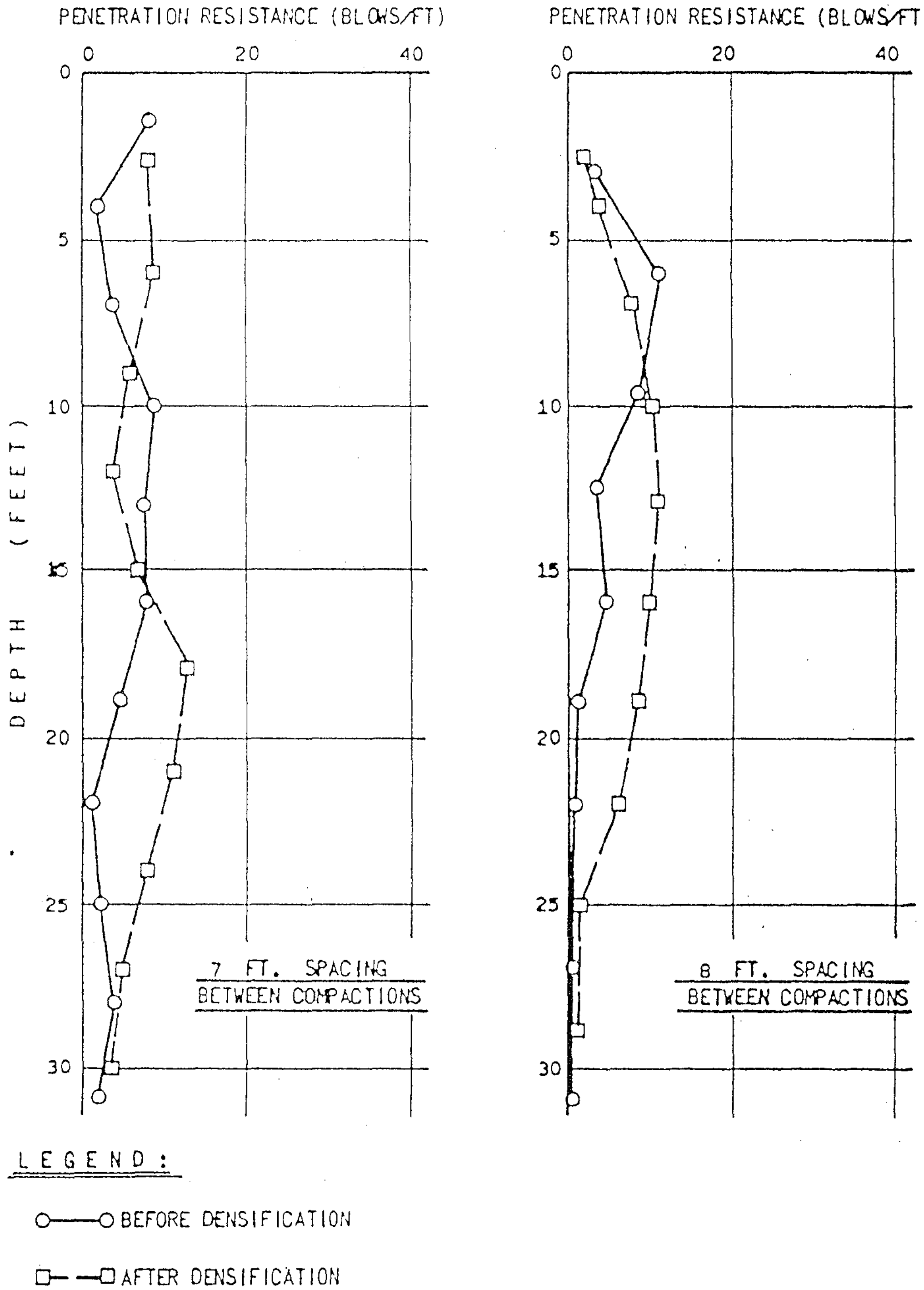


Fig. 14 Standard Penetration Resistance Before and After Densification by Vibroflotation. Office Building No. 450, Treasure Island (Basore and Boitano, 1968)

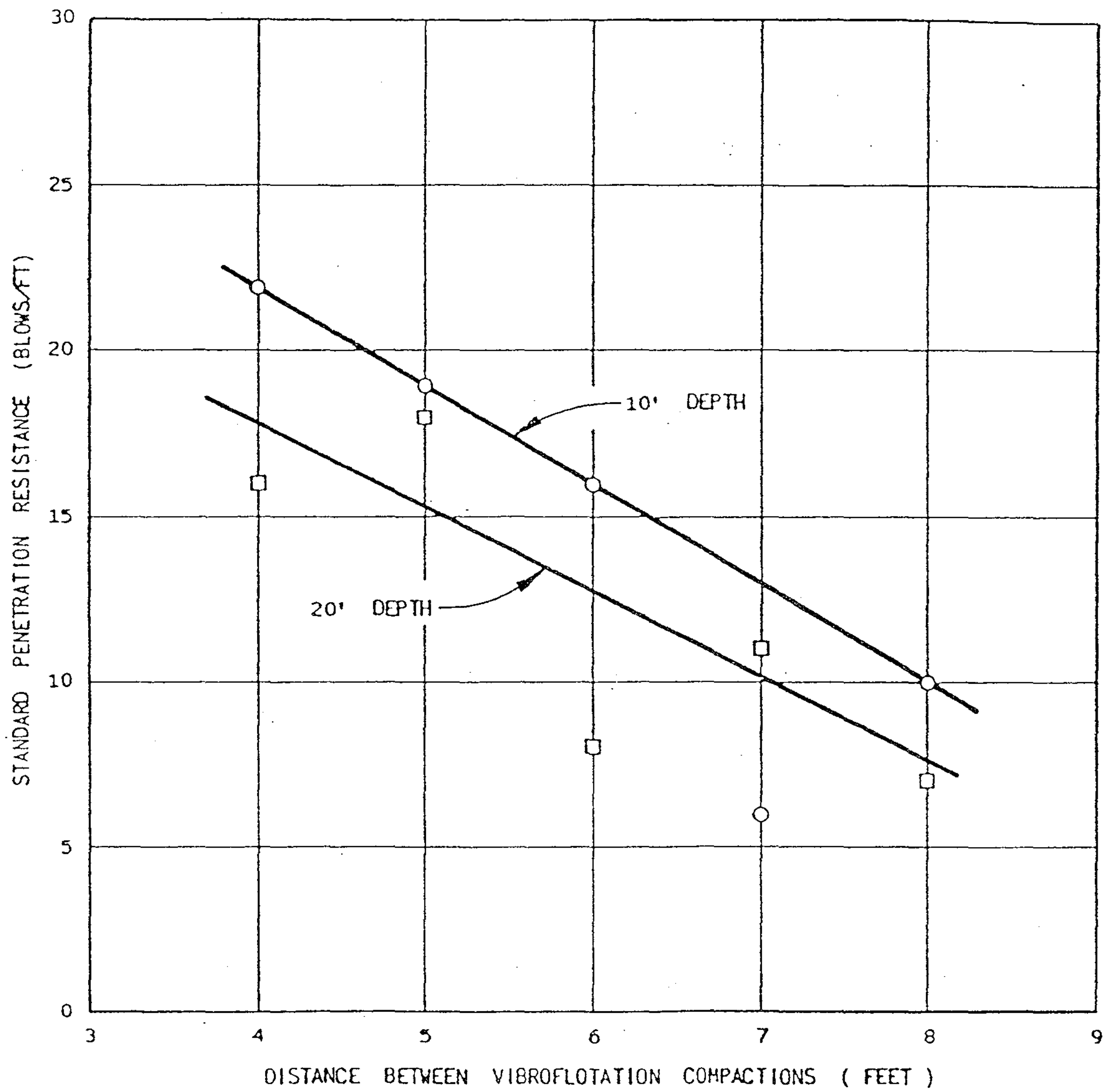


Fig. 15 Standard Penetration Resistance as a Function of Probe Spacing for Depths of 10 and 20 Feet. Office Building No. 450, Treasure Island (Basore and Boitano, 1968)

densification was adequate, isolated zones of silt and clay remained in the fill that were not densified to the specified minimum requirements.

Performance of the Project During the Loma Prieta Earthquake

Based on readings taken from a CDMG instrument on Treasure Island, the project site was subjected to a peak ground acceleration of approximately 0.16g within a bracketed duration of about 2.5 seconds during the Loma Prieta earthquake. A formal building inspection performed soon after the earthquake produced no evidence of damage. Some lateral spreading, sand boils, and localized settlement were observed outside of the improved ground areas adjacent to the buildings, thus suggesting that had the densification not been performed, the soil beneath the building foundation would have liquefied.

FACILITIES 487, 488, AND 489, TREASURE ISLAND

Project Description

Facilities 487, 488, and 489, located as shown on Fig. 6, were constructed in 1973. They consist of three story buildings with exterior and interior concrete block walls, precast concrete floor slabs, and concrete slab-on-grade first floors. Typical dead plus live loads for the longitudinal walls are approximately 3 kips per foot. Typical dead plus live loads for the exterior cross walls (bearing walls) are approximately 5 kips per foot, and for the interior cross walls (bearing walls) approximately 7 kips per foot. There are no columns on independent footings.

Initial Conditions

The site is essentially level and the ground surface is approximately 10 feet above MLLW. A subsurface section along the east-west centerline of the project site is shown on Fig. 16. The upper 24 to 33 feet of soil are composed of very loose to medium dense hydraulically

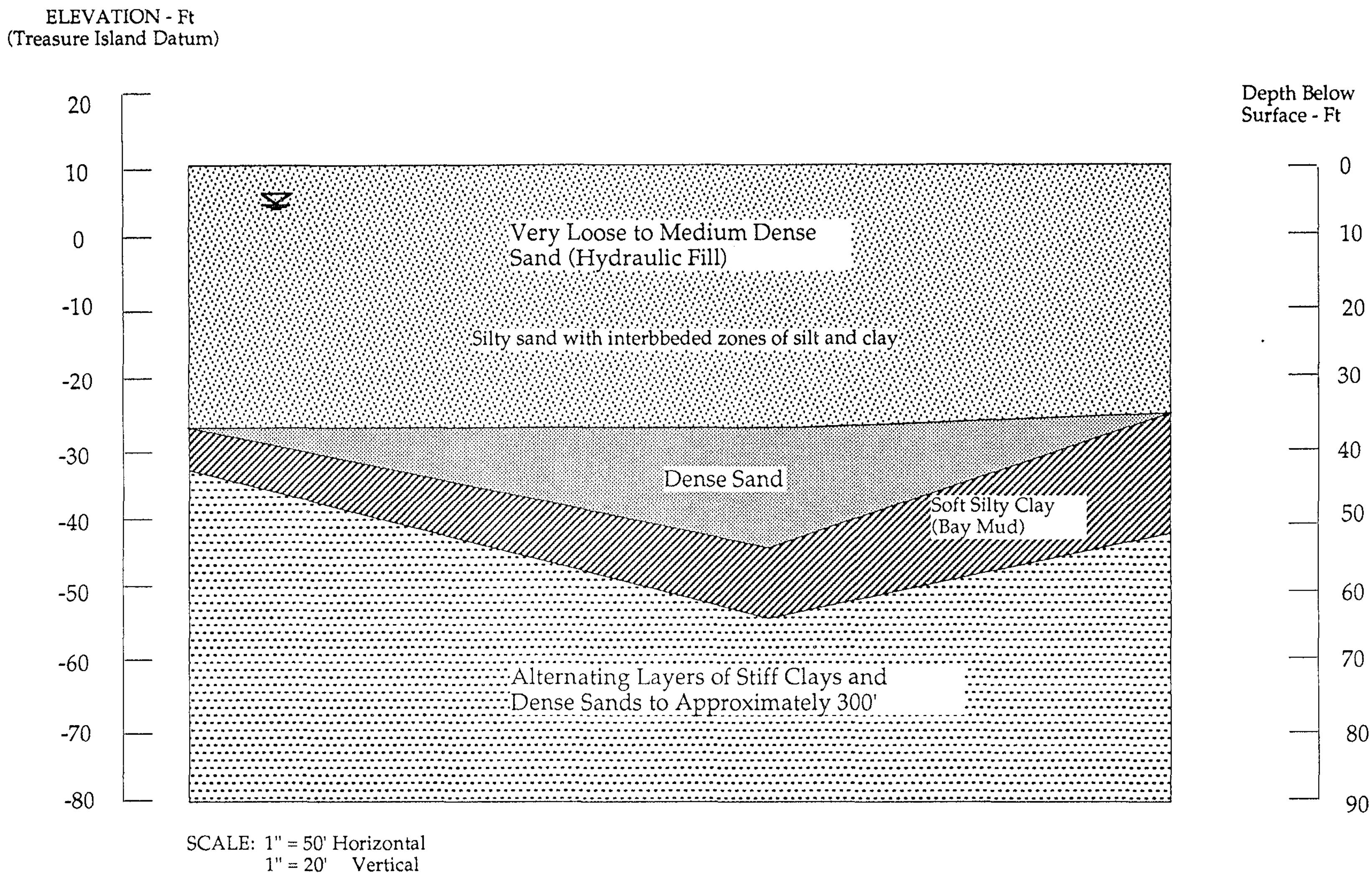


Fig. 16 Subsurface Profile at Facilities 487, 488, and 489, Treasure Island

placed sand fill. The sand is generally fine-grained and contains less than 12 percent of material finer than the #200 sieve. Occasional thin layers of soft compressible silts and clays were encountered in the fill below a depth of about 15 feet. The sand fill is underlain by a layer of soft, silty clay (Bay Mud) approximately 4 feet thick. The Bay Mud is underlain by alternating layers of dense to very dense sands and stiff clays to the depths penetrated. The groundwater level was approximately 5-1/2 ft below the ground surface (+4-1/2 ft MLLW) across the site at the time of drilling.

Liquefaction Potential - The liquefaction potential of the sand fill at the site was evaluated using the simplified procedure by Seed and Idriss (1971). According to the analysis, the existing saturated sand fill would be only marginally safe against liquefaction during earthquakes of "reasonably large magnitude" (peak ground accelerations of about 0.30 to 0.40g at the site).

Foundation Alternatives

It was concluded that the proposed structures could be supported on either spread footings bearing on sand fill that had been densified to prevent liquefaction or on driven piles extending through the fill and soft Bay Mud into bearing soils encountered below a depth of 90 feet. The use of pile foundations was considered impractical for three major reasons: 1) long piles would be very expensive; 2) there would be large downdrag forces exerted on the piles by the settlement of the Bay Mud layer; and 3) possible loss of the lateral resistance of the piles during an earthquake due to liquefaction of the sand. It was decided that spread footings bearing on densified sand fill would constitute the best foundation scheme for the proposed building.

Improvement Goals and Methods

Specified Level of Improvement - The specifications required that the hydraulic sand fill be improved to a minimum relative density of 75 percent to a depth of 30 feet beneath the buildings and to a distance of 10 feet beyond each building perimeter.

Improvement Method - Three methods of densifying the sand fill were investigated: vibrocompaction, Terra Probe, and timber displacement piles. The geotechnical engineer recommended either vibrocompaction or the Terra Probe method, and the owner selected vibrocompaction.

Construction Procedures and Problems

Trial Densification Tests - A testing program was implemented prior to production densification in order to establish the optimum spacing for the compaction points. Prior to and after densifying a test area, SPTs were performed to evaluate the effectiveness of the different spacings used. The minimum spacing between compaction points was 6-1/2 feet, forming a grid of equilateral triangles.

The specifications called for the vibrator to be inserted at each compaction point to a depth of 30 feet below the ground surface and maintained at that depth for a period of one minute, then withdrawn at a rate of not more than one foot per minute. Crushed rock no larger than 1-1/2 in in largest dimension was continuously placed around the vibrator and follow-up pipe during the densification and withdrawal process.

The specifications required that the site be allowed to dry out and then brought up to finish grade. The upper one foot of sand was compacted to at least 95% relative compaction using a vibratory compactor.

Final Results of Improvement

The increase in density of the fill was measured using SPT; however, no records of the field control testing are currently available. However, geotechnical engineers at the Western Division Naval Facilities Engineering Command have indicated that the average minimum relative densities achieved at the site equaled or exceeded the specified minimum relative density of 75 percent.

Performance of the Project During the Loma Prieta Earthquake

Based on readings taken from an a CDMG instrument located on Treasure Island, project site was subjected to a peak ground acceleration of approximately 0.16g within a bracketed duration of approximately 2.5 seconds during the Loma Prieta earthquake. A formal building inspection performed shortly after the earthquake revealed some minor cracking in the concrete floor of building 487 caused by differential settlement of the foundation; however, no repairs were required. The amount of settlement was not measured. Buildings 488 and 489 had no reported damage.

APPROACH TO BERTHING PIER, TREASURE ISLAND

Project Description

The general purpose/berthing pier, shown as Approach to Pier 1 on Fig. 6, was constructed in 1984. The project consists of a pile supported reinforced concrete general purpose/berthing pier and an approach area to the entrance of the pier. The approach to the pier is the part of the project with which this case history is concerned.

Initial Conditions

The surface of the approach is essentially level and fronts the water of the bay for approximately 100 feet. A subsurface section along the centerline of the pier, illustrating the soil conditions encountered in the test borings is shown in Fig. 17. The soil underlying the approach to the pier consists of a 43-foot thick layer of loose to medium dense hydraulically placed sand fill. The sand is generally fine- to medium-grained and contains less than 10 percent of material finer than the #200 sieve. Occasional thin lenses of soft compressible silt (Bay Mud) were encountered in the upper 20 feet of the fill. The sand fill is underlain by a layer of soft, compressible silty clay (Bay Mud) approximately 80 feet thick. The Bay Mud is underlain by alternating layers of very stiff sandy clays and dense sands extending to the depths penetrated.

Liquefaction Potential - The geotechnical engineer estimated that the peak ground surface acceleration at the site during the design earthquake would be at least 0.35g. A major concern was possible seismic instability of the waterfront slope of the approach area beneath and adjacent to the pier. The liquefaction potential of the sand fill the pier approach area was evaluated by the empirical method presented by Seed et al. (1983) that uses a correlation between observed liquefaction and Standard Penetration blow count data. Based on this analysis, it was concluded that the sand fill was susceptible to liquefaction. It was decided, therefore, that the sand fill underlying the approach area should be densified to minimize its liquefaction potential.

Improvement Goals and Methods

Specified Performance of Improvement. - The geotechnical engineer specified that at a minimum, the sand fill be densified to a relative density of at least 75 percent beneath the approach area to a depth of 40 feet. Since the top few feet of sand is not normally densifed adequately due to the lack of confining pressure at the surface, it was specified that this depth be determined by CPT. This layer of sand was to be compacted by conventional

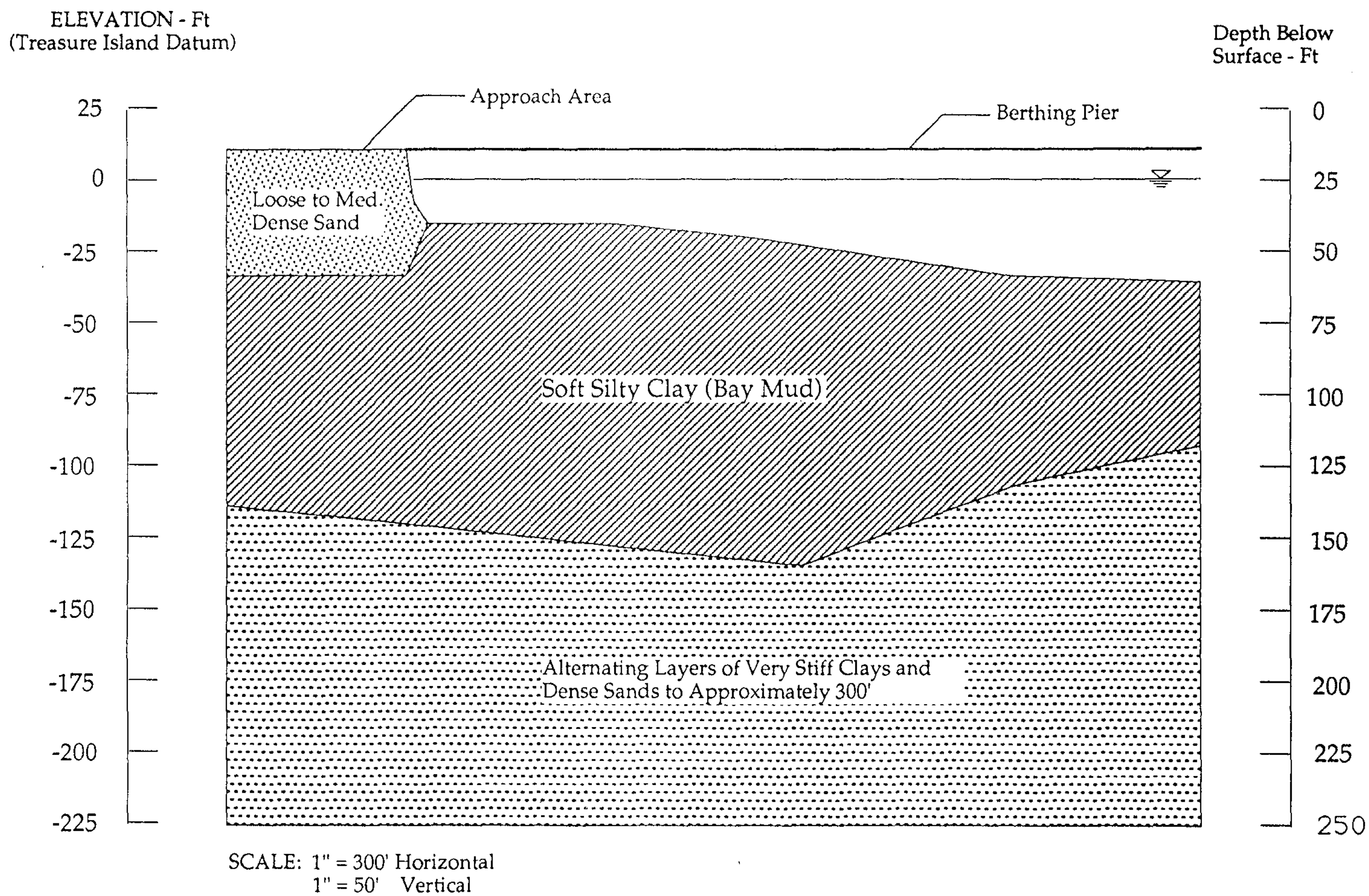


Fig. 17 **Subsurface Profile at the Approach Area of the Berthing Pier, Treasure Island**

methods to a minimum relative compaction of 95 percent based on ASTM Method D1557-78.

