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GROUND IMPROVEMENT ISSUES FOR THE POSEY & WEBSTER ST. TUBES SEISMIC RETROFIT PROJECT: LESSONS FROM CASE HISTORIES

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16. Abstract

The design of ground improvement work to mitigate liquefaction hazards at the Posey and Webster Street Tubes was complicated by some relatively unique aspects of the project. Therefore, a detailed review of case histories and physical modeling studies was requested to address issues of relevance to the Posey and Webster Street Tubes Seismic Retrofit project. This report addresses "Lessons From Case Histories," while a companion report addresses "Lessons From Physical Modeling Studies."

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LESSONS FROM CASE HISTORIES

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1. INTRODUCTION

Ground improvement work is currently being designed to mitigate liquefaction hazards at the Posey and Webster Street (Alameda) Tubes that connect Oakland and Alameda, California. The Alameda Tubes consist of a pair of 37 foot diameter reinforced concrete tubes traversing the channel between Oakland and Alameda. Both tubes were placed in trenches that were backfilled with: (1) loose to medium-dense clean sand along most of the Webster St. tube; and (2) soft, low-plasticity clay with zones of loose sand, silty sand, and sandy silt along most of the Posey tube. Native soil stratigraphy is quite complex, as described in the draft geotechnical report by Parsons Brinckerhoff. Liquefaction analyses for the Safety Evaluation Earthquake indicate that liquefaction would be expected in the sand backfill around the Webster St. tube and in the zones of sand and silty sand backfill around the Posey tube. In addition, there is some concern about potential liquefaction or softening of the lean, soft clay backfill around the Posey tube. Thus, a primary concern is the potential for deformation in the tubes due to earthquake-induced liquefaction or softening of the backfill materials.

Initial design proposals for mitigating the potential for liquefaction-induced deformations in the Alameda Tubes involved:

(1) three rows of stone columns along each side of large lengths of the Alameda tubes; and
(2) in-ground-walls formed by overlapping jet grout columns along each side of those portions of the Alameda tubes where stone column construction is not feasible.

The design of the ground improvement work, and an evaluation of its expected performance during future earthquakes, is complicated by some relatively unique aspects of this project. Thus, a detailed review of case histories and physical modeling studies was requested to address several issues of relevance to the Alameda Tubes project.

Findings of this study are presented in two companion reports, "Lessons From Case Histories" (this report) and "Lessons From Physical Modeling Studies." This report addresses lessons from case histories, and is arranged in the following order:

(1) a review and limited reevaluation of available case histories involving earthquake performance of liquefiable sites treated by vibro- and drain-techniques;
(2) a review of available case histories involving earthquake performance of liquefiable sites treated by in-ground-wall techniques;
(3) a review of recent construction experiences with vibro-replacement stone column techniques, with attention to aspects relevant to the Alameda Tubes project;
(4) a review is presented of other case histories and research of interest to the Alameda project, including the earthquake performance of tunnel-like structures, the applicability of SPT-based design procedures to improved ground, and analysis methods for gravel drains in liquefiable soils; and
(5) a summary is presented of the implications of the preceding studies for the Alameda Tubes project.

Recommendations regarding the proposed ground improvement work for the Alameda Tubes project are outlined by Boulanger, Idriss and Stewart (1997) and have been drawn from the findings presented in this report and the companion report on physical modeling studies.

2. EARTHQUAKE PERFORMANCE OF VIBRO- AND DRAIN-TREATED SITES

Mitchell et al. (1995) recently provided an excellent summary of known case histories involving earthquake performance of improved ground sites, and the lessons that these case histories provided. Our efforts focused on: (1) case histories involving ground improvement techniques that were closely related to those being considered at the Alameda Tubes; (2) case histories for which additional information was published or obtained since Mitchell et al.'s paper was published; and (3) case histories with SPT data for which the Mitchell et al.'s paper did not show analyses.

Fourteen case histories involving the earthquake performance of sites treated by vibro- or drain-techniques were re-evaluated. The location, method of treatment, earthquake event, and estimated peak ground acceleration for each case history are summarized in Table 2-1. Each case history is then individually summarized in Appendix A with tables that list:

- Site and location;
- Earthquake event;
- Facilities;
- Treatment method;
- Behavior of the treated areas;
- Behavior of the untreated areas;
- Subsurface conditions;
- Availability of in-situ test data;
- Comments; and
- Lessons learned.

In addition, Appendix A contains select figures for each case history immediately following their respective summary tables.

2.1. General Performance

In ten of the fourteen case histories, treatment was able to prevent movements or reduce them to harmless levels. Of these ten, at least four had some isolated cracking or damage to pavements or slabs, but the damage was much less than in untreated areas and it was easily repaired.

Five case histories had significant damage within some of the treated areas (Nos. 2, 3, 10, 13, 14 in Table 2-1). At the NTT Building in Niigata and the Paper Plant in Hachinohe (Nos. 2 and 3), damage occurred where the vibroflotation treatment zones were of insufficient vertical or lateral extent. At the Jensen Filtration Plant, (No. 10), damage appears to have occurred because the treated alluvium was too silty (and hence of low permeability) to be satisfactorily drained by the installed sand drain system. The cause of damage at the "Small building" on Port Island and the rubble mound breakwater (Nos. 13 and 14) are not known because sufficient in-situ test data and study of these sites have not yet been published or released.

2.2. Extent of Treatment and Damage Within Treated Zones

Damage within a treated zone may occur when liquefaction of the surrounding untreated soils causes:

(1) high excess pore pressures to migrate from the untreated soils into the treated soils, thereby softening the treatment zone;

(2) a reduction in the lateral or vertical support that the untreated soils provide for the treatment zone; or

(3) loads imposed on the treatment zone due to lateral spreading deformations in the surrounding untreated soils.

The effect of treatment extent on behavior is illustrated by the following case histories.

(1) The 0.5 m of settlement experienced by the NTT Building (No. 2 in Table 2-1) is attributed to liquefaction between depths of 7 and 12 m. The treatment zone only extended to a depth of 7 m, and thus was of insufficient vertical extent to protect the building from damage.

(2) Movements up to 0.4 m in the secondary buildings at the Paper Plant in Hachinohe (No. 3 in Table 2-1) is attributed to the fact that the treatment was limited to strips underneath the columns and footing beams. Liquefaction of the surrounding soils likely resulted in loss of lateral confinement and migration of high excess pore across the treatment strips.

(3) The three building sites on Treasure Island (Nos. 5, 6, and 7 in Table 2-1) all appeared to have experienced minimal differential settlements despite liquefaction having developed between depths of 6.7 and 12 m at least one site (No. 5; sand ejecta in elevator pits) and probably over similar depth intervals at the other two sites. This good performance illustrates that thick nonliquefiable crusts in non-laterally-spreading sites may provide adequate safety against excessive deformations or damage in some situations.

(4) Damage to slabs within the "Warehouse Facility" on Port Island (Case 11 in Table 2-1) may be attributed to: (1) liquefaction of the surrounding soils between depths of 4 and 16 m since the treatment zones only extended 2-5 m beyond the building edges; and (2) liquefaction beneath portions of some warehouses since treatment may have been obstructed by a preexisting buried sea wall.

(5) Liquefaction evidence along the south boundary of the amusement park on Port Island (Case 12 in Table 2-1) appears to have been within a distance equal to about 1/2 the "thickness of liquefiable soil" from the treatment boundary.

The effect that the extent of treatment has on behavior clearly has to be considered in design. Quantifying the required extent of treatment for a particular design purpose requires consideration of the thickness of liquefiable soils, the behavior of the surrounding liquefied soils, the potential for lateral spreading (proximity to a free face), the thickness of any nonliquefiable crust, the structural loads, and the tolerable amount of deformations.