Improvement Method. - The three methods considered for densifying the sand fill at the site were compaction piles, vibro-replacement, and Terraprobe. The final recommendation of the geotechnical engineer, based on a combination of cost, time, and effectiveness was vibro-replacement.

Construction Procedures and Specifications

Trial Densification Tests - A series of densification tests was performed to establish the spacing criteria for the compaction probes and to determine static cone penetration resistance correlations to SPT N-values. A multiplier of 4.0 was used to correlate the SPT N values with the q_c values. This multiplier was adjusted by the geotechnical engineer to suit the actual field findings. The modified values derived served as the production densification criteria for the contractor. The backfill material was granular and contained no stones larger than 1-1/2 inches in greatest dimension. In addition, the following grading was specified:

Sieve Size	Percent Passing
3/4"	100
No. 4	50-100
No. 40	0-5

It was required that the sand fill in the test sections be densified to the standard penetration resistance N-values or static cone penetration resistance, q_c, shown in Table 3:

In connection with these criteria the SPTs were made at depth intervals of 2.5 feet. The average of three consecutive standard penetration resistance values taken at the specified depth intervals above, at, and below any depth were to be equal to or greater than the value shown in Table 3. The borings were located at points which were equidistant from three probe locations. The CPTs were performed at depth intervals not exceeding one foot. The values of static cone readings were to meet or exceed the values in Table 3, except where

Table 3: Standard Penetration and Static Cone Resistance Requirements

Depth Below Ground Surface (ft)	Standard Penetration N-Values	Static Cone Resistance $q_c = 4N$ (tsf)
5	11	44
10	15	60
15	19	76
20	22	90
25	25	100
30	27	106
35	28	114
40	30	120

the friction ratio was greater than 2.0 percent. The static cone penetrations were taken at points which were equidistant from three probe locations.

Final Results of Improvement

Static cone penetration resistance q_c values were obtained for the full depth of densification. An average of one CPT was performed for every 1,500 square feet of ground surface within the area densified. The CPT data indicated that the lower 5 to 7 feet of the fill consisted mainly of silty sand and sandy silt that did not meet the minimum densification requirements. It was judged that this material was potentially liquefiable, but that reprobing the layer would unlikely result in significant improvement. Overall, it was concluded that the densification of the sand fill was achieved with respect to the minimum relative density requirements.

Performance of the Project During the Loma Prieta Earthquake

Based on readings taken from a CDMG instrument located on Treasure Island, the project site was subjected to a peak ground acceleration of approximately 0.16g within a bracketed duration of about 2.5 seconds during the Loma Prieta earthquake. A formal inspection of the pier approach performed shortly after the earthquake revealed no signs of ground movement in the densified areas. However, there were several sinkholes and sand boils observed in the adjacent, unimproved soil areas.

BUILDING 453, TREASURE ISLAND

Project Description

Building 453, located as shown in Fig. 6, was constructed in 1969. It consists of six, four-story wings radiating from a central core. Construction is reinforced concrete. The finish floor elevation is approximately 2-1/2 feet above the existing grade. Estimated dead plus live loads for the wings are approximately 6.2 kips per linear foot for the exterior walls and 7.3 kips per linear foot for the interior walls. The core loads are carried by a series of circumferential and radial walls. It is estimated that the building loads average 735 psf and 560 psf over the gross core and wing areas, respectively.

Initial Conditions

The site is essentially level. The ground surface was approximately 10 feet above Mean Lower Low Water (MLLW) at the time of drilling. A subsurface section along the north-south centerline of the project site, illustrating the soil conditions encountered in the test borings is shown in Fig. 18. The upper 45 feet of soil consists of loose to medium dense hydraulically placed sand fill. The sand is generally fine- to medium-grained and contains less than 12 percent of material finer than the #200 sieve. Occasional thin lenses of soft compressible silt (dredged Bay Mud) were encountered in the lower 20 feet of fill. The sand fill is underlain by a layer of soft to medium stiff clayey silt (Bay Mud) approximately 20 feet thick. The Bay Mud is underlain by alternating layers of stiff clays and dense sands to the depths penetrated. The groundwater level was approximately 6 feet below the ground surface (+4 MLLW) across the site at the time of drilling.

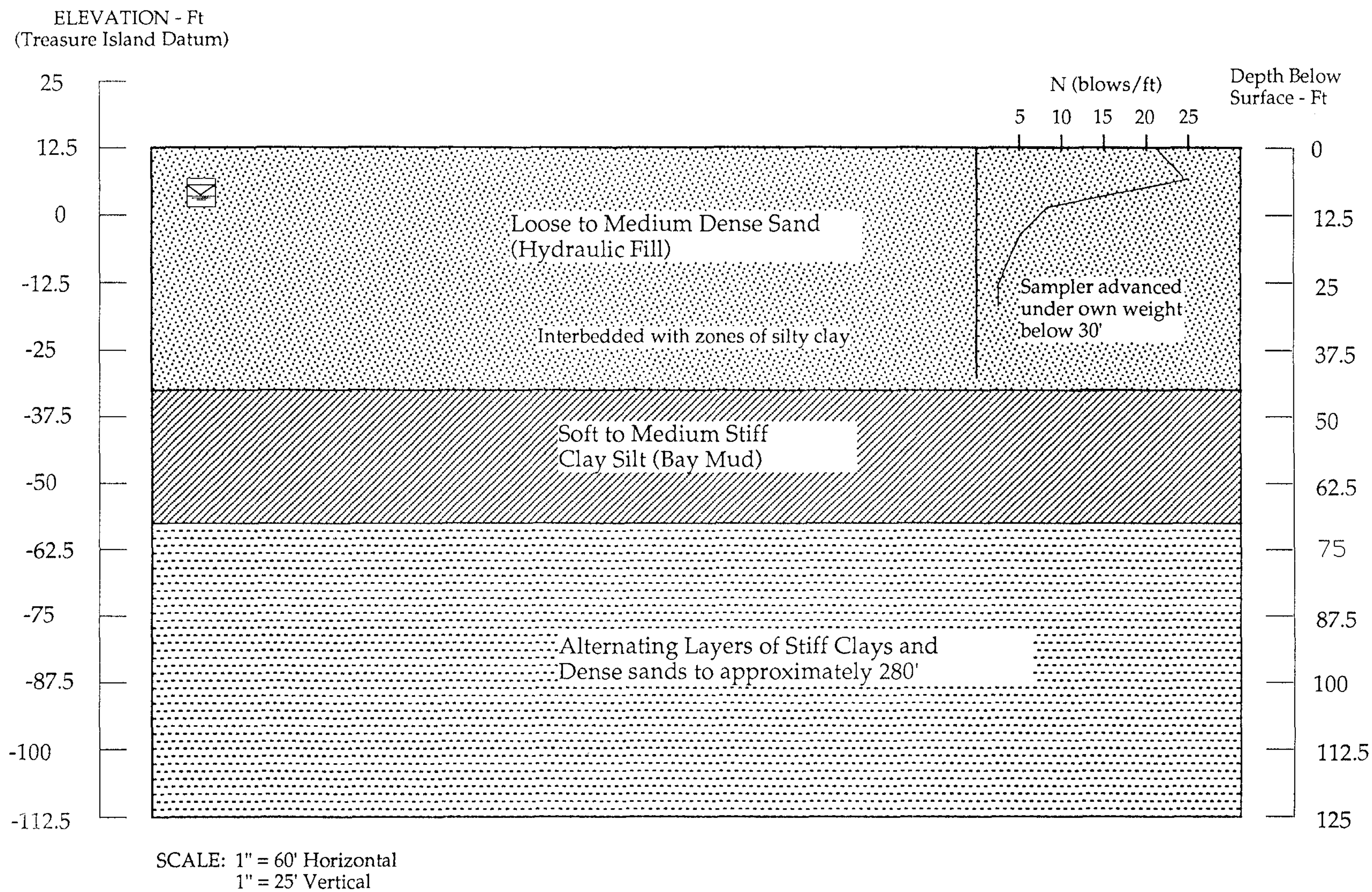


Fig. 18 Subsurface Profile at Building 453, Treasure Island

Liquefaction Potential. - At the time the project was being investigated, the Niigata earthquake was the only good prior case study for a liquefaction analysis. The initial liquefaction analysis proposed by Seed and Idriss (1967) was used to evaluate the liquefaction potential for the sand fill at the site. From this analysis, it was concluded that liquefaction of the upper 30 feet of sand fill was possible in a large earthquake (peak ground accelerations > 0.35g).

Foundation Alternatives

Based on the results of the geotechnical investigation, it was concluded that the building could be supported on either spread footings or piles. Because of the potential for liquefaction of the sand fill, it was decided that if spread footings were used, the sand fill should be densified. It was concluded that the use of piles for structural support would not be as economical as spread footings bearing on densified fill because of the modest strength of the supporting soils and the large downdrag forces that would be developed in the soft clay and fill.

Improvement Goals and Methods

Specified Level of Improvement. - The geotechnical engineer specified that at a minimum, the sand fill should be densified to a relative density of at least 70% to a depth of 30 feet under the building and to a distance 10 feet beyond the building perimeter in order to prevent liquefaction. It was noted that normally, sand densification methods are not effective in the upper few feet of the soil layer because of the lack of confining pressure at the surface. Because of this, it was specified that the upper four feet of fill be excavated, backfilled, and recompacted to a minimum relative compaction of 95% based on the ASTM D1557-78 compaction test method.

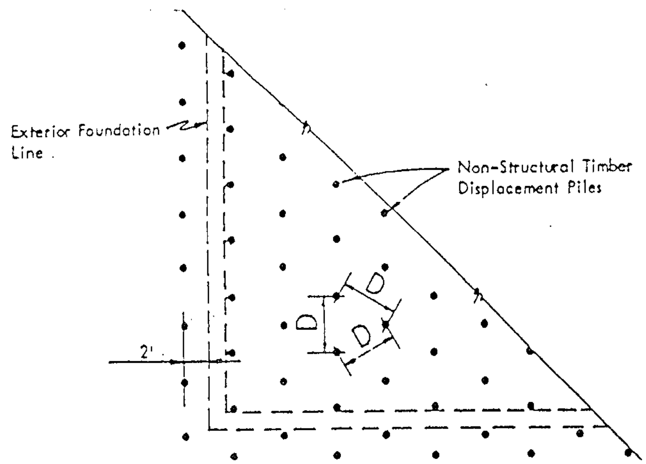
Improvement Method. - Three methods were considered for densification of the sand fill: vibrocompaction, sand compaction piles, and non-structural displacement piles. The sand compaction piles were to be 14 inches in diameter and spaced approximately 3-1/2 foot on centers. The engineer noted that compaction piles would probably not achieve a high degree of uniformity in density.

The non-structural timber piles were to be Class C piles with an 8-inch minimum tip diameter and 12-inch minimum diameter three feet from the butt. They were to be approximately 20 feet long and driven to a depth of approximately 25 feet into the fill. The tops of the piles would be driven below grade so that they would be below the permanent water table and therefore be immune to deterioration. The fill above the pile butts would then be excavated and recompacted to refill the voids created by the follower.

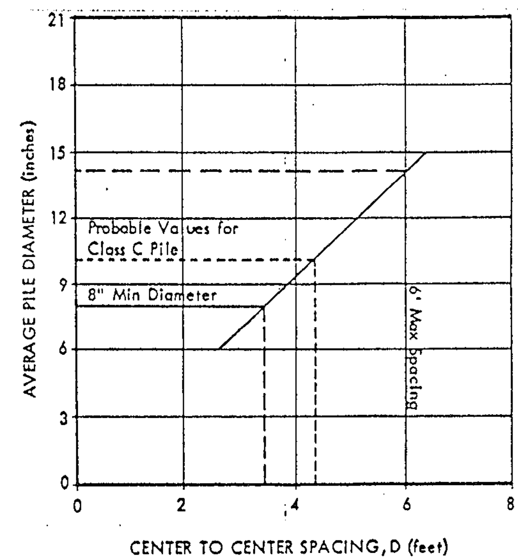
The cost in 1969 of vibrocompaction was estimated to be approximately $3.00 per sq. ft. of ground surface. The cost of the sand compaction piles and non-structural displacement piles were estimated to be $1.50 per sq. ft. and $2.60 per sq. ft., respectively. The owner decided to use non-structural displacement piles to densify the site.

<u>Construction Procedures</u>

Although field tests were performed in order to determine the required spacing of the displacement piles, the data are not currently available. A relationship between pile spacing and average pile diameter as developed by the geotechnical engineer is shown Fig. 19B. It was assumed that the Class C pile would yield an average diameter of 10 inches in the "loose zone" (16 to 25 feet below the ground surface) and therefore require a 4.3 foot center to center spacing in a triangular pattern as shown in Figure 19A. Since the non-structural displacement piles were driven approximately 6-1/2 feet below grade, the soil above the piles was excavated and recompacted to remove the voids created by the pile follower.



A.



B.

Fig. 19 A. Triangular Pattern of Displacement Piles
B. Spacing of Displacement Piles as a Function of Pile Diameter Building 453, Treasure Island (Harding-Miller-Lawson & Associates, 1968)

Final Results of Improvement

The increase in density of the fill was measured using SPT. The final average relative density of the sand fill is not known. However, it is concluded that the densification program was successful based on discussions with Naval Facilities Command engineers.

Performance of the Project During the Loma Prieta Earthquake

Based on readings taken from a CDMG instrument located on Treasure Island, the project site was subjected to a peak ground acceleration of approximately 0.16g within a bracketed duration of about 2.5 seconds during the Loma Prieta earthquake. The building was inspected for damage shortly after the earthquake, and no major structural damage was observed in the building although there was a concrete spall at the end of one wing. In addition, there was some cracking in the floor system and some repairs were required for the slab-on-grade due to minor ground settlements (less than 3/8 inch). No foundation repairs were required, however. No sinkholes, sand boils, or other evidence of liquefaction was observed around the building.

EASTERN SHORE, MARINA BAY, RICHMOND, CALIFORNIA

Project Description

The project, as shown in plan in Fig. 20, consisted of expanding the Marina Bay Esplanade by extending it approximately 1000 feet along the eastern shore of Marina Bay in 1987. The expansion includes walkways, landscape areas, and light standards. The walkways are supported at grade with a finish surface elevation of about +7.5 feet, National Geodetic Vertical Datum (NGVD). The adjacent shoreline is sloped 3:1 (horizontal : vertical) and protected with rock riprap. There is a large residential development just east of the Esplanade.

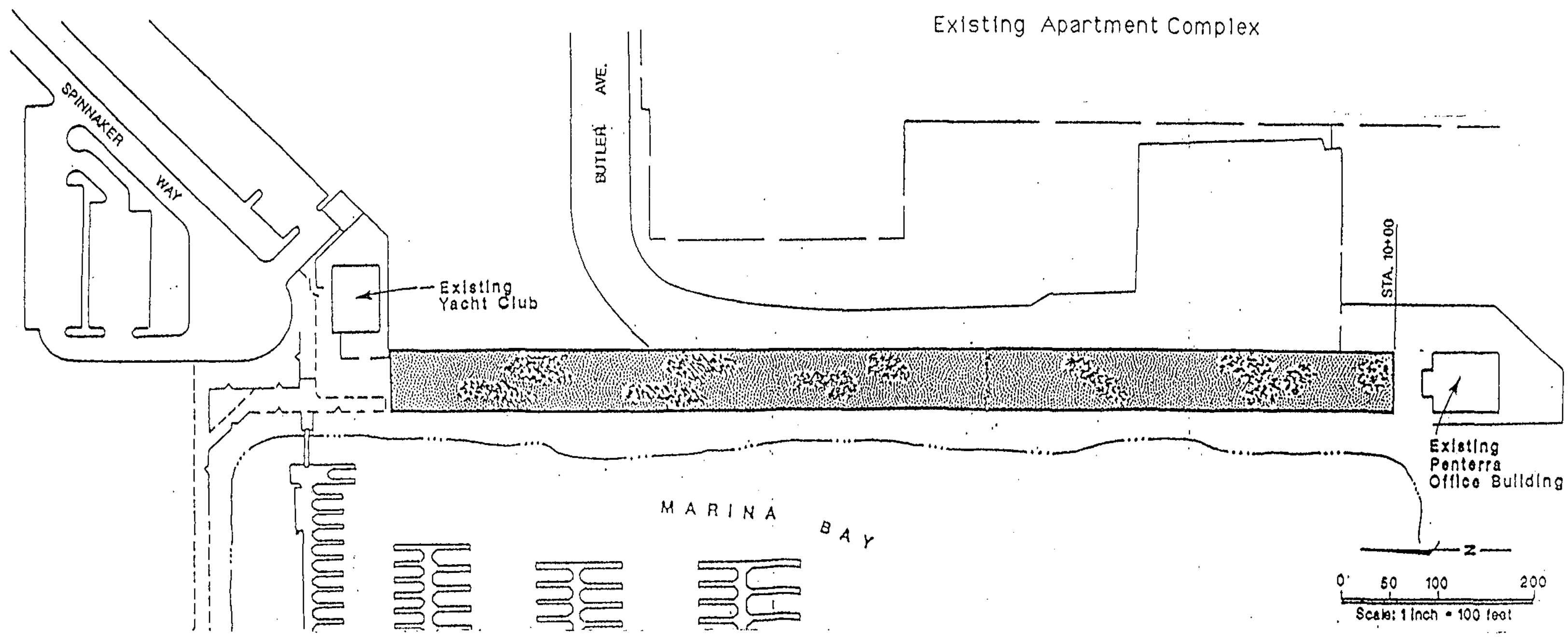


Fig. 20 Site Plan - Eastern Shore, Marina Bay, Richmond, CA (Harding Lawson Associates, 1987)

Initial Conditions

At time of the geotechnical investigation, the site was nearly level with a surface elevation of about +14 feet and paved with asphalt concrete. Adjacent to Marina Bay, the ground surface sloped on an average of 2:1 to the water. A subsurface section east-west across the project site, illustrating the soil conditions encountered in the test borings is shown on Fig. 21. The upper 13 feet of soil is composed of medium dense to dense sandy and gravelly artificial fill interspersed with clay inclusions and construction debris. The artificial fill is underlain by a layer of loose hydraulically placed silty sand and sandy silt approximately 11 feet thick. This deposit is generally fine-grained and contains up to 55 percent of material finer than the #200 sieve. The hydraulic fill is underlain by medium stiff to stiff clays to the depths penetrated. The groundwater level was approximately 10 feet below the ground surface (+5 NGVD) across the site at the time of drilling.

Liquefaction Potential. - The liquefaction potential of the hydraulically placed silt and sand fill was evaluated using the analytical-empirical procedure based on the liquefaction behavior of saturated clean and silty sands during historic earthquakes by Seed, et al. (1984). A design earthquake of magnitude 6.5 resulting in a peak ground acceleration of 0.30g at the site was used in all liquefaction analyses. The primary data used in the evaluation consisted of SPT N values obtained from field investigations and corrected for fines content (Seed, 1987). The resulting $(N_1)_{60}$ values varied between 11 and 22 with an average value of 15.

Based on the results of the liquefaction analysis, it was concluded that a continuous deposit of liquefiable soils existed between elevations +5 and -11 feet along the entire length of the proposed project. This deposit was overlain by a dense surface layer of liquefaction-resistant material approximately 9 feet thick.

In the event of liquefaction of the underlying deposit, the surface layer would prevent complete loss of bearing capacity. However, because the liquefiable deposit is adjacent to

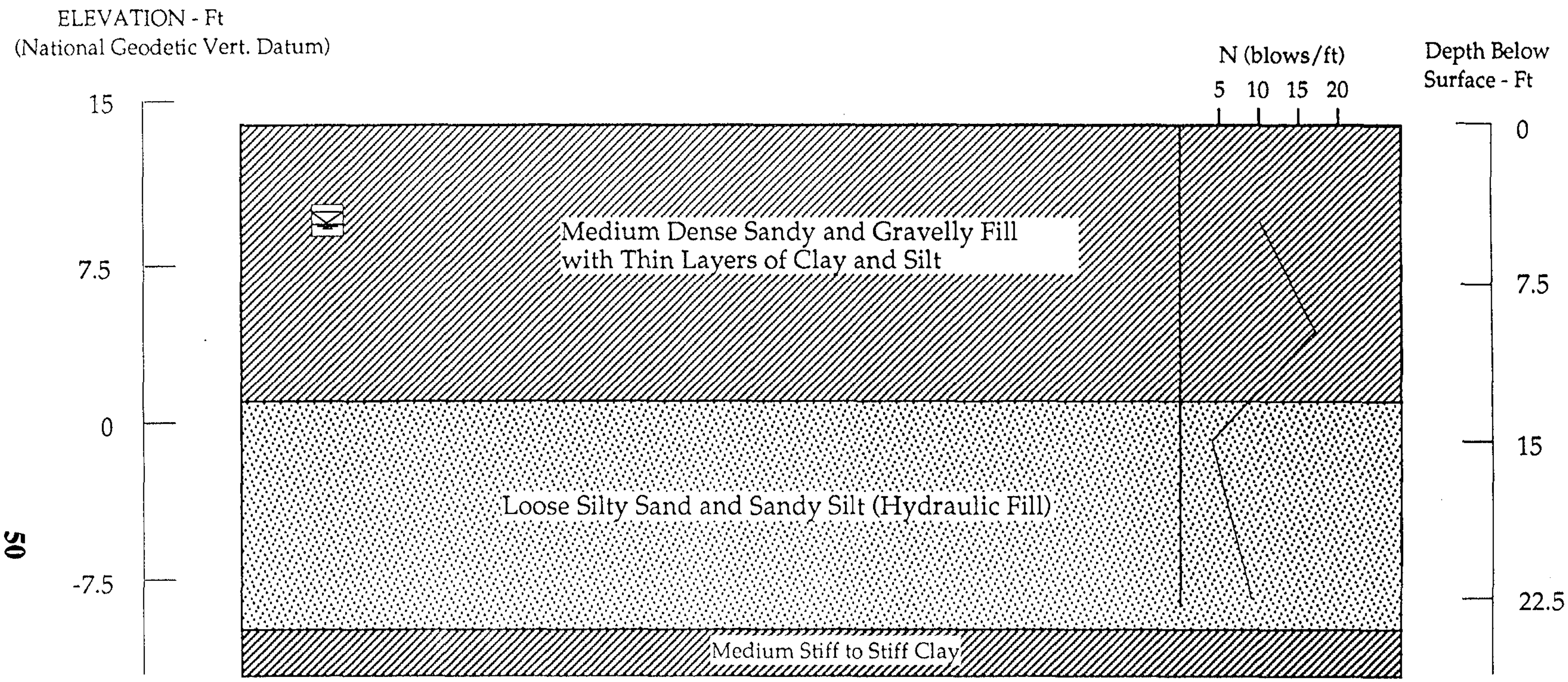


Fig. 21 Subsurface Profile at the Eastern Shore of Marina Bay Esplanade, Richmond, CA

Marina Bay, and lateral spreading during an earthquake was a strong possibility, there was a significant risk to inland development. Both the City of Richmond and the developer agreed that it would be uneconomical to eliminate the liquefaction potential of the entire deposit under the site. Therefore, it was decided to construct a "buttress" through the liquefiable deposit along the shoreline boundary to resist lateral spreading.