2.3. Observed Performance Versus Current Analysis Methods

Seven case histories (Nos. 1, 2, 3, 4, 6, 11, and 12 in Table 2-1) included sufficient in-situ test data to enable a limited re-evaluation of the liquefaction potential at these sites. In all cases, the available in-situ test data were from Standard Penetration Tests (SPT). Testing procedures were generally not well described, and thus assumptions regarding the conversion of reported SPT N-values to equivalent N_{60}-values are given in the Tables in Appendix A. The potential for liquefaction was estimated using the simplified procedure developed by Seed and Idriss (1971) and the updated charts of Seed et al. (1985). Mitchell et al. (1995) present similar analyses for three of these case histories (Nos. 6, 11, and 12).

Analysis results for these case histories are presented in Appendix A along with their corresponding summary Table and select figures (showing the in-situ test data). The analysis results are presented as plots of earthquake-induced cyclic stress ratios versus $N_{1\text{-}60}$ values, and include the curves proposed by Seed et al. (1985) for direct comparison. Data were sometimes grouped by depth interval, with the depth intervals selected to be representative of the stratigraphic units or to better represent variations in either cyclic stress ratios or $N_{1\text{-}60}$ values. This limited reevaluation of the SPT data was consistent with the observations, provided that the role of overlying crusts, the extent of treatment, the magnitude of ground surface deformations, and the uncertainty in the in-situ test data and peak ground accelerations are considered. The data were not sufficiently detailed to evaluate: (1) the potential benefits from drainage during earthquake shaking, or (2) the potential benefits of the shear stiffness and strength of any vibro-replacement columns.

2.4. Relative Roles of Densification and Drainage

Vibro-replacement techniques can improve liquefiable soils by densifying them, by providing drainage paths for excess pore pressures to dissipate during earthquake shaking, and by reinforcing them. In design practice, the potential benefits of drainage and reinforcement are often not quantified or explicitly relied upon, although they may be referred to as an additional benefit of the vibro-replacement technique. The relative roles of these three mechanisms at sites treated by vibro-replacement techniques and shaken in past earthquakes is impossible to evaluate based on the available case history data.

Two case histories (Nos. 9 and 10 in Table 2-1), however, involved the use of sand or gravel drains for the primary purpose of dissipating excess pore pressures during earthquake shaking. Woodward-Clyde Consultants anticipates completing their study of the Jensen Filtration Plant case (No. 10 in Table 2-1) sometime next year, and thus there is currently insufficient publicly available data to allow an independent, detailed evaluation of that site's performance. However, information is available from papers describing the lateral spreading damage at this site after the 1971 earthquake (Youd 1971, Dixon and Burke 1977). The liquefiable layer at this site is between 1.5 and 7.6 m thick, and consists of silty sands and sandy silts with occasional zones of clayey silt and silty clay materials. SPT $N_{1\text{-}60}$ values in this liquefiable layer were about 10-15 before treatment. This layer typically had more than 50% fines (passing #200 sieve), and thus it is possible that these soils might not have drained rapidly enough during earthquake shaking. In addition, the drains were constructed prior to the availability of research results (e.g., Iai and Koizumi 1986; Onoue 1988) showing that drain resistance is more important than indicated by Seed and Booker (1977). The detailed study of the Jensen Filtration plants' performance currently underway is expected to clarify the important factors in the observed damage at this site.

The use of gravel drains at the Port of Kushiro (No. 9 in Table 2-1) appears to have been successful in preventing significant movements. However, the treatment zone was also confined largely within steel cellular cofferdams, and this may also have contributed to the good behavior at this site.

Table 1. Summary of Case Histories

No. (1)	Site (2)	Location (3)	Method of treatment[b] (4)	Earthquake event (5)	Peak accel. (6)	Damage (7)
1	Nippon Oil Co.	Niigata	Vibroflotation	1964 Niigata	0.16 g	None; Minor
2	NTT building	Niigata	Vibroflotation	1964 Niigata	0.16 g	S_{max}≈0.5 m
3	Paper plant: (i) Group I (ii) Group II	Hachinohe	Vibroflotation	1968 Tokachioki	0.225 g	(i) None. (ii) S_{max}≈0.4 m
4	Group of oil tanks	Ishinomaki Port	Sand compaction piles	1978 Miyagiken-oki	0.18 g[a]	None
5	Med/Dental clinic	Treasure Island, CA	Vibroreplacement stone columns	1989 Loma Prieta	0.16 g	None
6	Building 450	Treasure Island, CA	Sand compaction piles	1989 Loma Prieta	0.16 g	None
7	Facilities 487-489	Treasure Island, CA	Vibrocompaction (vibroflotation)	1989 Loma Prieta	0.16 g	Minor cracking in floor of bldg. 487.
8	Approach to Pier 1	Treasure Island, CA	Vibroreplacement stone columns	1989 Loma Prieta	0.16 g	None
9	Wharves (6 locations)	Port of Kushiro	Gravel drains	1993 Kushiro-Oki	0.47 g	None, ranging to S_{max}≈20-40 mm
10	Jensen Filtration Plant	Northridge, CA	Sand drains	1994 Northridge	0.98 g	Cracks to 80 mm, offsets to 200 mm.
11	Warehouses (5 buildings)	Port Island, Kobe	Vibro-rod	1995 Hyogo-Ken Nanbu	0.34 g[a]	None, ranging to offsets of 100 mm.
12	Amusement park	Port Island, Kobe	Vibro-rod	1995 Hyogo-Ken Nanbu	0.34 g[a]	None; some cracks to 25 mm and ejecta along south side.
13	Small building	Port Island, Kobe	Vibro-rod	1995 Hyogo-Ken Nanbu	0.34 g[a]	S_{diff}≈150 mm beside building.
14	Rubble mound breakwater	Nishinomiya area	Sand compaction piles	1995 Hyogo-Ken Nanbu		S_{max}≈1-2 m.

[a] See the respective Table for this case history for more detail.
[b] Brief explanations of the different terms are given below.

Vibroflotation (or vibrocompaction): a probe that vibrates laterally due to rotating ecentric weights.

Vibroreplacement stone columns: similar to vibroflotation but with the cavity being infilled with stone or gravel by either a top- or bottom-feed method to produce a coherent column of compacted stone.

Vibro-rod: a vibrating probe that is vibrated vertically, with the vibration applied from above the ground surface; the Vibro-rod, Terraprobe, Vibro-Wing, and Franki Y-probe are variants of the vibrating probe approach.

Sand compaction piles: a closed-end pipe pile is driven to the desired depth, and the resulting hole filled with sand during withdrawal of the casing; may include redriving the casing several times during withdrawal to improve densification (e.g., the vibrocompozer method, common in Japan, involves redriving the casing; redriving was not reported for Building 450 on Treasue Island).

Gravel drains: a casing auger is advanced to the desired depth, gravel is progressively poured in the casing and compacted by a tamper as the casing is slowly withdrawn.

Sand drains at the Jensen Filtration Plant: details not yet released..

3. EARTHQUAKE PERFORMANCE OF IN-GROUND-WALLS

The Kobe earthquake produced several case histories involving the performance of sites improved by in-ground-walls. According to a personal communication with O. Taki by Mitchell et al. (1995), there were eight such case histories that all showed very good performance. No details for these case histories were provided at that time, however. Details have been published by Hamada and Wakamatsu (1996) for the following four cases.

3.1. Shimagami Pumping Station in Kobe

The Shimagami Pumping Station in Nagata Ward in Kobe experienced hardly any damage during the 1995 Hyogo-ken Nanbu earthquake despite extensive liquefaction in the surrounding soils (Hamada and Wakamatsu 1996). The Station building is located within 8 m of a quay wall that moved outwards up to 3 m, and the ground surface settled more than 1 m adjacent to the building (Figure 3-1). This 16 m high building has a basement that extends 8 m below the ground surface on the north side, and 11 m below the ground surface on the south side (Figure 3-2). The basement excavation involved construction of a perimeter in-ground-wall (Soil Mixed Wall, or SMW type) that was 0.45 m wide, about 18 m deep, and reinforced with steel H-section piles. This in-ground-wall was left in place after construction was completed. The building itself was supported by 1.0 to 1.5 m diameter cast-in-place concrete piles. The soil profile consisted of about 10.5 m of loose sandy fill, overlying sand with gravel and gravel that extends to depths of more than 20 m. This underlying gravel layer is considered nonliquefiable, and has $N_{1\text{-}60}$ values greater than 30 below about 12 m depth. Hamada and Wakamatsu (1996) suggested that the "in-ground wall had a great influence to prevent damage to the foundation piles, in addition to existence of the basement." Note that Hamada and Wakamatsu also had presented data showing that buildings with basements had generally experienced less differential settlements than buildings without basements.

The presence of the basement and piles likely had a very positive role in the good performance of this building. On the south side of the building, the 11 m deep basement has essentially no liquefiable soil beneath it. On the north side of the building, the 8 m deep basement has only 2 to 3.5 m of liquefiable soil beneath it. Potential deformations within this liquefiable layer beneath the north basement would have been less than in the much thicker surrounding fills. Resistance to these deformations would also have been provided by the 1.0-1.5 m diameter piles and the fact that the south side of the building was resting on nonliquefied soil. Therefore, the in-ground-wall may be considered to have had a potentially positive influence on the behavior of this building, but it seems that the presence of the basement and piles had as strong, or possibly stronger, influence.

3.2. Seibu Sewage Treatment Plant in Kobe

The Seibu Sewage Treatment Plant in Kobe has three sedimentation basins, of which basins A and B experienced large separations in concrete casting joints during the 1995 Hyogo-ken Nanbu earthquake, while basin C experienced no damage (Hamada and Wakamatsu 1996). The basins are located near quay walls that moved about 2 m outwards due to liquefaction of the surrounding fill (Figure 3-3). Basin C had 2 basements that together extended about 6 m below the ground surface, which is much deeper than for the other two basins. Excavation of the basement for basin C involved construction of an in-ground-wall around the perimeter (Figure 3-4). The in-ground-wall was shown to extend to about 17 m deep, but details of the wall thickness, construction method, or possible reinforcement were not given. The basin building was founded on 1.5 m diameter cast-in-place concrete piles. The soil profile consists of about 10 m of loose sandy fill, overlying 4 m of silt (N values ranging from 8 to 35), and then underlain by inter-layered sand and silt to the depths of interest. As for the Shimagami Pumping Station, it appears that the good performance of this building may be partly attributed to the presence of the in-ground-wall, but that the presence of the basement and large-diameter piles would also be expected to have played an important role.

3.3. The Oriental Hotel in Kobe

The Oriental Hotel is a 60 m high building located on the Central Pier of Kobe Port (Hamada and Wakamatsu 1996). The quay walls around the perimeter of the pier were heavily damaged with movements greater than 1 or 2 m due to liquefaction of the surrounding fill. The soil profile consists of about 12 m of loose sandy fill, overlying about 18 m of Holocene clay, and underlain by firm Pleistocene gravel. The hotel has no basement, and was supported by 1.0 m diameter, 30 m long, cast-in-place concrete piles bearing in the firm gravel. In-ground-walls, about 1.0 m wide, were constructed by overlapping 1.0 m diameter, 12-19 m long, mixed-in-place soil-cement piles (Figure 3-5). There was no structural damage to the building and no settlement within the in-ground-wall area after the 1995 Hyogo-ken Nanbu earthquake. In addition, there was no evidence of liquefaction on the ground surface when the building foundation was excavated after the earthquake. Thus, the evidence indicates that the in-ground-walls were very effective in protecting the foundation of this building from liquefaction-induced deformations despite the large deformations that developed around, and immediately adjacent to, the building's perimeter.

3.4. Highway Bridges of No. 5 Bay Highway in Uozaki-hama

Hamada and Wakamatsu (1996) reported that the highway bridges of No. 5 Bay Highway in Uozaki-hama had in-ground-wall foundations. A comparison of lateral movements of bridge foundations versus the surrounding ground surface suggest that in-ground-walls and caissons were more effective than pile foundations in minimizing the permanent lateral deformations induced by liquefaction (Figure 3-6).

3.5. Discussion of Design Issues

In-ground-walls are a potentially economical ground improvement technique, particularly for soils with significant fines contents for which vibro-densification and drain techniques may be ineffective. In-ground walls improve liquefiable sites by: (1) reducing earthquake-induced shear strains in the treatment zone, thereby limiting pore pressure generation in the enclosed soils; (2) containing the enclosed soil should liquefaction develop, and thus contributing to the composite shear strength of the treatment zone; and (3) acting as a barrier to the migration of high excess pore pressures from the surrounding untreated soils.

Design methods for quantifying the behavior of in-ground-walls during earthquake shaking are not yet well established. The rehabilitation of Jackson Lake Dam included one of the first applications of in-ground walls in the U.S. (Ryan and Jasperse 1989, Taki and Yang 1991). The in-ground walls were not designed to prevent the triggering of liquefaction, but rather to provide sufficient strength in the treatment zone to ensure slope stability even if the enclosed soils did liquefy. The three physical modeling studies described in the accompanying report provide a starting point for understanding the earthquake behavior of in-ground walls and developing design methodologies. For example, the experiments by Suzuki et al. (1991) and Babasaki et al. (1991) showed that the ability of in-ground walls to reduce excess pore pressure generation is dependent on the shear stiffness of the walls aligned with the direction of shaking. Increasing the shear stiffness of the in-ground wall system (by reducing the spacing perpendicular to the direction of shaking) reduces the shear strains induced on the enclosed soil during earthquake shaking and hence reduces the generation of excess pore pressures. Babasaki et al. (1991) also described some limited results from three-dimensional, linear-elastic, finite element analyses of the in-ground wall system. Additional research on the effectiveness, and design, of in-ground walls for mitigating liquefaction hazards is needed.