Improvement Goals and Methods

Specified Level of Improvement. - The goal was to construct a liquefaction resistant buttress along the shoreline to contain the liquefied soil on the inland side thereby preventing bayward lateral spreading. The specified level of soil improvement required that the liquefiable soils in the buttress area be densified enough to prevent liquefaction based on CPT correlations proposed by Robertson and Campanella (1985), and Seed, et al. (1983), and by using SPT data (Seed et al., 1984).

Improvement Method. - Several densification methods were considered for developing a buttress that would be stable and buildable given the economic and time constraints of the project. Because of the high fines content of the liquefiable deposit, it was believed that ground improvement methods such as deep dynamic compaction or vibrocompaction would not densify the soil adequately to form an effective buttress. Therefore, it was decided to construct the buttress using the vibro-replacement stone column technique (Rinne, et al, 1988).

The buttress consists of 42-inch diameter stone columns placed 6 feet on center in a square grid, extending about 1 foot below the bottom of the liquefiable zone at elevation -12 feet, as shown on Fig. 22. The buttress is trapezoidal in cross section with crest and base widths of about 16 and 58 feet, respectively. The inland face of the buttress slopes at about 1:1 and the outboard (bay side) face slopes at about 2:1. The crest elevation is +6 feet.

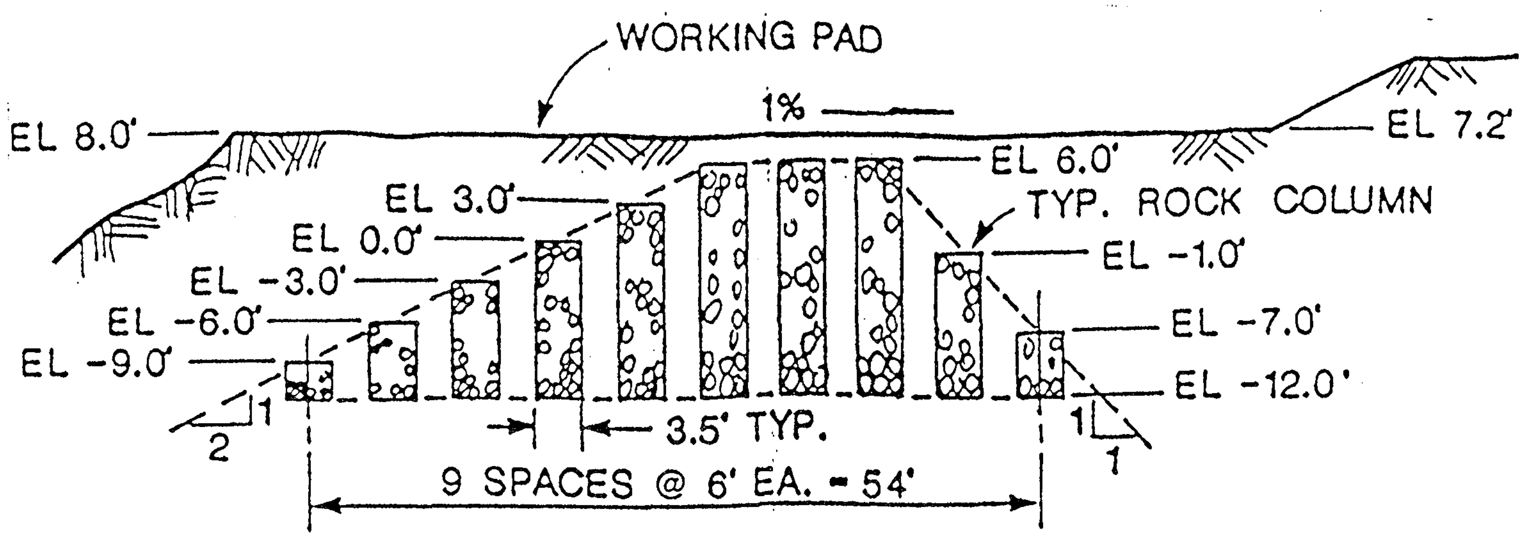


Fig. 22 Typical Buttress Cross Section, Marina Bay Esplanade, Richmond, CA (Harding Lawson Associates, 1987)

The stone column buttress is intended to limit lateral spreading in two ways. First, the columns, which consist of dense, liquefaction resistant crushed-rock, reinforce the slope along the shoreline. Secondly, the installation of the columns increases the liquefaction resistance of the sands and silts around the columns by densification and provides both increased lateral confinement and a drainage path to dissipate earthquake-induced pore pressures.

Construction Procedures and Problems

Construction Method. - The stone columns were constructed using two 1-1/2-foot diameter, 12-foot long, downhole vibrators suspended from cranes. The vibrators were advanced into the ground dry, primarily by their vibratory energy. The hole created by the vibrator was backfilled with 3/8-inch by 1-inch crushed rock placed in about 3-foot thick lifts using a bottom-feed system. The amount of crushed rock required for each lift was determined by assuming an in-place relative density and computing the weight of rock required to achieve a 42-inch diameter column at this density.

Problems. - For the first several days of construction, the cranes were working from a pad excavated to an elevation of +8 feet and frequently became stuck in the soft subgrade soils. Consequently, they were moved to the paved area just east of the buttress alignment. Minor slope failures and settlement occurred in the asphalt-paved area north of the Penterra office building (see Fig. 20) due to construction induced vibrations. In addition, tension cracks appeared in several areas behind the vertical slope parallel to the centerline of the buttress. Because of concern that the construction-induced vibrations might damage the Yacht Club at the north end of the buttress, the northernmost row of columns was eliminated. In addition, the vibrators were not able to penetrate the ground surface at five locations along the western most row of columns because of concrete debris in the fill. Therefore, the entire row of columns was shifted 6 feet to the east at these locations.

Field Control

During the installation of each stone column, the as-built column tip elevation and column length were recorded, and the amount of rock placed in each column was monitored. In addition, the maximum drive motor resistance of the downhole vibrator was recorded for each column to give a qualitative indication of the increase in the relative density of the rock.

Final Results of Improvement

To determine the effect of placing stone columns in the liquefiable deposit, 10 test borings were drilled and 12 CPT probes were advanced in various locations between the stone columns. CPT tip-resistance values were converted to q_{c1} using overburden pressures using correction factors proposed by Robertson and Campanella (1985). The correlations between liquefaction resistance and q_{c1} values proposed by Robertson and Campanella (1985) and Seed, et al (1983), and SPT data (Seed et al., 1984) were used to evaluate the post-construction liquefaction potential of the sand and silt in the buttress zone.

Average pre- and post-construction q_{cl} values are shown in Fig. 23. It can be seen that the liquefaction potential of the deposits prior to densification was moderate to high using the Seed correlation, and high using the Robertson and Campanella correlation. The average q_{cl} value increased by approximately 45 kg/cm2 following column installation. It was concluded, based on the CPT correlations, that the post-construction liquefaction potential of the hydraulic fill between the stone columns was low for the design earthquake. It should be noted that increases in CPT tip resistance were greatest in zones with lower silt contents.

Average pre- and post-construction $(N_1)_{60}$ values are plotted in Fig. 24, which presents the correlation between liquefaction resistance and $(N_1)_{60}$ values for the maximum credible earthquake (Magnitude 7.5). As shown, the average $(N_1)_{60}$ value increased by 7 blows/foot. Despite the increase, some potential for liquefaction of the soil between the stone columns

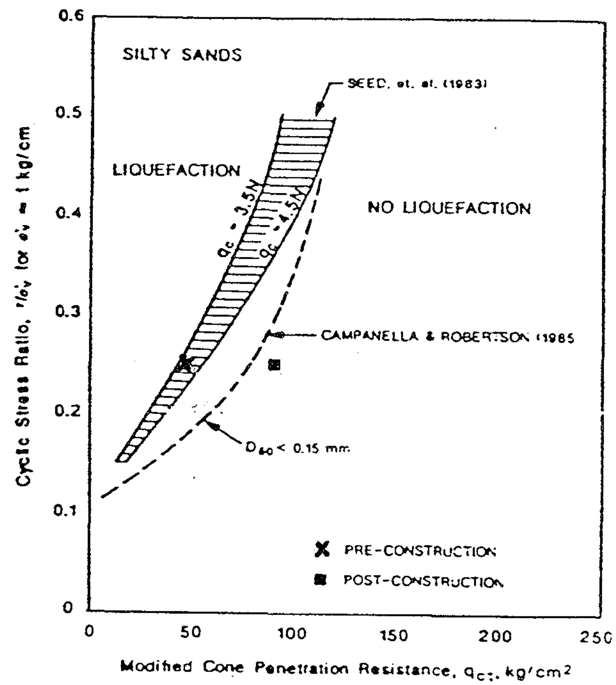


Fig. 23 Comparison of Pre-Construction and Post-Construction CPT Data (M = 6.5 Earthquake), Marina Bay Esplanade (Rinne, et al., 1988)

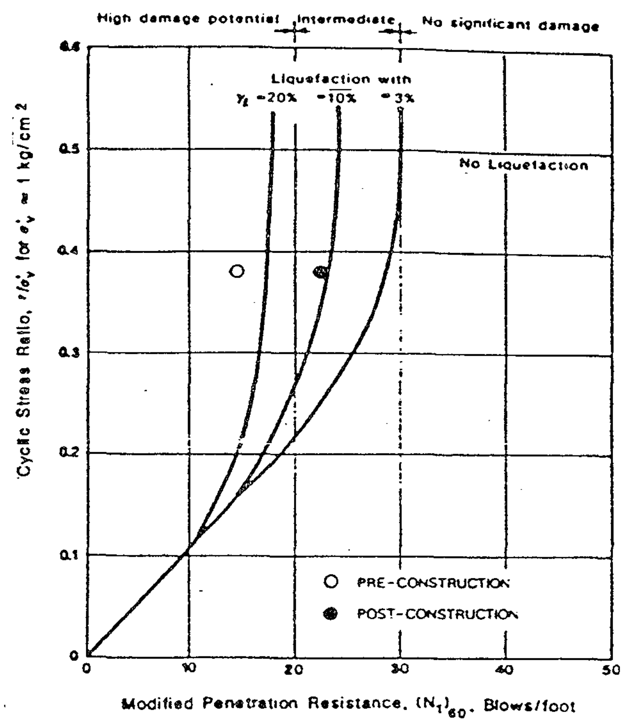


Fig. 24 Comparison of Pre-Construction and Post-Construction SPT Data (M = 7.5 Earthquake), Marina Bay Esplanade (Rinne, et al., 1988)

during a maximum credible earthquake remains. However, the estimated shear strain potential of the liquefiable deposits decreased from greater than 20 percent to approximately 10 percent as a result of treatment. Overall, the combined effects of the stone column reinforcement and the improvement in the liquefaction resistance of the soil between the columns was considered sufficient to prevent lateral spreading into the Bay in the event of liquefaction of the untreated ground on the inland side of the buttress.

Performance of the Project During the Loma Prieta Earthquake

Based on readings taken from a USGS instrument located in Richmond, the project site was subjected to a peak ground acceleration of approximately 0.11g within a bracketed duration of about one second during the Loma Prieta earthquake. No evidence of liquefaction or lateral spreading within or behind the buttress area was detected. Some small sand boils were observed in undeveloped areas within about one mile of the project site, however.

EAST BAY PARK CONDOMINIUMS, EMERYVILLE, CALIFORNIA

Project Description

A condominium development, consisting of a 30-story "tripod-shaped" tower and a 4-story parking garage, was constructed in 1983. The condominium tower is a reinforced concrete, ductile frame structure. The three wings of the tower measure approximately 70 feet by 140 feet in plan dimensions. Interior and exterior column loads in the tower are approximately 3000 kips and 2300 kips, respectively, for combined dead-plus-live loads. The parking garage consists of a reinforced concrete, shear wall structure, measuring approximately 125 feet by 514 feet in plan dimension. Column loads in the garage are approximately 800 kips and 500 kips, respectively, for combined dead-plus-live loads.

Based on bearing capacity and settlement considerations deep pile foundations were specified for support of the heavy column loads of both the tower and the garage. Other foundation types, including a mat foundation for the tower and footing foundations for the garage, were considered; however, settlement analyses indicated that they would be subject to marginal or excessive settlements. Thus, it was decided to support the tower on 14-inch square prestressed concrete piles, and the garage on 12-inch square prestressed piles.

Initial Conditions

At the time of the geotechnical investigation, the site was essentially level and cleared of all existing structures. The upper 10 to 20 feet of soil consist of medium dense hydraulically placed sand interspersed with occasional lenses and thin layers of soft silty and sandy clays and containing minor amounts of concrete, brick, and roofing paper. The sand fill is generally fine-grained and contains less than 5 percent of material finer than the #200 sieve. The fill is underlain by a layer of soft to medium stiff clayey silt (Bay Mud) approximately 5 to 12 feet thick. The Bay Mud is underlain by alternating layers of very stiff clays and dense sands and gravels to the depths penetrated. Free groundwater was encountered across the site at a depth of approximately 5 feet below the ground surface at the time of drilling. An idealized soil profile is shown in Fig. 25.

Reason for Soil Improvement

This project is unusual in that both deep foundations and ground improvement were used. The liquefaction potential of the sand fill was evaluated based on SPT blow count data obtained in the exploratory borings, and using applicable simplified liquefaction analysis procedures (Seed, 1979). The results indicated that because the sand fill is typically in a medium dense condition, liquefaction and a corresponding complete loss of soil strength was unlikely. However, the sands below the groundwater level could experience some cyclic mobility (Seed, 1979) during moderate to strong earthquake ground shaking at the site, and such movements could adversely affect the pile foundations.

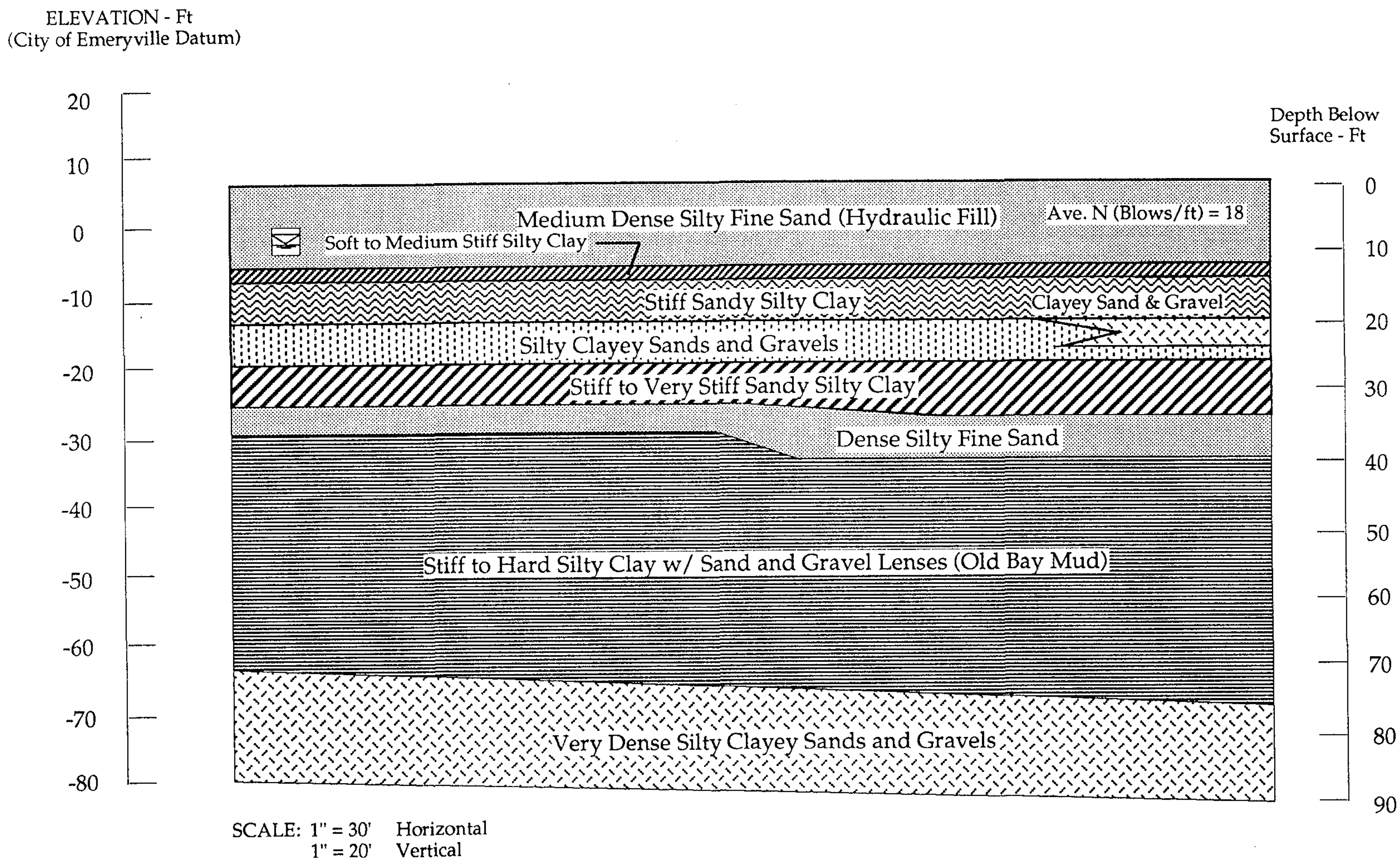


Fig. 25 Subsurface Profile at the East Bay Park Condominium

In addition, some areal settlements were expected to occur due to densification of the sand fill during moderate to strong earthquake ground shaking. Based on data presented by Lee and Albaisa (1974), and using SPT data for the site, it was estimated that settlements of up to 1 to 1.5 inches could occur. Although these settlements would not affect the pile foundation, they could damage underground utilities, pavements, and other surface improvements. Based on these considerations, the geotechnical engineer recommended that the sand fill be densified to minimize the potential for cyclic mobility and seismic compaction settlements and to optimize the overall seismic performance of the site.

Ground Improvement

Densification of the sand fill was done using vibrocompaction. The authors have been unable to locate final construction reports; however, the following information was provided by engineers who worked on the project[2]. More than 1000 vibro-compaction probe points were spaced in a triangular pattern on 8-ft centers that extended a minimum of 20-ft beyond the foot print of the tower structure. Pea gravel was used as the backfill in the vibro-compaction holes. The relative density of the hydraulic sand fill was increased to a value greater than 100 percent as inferred from SPT N-values determined after densification. The sand was so dense after treatment that it was necessary to predrill through it in order to drive the foundation piles for the high-rise structure.

Performance of the Project During the Loma Prieta Earthquake

The site of the East Bay Park Condominiums was subjected to a peak ground surface acceleration of about 0.26 g within a bracketed duration of about two seconds. No ground settlement or damage to the 30 storey tower was observed.

---

[2] Personal communications, September 1991, from Laurence R. Houps, Woodward-Clyde Consultants, Oakland, California, and Tom Graf, Geomatrix Consulatants, San Francisco, California.

PERIMETER SAND DIKE, HARBOR BAY ISLE DEVELOPMENT, ALAMEDA, CALIFORNIA

Project Description

Perimeter dikes, shown on Fig. 26, were constructed beginning in 1965 around the Bay-side of a reclaimed land site to retain hydraulic fill pumped into the area behind the dikes. The dikes were constructed by excavating a trench with a clamshell or dragline in the bottom of the Bay adjacent to and generally outboard of the dike alignment. The excavated material was then placed as fill adjacent to the excavation until a dike extending above the high-water mark was formed. In some cases, the excavated materials consisted of silty clay (Bay Mud), and in other cases of sand. The area filled behind the dikes encompasses approximately 900 acres.

Initial Conditions

At the time of the geotechnical investigation, the average ground surface elevation along the dikes was about 8 ft above Bay level, or 108 referenced to Harbor Bay Isle Datum. Typically, the outboard (bayside) face of the dikes was covered with riprap and three distinct slope inclinations were visible in any one profile. The top part of the dikes were sloped downward at about 1.5 : 1 (horizontal : vertical) or steeper for a height of about 10 feet. Below 10 feet, the slope became flatter and was inclined at about 3 : 1 for an additional height of about 10 feet. Below the toe of the second slope, there existed a very long, gradual slope inclined at about 10 : 1 or flatter for about 200 feet. Typical profiles along the dikes are shown on Fig. 27.