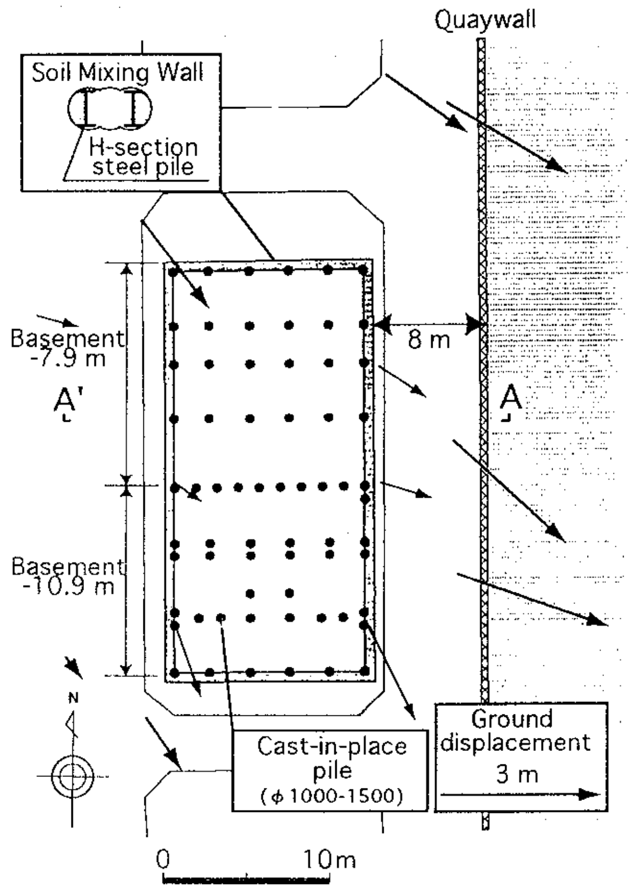


Fig. 3-1. Plan of Shimagami Pumping Station and Liquefaction-Induced Ground Displacement (Hamada and Wakamatsu 1996)

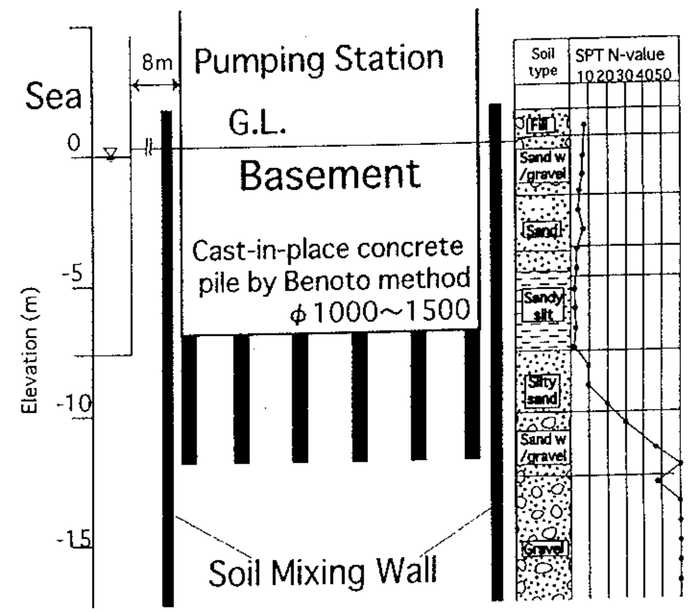


Fig. 3-2. Cross-Section of Shimagami Pumping Station (Hamada and Wakamatsu 1996)

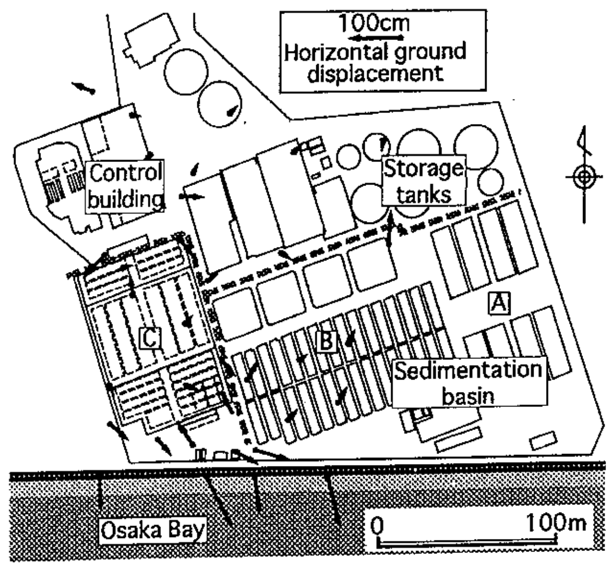


Fig. 3-3. Plan of Seibu Sewage Treatment Plant and Liquefaction-Induced Ground Displacement (Hamada and Wakamatsu 1996)

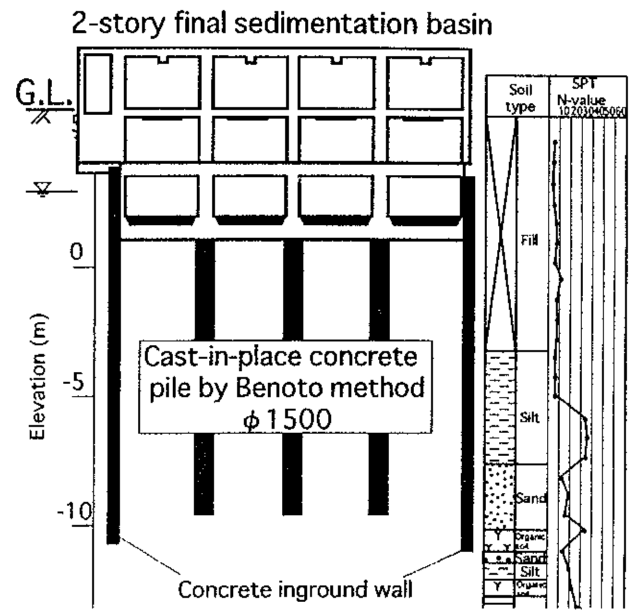


Fig. 3-4. Cross-Section of Basin C at Seibu Sewage Treatment Plant (Hamada and Wakamatsu 1996)

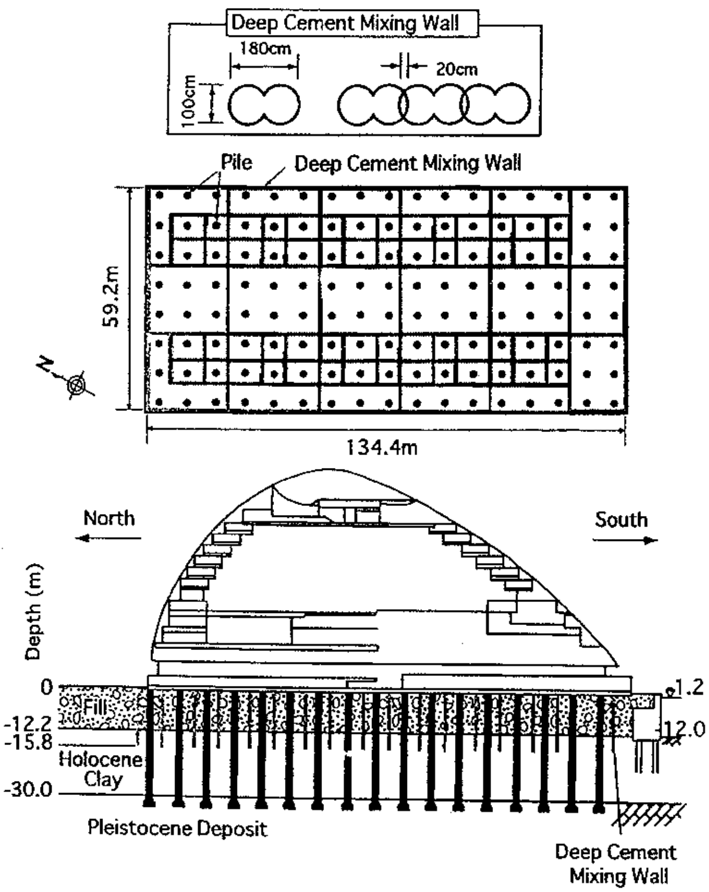


Fig. 3-5. Cross-Section and Plan of Oriental Hotel (Hamada and Wakamatsu 1996)

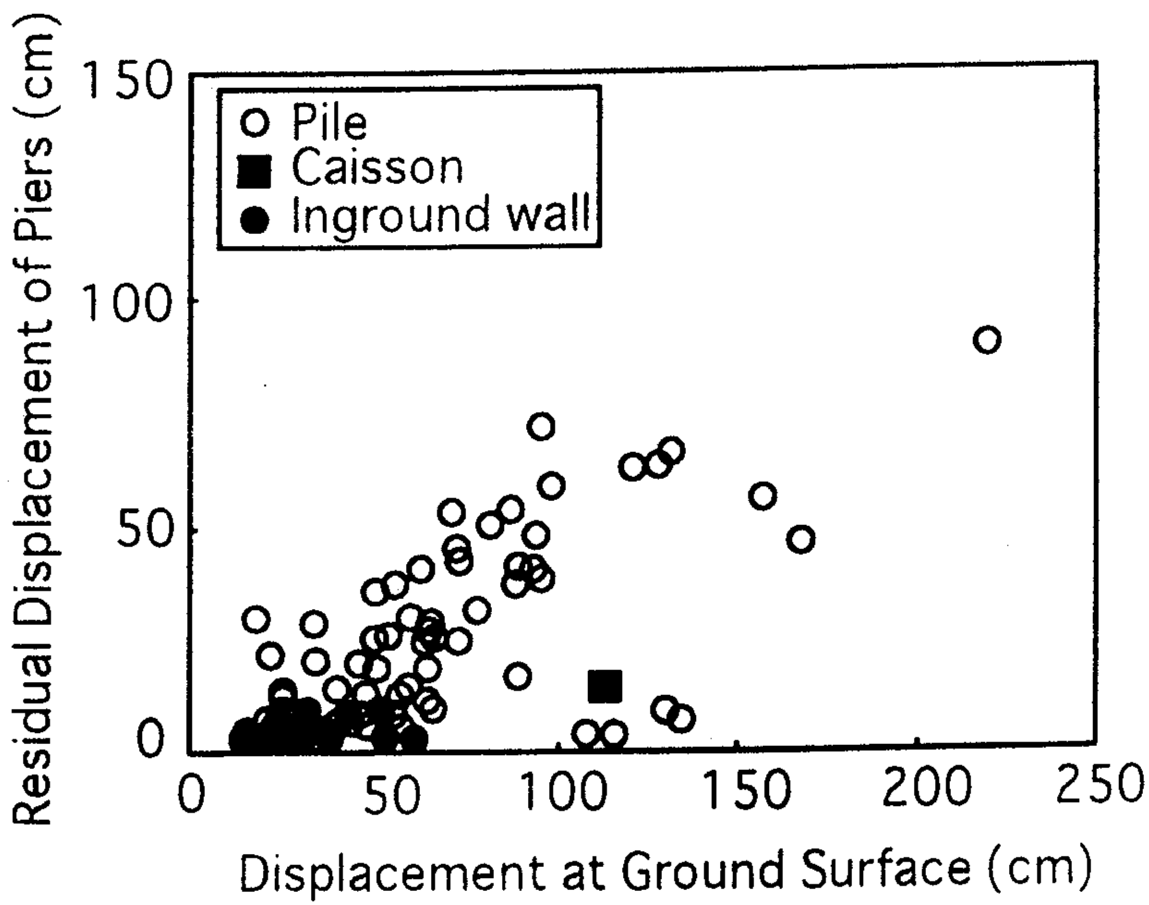


Fig. 3-6. Displacements of Bridge Piers Versus Displacements of Ground Surface (Hamada and Wakamatsu 1996)

4. RECENT CONSTRUCTION EXPERIENCE WITH STONE COLUMNS

Recent construction experiences with stone columns were reviewed with regard to lessons that might prove valuable for the Alameda Tubes project. Issues that were given attention include:

(1) the ability to construct stone columns that can reliably act as drainage elements;
(2) the ability to penetrate hard layers, cobbly soils, or any other material that resembles the magnetite (iron ore) materials overlying the Alameda Tubes; and
(3) experience in soils similar to the backfill materials around the Alameda Tubes.

4.1. Redondo Beach King Harbor

The permeability of vibro-replacement stone columns at Redondo Beach King Harbor were studied by Baez and Martin (1995). Two different construction methods were used (two columns by top feed, and two columns by bottom feed), and two different stone gradations were used (a 19 mm uniform blend classifying as GP, and a #40 sieve to 19 mm blend classifying as GW). Water jetting was used with the top feed method, and water and air jetting were used with the bottom feed method due to difficulties in penetration and in maintaining the probe free (Juan Baez, personal communication). The soil profile consisted of about 6.5 m of loose hydraulic sand fill (typically <5% fines), overlying about 2.5 m of silty clay, overlying medium-dense to dense silty sand and sand to depths of about 12 m (the depth of treatment). Permeabilities were measured using field pumping tests and laboratory constant head tests.

Field injection tests gave permeabilities of about 6.2×10^{-4} cm/s for the native sand and 0.9×10^{-2} to 2.5×10^{-2} cm/s for the columns; the ratio of column to native sand permeabilities being about 15 to 40. Constant head tests were performed on samples of the native sand and of the column materials. Samples were obtained from between depths of about 3 and 4 feet by hand-excavation (Juan Baez, personal communication). Column materials were found to contain about 80% imported stone and about 20% native sand, regardless of the feed method or stone gradation (Figure 4-1). The mix of native sand and imported stone appeared uniform across the column (Juan Baez, personal communication). The laboratory tests gave permeabilities of about 2.4×10^{-2} cm/s for the native sand, and 1.1 to 2.1 cm/s for the column materials; the ratio of column to native sand permeabilities being about 40 to 100. Thus, the laboratory tests gave permeabilities that were about two orders of magnitude greater than from the field tests, regardless of whether the tests were for the native soils or column materials. The reasons for these discrepancies were not understood. Nonetheless, a comparison of lab-to-lab or field-to-field test results give a reasonably consistent range for the ratio of column permeability to native soil permeability (15-40 versus 40-100).

An important issue not addressed in these experiments is the vertical permeability of the stone columns. The field experiments were interpreted as producing predominantly radial flow in the stone columns, and thus were supposedly measuring the horizontal permeability. It seems very likely that the vertical permeability of the stone columns will vary considerably due to variations in the degree of mixing between the imported stone and the native soil, and in the native soil type. Vertical flow within a stone column would be effectively impeded by the lowest permeability intervals. Therefore, the thorough mixing of the imported stone with a layer of low-permeability native soil over a relatively small depth interval could greatly reduce the drainage capacity of that stone column during earthquake shaking.

4.2. Mormon Island Auxiliary Dam

Bottom-feed stone columns were used to treat old dredge spoils in the foundation of the Mormon Island Auxiliary Dam (Allen et al. 1995). These dredge spoils are irregularly layered with fine and coarse fractions ranging from sandy silt to cobbles. The upper 4.5-6 m was coarse sand to cobble size, and the lower 3-6 m was silty sand to silty clay, with 10-77% fines (30% on average).

Previously, there was little experience with constructing stone columns in coarse sands and gravels, so a test program evaluated different construction methods: rotary displacement stone columns, vibro-pipe drains, vibro-rod compaction, bottom-feed stone columns, and vibro-rod/vibropipe stone columns. All methods proved constructible over the full 20-m layer at the toe of the dam; i.e., where they did not have to penetrate the compacted materials of the downstream shell. Only the vibropipe method proved constructible through the downstream shell of the dam. Water jetting was required to achieve the required treatment depths. Vibropipe and bottom-feed stone columns were selected for Phase I construction. The design called for a wide zone of bottom-feed stone columns (diameter > 1.2 m) with a 6-m-wide upstream curtain, and 2.4-m-wide downstream curtain, of 0.25 m diameter vibropipe mini-columns on 1.0 m spacing. These curtains are to act as drains to prevent migration of excess pore pressures from the surrounding liquefiable zones into the treatment zone.