Based on the results of extensive geotechnical studies, the 4,000' of dikes can be divided into three different sections as shown on Fig. 26. An idealized subsurface profile of each section is presented on Fig. 27. The sand dike in section I consists of of a 5 to 7 foot upper layer of very dense sand with SPT blow counts ranging between 15 and 35. The dense sand is

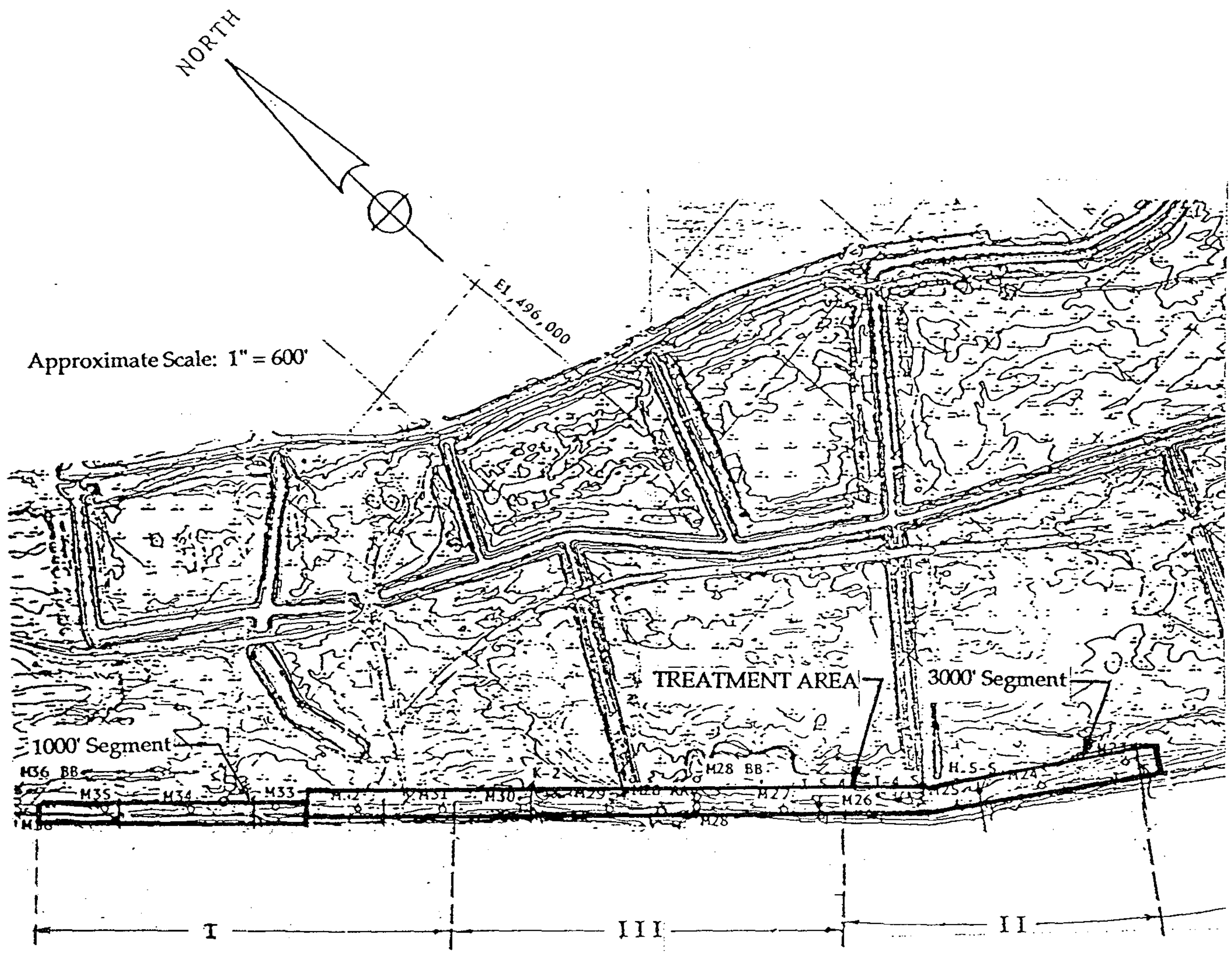


Fig. 26 Site Plan - Perimeter Dikes at Harbor Bay Isle Development (Hallenbeck & Associates, 1985)

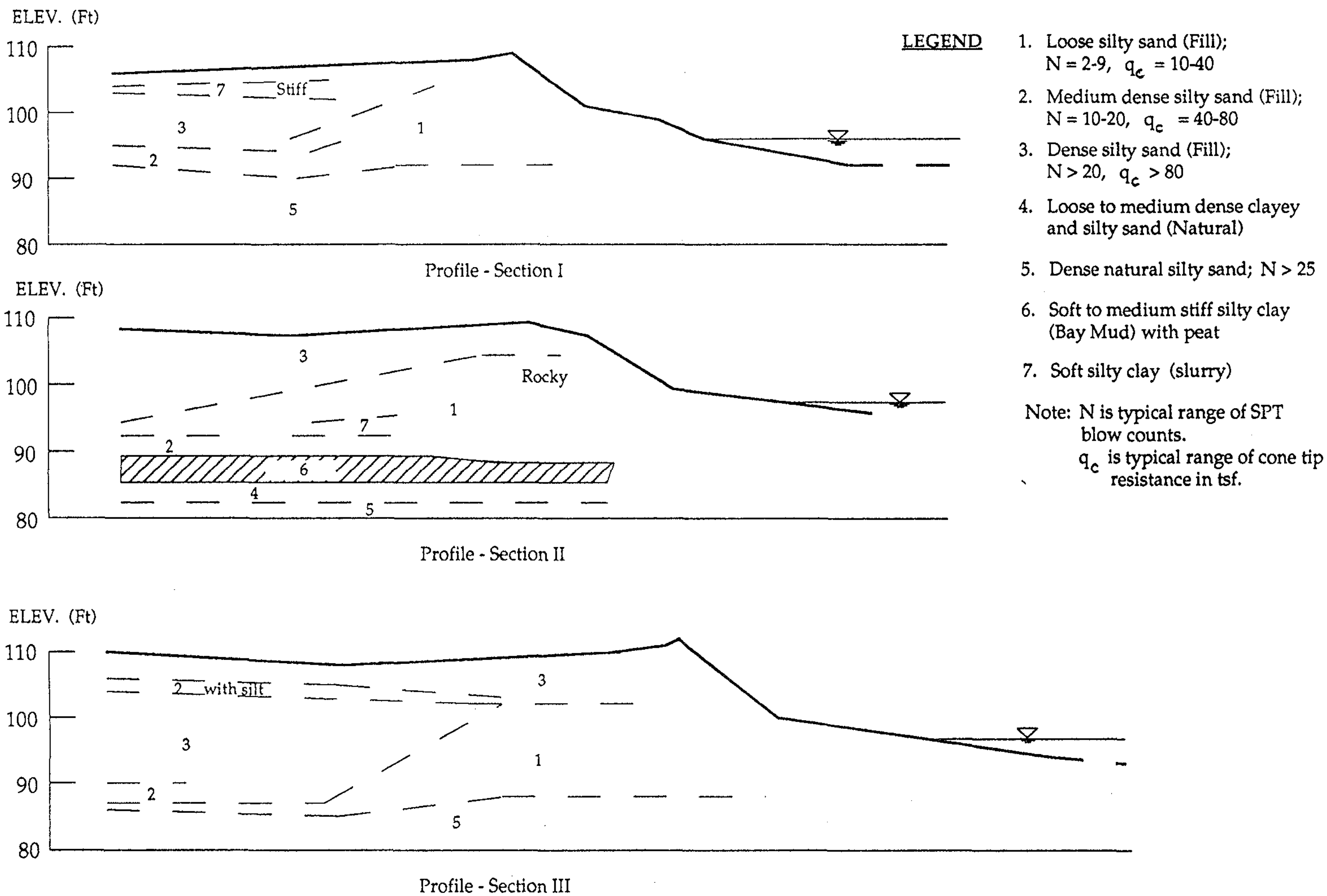


Fig. 27 Typical Subsurface Profiles of Dikes at Harbor Bay Isle

underlain by a layer of loose to medium dense silty sand approximately 12 feet thick. Blow counts in this layer ranged from 3 to 10. Both the surface dense sand and the underlying loose sand are hydraulically placed fill. Very dense silty and clayey sand with blow counts above 25 is encountered below a depth of 17 feet.

Section II similarly consists of 5 to 12 feet of dense sand crust over a layer of loose sand fill. The dense sand has SPT blow counts ranging between 10 and 20. The loose sand layer is approximately 6 to 13 feet thick and is interspersed with thin clay lenses. This layer has blow counts ranging from 3 to 8. The loose sand is underlain by a layer of soft Bay Mud approximately 3 feet thick. The Bay Mud is underlain by a layer of natural loose to medium dense sand extending to a maximum depth of 26 feet below the ground surface. This sand layer has blow counts of 8 to 15. Below a depth of 26 feet, the sand becomes very dense with blow counts in excess of 30.

Section III consists of a surface fill layer of 10 to 16 feet of very dense sand with SPT blow counts ranging between 30 and 40. The dense sand is underlain by a layer of loose to medium dense sand 6 to 12 feet thick having blow counts of between 5 and 10. The lower 5 feet of the loose sand layer consist of natural sand. Below a depth of 22 feet, the natural sand becomes very dense, with blow counts above 30. In all cases, the hydraulic sand fill is generally fine-grained and averages less than 11 percent finer than the #200 sieve. The natural sand is also generally fine-grained, with an average fines content of about 22 percent. The groundwater level was approximately at the same elevation as mean sea level.

Liquefaction Potential. The liquefaction potential of the loose hydraulic sand fill was evaluated using the analytical-empirical procedure based on SPT N values corrected for fines content (Seed, et al, 1987). A M 8.25 earthquake occurring on the San Andreas Fault was used as the design earthquake. The results of the analysis showed that the liquefaction potential of the loose and medium dense sand fill in the dikes was high, and it was concluded that the dikes could experience excessive yielding or slope failure during a major earthquake. Such yielding or failure would reduce confinement of the interior soils and might allow lateral spreading in the interior of the Harbor Bay Isle development. Based on these conclusions, it was recommended that the loose sand fill in the dikes be densified to minimize their liquefaction potential.

Improvement Goals and Methods

Specified Level of Improvement. - It was specified that the sand fill dike be densified between the depths of 5 and 26 feet below the ground surface. After treatment, it was required that the average CPT-tip resistance in the sands be at least 100 kg/cm2. In addition, it was required that no more than 10 percent of the recorded CPT-tip resistances in any one layer be less than 90 kg/cm2. A layer was defined as "any continuous zone of sand or silty sand within the treatment area lying between any two elevations 3 feet apart".

Improvement Method. - In the geotechnical engineer's opinion, Deep Dynamic Compaction (DDC) would be the most cost effective method of densifying the sand fill and this was the method chosen. DDC had been used to densify the southeast section of dike in 1983 with very good results. Other ground improvement methods considered were chemical grouting and recompaction of the dikes using standard grading equipment.

Construction Procedures

The treatment area of the 3,000-foot long segment of dike was 90 feet wide, and the densification was carried out from June to October 1985. The project specifications called for a total of five passes of the pounder throughout the entire treatment area. The number and location of pounder drops varied with each pass. The required number of drops for each pass and the pattern of drop points for the 3,000 foot segment is shown on Fig. 28. For this segment, a 7 by 7 foot, 20 ton pounder was dropped from a height of 100 feet. The energy applied throughout the 3,000 foot segment was about 360 foot-kips per square foot of area treated.

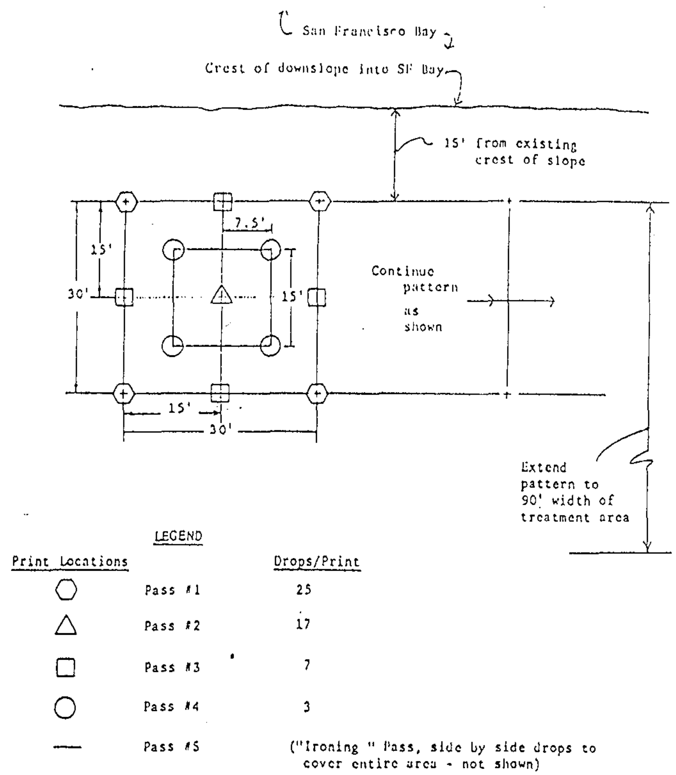


Fig. 28 Typical Drop Pattern - 3000' Segment, Harbor Bay Isle (Hallenbeck & Associates, 1985)

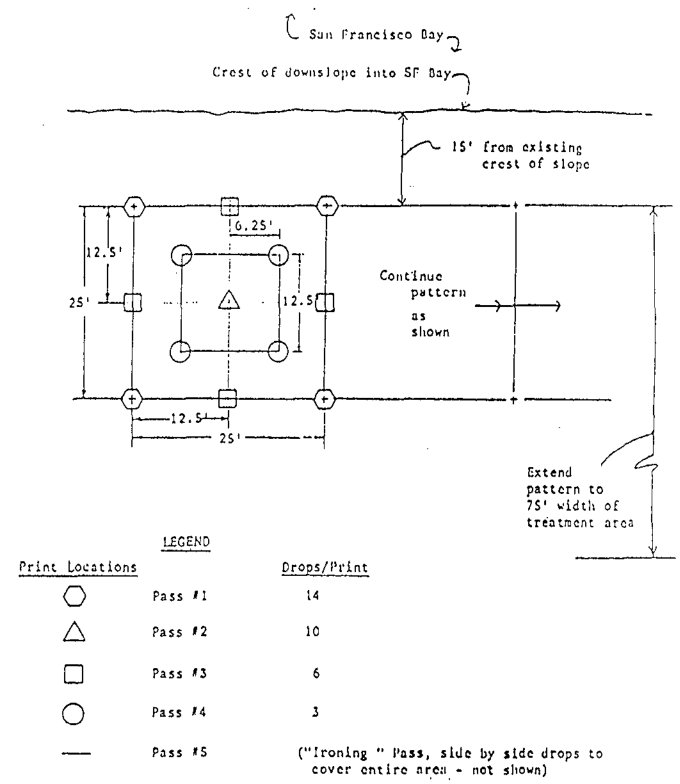


Fig. 29 Typical Drop Pattern - 1000' Segment, Harbor Bay Isle (Hallenbeck & Associates, 1985)

The required drop pattern for the 1,000 foot segment, shown on Fig. 29, was essentially the same at that for the 3,000 foot segment. The treatment area was narrowed to 75 feet, and because the soil conditions were somewhat different in the 1,000 foot segment than those in the 3000 foot segment, the total number of drops was reduced. For this segment, a 5 by 5 foot, 20 ton pounder was dropped from a height of 95 feet. The total energy applied to the 1,000 foot segment was approximately 310 foot-kips per square foot of area treated. The work was done during December 1985 and January 1986.

Final Results of Improvement

CPT testing was performed before, during, and after DDC treatment. In addition, four test borings were drilled at various locations in the 3,000 foot segment of dike when the DDC treatment was about half completed, and SPTs were performed to provide a correlation between CPT tip resistance and SPT blow count at the site. Changes in pore water pressures in the sand fill after treatment were measured using both open standpipe and closed porous stone (hydraulic) piezometers. These measurements enabled the geotechnical engineer to moniter the rate of pore pressure dissipation in the fill after the DDC treatment.

It was concluded that the project specifications were met or exceeded in the treated areas of the dike. The average CPT tip resistances in the densified sand were at least 100 kg/cm2; except in the southern one-third of the 3,000 foot segment. CPTs performed in this section indicated lenses of soil within the profile with tip resistances less than 80 kg/cm2. It was concluded, based on previous CPT data and sampling, that these lenses consist of clays or silts not susceptible to liquefaction. Therefore, no further treatment was required.

Performance of the Project During the Loma Prieta Earthquake

Based on readings taken from a CDMG instrument located in downtown Oakland, and a strong motion instrument at the Alameda Naval Air Station, the project site was estimated to have been subjected to a peak ground acceleration of approximately 0.25g within a bracketed duration of about four seconds during the Loma Prieta earthquake. Evidence of extensive liquefaction (large sand boils, sink holes) was observed in areas of Bay Farm Island behind the dikes, the adjacent Oakland International Airport runways, and Alameda Naval Air Station. No liquefaction or permanent movement of the perimeter dikes was detected.

HANOVER PROPERTIES, UNION CITY, CALIFORNIA

Project Description

The Hanover properties, consisting of five relatively lightly loaded tilt-up panel buildings, were constructed in 1988. The buildings cover an an area of approximately 200,000 square feet.

Initial Conditions

The site was essentially level and unimproved at the time of the geotechnical investigation. The upper eight to twelve feet of soil is composed of two to three feet of hard clayey silt fill underlain by alternating layers of loose sand and firm silt approximately two feet thick. The sand is generally fine- to medium-grained. The sand and silt are underlain by a layer of soft to medium stiff silty clay (Bay Mud) interspersed with organics approximately 15 feet thick. The Bay Mud is underlain by stiff to very stiff clays to the depths penetrated. The groundwater surface was approximately seven feet below the ground surface at the time of drilling.

Liquefaction Potential. - The liquefaction potential of the loose sand layers was evaluated using both SPT and CPT data. It was concluded that the liquefaction potential of the sand layers was moderate to high for a moderate to large earthquake occurring on either the Hayward or San Andreas Faults. Based on these findings, it was decided that the buildings should be supported on spread footings bearing on soil densified to reduce the liquefaction potential.

Improvement Goals and Methods

Specified Level of Improvement - The geotechnical engineer specified that at a minimum, the loose sand layers between a depth of eight and twelve feet be densified to a minimum relative density of 75 percent beneath the buildings and to a distance of 10 feet beyond each building perimeter.

Improvement Method. - Several methods of ground improvement believed to be appropriate for the soil conditions at the site including, vibrocompaction, Terraprobe densification, deep dynamic compaction (DDC), and excavating and recompacting the sand, were investigated. It was ultimately decided that the loose sands would be densified using DDC. This method was chosen because it was estimated to provide the most effective densification for the least cost. The only concern arising from the use of DDC was that treatment would be necessary within 60 feet of an existing warehouse structure and within approximately 25 to 50 feet of existing pavement, curbs, and utility lines. Because of this, it was necessary to provide vibration monitoring in the vicinity of the existing structures and utilities.

Construction Procedures

Trial Densification Tests. - Prior to production densification, two test areas of approximately 2,500 square feet each were treated to establish a drop pattern and the number of drops at each point. The relative success of the densification was determined based upon three factors: 1) CPT performed both before and after the DDC treatment; 2) elevation drop over the test area, indicating the soil volume reduction; and 3) the amount of energy imparted to the ground. It was found that a 10-ton pounder dropped 10 times at primary drop points and six times at secondary drop points from a height of 25 feet satisfactorily densified the loose sand layers in the test area. The drop pattern is not known.

Construction Procedure. - The site was densified using the same drop procedure as in the test areas. Treatment of the entire area was completed in four weeks. No major problems were encountered during the treatment.

Final Results of Improvement

The increase in density of the sand layer was evaluated by comparing SPT and CPT data taken before DDC treatment with CPT taken after treatment, and by calculating the total soil volume reduction. The results of the CPT indicated that the liquefiable sand layer underlying the site had been densified to the minimum specifications required. The average elevation drop across the site after the DDC treatment was one and one-half to two feet.

Performance of the Project During the Loma Prieta Earthquake

Based on readings taken from a USGS instrument located in Fremont, the project site was subjected to a peak ground acceleration of approximately 0.16g within a bracketed duration of about three seconds during the Loma Prieta earthquake. No evidence of liquefaction or ground settlement was observed during a post-earthquake inspection of the site.

KAISER HOSPITAL ADDITION, SOUTH SAN FRANCISCO, CALIFORNIA

Project Description

The project, completed in 1979, consists of a one-story addition immediately adjacent to an existing hospital. Following the partial collapse of the Veteran's Administration Hospital during the 1971 San Fernando Valley earthquake, the State of California required more stringent conditions for seismic design in hospitals with the 1973 Hospital Act. Although the new code was not retroactive, the provisions were applied to hospital structures modified more than 10 percent and therefore covered the planned expansion of Kaiser Hospital.

Initial Conditions

The upper eight feet of soil is unconsolidated fill consisting of sands, gravels, clays, and construction debris. The fill is underlain by a layer of loose to medium dense hydraulically placed sand fill extending from 8 to 35 feet below the ground surface. The sand is generally fine- to medium-grained.

Liquefaction Potential. - The liquefaction potential of the loose to medium dense, hydraulically placed sand fill was considered to be moderate during a large earthquake. The liquefaction potential of the upper eight feet of fill was considered to be low.

Foundation Alternatives

It was concluded that the proposed expansion could be supported on either spread footings or piles. Because the site is underlain by an approximately 27-foot thick layer of potentially liquefiable sand fill, it was decided that if spread footings were to be used, the sand fill should be densified to reduce the liquefaction potential.
The pile alternative was eliminated because it was thought that the noise of pile driving would be too disruptive to continuing hospital operations.

Improvement Goals and Methods

Specified Level of Improvement. - The geotechnical engineer specified that at a minimum, the potentially liquefiable deposit be densified to a minimum relative density of 70% beneath the building and to a distance of 10 feet beyond the building perimeter.

Improvement Method. - Three methods of densifying the potentially liquefiable layer were considered including timber displacement piles, compaction grouting, and excavation and recompaction of the liquefiable soils. From a technical standpoint the preferred method was to use timber displacement piles. However, it was required that the hospital remain in operation during the densification process and the pile driving operations would be unacceptable.

Excavation and recompaction of the liquefiable soils would have been very expensive and time consuming and, therefore, was not seriously considered. It was concluded that compaction grouting of the potentially liquefiable sand layer was the optimum method from both an economic and environmental standpoint given the project constraints of budget and continued operation of the hospital.

Construction Procedures and Problems

Trial Densification Tests. - At the time of the project (1979), little was known quantitatively about the effects of compaction grouting in soils that were not initially loose. In view of this, a pilot test section was constructed at the site to evaluate the effectiveness of compaction grouting in medium dense sand. Concrete grout (two-inch slump) was injected into the sand layer located between depths of 8 and 35 feet. A varying injection point spacing was used to determine the optimum grid pattern and spacing. Injection pressures and grout intake volumes were also monitored.

Each injection point was grouted from the top of the sand layer downward uniformly in stages to the bottom of the layer. Casing was first installed at each injection point to a depth of eight feet. Grout was injected in all locations until a slight ground heave (about 1/8 inch) was observed or until the grout refused to flow at injection pressures higher than 600 psi. The grout was then allowed to harden and the hole advanced by drilling through the hardened grout to the next stage to be grouted. The stage lengths varied from three to four feet. This procedure was repeated to the bottom of the sand layer. In effect, a grout "cap" was formed over the test section which helped prevent ground heave; thus the grout at each level was progressively more effective in compacting the soil. The total surveyed ground heave after grouting averaged about 1/2 inch, corresponding to about 10 percent of the grout take.

To evaluate the effectiveness of the procedure in densifying the fill layer, CPTs and SPTs were performed before and after the grouting. The CPT values were converted to equivalent SPT values as most available liquefaction potential relationships were based on SPT blow counts at the time. A comparison of the equivalent SPT blow counts before and after grouting in the test section using an eight-foot on center injection point spacing in a triangular grid pattern is shown on Fig. 30. This pattern was considered to be the optimum configuration to achieve the required relative density.

Construction Method. - The specifications called for the injection points to spaced at a maximum of eight feet on centers and that a peripheral row of points be located at least five feet beyond the planned building perimeter. Alternate peripheral points were to be injected first. Grouting was to continue at each point until either a drop in injection pressure indicated shearing of the soil, the injection pressure remained at 400 psi with less than 0.75 ft^3/min. grout take, or a surface heave of 1/8 inch occurred. It was specified that the grout pumping equipment be capable of injecting at pressures of up to 600 psi and have a minimum flow rate of 2 ft^3/min.

Problems. - Prior to production grouting, the contractor requested, and was granted, a modification of the injection procedures. This consisted of injecting the grout from the top downward continuously at each injection point without allowing the grout to harden between stages. He also proposed leaving all but the uppermost section of grout pipe in the ground to further reinforce the soil. Using this method, the contractor experienced considerable difficulty in keeping the injection pipe open while driving between stages. This resulted in volumes of grout injected that were insufficient to provide the required compaction.

The contractor then requested another procedure consisting of changing the direction in which grouting was to proceed through the sand layer. The casing was first installed to the bottom of the liquefiable layer at a depth of 35 feet and gradually withdrawn in three foot stage intervals while grout was continuously injected. The results of extensive field density testing performed in the area grouted using this procedure showed that below a depth of

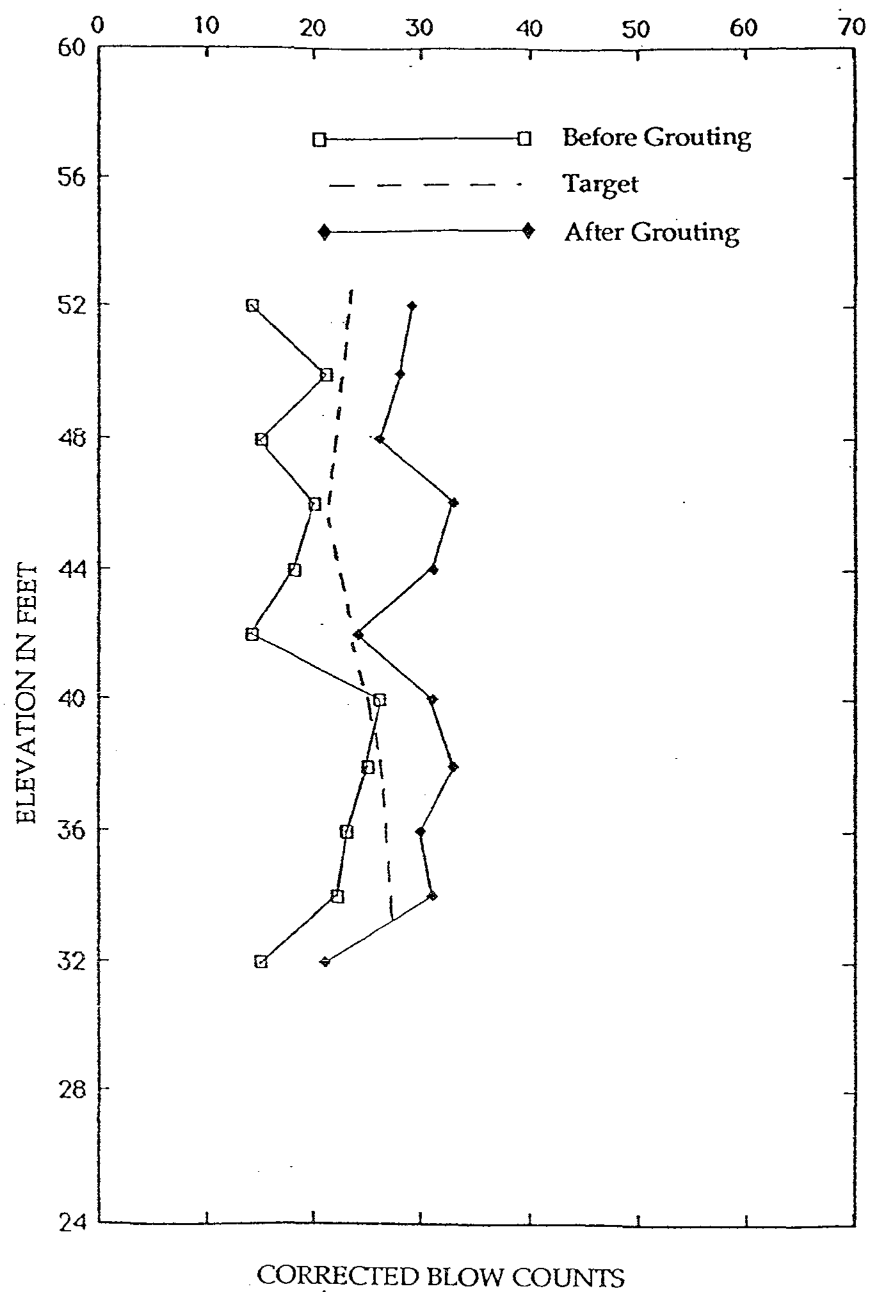


Fig. 30 Mean Corrected Blow Count Values in Test Section Before and After Compaction Grouting, Kaiser Hospital, South San Francisco (N. C. Donovan, 1978)

about 17 feet, the degree of compaction was adequate, while above 17 feet the level of improvement was below the minimum required. The reasons for the lower compaction at the shallow depths were not clearly evident, but may have been influenced by either the lower density fill placed over the liquefiable sands in this area, or by the grouting procedure.

Several alternative grouting schemes were proposed by the grout contractor. Ultimately, it was decided to grout the liquefiable layer in two phases; from 14 to 7 feet and from 35 to 7 feet, using three foot stage lengths. The final spacing between injection points was four feet on centers.

Final Results of Improvement

Extensive CPT and SPT testing was performed during production grouting to evaluate the increase in densification. Surface heave was monitored throughout the grouting process. Surveyed heave contours (conical in shape) were observed across the site after the completion of production grouting. Maximum heave across the site averaged less than 1/2 inch.

The results of the CPT and SPT are summarized in Fig. 31. It was concluded that the hydraulic fill sand layer was adequately densified to prevent most liquefaction from occurring. The actual cost of the densification project was higher than originally estimated, but still less than one-third that of the alternative methods considered.

Performance of the Project During the Loma Prieta Earthquake

Based on readings taken from a CDMG instrument located on Sierra Point in South San Francisco, the project site was subjected to a peak ground acceleration of approximately 0.11g within a bracketed duration of about 2 seconds during the Loma Prieta earthquake. There were no reports of damage to the facility or surrounding paved areas caused by the earthquake.

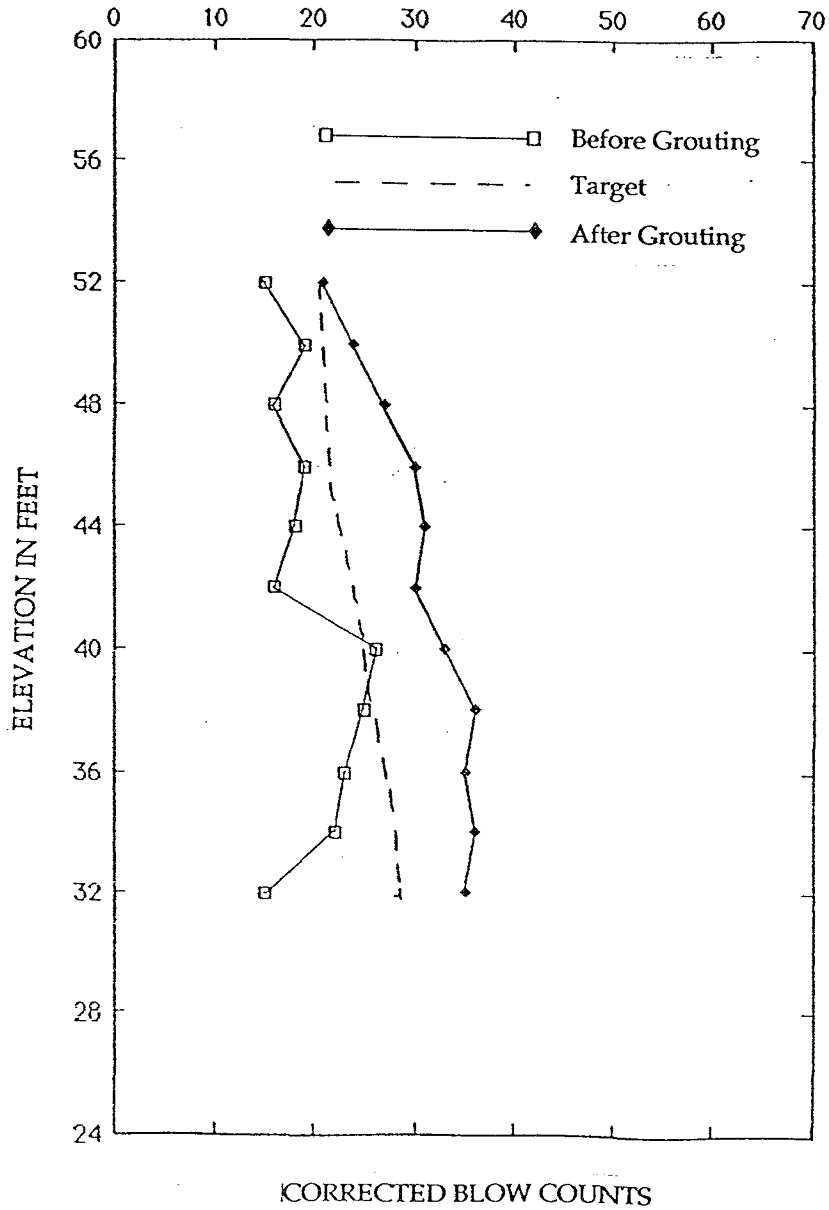


Fig. 31 Summary of Mean Corrected Blow Counts at the Project Site at the end of Production Grouting, Kaiser Hospital, South San Francisco (N. C. Donovan, 1978)

RIVERSIDE AVENUE BRIDGE, SANTA CRUZ, CALIFORNIA

Project Description

The Riverside Avenue Bridge consists of a reinforced concrete, two-lane traffic bridge spanning the San Lorenzo River. The bridge is supported by reinforced concrete nose piers on each side of the bridge. In addition, a concrete slab-apron lines the river channel beneath the bridge and nose piers. The soil area under the south nose pier, below the concrete slab-apron, as shown in Fig. 32, is the subject of this case history. Although the ground improvement was not undertaken for seismic strengthening, the behavior of the treated ground during strong shaking is nonetheless important.

Initial Conditions

The upper five feet of soil (beneath the concrete slab-apron) is composed of saturated, loose to medium dense sandy gravel up to one-inch diameter. The gravel is underlain by a layer of dense gravelly sand approximately 11 feet thick. The sand is generally medium- to fine-grained and contains less than five percent of material finer than the #200 sieve. The gravels are one- to two-inches in diameter. The sand is underlain by alternating layers of soft to medium stiff silty clays and sandy silts to the depths penetrated. The water level of the river is approximately nine feet above the bottom of the concrete slab-apron at high tide.

The granular bearing soils underneath the south nose pier were being eroded away by the river thereby undermining the pier. The resulting settlement of the pier was causing damage to the bridge decking above. Over time, the erosion and resulting settlement appeared to be increasing, and it was decided that some method of improving the granular soils to prevent further erosion must be implemented.

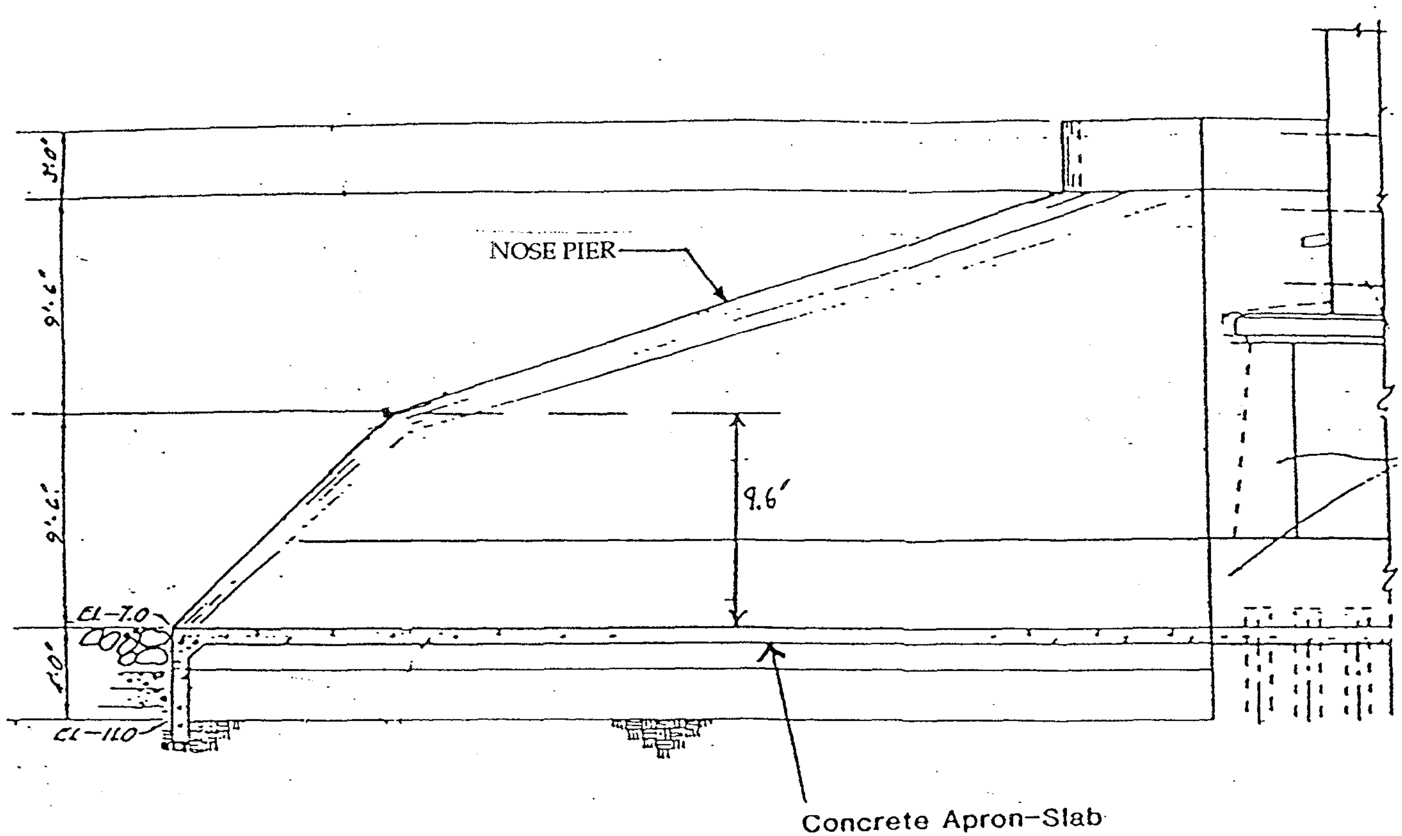


Fig. 32 Riverside Avenue Bridge Nose Pier (Geo/Resources Consultants, Inc., 1986)

Soil Improvement Goals and Methods

Specified Level of Improvement. - It was specified that the method of improvement must prevent additional settlement of the pier. In addition, the method chosen needed to be performed with the existing nose pier, slab-apron, bridge deck, etc., in place. Santa Cruz City officials required that one lane of traffic remain open at all times during construction. Work was to proceed only between the hours of 8 a.m. and 6 p.m. and was required to be completed within 15 days of the start of construction.

Improvement Method. - Based on the nature of the problem and the space constraints at the site, grouting was the only ground improvement method seriously considered. It was decided to use chemical grout to "cement" the sand grains into a single, erosion resistant mass.

Construction Procedures

The chemical grout was composed of sodium silicate N grade and MC 500 micro-fine cement. Less than one-tenth of a percent by volume of phosphoric acid was used to control set time. The grouting was accomplished by placing sleeve port grout pipes (SPGP) into the granular bearing soils and injecting grout, in a zone around and underneath the nose pier, as shown in Figs. 33 and 34.

Casing pipe was set through the river sediment and holes were drilled through the eight-inch thick concrete slab for SPGP access. Twelve vertical holes were drilled through the nose pier for grout injection directly beneath the footing. The steel SPGPs were vibrated or jetted into the granular soil. The grout was pumped into the SPGP through an internal packer in multiple stages at each injection point. At the completion of grouting operations, the lower portion of the SPGP was backfilled with cement grout and the remaining portion removed, along with the casing pipe. All holes in the nose pier were also grouted. No major problems were reported during the grouting process and the job was completed within the 15 day time limit.

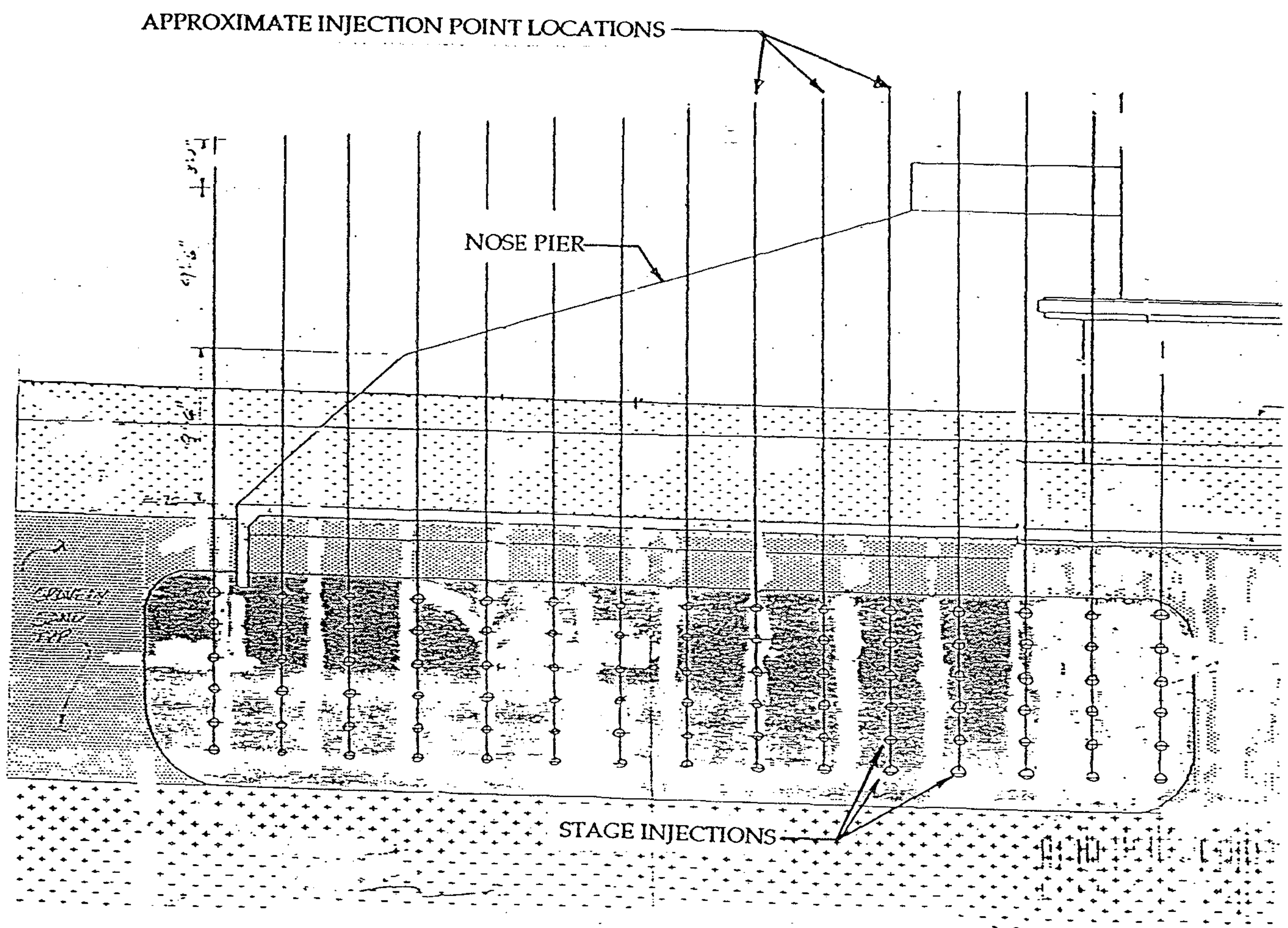


Fig. 33 Typical Grout Pattern - Riverside Ave. Bridge (Cross Section) (GKN Hayward Baker, 1986)

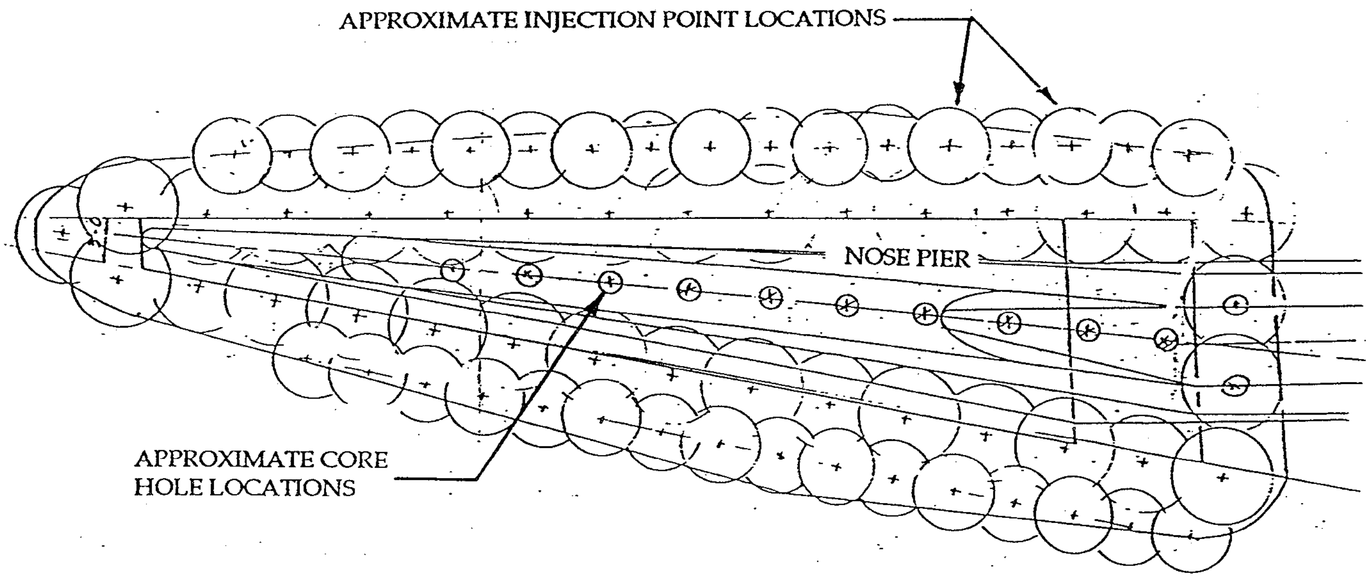


Fig. 34 Typical Grout Pattern - Riverside Ave. Bridge (Plan)(GKN Hayward Baker, 1986)

Field Control and Evaluation

Field Control. - The contractor was required to establish a quality control program consisting of sampling and testing the water, soil and grout admixture to assure proper placement and consistency of the grout. In addition, the contractor was required to keep records of all grouting operations including:

a. Geologic logs of the grout holes.
b. Hole location and depth.
c. Time of each change in grouting operation.
d. Injection pressure at each hole.
e. Rate of pumping.
f. Amount of chemical used for each hole.