Baez and Martin (1995) reported, based on a personal communication with Matt Allen in 1994, that the constructed bottom-feed stone columns were found to consist of about 77% imported stone plus about 23% native sand, based on bulk samples obtained using open-ended Becker hammer borings. Mr. Allen (1997, personal communication) provided the following additional information: (i) the open-ended Becker hammer tests were performed in the centers of one vibro-replacement bottom-feed stone column and one rotary-displacement stone column during the demonstration section work, with samples being intermittently drawn from the cyclone; (ii) the vibro-replacement column had a reasonably consistent mix of native soil and imported stone over its full length, with the exception that the top 10-15 feet contained a greater percentage of fines; (iii) the greater percentage of fines in the top 10-15 feet of the vibro-replacement stone column was attributed to the high water flows during construction which brought up large quantities of fines from the lower portions of the treatment zone; and (iv) that the rotary displacement column appeared to have a more variable mix of native soil and imported stone along its length.

Baez and Martin (1995) reported further that the stone backfill was a rounded, minus 25 mm stone, and that the permeability of the stone columns samples was about 26 times less than that of the imported stone (based on laboratory, constant head tests). Furthermore, field pumping tests showed that treatment had reduced the average permeability of the site by 1/5 to 1/2 of the typical free-field values. This reduction in the average permeability is consistent with the expected effects of densification on the permeability of the native soil, and with the fact that the average permeability for horizontal flow across the treatment zone would be dominated by the native soil matrix and not by the isolated stone columns. Free-field permeabilities of the dredge spoils were typically 0.6-1.5 $x10^{-2}$ cm/s (from pumping tests), while the lab tests on the stone column mixtures gave permeabilities of about 1.2-1.4 cm/s. These data suggest that the stone column mix was still about 100-200 times more permeable than the native sand. However, the field pumping results could be biased by the layering of lower and higher permeability soils, and thus the relative permeabilities of the stone column and native sand could vary vertically within the deposit. Thus, some caution is warranted in evaluating these results because they involve a comparison of small-scale laboratory tests against large-scale field tests.

4.3. Fraser Delta in British Columbia

Unpublished information on the mixing of native soil and imported stone by different stone column construction methods was presented by Mr. Ernest Naesgaard of Macleod Geotechnical Ltd. in West Vancouver, British Columbia, at the 7th Annual Symposium of the Vancouver Geotechnical Society in 1993. Naesgaard graciously provided the following information in personal communications, excerpts from project files, and written correspondence (December 1996, January 1997, February 1997). His efforts are greatly appreciated.

At a site in the Fraser Delta, vibro-replacement stone columns were constructed for the purpose of densification. The stone columns were later exposed when a 12-foot deep excavation, including de-watering, was made for construction of a raft foundation. Voids within the stone column were filled with the native sand, as shown by photographs and grain size analyses. The imported stone was uniformly graded 1"-1.5" rounded gravel. The stone columns were constructed using the top feed method with water jetting. A lot of extra vibrational effort was needed within the 10-20 foot depth interval to densify the sand because of the presence of some silt; this extra effort may have aggravated the mixing of the native soil with the imported stone.

Naesgaard also described another site where they constructed vibro-replacement stone columns for the purpose of drainage during earthquake shaking. Columns were constructed by washing and vibrating a hole, thoroughly flushing the hole until the return water was clean, and then dumping pea gravel into the hole from the top (i.e., top feed method). Only nominal vibration was applied to the pea gravel as the column was built to avoid mixing the pea gravel with the native soil. Samples of the constructed stone columns were obtained using a hollow-stem auger and a SPT sampler with a core-catcher. Grain size tests on the samples showed that the column materials were clean (i.e., negligible mixing with the native soil). They also stripped the top 2 feet of soil, and saw that the exposed stone columns were clean. It was believed that the stone columns were clean because they had thoroughly cleaned the hole and then used minimal vibration during placement of the pea gravel (i.e., their main goal was a clean column, not densification of the native soil).

4.4. Seventh St. Marine Terminal at the Port of Oakland

Stone columns were used at the Seventh St. Marine Terminal at the Port of Oakland, after it was damaged in the Loma Prieta earthquake, to mitigate liquefaction hazards during future earthquakes (Egan et al. 1992). A 12-m wide zone of stone columns was constructed along the rear of the wharf. The stone columns extended mostly through a 4 to 7 m thick rock fill dike that had been used as a perimeter dike to retain hydraulic filling of the terminal area (Figure 4-2). The dike rests on a 4.3 to 8.2 m thick hydraulically-placed sand base. These sands are predominantly clean, fine-to-coarse grained with N_{1-60} of 9 to 20 (average 14). The dike was constructed of sound rock (<15" and <6" source materials), and not considered susceptible to liquefaction. Underneath the dike and sand base, there is a 11 to 15 m thick layer of native silty or clayey sand. The upper 1.5 m of this layer has 12-35% fines with N_{1-60} of 13 to 18 (average 15), and thus was considered potentially liquefiable under the design earthquake. The deeper portions of this native sand layer were not considered susceptible to liquefaction.

The design required that N_{1-60} values in the liquefiable sands beneath the dike be increased to about 25 to 30, based on liquefaction analyses using the charts by Seed et al. (1985). However, analyses were also performed to identify a stone column spacing that would minimize excess pore pressures during earthquake shaking, using the procedures of Seed and Booker (1977). Based on these analyses, a maximum spacing of 2.4 m was specified for stone column diameters of 0.9 m.

Vibro-replacement stone columns were constructed with the stone being introduced at the top of the vibrator's hole by a front-end loader. Representative pre- and post-treatment N_{1-60} values were shown to have increased to between 25 and 35 (excluding 1 higher and 1 lower outlying data point). Penetration through the rockfill dike was accomplished using water jetting at the vibrator tip. No problems with penetrating the rockfill were reported.

4.5. Medical/Dental Clinic on Treasure Island

Stone columns were used in 1989 to mitigate liquefaction hazards at the Medical/Dental Clinic on Treasure Island (Mitchell and Wentz 1991). The soil profile consists of hydraulic sand fill (typically <10% fines) to a depth of about 13 m. This sand includes some thin clayey silt lenses between the depths of about 6.7 and 12.2 m. Trial treatment sections were constructed with stone column spacings of 2.44 to 3.05 m, to a depth of 12.2 m. For the zone between 6.7 and 12.2 m depth, N values were reported to be 2 to 5 before treatment and 3 to 19 after treatment, and CPT tip resistances were unchanged by treatment. It was subsequently decided to only treat the hydraulic sand fill to a depth of 6.7 m. During the Loma Prieta earthquake, the bottom 2.4 m of two 6.7 m deep elevator shafts filled with sand ejecta, indicating that the sand between depths of 6.7 and 12.2 m must have liquefied.

4.6. Approach Area to Pier 1 On Treasure Island

A vibrating probe was used in 1985 to treat hydraulic sand fill (typically <10% fines) to depths of 13 m at the approach area for Pier 1 on Treasure Island (Mitchell and Wentz 1991). The lower 2 m of the fill was silty sand and sandy silt. Treatment of this lower 2 m was reported as not meeting the specified "minimum relative density;" however, the measurement methods for determining relative density of silty sand were not stated.

4.7. Terminal Island in San Pedro Bay

Yourman et al. (1995) describe the use of bottom-feed vibro-replacement stone columns in a variable hydraulic fill consisting of silty sands inter-layered with silts and clays up to 3 m thick. Increases in SPT N values were obtained in thicker zones of silty sand with less than about 15% fines, but no significant increase in N-values were obtained in silty sands with greater than 15% fines or in the inter-layered zones of fine and coarse grained soils. Their experiences serve as a reminder that stone columns generally do not produce increase in N values in predominantly fine grained soils, and that the effectiveness of stone columns in soils with clay interlayers can be hampered or difficult to evaluate.

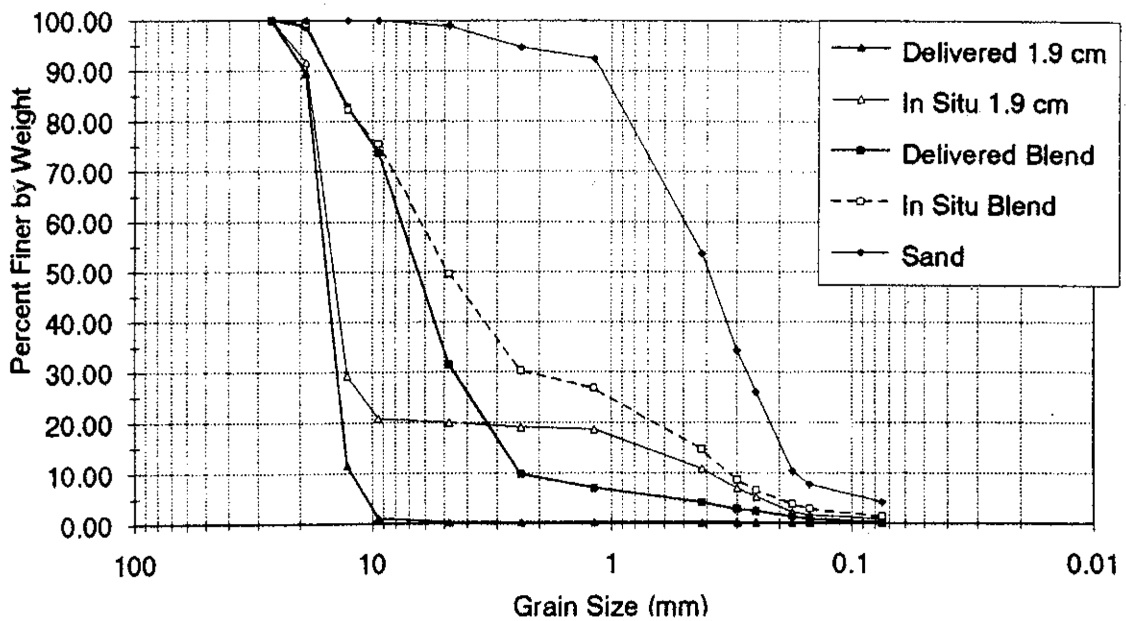


Fig. 4-1. Gradation for Stone Column Materials at King Harbor (Baez and Martin 1995)

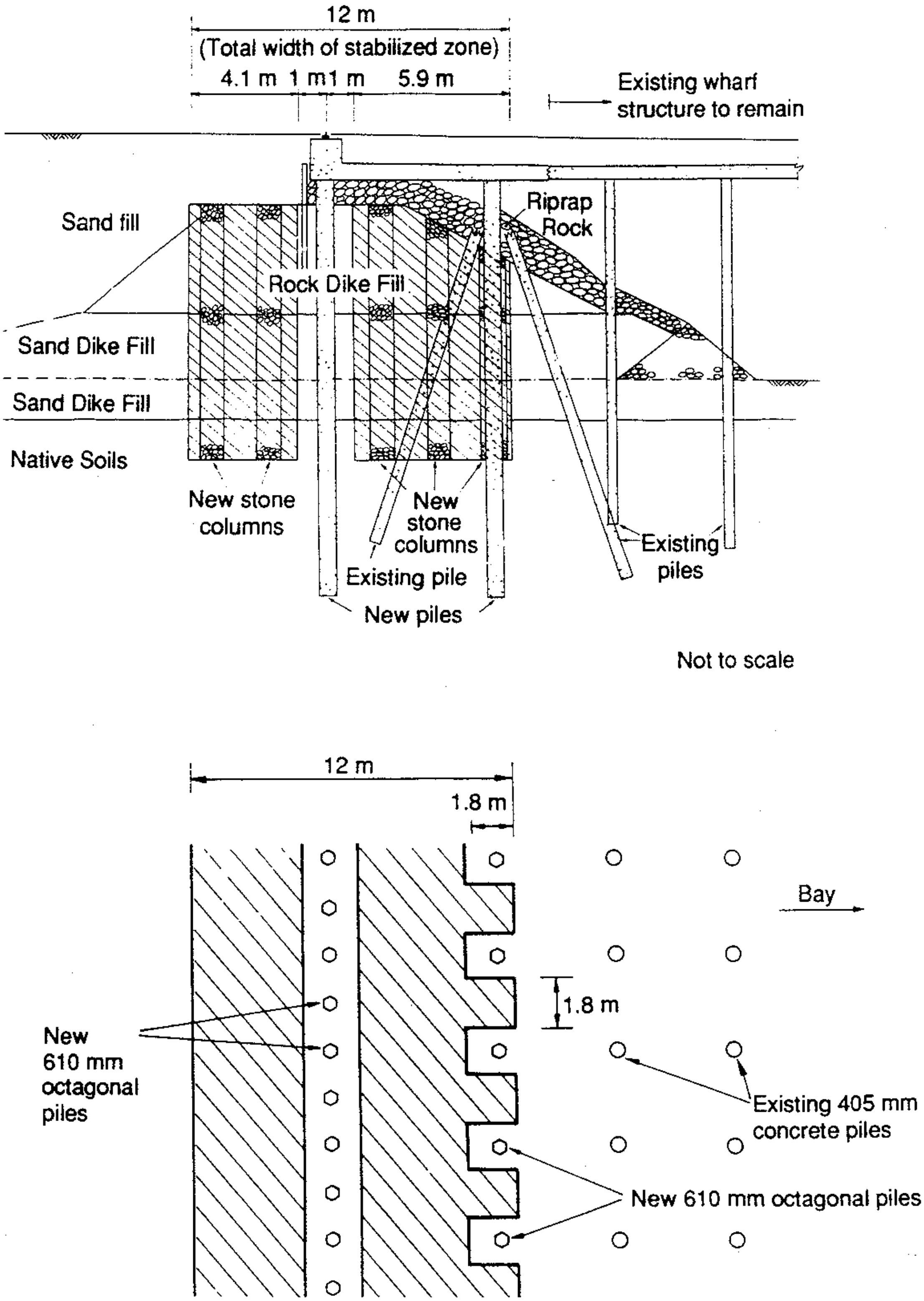


Fig. 4-2. Stone Column and New Piling Configuration for Seismic Repair at Seventh Street Marine Terminal (Egan et al. 1992)

5. OTHER CASE HISTORIES AND RESEARCH OF INTEREST

5.1. Tunnel-Like Structures in Liquefiable Soil

The collapse of the Daikai subway station, a cut-and-cover underground metro station, in the 1995 Kobe earthquake was the first reported failure of an underground metro station during an earthquake (Iida et al. 1996, Matsuda et al. 1996, Nakamura et al. 1996). This 17 m wide, 7.2 m high, 120 m long box frame structure suffered complete collapse of more than half its center columns, followed by collapse of the ceiling slab (Figures 5-1 and 5-2). This structure was constructed in 1964 without consideration of seismic loads. Backfill around the structure was decomposed granite with corrected N-values of about 10 to depths of about 12 m. The water table was between depths of 6 and 8 m. Two-dimensional finite element analyses using equivalent-linear properties for the soil (Matsuda et al. 1996, Nakamura et al. 1996) suggest that the damage was primarily caused by strong horizontal forces due to relative displacements of the surrounding soils and inertial loads from the overburden soil during the earthquake. The presence of the loose backfill immediately adjacent to the structure was included in the analyses of Nakamura et al. (1996) but not in the analyses by Matsuda et al. (1996). The potential for liquefaction in the lower portions of the backfill, and the resulting influence on dynamic response, was not addressed in either analysis. Both analyses showed that the shear and bending capacity of the center columns was exceeded during the earthquake.