All records were submitted to the U.S. Army Corps of Engineers for approval.

Sampling and Testing. - Originally, it was required that the unconfined compressive strength and relative density of the soil after grouting be obtained from undisturbed samples of the soil. The samples were to be a minimum of three inches in diameter and 30 inches long. This requirement was eventually dropped because the required method of sampling would have been very difficult and expensive.

To evaluate the effectiveness of the grouting, 76 additional SPGPs were incorporated into the original grouting plan and field grouted specifically for testing purposes. From these, grouted sand samples were made and strength tested. In addition, the nose pier and bridge deck were surveyed before and after the grouting to check for movement due to ground heave.

Final Results of Improvement

Approximately 40,000 gallons of chemical grout were injected into 77 locations around and beneath the nose pier. A total of about 550 points of injection were used in the zone to be stabilized. Based upon the unconfined compressive strengths of the field samples, and the volume of grout injected, it was concluded that the sand underneath the nose pier was suitably strengthened and that settlement of the pier would no longer occur.

Performance of the Project During the Loma Prieta Earthquake

Based on readings taken from a CDMG instrument located on the University of California, Santa Cruz campus, the project site was subjected to a peak ground acceleration of approximately 0.45g within a bracketed duration of 15 seconds during the Loma Prieta Earthquake. According to the Santa Cruz city engineer, no settlement of the bridge pier or other detrimental ground movements were observed.

SANTA CRUZ COUNTY DETENTION FACILITY, SANTA CRUZ, CALIFORNIA

Project Description

The Santa Cruz County Detention Facility, constructed in 1979, consists of a one- and two-story building, a recreation yard, and two buffer zones. The structure is of a modular, split-level design with maximum plan dimensions of 200 feet by 220 feet.

Initial Conditions

The site was essentially level and paved for use as a parking lot at the time of the geotechnical investigation. The upper 4 to 12 feet of soil is composed of firm to very stiff clays and silts and medium to very dense sands and gravels. These materials are engineered fill placed during a redevelopment project in 1964. The fill is underlain by a layer of soft to stiff sandy silts and loose to medium dense silty sands varying in thickness from 20 to 70 feet. This layer varies in thickness from 20 to 70 feet. The silt and sand are underlain by siltstone bedrock to the depths penetrated. The groundwater level was 12 to 17 feet below the ground surface across the site at the time of drilling.

Liquefaction Potential. - A seismic study of the site concluded that an earthquake occurring on either the San Gregorio or San Andreas Faults (M 6.0 to 8.0), would result in peak ground surface accelerations at the site of 0.15 to 0.45g. A liquefaction analysis of the silt and sand layer below the water table using the simplified procedure developed by Seed, et al (1983) indicated that widespread liquefaction would likely occur in the layer with peak ground surface accelerations of 0.15 to 0.20g. Liquefaction of the upper 4 to 12 feet of fill was considered very unlikely because it is located above the groundwater table.

Foundation Alternatives

The primary consideration for foundation design was the highly compressible and liquefiable nature of the soft, clayey and sandy silts underlying the site. It was concluded that total settlements of approximately one to two inches, and differential settlements of about one inch could occur due to consolidation of the silt layer by the foundation loads of the proposed building. In addition, settlements of up to 5 to 10 inches were expected to occur should the silts liquefy during an earthquake. Because of these potential settlements, the geotechnical engineer recommended that the building, including its ground floor slab, be supported on driven piles end-bearing in the siltstone bedrock.

However, the County of Santa Cruz decided that the proposed pile foundation was too expensive and asked for an alternative foundation design. Consequently, it was decided that the building would be supported on spread footings and that the silt layer be densified to minimize consolidation and liquefaction potential.

Soil Improvement Goals and Methods

Specified Level of Improvement. - The geotechnical engineer specified that at a minimum, the silty sand fill be densified to a minimum relative density of 70% between the depths of 5 and 35 feet beneath the building and to a distance of 10 feet beyond the building perimeter in order to prevent liquefaction.

Improvement Method. - Because the site was level and open with no existing structures nearby, Deep Dynamic Compaction (DDC) was well-suited to densify the site because of its simplicity and relative low cost.

<u>Construction Procedures</u>

Trial Densification Tests. - Prior to production densification a test area forming a square with 20-foot long sides, was densified by DDC in order to evaluate the effectiveness of the method in the soils between 5 and 35 feet depth. Prior to densification, three test borings were drilled in the test area and SPTs were performed at various depths. In addition, a piezometer was installed in the center the square.

The DDC contractor determined that a pounder about six feet square and weighing 20 tons, would be dropped from a height of 60 feet to produce the densification. The drop points were along the perimeter of the square test area as shown in Fig. 35. Twenty drops were made at each corner of the square, and eight drops were made at the midpoint of each side of the square.

A comparison of the subsurface profile at the test section before and after treatment, illustrating that a substantial amount of densification was achieved in the layers above depths of about 25 to 30 feet, is shown on Fig. 36. A significant increase in blow count values was achieved in the silty sand layer above a depth of about 30 feet as shown in Fig. 37. Overall, it was concluded that liquefiable layer in the test section was adequately densified.

Production Densification. - Production densification of the project site proceeded using the same drop pattern and number of drops that was used in the test section. No problems were reported during the densification.

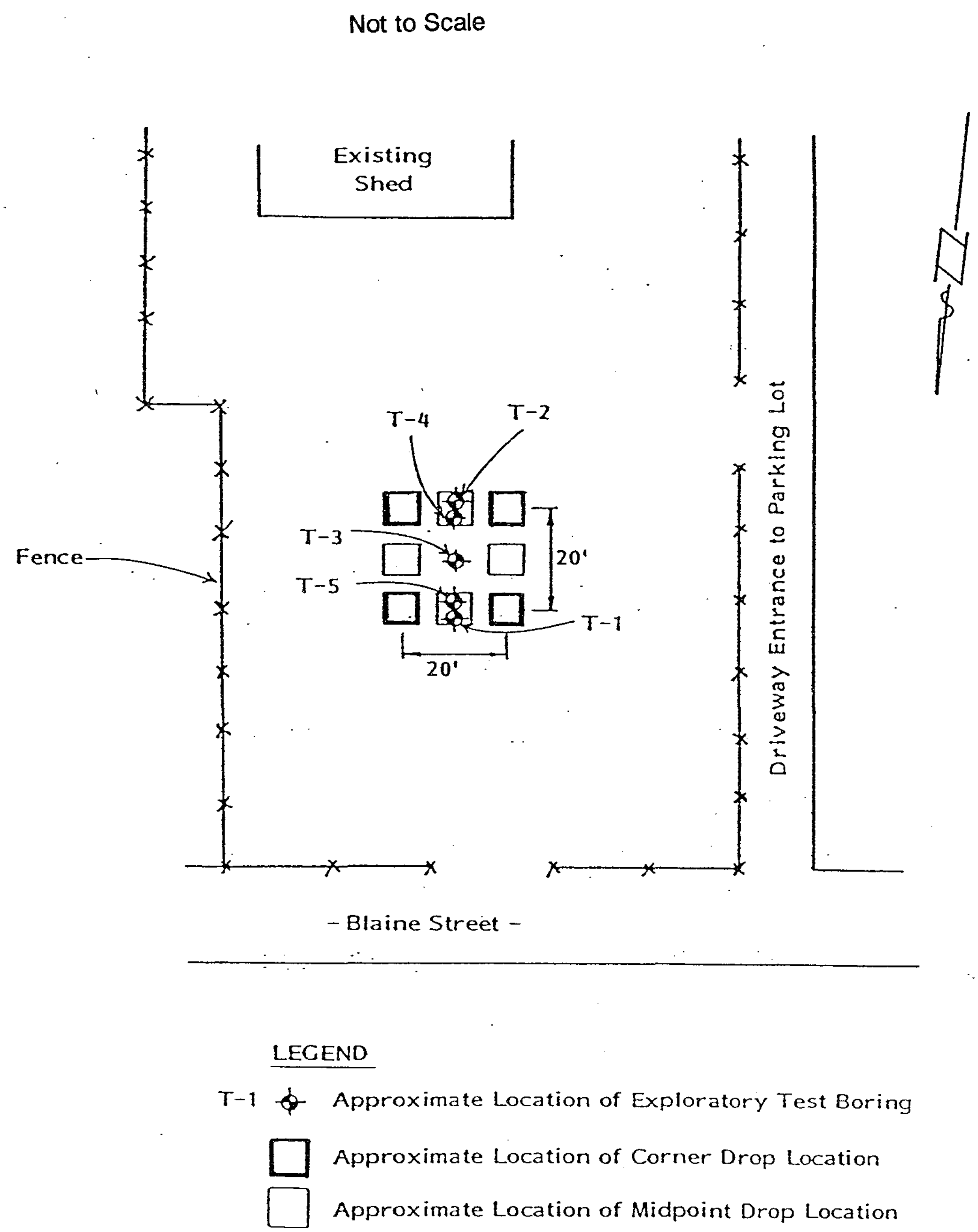


Fig. 35 Drop Pattern for Deep Dynamic Compaction at the Santa Cruz Detention Facility (Kaldveer & Associates, 1977)

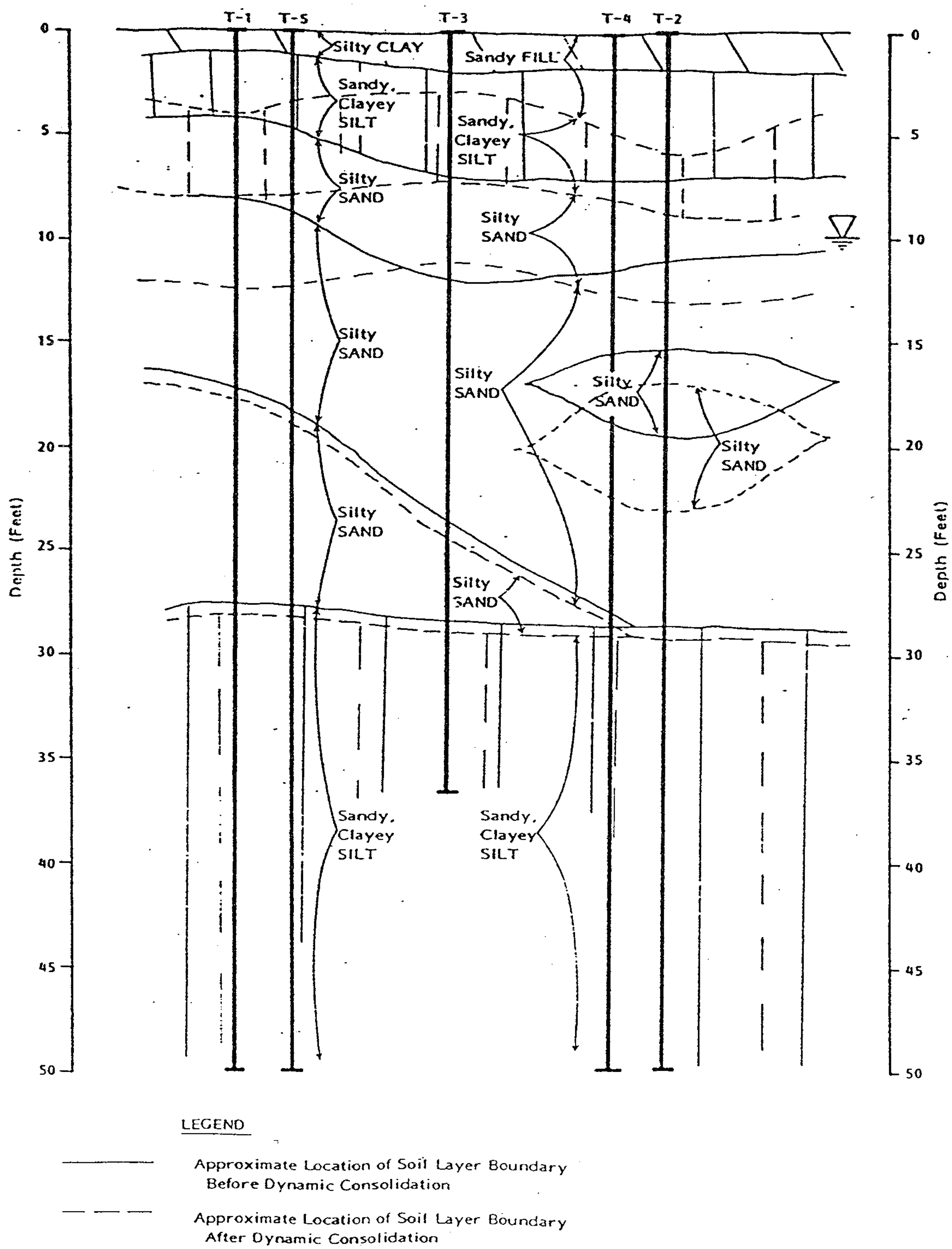


Fig. 36 Comparison of Subsurface Profile Before and After Deep Dynamic Compaction, Santa Cruz Detention Facility (Kaldveer & Associates, 1977)

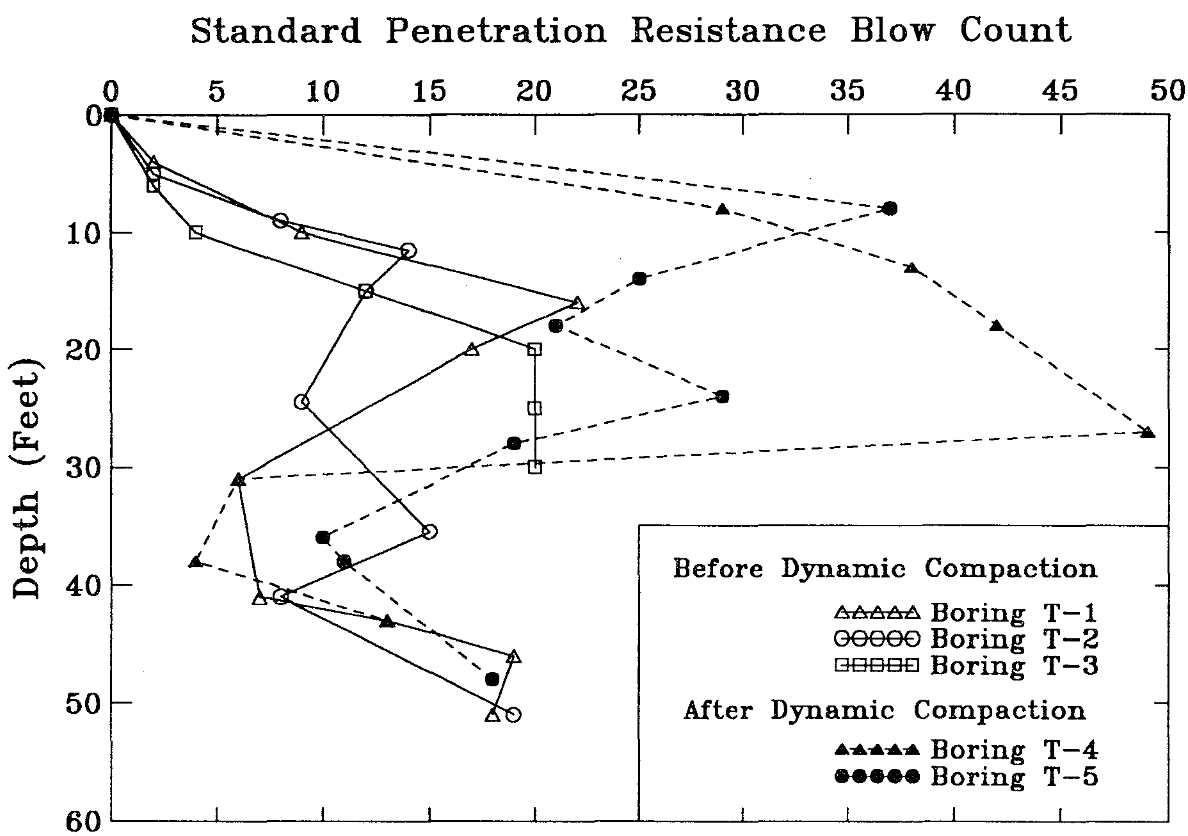


Fig. 37 Comparison of Standard Penetration Resistance Before and After Dynamic Compaction, Santa Cruz Detention Facility (Kaldveer & Associates, 1977)

Final Results of Improvement

Twelve borings were drilled across the site and SPTs were performed to evaluate the resulting densification of DDC. Induced ground settlements were also measured across the site. Based on the SPT results, it was concluded that the potentially liquefiable soils extending from a depth of 5 to 35 feet were densified to an average minimum relative density of 75 percent. Total induced ground surface settlements across the site ranged from 12 to 31 inches.

Performance of the Project During the Loma Prieta Earthquake

Based on readings taken from a CDMG instrument located on the University of California, Santa Cruz campus, the project site was subjected to a peak ground surface acceleration of approximately 0.45g within a bracketed duration of 15 seconds during the Loma Prieta earthquake. There were no reports of any damage to the building or surrounding facilities due to liquefaction or associated ground failure phenomena.

DISCUSSION AND CONCLUSIONS

All of the sites with improved ground that were studied performed very well during the Loma Prieta earthquake. Without exception, there was little or no distress or damage due to ground shaking to either the improved ground or to the facilities and structures built upon it. In many cases, untreated ground adjacent to the improved ground badly cracked and/or settled, due primarily to liquefaction. This resulted in some damage to the facilities and structures built upon the untreated ground. In every case studied in which the ground accelerations were great enough that liquefaction of the untreated ground would have been predicted to occur, it did occur. Together these results support the conclusions that (1) the procedures used for prediction of liquefaction were reliable, and (2) ground improvement is effective for mitigation of liquefaction risk.

However, in assessing these results and their implications for the future, it is important to note that the Loma Prieta earthquake was of only moderate intensity and short duration. On average, each of the sites experienced ground accelerations of only about 25 to 75% of the design earthquake values (Table 2), and the durations were very short compared to the usual values for a magnitude 7 event. How the improved ground sites would perform during an earthquake of larger magnitude and longer duration is not known; however, almost certainly, soil liquefaction and related effects at the sites would be reduced. The question is by what amount?

Detailed ground response analyses have not yet been made for the sites studied. Thus the analyses and interpretations of behavior are based on estimated maximum ground accelerations and durations of ground shaking that were obtained from the nearest available ground motion records. Additional ground response studies would be useful to establish more exactly the actual ground motions that occurred and the influence, if any, of the ground improvement on the surface motions.

One of the most important aspects of any ground improvement project is accurate measurement of the improvement achieved. In almost every case studied, there were questions as to the overall amount of increase in relative density achieved. As CPT and shear wave velocity methods for liquefaction potential assessment become more established and better validated, it will become less expensive and simpler to perform testing programs for assessment of the overall improvement of the soil at a given site. It is hoped that more complete quantitative documentation of post-treatment properties will be retained for all future ground improvement projects so that more quantitative studies of behavior during future earthquakes will be possible.

Finally, it must be emphasized that ground improvement, in spite of the great benefits demonstrated by the Loma Prieta earthquake, is not a panacea for mitigation of all earthquake risk at a site. Its functions are mitigation of liquefaction potential and the prevention of lateral spreading. Analyses have indicated that the improvement has little effect on the ground surface response. Thus surface shaking remains a function of the input rock motions and the characteristics of the soil profile. As soft soil sites generally amplify rock motions, and ground improvement is most frequently used at soft soil sites, strucures that are at risk from shaking before ground treatment will remain so after treatment unless structural strengthening is carried out.

REFERENCES

Basore, C.E., and Boitano, J.D., (1968), "Densification of Sand Fill with Compaction Piles and Vibroflotation - A Case History," Journal of the Soil Mechanics and Foundations Division, A.S.C.E., Vol. 95, No. SM6, pp. 1303-1323.

GeoMatrix Consultants, (1988), "Geotechnical Study - Theatre Complex", Emeryville, Calif., Proj. No. 1223C, Feb. 10, 1988.

GeoMatrix Consultants, (1990), "Draft Report - Evaluation of Interior Area Performance Naval Air Station Treasure Island", San Francisco, Calif., Proj. No. 1539, Aug. 13, 1990.

Geo/Resource Consultants, (1986), "Field and Laboratory Data Report - Riverside Avenue Bridge", Santa Cruz, Calif., Proj. No. 447-055-00-1, Sept. 24, 1986.

Harding Lawson Associates, (1983), "Geotechnical Investigation - Proposed General Purpose/Berthing Pier, Treasure Island", San Francisco, Calif., Proj. No. 2177,057.04, May 13, 1983.

Harding Lawson Associates, (1986), "Soil Investigation - Medical/Dental Clinic, Treasure Island", San Francisco, Calif., Proj. No. 17916,002.04, Oct. 13, 1986.

Harding Lawson Associates, (1986), "Geotechnical Investigation - Proposed Esplanade Extension East Shore, Marina Bay", Richmond, Calif., Proj. No. 8003,069.04, July 25, 1986.

Harding Lawson Associates, (1987), "Final Report - Geotechnical Engineering Services During Construction Stone Column Installation, Eastern Shore, Marina Bay", Richmond, Calif., Proj. No. 8003,071.04, March 18, 1987.

Harding-Miller-Lawson and Associates, (1968), "Foundation Investigation - 1600 Man Barracks (Increment II), Treasure Island", San Francisco, Calif., Proj. No. 2176.5, Jan. 26, 1968.

Hausmann, M.R., (1990), Engineering Principals of Ground Modification, 632 pp., McGraw - Hill.

Peter Kaldveer and Associates, (1977), "Geotechnical Investigation - Santa Cruz County Detention Facility", Santa Cruz, Calif., Proj. No. K480-2A, April 26, 1977.

Peter Kaldveer and Associates, (1978) "Evaluation of Dynamic Consolidation Test Section - Santa Cruz County Detention Facility", Santa Cruz, Calif., Proj. No. K480-2D, May 18, 1978.

Lee, K.L. and Albaisa, A. (1974), "Earthquake Induced Settlements in Saturated Soils," Journal of the Geotechnical Engineering Division, A.S.C.E., Vol. 100, No. GT4, pp.387-406.

Mitchell, J.K., (1981), "Soil Improvement: State-of-the-Art," Proc. 10th ICSMFE, Vol. 4, pp. 509-565, Stockholm.

Rinne, E.E., Shields, C.S., and Nardi, C.R., (1988), "Mitigation of Liquefaction Potential," Proc Int Conf. on Earthquake Engineering, Tokyo.

Robertson, P.K., and Campanella, R.G., (1985), "Liquefaction Potential of Sands Using the CPT," Journ. Geotech. Engineering, ASCE, Vol. 111, No. GT 3, pp. 384-403.

Seed, H.B., and Idriss, I.M., (1967), "Analysis of Soil Liquefaction: Nigata Earthquake," Journ. Soil Mech. and Found. Div., ASCE, Vol. 93, No.4.

Seed, H.B., and Idriss, I.M., (1971), "Simplified Procedure for Evaluating Soil Liquefaction Potential," Journ. Soil Mech. and Found. Div., ASCE, Vol. 97, No. 9, pp. 1249-1274.

Seed, H.B., (1979), "Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground during Earthquakes", Journ. Geotech Engineering, ASCE, Vol. 105, No. 2, pp. 201-255.

Seed, H.B., Idriss, I.M., and Arango, I., (1983), "Evaluation of Liquefaction Potential using Field Performance Data," Journ. Geotech. Engineering, ASCE, Vol.190, No. 3, pp. 458-482.

Seed, H.B., Tokimatsu, K., and Harder, L.F., (1984), "The Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations," Report UCB/EERC - 84/15, University of California, Berkeley.

Seed, H.B., "Design Problems in Soil Liquefaction, (1987), "Journ. Geotech. Engineering, ASCE, Vol. 113, No. 8, pp. 827-845.

Welsh, J.P., "Construction Considerations for Ground Modification Projects," (1986), Proc Int Conf. on Deep Foundations, Bejing.

Wightman, A. (1991), "Ground Improvement by Vibrocompaction," Geotechnical News, Vol. 9, No. 2, pp. 39-41.

Woodward-Clyde-Sherard and Associates, (1966), "Soil Investigation for the Proposed Office Building No. 450", Treasure Island, Proj. No. S-10439, June 14, 1966.

Woodward-Clyde Consultants, (1981), "Geotechnical Engineering Study Final Design Report - East Bay Park
Condominium", Emeryville, Calif., Proj. No. 14884 B,C,D, June 15, 1981.

Woodward-Lundgren and Associates, (1972), "Foundation Design Recommendations for the Proposed Barracks Buildings", Treasure Island, Proj. No. S-12610, 1972.

EARTHQUAKE ENGINEERING RESEARCH CENTER REPORT SERIES

EERC reports are available from the National Information Service for Earthquake Engineering(NISEE) and from the National Technical Information Service(NTIS). Numbers in parentheses are Accession Numbers assigned by the National Technical Information Service; these are followed by a price code. Contact NTIS, 5285 Port Royal Road, Springfield Virginia, 22161 for more information. Reports without Accession Numbers were not available from NTIS at the time of printing. For a current complete list of EERC reports (from EERC 67-1) and availablity information, please contact University of California, EERC, NISEE, 1301 South 46th Street, Richmond, California 94804.

UCB/EERC-81/01 "Control of Seismic Response of Piping Systems and Other Structures by Base Isolation," by Kelly, J.M., January 1981, (PB81 200 735)A05.

UCB/EERC-81/02 "OPTNSR- An Interactive Software System for Optimal Design of Statically and Dynamically Loaded Structures with Nonlinear Response," by Bhatti, M.A., Ciampi, V. and Pister, K.S., January 1981, (PB81 218 851)A09.

UCB/EERC-81/03 "Analysis of Local Variations in Free Field Seismic Ground Motions," by Chen, J.-C., Lysmer, J. and Seed, H.B., January 1981, (AD-A099508)A13.

UCB/EERC-81/04 "Inelastic Structural Modeling of Braced Offshore Platforms for Seismic Loading," by Zayas, V.A., Shing, P.-S.B., Mahin, S.A. and Popov, E.P., January 1981, INEL4, (PB82 138 777)A07.

UCB/EERC-81/05 "Dynamic Response of Light Equipment in Structures," by Der Kiureghian, A., Sackman, J.L. and Nour-Omid, B., April 1981, (PB81 218 497)A04.

UCB/EERC-81/06 "Preliminary Experimental Investigation of a Broad Base Liquid Storage Tank," by Bouwkamp, J.G., Kollegger, J.P. and Stephen, R.M., May 1981, (PB82 140 385)A03.

UCB/EERC-81/07 "The Seismic Resistant Design of Reinforced Concrete Coupled Structural Walls," by Aktan, A.E. and Bertero, V.V., June 1981, (PB82 113 358)A11.

UCB/EERC-81/08 "Unassigned," by Unassigned, 1981.

UCB/EERC-81/09 "Experimental Behavior of a Spatial Piping System with Steel Energy Absorbers Subjected to a Simulated Differential Seismic Input," by Stiemer, S.F., Godden, W.G. and Kelly, J.M., July 1981, (PB82 201 898)A04.

UCB/EERC-81/10 "Evaluation of Seismic Design Provisions for Masonry in the United States," by Sveinsson, B.I., Mayes, R.L. and McNiven, H.D., August 1981, (PB82 166 075)A08.

UCB/EERC-81/11 "Two-Dimensional Hybrid Modelling of Soil-Structure Interaction," by Tzong, T.-J., Gupta, S. and Penzien, J., August 1981, (PB82 142 118)A04.

UCB/EERC-81/12 "Studies on Effects of Infills in Seismic Resistant R/C Construction," by Brokken, S. and Bertero, V.V., October 1981, (PB82 166 190)A09.

UCB/EERC-81/13 "Linear Models to Predict the Nonlinear Seismic Behavior of a One-Story Steel Frame," by Valdimarsson, H., Shah, A.H. and McNiven, H.D., September 1981, (PB82 138 793)A07.

UCB/EERC-81/14 "TLUSH: A Computer Program for the Three-Dimensional Dynamic Analysis of Earth Dams," by Kagawa, T., Mejia, L.H., Seed, H.B. and Lysmer, J., September 1981, (PB82 139 940)A06.

UCB/EERC-81/15 "Three Dimensional Dynamic Response Analysis of Earth Dams," by Mejia, L.H. and Seed, H.B., September 1981, (PB82 137 274)A12.

UCB/EERC-81/16 "Experimental Study of Lead and Elastomeric Dampers for Base Isolation Systems," by Kelly, J.M. and Hodder, S.B., October 1981, (PB82 166 182)A05.

UCB/EERC-81/17 "The Influence of Base Isolation on the Seismic Response of Light Secondary Equipment," by Kelly, J.M., April 1981, (PB82 255 266)A04.

UCB/EERC-81/18 "Studies on Evaluation of Shaking Table Response Analysis Procedures," by Blondet, J. M., November 1981, (PB82 197 278)A10.

UCB/EERC-81/19 "DELIGHT.STRUCT: A Computer-Aided Design Environment for Structural Engineering," by Balling, R.J., Pister, K.S. and Polak, E., December 1981, (PB82 218 496)A07.

UCB/EERC-81/20 "Optimal Design of Seismic-Resistant Planar Steel Frames," by Balling, R.J., Ciampi, V. and Pister, K.S., December 1981, (PB82 220 179)A07.

UCB/EERC-82/01 "Dynamic Behavior of Ground for Seismic Analysis of Lifeline Systems," by Sato, T. and Der Kiureghian, A., January 1982, (PB82 218 926)A05.

UCB/EERC-82/02 "Shaking Table Tests of a Tubular Steel Frame Model," by Ghanaat, Y. and Clough, R.W., January 1982, (PB82 220 161)A07.

UCB/EERC-82/03 "Behavior of a Piping System under Seismic Excitation: Experimental Investigations of a Spatial Piping System supported by Mechanical Shock Arrestors," by Schneider, S., Lee, H.-M. and Godden, W. G., May 1982, (PB83 172 544)A09.

UCB/EERC-82/04 "New Approaches for the Dynamic Analysis of Large Structural Systems," by Wilson, E.L., June 1982, (PB83 148 080)A05.

UCB/EERC-82/05 "Model Study of Effects of Damage on the Vibration Properties of Steel Offshore Platforms," by Shahrivar, F. and Bouwkamp, J.G., June 1982, (PB83 148 742)A10.

UCB/EERC-82/06 "States of the Art and Pratice in the Optimum Seismic Design and Analytical Response Prediction of R/C Frame Wall Structures," by Aktan, A.E. and Bertero, V.V., July 1982, (PB83 147 736)A05.

UCB/EERC-82/07 "Further Study of the Earthquake Response of a Broad Cylindrical Liquid-Storage Tank Model," by Manos, G.C. and Clough, R.W., July 1982, (PB83 147 744)A11.

UCB/EERC-82/08 "An Evaluation of the Design and Analytical Seismic Response of a Seven Story Reinforced Concrete Frame," by Charney, F.A. and Bertero, V.V., July 1982, (PB83 157 628)A09.

UCB/EERC-82/09 "Fluid-Structure Interactions: Added Mass Computations for Incompressible Fluid," by Kuo, J.S.-H., August 1982, (PB83 156 281)A07.

UCB/EERC-82/10 "Joint-Opening Nonlinear Mechanism: Interface Smeared Crack Model," by Kuo, J.S.-H., August 1982, (PB83 149 195)A05.

Preceding page blank

UCB/EERC-82/11 "Dynamic Response Analysis of Techi Dam," by Clough, R.W., Stephen, R.M. and Kuo, J.S.-H., August 1982, (PB83 147 496)A06.

UCB/EERC-82/12 "Prediction of the Seismic Response of R/C Frame-Coupled Wall Structures," by Aktan, A.E., Bertero, V.V. and Piazzo, M., August 1982, (PB83 149 203)A09.

UCB/EERC-82/13 "Preliminary Report on the Smart 1 Strong Motion Array in Taiwan," by Bolt, B.A., Loh, C.H., Penzien, J. and Tsai, Y.B., August 1982, (PB83 159 400)A10.

UCB/EERC-82/14 "Seismic Behavior of an Eccentrically X-Braced Steel Structure," by Yang, M.S., September 1982, (PB83 260 778)A12.

UCB/EERC-82/15 "The Performance of Stairways in Earthquakes," by Roha, C., Axley, J.W. and Bertero, V.V., September 1982, (PB83 157 693)A07.

UCB/EERC-82/16 "The Behavior of Submerged Multiple Bodies in Earthquakes," by Liao, W.-G., September 1982, (PB83 158 709)A07.

UCB/EERC-82/17 "Effects of Concrete Types and Loading Conditions on Local Bond-Slip Relationships," by Cowell, A.D., Popov, E.P. and Bertero, V.V., September 1982, (PB83 153 577)A04.

UCB/EERC-82/18 "Mechanical Behavior of Shear Wall Vertical Boundary Members: An Experimental Investigation," by Wagner, M.T. and Bertero, V.V., October 1982, (PB83 159 764)A05.

UCB/EERC-82/19 "Experimental Studies of Multi-support Seismic Loading on Piping Systems," by Kelly, J.M. and Cowell, A.D., November 1982, (PB90 262 684)A07.

UCB/EERC-82/20 "Generalized Plastic Hinge Concepts for 3D Beam-Column Elements," by Chen, P. F.-S. and Powell, G.H., November 1982, (PB83 247 981)A13.

UCB/EERC-82/21 "ANSR-II: General Computer Program for Nonlinear Structural Analysis," by Oughourlian, C.V. and Powell, G.H., November 1982, (PB83 251 330)A12.

UCB/EERC-82/22 "Solution Strategies for Statically Loaded Nonlinear Structures," by Simons, J.W. and Powell, G.H., November 1982, (PB83 197 970)A06.

UCB/EERC-82/23 "Analytical Model of Deformed Bar Anchorages under Generalized Excitations," by Ciampi, V., Eligehausen, R., Bertero, V.V. and Popov, E.P., November 1982, (PB83 169 532)A06.

UCB/EERC-82/24 "A Mathematical Model for the Response of Masonry Walls to Dynamic Excitations," by Sucuoglu, H., Mengi, Y. and McNiven, H.D., November 1982, (PB83 169 011)A07.

UCB/EERC-82/25 "Earthquake Response Considerations of Broad Liquid Storage Tanks," by Cambra, F.J., November 1982, (PB83 251 215)A09.

UCB/EERC-82/26 "Computational Models for Cyclic Plasticity, Rate Dependence and Creep," by Mosaddad, B. and Powell, G.H., November 1982, (PB83 245 829)A08.

UCB/EERC-82/27 "Inelastic Analysis of Piping and Tubular Structures," by Mahasuverachai, M. and Powell, G.H., November 1982, (PB83 249 987)A07.

UCB/EERC-83/01 "The Economic Feasibility of Seismic Rehabilitation of Buildings by Base Isolation," by Kelly, J.M., January 1983, (PB83 197 988)A05.

UCB/EERC-83/02 "Seismic Moment Connections for Moment-Resisting Steel Frames.," by Popov, E.P., January 1983, (PB83 195 412)A04.

UCB/EERC-83/03 "Design of Links and Beam-to-Column Connections for Eccentrically Braced Steel Frames," by Popov, E.P. and Malley, J.O., January 1983, (PB83 194 811)A04.

UCB/EERC-83/04 "Numerical Techniques for the Evaluation of Soil-Structure Interaction Effects in the Time Domain," by Bayo, E. and Wilson, E.L., February 1983, (PB83 245 605)A09.

UCB/EERC-83/05 "A Transducer for Measuring the Internal Forces in the Columns of a Frame-Wall Reinforced Concrete Structure," by Sause, R. and Bertero, V.V., May 1983, (PB84 119 494)A06.

UCB/EERC-83/06 "Dynamic Interactions Between Floating Ice and Offshore Structures," by Croteau, P., May 1983, (PB84 119 486)A16.

UCB/EERC-83/07 "Dynamic Analysis of Multiply Tuned and Arbitrarily Supported Secondary Systems," by Igusa, T. and Der Kiureghian, A., July 1983, (PB84 118 272)A11.

UCB/EERC-83/08 "A Laboratory Study of Submerged Multi-body Systems in Earthquakes," by Ansari, G.R., June 1983, (PB83 261 842)A17.

UCB/EERC-83/09 "Effects of Transient Foundation Uplift on Earthquake Response of Structures," by Yim, C.-S. and Chopra, A.K., June 1983, (PB83 261 396)A07.

UCB/EERC-83/10 "Optimal Design of Friction-Braced Frames under Seismic Loading," by Austin, M.A. and Pister, K.S., June 1983, (PB84 119 288)A06.

UCB/EERC-83/11 "Shaking Table Study of Single-Story Masonry Houses: Dynamic Performance under Three Component Seismic Input and Recommendations," by Manos, G.C., Clough, R.W. and Mayes, R.L., July 1983, (UCB/EERC-83/11)A08.

UCB/EERC-83/12 "Experimental Error Propagation in Pseudodynamic Testing," by Shiing, P.B. and Mahin, S.A., June 1983, (PB84 119 270)A09.

UCB/EERC-83/13 "Experimental and Analytical Predictions of the Mechanical Characteristics of a 1/5-scale Model of a 7-story R/C Frame-Wall Building Structure," by Aktan, A.E., Bertero, V.V., Chowdhury, A.A. and Nagashima, T., June 1983, (PB84 119 213)A07.

UCB/EERC-83/14 "Shaking Table Tests of Large-Panel Precast Concrete Building System Assemblages," by Oliva, M.G. and Clough, R.W., June 1983, (PB86 110 210/AS)A11.

UCB/EERC-83/15 "Seismic Behavior of Active Beam Links in Eccentrically Braced Frames," by Hjelmstad, K.D. and Popov, E.P., July 1983, (PB84 119 676)A09.

UCB/EERC-83/16 "System Identification of Structures with Joint Rotation," by Dimsdale, J.S., July 1983, (PB84 192 210)A06.

UCB/EERC-83/17 "Construction of Inelastic Response Spectra for Single-Degree-of-Freedom Systems," by Mahin, S. and Lin, J., June 1983, (PB84 208 834)A05.

UCB/EERC-83/18 "Interactive Computer Analysis Methods for Predicting the Inelastic Cyclic Behaviour of Structural Sections," by Kaba, S. and Mahin, S., July 1983, (PB84 192 012)A06.

UCB/EERC-83/19 "Effects of Bond Deterioration on Hysteretic Behavior of Reinforced Concrete Joints," by Filippou, F.C., Popov, E.P. and Bertero, V.V., August 1983, (PB84 192 020)A10.

UCB/EERC-83/20 "Correlation of Analytical and Experimental Responses of Large-Panel Precast Building Systems," by Oliva, M.G., Clough, R.W., Velkov, M. and Gavrilovic, P., May 1988, (PB90 262 692)A06.

UCB/EERC-83/21 "Mechanical Characteristics of Materials Used in a 1/5 Scale Model of a 7-Story Reinforced Concrete Test Structure," by Bertero, V.V., Aktan, A.E., Harris, H.G. and Chowdhury, A.A., October 1983, (PB84 193 697)A05.

UCB/EERC-83/22 "Hybrid Modelling of Soil-Structure Interaction in Layered Media," by Tzong, T.-J. and Penzien, J., October 1983, (PB84 192 178)A08.

UCB/EERC-83/23 "Local Bond Stress-Slip Relationships of Deformed Bars under Generalized Excitations," by Eligehausen, R., Popov, E.P. and Bertero, V.V., October 1983, (PB84 192 848)A09.

UCB/EERC-83/24 "Design Considerations for Shear Links in Eccentrically Braced Frames," by Malley, J.O. and Popov, E.P., November 1983, (PB84 192 186)A07.

UCB/EERC-84/01 "Pseudodynamic Test Method for Seismic Performance Evaluation: Theory and Implementation," by Shing, P.-S.B. and Mahin, S.A., January 1984, (PB84 190 644)A08.

UCB/EERC-84/02 "Dynamic Response Behavior of Kiang Hong Dian Dam," by Clough, R.W., Chang, K.-T., Chen, H.-Q. and Stephen, R.M., April 1984, (PB84 209 402)A08.

UCB/EERC-84/03 "Refined Modelling of Reinforced Concrete Columns for Seismic Analysis," by Kaba, S.A. and Mahin, S.A., April 1984, (PB84 234 384)A06.

UCB/EERC-84/04 "A New Floor Response Spectrum Method for Seismic Analysis of Multiply Supported Secondary Systems," by Asfura, A. and Der Kiureghian, A., June 1984, (PB84 239 417)A06.

UCB/EERC-84/05 "Earthquake Simulation Tests and Associated Studies of a 1/5th-scale Model of a 7-Story R/C Frame-Wall Test Structure," by Bertero, V.V., Aktan, A.E., Charney, F.A. and Sause, R., June 1984, (PB84 239 409)A09.

UCB/EERC-84/06 "Unassigned," by Unassigned, 1984.

UCB/EERC-84/07 "Behavior of Interior and Exterior Flat-Plate Connections subjected to Inelastic Load Reversals," by Zee, H.L. and Moehle, J.P., August 1984, (PB86 117 629/AS)A07.

UCB/EERC-84/08 "Experimental Study of the Seismic Behavior of a Two-Story Flat-Plate Structure," by Moehle, J.P. and Diebold, J.W., August 1984, (PB86 122 553/AS)A12.

UCB/EERC-84/09 "Phenomenological Modeling of Steel Braces under Cyclic Loading," by Ikeda, K., Mahin, S.A. and Dermitzakis, S.N., May 1984, (PB86 132 198/AS)A08.

UCB/EERC-84/10 "Earthquake Analysis and Response of Concrete Gravity Dams," by Fenves, G. and Chopra, A.K., August 1984, (PB85 193 902/AS)A11.

UCB/EERC-84/11 "EAGD-84: A Computer Program for Earthquake Analysis of Concrete Gravity Dams," by Fenves, G. and Chopra, A.K., August 1984, (PB85 193 613/AS)A05.

UCB/EERC-84/12 "A Refined Physical Theory Model for Predicting the Seismic Behavior of Braced Steel Frames," by Ikeda, K. and Mahin, S.A., July 1984, (PB85 191 450/AS)A09.

UCB/EERC-84/13 "Earthquake Engineering Research at Berkeley - 1984," by EERC, August 1984, (PB85 197 341/AS)A10.

UCB/EERC-84/14 "Moduli and Damping Factors for Dynamic Analyses of Cohesionless Soils," by Seed, H.B., Wong, R.T., Idriss, I.M. and Tokimatsu, K., September 1984, (PB85 191 468/AS)A04.

UCB/EERC-84/15 "The Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations," by Seed, H.B., Tokimatsu, K., Harder, L.F. and Chung, R.M., October 1984, (PB85 191 732/AS)A04.

UCB/EERC-84/16 "Simplified Procedures for the Evaluation of Settlements in Sands Due to Earthquake Shaking," by Tokimatsu, K. and Seed, H.B., October 1984, (PB85 197 887/AS)A03.

UCB/EERC-84/17 "Evaluation of Energy Absorption Characteristics of Highway Bridges Under Seismic Conditions - Volume I (PB90 262 627)A16 and Volume II (Appendices) (PB90 262 635)A13," by Imbsen, R.A. and Penzien, J., September 1986.

UCB/EERC-84/18 "Structure-Foundation Interactions under Dynamic Loads," by Liu, W.D. and Penzien, J., November 1984, (PB87 124 889/AS)A11.

UCB/EERC-84/19 "Seismic Modelling of Deep Foundations," by Chen, C.-H. and Penzien, J., November 1984, (PB87 124 798/AS)A07.

UCB/EERC-84/20 "Dynamic Response Behavior of Quan Shui Dam," by Clough, R.W., Chang, K.-T., Chen, H.-Q., Stephen, R.M., Ghanaat, Y. and Qi, J.-H., November 1984, (PB86 115177/AS)A07.

UCB/EERC-85/01 "Simplified Methods of Analysis for Earthquake Resistant Design of Buildings," by Cruz, E.F. and Chopra, A.K., February 1985, (PB86 112299/AS)A12.

UCB/EERC-85/02 "Estimation of Seismic Wave Coherency and Rupture Velocity using the SMART 1 Strong-Motion Array Recordings," by Abrahamson, N.A., March 1985, (PB86 214 343)A07.

UCB/EERC-85/03 "Dynamic Properties of a Thirty Story Condominium Tower Building," by Stephen, R.M., Wilson, E.L. and Stander, N., April 1985, (PB86 118965/AS)A06.

UCB/EERC-85/04 "Development of Substructuring Techniques for On-Line Computer Controlled Seismic Performance Testing," by Dermitzakis, S. and Mahin, S., February 1985, (PB86 132941/AS)A08.

UCB/EERC-85/05 "A Simple Model for Reinforcing Bar Anchorages under Cyclic Excitations," by Filippou, F.C., March 1985, (PB86 112 919/AS)A05.

UCB/EERC-85/06 "Racking Behavior of Wood-framed Gypsum Panels under Dynamic Load," by Oliva, M.G., June 1985, (PB90 262 643)A04.

UCB/EERC-85/07 "Earthquake Analysis and Response of Concrete Arch Dams," by Fok, K.-L. and Chopra, A.K., June 1985, (PB86 139672/AS)A10.

UCB/EERC-85/08 "Effect of Inelastic Behavior on the Analysis and Design of Earthquake Resistant Structures," by Lin, J.P. and Mahin, S.A., June 1985, (PB86 135340/AS)A08.

UCB/EERC-85/09 "Earthquake Simulator Testing of a Base-Isolated Bridge Deck," by Kelly, J.M., Buckle, I.G. and Tsai, H.-C., January 1986, (PB87 124 152/AS)A06.

UCB/EERC-85/10 "Simplified Analysis for Earthquake Resistant Design of Concrete Gravity Dams," by Fenves, G. and Chopra, A.K., June 1986, (PB87 124 160/AS)A08.

UCB/EERC-85/11 "Dynamic Interaction Effects in Arch Dams," by Clough, R.W., Chang, K.-T., Chen, H.-Q. and Ghanaat, Y., October 1985, (PB86 135027/AS)A05.

UCB/EERC-85/12 "Dynamic Response of Long Valley Dam in the Mammoth Lake Earthquake Series of May 25-27, 1980," by Lai, S. and Seed, H.B., November 1985, (PB86 142304/AS)A05.

UCB/EERC-85/13 "A Methodology for Computer-Aided Design of Earthquake-Resistant Steel Structures," by Austin, M.A., Pister, K.S. and Mahin, S.A., December 1985, (PB86 159480/AS)A10 .

UCB/EERC-85/14 "Response of Tension-Leg Platforms to Vertical Seismic Excitations," by Liou, G.-S., Penzien, J. and Yeung, R.W., December 1985, (PB87 124 871/AS)A08.

UCB/EERC-85/15 "Cyclic Loading Tests of Masonry Single Piers: Volume 4 - Additional Tests with Height to Width Ratio of 1," by Sveinsson, B., McNiven, H.D. and Sucuoglu, H., December 1985.

UCB/EERC-85/16 "An Experimental Program for Studying the Dynamic Response of a Steel Frame with a Variety of Infill Partitions," by Yanev, B. and McNiven, H.D., December 1985, (PB90 262 676)A05.

UCB/EERC-86/01 "A Study of Seismically Resistant Eccentrically Braced Steel Frame Systems," by Kasai, K. and Popov, E.P., January 1986, (PB87 124 178/AS)A14.

UCB/EERC-86/02 "Design Problems in Soil Liquefaction," by Seed, H.B., February 1986, (PB87 124 186/AS)A03.

UCB/EERC-86/03 "Implications of Recent Earthquakes and Research on Earthquake-Resistant Design and Construction of Buildings," by Bertero, V.V., March 1986, (PB87 124 194/AS)A05.

UCB/EERC-86/04 "The Use of Load Dependent Vectors for Dynamic and Earthquake Analyses," by Leger, P., Wilson, E.L. and Clough, R.W., March 1986, (PB87 124 202/AS)A12.

UCB/EERC-86/05 "Two Beam-To-Column Web Connections," by Tsai, K.-C. and Popov, E.P., April 1986, (PB87 124 301/AS)A04.

UCB/EERC-86/06 "Determination of Penetration Resistance for Coarse-Grained Soils using the Becker Hammer Drill," by Harder, L.F. and Seed, H.B., May 1986, (PB87 124 210/AS)A07.

UCB/EERC-86/07 "A Mathematical Model for Predicting the Nonlinear Response of Unreinforced Masonry Walls to In-Plane Earthquake Excitations," by Mengi, Y. and McNiven, H.D., May 1986, (PB87 124 780/AS)A06.

UCB/EERC-86/08 "The 19 September 1985 Mexico Earthquake: Building Behavior," by Bertero, V.V., July 1986.

UCB/EERC-86/09 "EACD-3D: A Computer Program for Three-Dimensional Earthquake Analysis of Concrete Dams," by Fok, K.-L., Hall, J.F. and Chopra, A.K., July 1986, (PB87 124 228/AS)A08.

UCB/EERC-86/10 "Earthquake Simulation Tests and Associated Studies of a 0.3-Scale Model of a Six-Story Concentrically Braced Steel Structure," by Uang, C.-M. and Bertero, V.V., December 1986, (PB87 163 564/AS)A17.

UCB/EERC-86/11 "Mechanical Characteristics of Base Isolation Bearings for a Bridge Deck Model Test," by Kelly, J.M., Buckle, I.G. and Koh, C.-G., November 1987, (PB90 262 668)A04.

UCB/EERC-86/12 "Effects of Axial Load on Elastomeric Isolation Bearings," by Koh, C.-G. and Kelly, J.M., November 1987.

UCB/EERC-87/01 "The FPS Earthquake Resisting System: Experimental Report," by Zayas, V.A., Low, S.S. and Mahin, S.A., June 1987.

UCB/EERC-87/02 "Earthquake Simulator Tests and Associated Studies of a 0.3-Scale Model of a Six-Story Eccentrically Braced Steel Structure," by Whittaker, A., Uang, C.-M. and Bertero, V.V., July 1987.

UCB/EERC-87/03 "A Displacement Control and Uplift Restraint Device for Base-Isolated Structures," by Kelly, J.M., Griffith, M.C. and Aiken, I.D., April 1987.

UCB/EERC-87/04 "Earthquake Simulator Testing of a Combined Sliding Bearing and Rubber Bearing Isolation System," by Kelly, J.M. and Chalhoub, M.S., 1987.

UCB/EERC-87/05 "Three-Dimensional Inelastic Analysis of Reinforced Concrete Frame-Wall Structures," by Moazzami, S. and Bertero, V.V., May 1987.

UCB/EERC-87/06 "Experiments on Eccentrically Braced Frames with Composite Floors," by Ricles, J. and Popov, E., June 1987.

UCB/EERC-87/07 "Dynamic Analysis of Seismically Resistant Eccentrically Braced Frames," by Ricles, J. and Popov, E., June 1987.

UCB/EERC-87/08 "Undrained Cyclic Triaxial Testing of Gravels-The Effect of Membrane Compliance," by Evans, M.D. and Seed, H.B., July 1987.

UCB/EERC-87/09 "Hybrid Solution Techniques for Generalized Pseudo-Dynamic Testing," by Thewalt, C. and Mahin, S.A., July 1987.

UCB/EERC-87/10 "Ultimate Behavior of Butt Welded Splices in Heavy Rolled Steel Sections," by Bruneau, M., Mahin, S.A. and Popov, E.P., September 1987.

UCB/EERC-87/11 "Residual Strength of Sand from Dam Failures in the Chilean Earthquake of March 3, 1985," by De Alba, P., Seed, H.B., Retamal, E. and Seed, R.B., September 1987.

UCB/EERC-87/12 "Inelastic Seismic Response of Structures with Mass or Stiffness Eccentricities in Plan," by Bruneau, M. and Mahin, S.A., September 1987, (PB90 262 650)A14.

UCB/EERC-87/13 "CSTRUCT: An Interactive Computer Environment for the Design and Analysis of Earthquake Resistant Steel Structures," by Austin, M.A., Mahin, S.A. and Pister, K.S., September 1987.

UCB/EERC-87/14 "Experimental Study of Reinforced Concrete Columns Subjected to Multi-Axial Loading," by Low, S.S. and Moehle, J.P., September 1987.

UCB/EERC-87/15 "Relationships between Soil Conditions and Earthquake Ground Motions in Mexico City in the Earthquake of Sept. 19, 1985," by Seed, H.B., Romo, M.P., Sun, J., Jaime, A. and Lysmer, J., October 1987.

UCB/EERC-87/16 "Experimental Study of Seismic Response of R. C. Setback Buildings," by Shahrooz, B.M. and Moehle, J.P., October 1987.

UCB/EERC-87/17 "The Effect of Slabs on the Flexural Behavior of Beams," by Pantazopoulou, S.J. and Moehle, J.P., October 1987, (PB90 262 700)A07.

UCB/EERC-87/18 "Design Procedure for R-FBI Bearings," by Mostaghel, N. and Kelly, J.M., November 1987, (PB90 262 718)A04.

UCB/EERC-87/19 "Analytical Models for Predicting the Lateral Response of R C Shear Walls: Evaluation of their Reliability," by Vulcano, A. and Bertero, V.V., November 1987.

UCB/EERC-87/20 "Earthquake Response of Torsionally-Coupled Buildings," by Hejal, R. and Chopra, A.K., December 1987.

UCB/EERC-87/21 "Dynamic Reservoir Interaction with Monticello Dam," by Clough, R.W., Ghanaat, Y. and Qiu, X-F., December 1987.

UCB/EERC-87/22 "Strength Evaluation of Coarse-Grained Soils," by Siddiqi, F.H., Seed, R.B., Chan, C.K., Seed, H.B. and Pyke, R.M., December 1987.

UCB/EERC-88/01 "Seismic Behavior of Concentrically Braced Steel Frames," by Khatib, I., Mahin, S.A. and Pister, K.S., January 1988.

UCB/EERC-88/02 "Experimental Evaluation of Seismic Isolation of Medium-Rise Structures Subject to Uplift," by Griffith, M.C., Kelly, J.M., Coveney, V.A. and Koh, C.G., January 1988.

UCB/EERC-88/03 "Cyclic Behavior of Steel Double Angle Connections," by Astaneh-Asl, A. and Nader, M.N., January 1988.

UCB/EERC-88/04 "Re-evaluation of the Slide in the Lower San Fernando Dam in the Earthquake of Feb. 9, 1971," by Seed, H.B., Seed, R.B., Harder, L.F. and Jong, H.-L., April 1988.

UCB/EERC-88/05 "Experimental Evaluation of Seismic Isolation of a Nine-Story Braced Steel Frame Subject to Uplift," by Griffith, M.C., Kelly, J.M. and Aiken, I.D., May 1988.

UCB/EERC-88/06 "DRAIN-2DX User Guide.," by Allahabadi, R. and Powell, G.H., March 1988.

UCB/EERC-88/07 "Theoretical and Experimental Studies of Cylindrical Water Tanks in Base-Isolated Structures," by Chalhoub, M.S. and Kelly, J.M., April 1988.

UCB/EERC-88/08 "Analysis of Near-Source Waves: Separation of Wave Types using Strong Motion Array Recordings," by Darragh, R.B., June 1988.

UCB/EERC-88/09 "Alternatives to Standard Mode Superposition for Analysis of Non-Classically Damped Systems," by Kusainov, A.A. and Clough, R.W., June 1988.

UCB/EERC-88/10 "The Landslide at the Port of Nice on October 16, 1979," by Seed, H.B., Seed, R.B., Schlosser, F., Blondeau, F. and Juran, I., June 1988.

UCB/EERC-88/11 "Liquefaction Potential of Sand Deposits Under Low Levels of Excitation," by Carter, D.P. and Seed, H.B., August 1988.

UCB/EERC-88/12 "Nonlinear Analysis of Reinforced Concrete Frames Under Cyclic Load Reversals," by Filippou, F.C. and Issa, A., September 1988.

UCB/EERC-88/13 "Implications of Recorded Earthquake Ground Motions on Seismic Design of Building Structures," by Uang, C.-M. and Bertero, V.V., November 1988.

UCB/EERC-88/14 "An Experimental Study of the Behavior of Dual Steel Systems," by Whittaker, A.S. , Uang, C.-M. and Bertero, V.V., September 1988.

UCB/EERC-88/15 "Dynamic Moduli and Damping Ratios for Cohesive Soils," by Sun, J.I., Golesorkhi, R. and Seed, H.B., August 1988.

UCB/EERC-88/16 "Reinforced Concrete Flat Plates Under Lateral Load: An Experimental Study Including Biaxial Effects," by Pan, A. and Moehle, J., October 1988.

UCB/EERC-88/17 "Earthquake Engineering Research at Berkeley - 1988," by EERC, November 1988.

UCB/EERC-88/18 "Use of Energy as a Design Criterion in Earthquake-Resistant Design," by Uang, C.-M. and Bertero, V.V., November 1988.

UCB/EERC-88/19 "Steel Beam-Column Joints in Seismic Moment Resisting Frames," by Tsai, K.-C. and Popov, E.P., November 1988.

UCB/EERC-88/20 "Base Isolation in Japan, 1988," by Kelly, J.M., December 1988.

UCB/EERC-89/01 "Behavior of Long Links in Eccentrically Braced Frames," by Engelhardt, M.D. and Popov, E.P., January 1989.

UCB/EERC-89/02 "Earthquake Simulator Testing of Steel Plate Added Damping and Stiffness Elements," by Whittaker, A., Bertero, V.V., Alonso, J. and Thompson, C., January 1989.

UCB/EERC-89/03 "Implications of Site Effects in the Mexico City Earthquake of Sept. 19, 1985 for Earthquake-Resistant Design Criteria in the San Francisco Bay Area of California," by Seed, H.B. and Sun, J.I., March 1989.

UCB/EERC-89/04 "Earthquake Analysis and Response of Intake-Outlet Towers," by Goyal, A. and Chopra, A.K., July 1989.

UCB/EERC-89/05 "The 1985 Chile Earthquake: An Evaluation of Structural Requirements for Bearing Wall Buildings," by Wallace, J.W. and Moehle, J.P., July 1989.

UCB/EERC-89/06 "Effects of Spatial Variation of Ground Motions on Large Multiply-Supported Structures," by Hao, H., July 1989.

UCB/EERC-89/07 "EADAP - Enhanced Arch Dam Analysis Program: Users's Manual," by Ghanaat, Y. and Clough, R.W., August 1989.

UCB/EERC-89/08 "Seismic Performance of Steel Moment Frames Plastically Designed by Least Squares Stress Fields," by Ohi, K. and Mahin, S.A., August 1989.

UCB/EERC-89/09 "Feasibility and Performance Studies on Improving the Earthquake Resistance of New and Existing Buildings Using the Friction Pendulum System," by Zayas, V., Low, S., Mahin, S.A. and Bozzo, L., July 1989.

UCB/EERC-89/10 "Measurement and Elimination of Membrane Compliance Effects in Undrained Triaxial Testing," by Nicholson, P.G., Seed, R.B. and Anwar, H., September 1989.

UCB/EERC-89/11 "Static Tilt Behavior of Unanchored Cylindrical Tanks," by Lau, D.T. and Clough, R.W., September 1989.

UCB/EERC-89/12 "ADAP-88: A Computer Program for Nonlinear Earthquake Analysis of Concrete Arch Dams," by Fenves, G.L., Mojtahedi, S. and Reimer, R.B., September 1989.

UCB/EERC-89/13 "Mechanics of Low Shape Factor Elastomeric Seismic Isolation Bearings," by Aiken, I.D., Kelly, J.M. and Tajirian, F.F., November 1989.

UCB/EERC-89/14 "Preliminary Report on the Seismological and Engineering Aspects of the October 17, 1989 Santa Cruz (Loma Prieta) Earthquake," by EERC, October 1989.

UCB/EERC-89/15 "Experimental Studies of a Single Story Steel Structure Tested with Fixed, Semi-Rigid and Flexible Connections," by Nader, M.N. and Astaneh-Asl, A., August 1989.

UCB/EERC-89/16 "Collapse of the Cypress Street Viaduct as a Result of the Loma Prieta Earthquake," by Nims, D.K., Miranda, E., Aiken, I.D., Whittaker, A.S. and Bertero, V.V., November 1989.

UCB/EERC-90/01 "Mechanics of High-Shape Factor Elastomeric Seismic Isolation Bearings," by Kelly, J.M., Aiken, I.D. and Tajirian, F.F., March 1990.

UCB/EERC-90/02 "Javid's Paradox: The Influence of Preform on the Modes of Vibrating Beams," by Kelly, J.M., Sackman, J.L. and Javid, A., May 1990.

UCB/EERC-90/03 "Earthquake Simulator Testing and Analytical Studies of Two Energy-Absorbing Systems for Multistory Structures," by Aiken, I.D. and Kelly, J.M., October 1990.

UCB/EERC-90/04 "Damage to the San Francisco-Oakland Bay Bridge During the October 17, 1989 Earthquake," by Astaneh, A., June 1990.

UCB/EERC-90/05 "Preliminary Report on the Principal Geotechnical Aspects of the October 17, 1989 Loma Prieta Earthquake," by Seed, R.B., Dickenson, S.E., Riemer, M.F., Bray, J.D., Sitar, N., Mitchell, J.K., Idriss, I.M., Kayen, R.E., Kropp, A., Harder, L.F., Jr. and Power, M.S., April 1990.

UCB/EERC-90/06 "Models of Critical Regions in Reinforced Concrete Frames Under Seismic Excitations," by Zulfiqar, N. and Filippou, F., May 1990.

UCB/EERC-90/07 "A Unified Earthquake-Resistant Design Method for Steel Frames Using ARMA Models," by Takewaki, I., Conte, J.P., Mahin, S.A. and Pister, K.S., June 1990.

UCB/EERC-90/08 "Soil Conditions and Earthquake Hazard Mitigation in the Marina District of San Francisco," by Mitchell, J.K., Masood, T., Kayen, R.E. and Seed, R.B., May 1990.

UCB/EERC-90/09 "Influence of the Earthquake Ground Motion Process and Structural Properties on Response Characteristics of Simple Structures," by Conte, J.P., Pister, K.S. and Mahin, S.A., July 1990.

UCB/EERC-90/10 "Experimental Testing of the Resilient-Friction Base Isolation System," by Clark, P.W. and Kelly, J.M., July 1990.

UCB/EERC-90/11 "Seismic Hazard Analysis: Improved Models, Uncertainties and Sensitivities," by Araya, R. and Der Kiureghian, A., March 1988.

UCB/EERC-90/12 "Effects of Torsion on the Linear and Nonlinear Seismic Response of Structures," by Sedarat, H. and Bertero, V.V., September 1989.

UCB/EERC-90/13 "The Effects of Tectonic Movements on Stresses and Deformations in Earth Embankments," by Bray, J. D., Seed, R. B. and Seed, H. B., September 1989.

UCB/EERC-90/14 "Inelastic Seismic Response of One-Story, Asymmetric-Plan Systems," by Goel, R.K. and Chopra, A.K., October 1990.

UCB/EERC-90/15 "Dynamic Crack Propagation: A Model for Near-Field Ground Motion.," by Seyyedian, H. and Kelly, J.M., 1990.

UCB/EERC-90/16 "Sensitivity of Long-Period Response Spectra to System Initial Conditions," by Blasquez, R., Ventura, C. and Kelly, J.M., 1990.

UCB/EERC-90/17 "Behavior of Peak Values and Spectral Ordinates of Near-Source Strong Ground-Motion over a Dense Array," by Niazi, M., June 1990.

UCB/EERC-90/18 "Material Characterization of Elastomers used in Earthquake Base Isolation," by Papoulia, K.D. and Kelly, J.M., 1990.

UCB/EERC-90/19 "Cyclic Behavior of Steel Top-and-Bottom Plate Moment Connections," by Harriott, J.D. and Astaneh, A., August 1990.

UCB/EERC-90/20 "Seismic Response Evaluation of an Instrumented Six Story Steel Building," by Shen, J.-H. and Astaneh, A., December 1990.

UCB/EERC-90/21 "Observations and Implications of Tests on the Cypress Street Viaduct Test Structure," by Bollo, M., Mahin, S.A., Moehle, J.P., Stephen, R.M. and Qi, X., December 1990.

UCB/EERC-91/01 "Experimental Evaluation of Nitinol for Energy Dissipation in Structures," by Nims, D.K., Sasaki, K.K. and Kelly, J.M., 1991.

UCB/EERC-91/02 "Displacement Design Approach for Reinforced Concrete Structures Subjected to Earthquakes," by Qi, X. and Moehle, J.P., January 1991.

UCB/EERC-91/03 "Shake Table Tests of Long Period Isolation System for Nuclear Facilities at Soft Soil Sites," by Kelly, J.M., March 1991.

UCB/EERC-91/04 "Dynamic and Failure Characteristics of Bridgestone Isolation Bearings," by Kelly, J.M., April 1991.

UCB/EERC-91/05 "Base Sliding Response of Concrete Gravity Dams to Earthquakes," by Chopra, A.K. and Zhang, L., May 1991.

UCB/EERC-91/06 "Computation of Spatially Varying Ground Motion and Foundation-Rock Impedance Matrices for Seismic Analysis of Arch Dams," by Zhang, L. and Chopra, A.K., May 1991.

UCB/EERC-91/07 "Estimation of Seismic Source Processes Using Strong Motion Array Data," by Chiou, S.-J., July 1991.

UCB/EERC-91/08 "A Response Spectrum Method for Multiple-Support Seismic Excitations," by Der Kiureghian, A. and Neuenhofer, A., August 1991.

UCB/EERC-91/09 "A Preliminary Study on Energy Dissipating Cladding-to-Frame Connections," by Cohen, J.M. and Powell, G.H., September 1991.

UCB/EERC-91/10 "Evaluation of Seismic Performance of a Ten-Story RC Building during the Whittier Narrows Earthquake," by Miranda, E. and Bertero, V.V., October 1991.

UCB/EERC-91/11 "Seismic Performance of an Instrumented Six Story Steel Building," by Anderson, J.C. and Bertero, V.V., November 1991.

UCB/EERC-91/12 "Performance of Improved Ground during the Loma Prieta earthquake," by Mitchell, J.K. and Wentz, Jr., F.J., October 1991.