Most pipelines are intentionally constructed in soil above the water table for ease of construction and maintenance. Hence, case histories of pipelines and other conduits floating to the surface as a result of earthquake-induced liquefaction are scarce.

Iai and Matsunaga (1992) analyzed the uplift of a 20 m wide by 7.5 m high underground structure embedded in a 19 m thick deposit of loose sand. Results of the effective stress-based, finite element analyses were presented, with emphasis on the stresses and strains in the soil surrounding the tunnel. The results indicated that the primary mechanism causing uplift of the buried structure is the extensional shear deformation in the soil below the structure. This conclusion is in agreement with observations from physical modeling studies, the results of which are summarized and discussed further in the companion report.

5.2. Applicability of SPT-Based Liquefaction Relationships to Improved Ground

SPT-based liquefaction relationships were developed using case histories for non-improved soil sites, and thus there may be some question as to whether they are directly applicable to improved soil sites (e.g., different fabric, lateral stress states, stress and strain histories). Tokimatsu et al. (1990) addressed this issue by performing cyclic laboratory tests on high-quality, undisturbed samples of sand obtained by in-situ freezing from a deposit that was treated by the vibratory sand compaction pile method. The correction of cyclic triaxial strengths to field loading conditions (e.g., simple shear loading conditions) required an estimate of the lateral earth pressure coefficient at rest (K_o). Tokimatsu et al. assumed that K_o was between 0.5 and 1.0 after treatment, although K_o was not directly measured. They concluded that: (1) the relationship between cyclic strength and N_1-values for this improved site was consistent with the relationships for natural sand deposits over the range of N_1-values between 21 and 28; and (2) that the liquefaction resistance of compacted sands could be evaluated using the SPT-based relationships developed for natural sand deposits.

5.3. Analysis of Gravel Drains

Seed and Booker (1977) presented design diagrams for the use of gravel drains to prevent liquefaction of sands during earthquake loading. In evaluating the effect of drain permeability, seepage in the drain column was assumed to occur radially towards an infinitely pervious pipe at the center of the column. Based on this approximation, Seed and Booker concluded that the drain functioned as if it were perfectly pervious if its permeability is on the order of 200 times that of the native soil (Figures 5-3 and 5-4). In the field, however, seepage in the drain columns will be primarily vertical towards the drainage boundaries and the drainage-path length can be very large for thick soil deposits. Consequently, the analyses by Seed and Booker (1977) under-estimate the detrimental effects of drain (or well) resistance on the performance of gravel drains.

Onoue (1988) presented design diagrams for gravel drains taking into account the effects of drain resistance. The boundary conditions for Onoue's analyses are shown in Figure 5-5, and the resulting design diagrams are shown in Figure 5-6. Onoue used the same pore pressure generation function as Seed and Booker (1977). Seed and Booker presented two sets of diagrams, one in terms of the maximum pore pressure ratio ($r_{u,max}$) and the other in terms of the maximum average pore pressure ratio [$(r_{u,ave})_{max}$; accounts for variation in r_u with radial position]. In the analyses by Onoue (1988), $r_{u,max}$ is a function of radial and vertical position, and thus Onoue presented diagrams for the overall average of the maximum excess pore pressure ratio [$(r_{u,max})_{ave}$]. Onoue showed that $r_{u,max}$ at an arbitrary point will not exceed 0.70 if $(r_{u,max})_{ave}$ is 0.60 or less. The drain (well) resistance is represented by the dimensionless constant:

$$L_W = 3.24\,(K_S/K_W)(H/d_W)^2$$

where

K_S = permeability of the native soil
K_W = permeability of the well or drain
H = thickness of liquefiable layer (Figure 5-5)
d_W = diameter of the well or drain (Figure 5-5)

Onoue concluded that drain resistance is always significant for practical problems, and must be considered in design.

For the Alameda Tubes, the potential significance of drain resistance is illustrated by the following hypothetical example. Typical values of H and d_W are about 50 feet and 1.5 feet, respectively. If the drain is 200 times more permeable than the native soil, then L_W is about 18. Suppose also that:

$r_N = 2$
$T_d = 50$

where

$r_N = N_{eq}/N_l$
N_{eq}= equivalent number of earthquake cycles
N_l = number of loading cycles to cause liquefaction
T_d= time factor for radial flow

and it was required that the overall average of the maximum excess pore pressure ratios [i.e., $(r_{u,max})_{ave}$] be kept below 60%. Then the required spacing ratio

$r_S = d_W/d_e$

where

d_W = diameter of well or drain

d_e = diameter of the tributary volume for the drain

would be about r_S= 0.18 for L_W=0 (perfect drain) and about r_S= 0.56 for L_W=18. For 18 inch diameter drains, this corresponds to

$d_e \approx 8.3$ feet if drain resistance is ignored (i.e., L_W=0), versus

$d_e \approx 2.7$ feet if drain resistance is accounted for (i.e., L_W=18).

If the diagrams by Seed and Booker (1977) were used instead, the required drain spacing would be about 8.5 feet to keep $r_{u,max}$ less than 0.60, and about 8.7 feet to keep $(r_{u,ave})_{max}$ less than 0.60. These spacings are in good agreement with the 8.3 foot spacing obtained using Onoue's diagrams with L_W=0; the small difference may be due to the different pore pressure criteria and the resolution of the diagrams. Thus, the diagrams by Onoue (1988) and Seed and Booker (1977) are in agreement if drain resistance is ignored.

More importantly, Seed and Booker (1977) suggest that spacings obtained from their diagrams would be unaffected by drain resistance for K_W/K_S ratios of 200 or greater, and thus the spacing for this example would remain at 8.5-8.7 feet even if drain resistance were accounted for. The diagrams by Onoue (1988), however, suggest that the spacing would have to be reduced to 2.7 feet because of drain resistance. Thus, a K_W/K_S ratios of 200 or greater is not necessarily sufficient to eliminate the effects of drain resistance, and hence diagrams like those of Onoue (1988) are needed to realistically include the effects of drain resistance.

If the above example is reconsidered with the drain being only 40 times more permeable than the native soil, such as might be obtained if the native soil and imported stone are mixed by the construction process, then L_W is about 88. Such large L_W values are not even shown on the design diagrams by Onoue (1988) since the effectiveness of drains is negligible under such conditions.

Drains are clearly more effective when the drain resistance is low. Increasing the drain's permeability will reduce the drain resistance ($L_W \propto 1/K_W$), but there are practical limits to what can be achieved in this way. The thickness of the liquefiable layer has a much greater influence on the drain resistance ($L_W \propto H^2$), and thus drains are more likely to be effective at sites with relatively thin treatment zones.

Iai and Koizumi (1986) also developed design diagrams for gravel drains (available in English in Iai and Matsunaga 1989). Iai and Koizumi (1986) compared their diagrams with those of Onoue (1988), and concluded that they were in good agreement when the earthquake duration is greater than about twice the duration required to cause liquefaction under undrained conditions (i.e., $r_N \geq 2$). This good agreement for $r_N \geq 2$ is because Iai and Koizumi's diagrams are based on the excess pore pressures reaching a steady state condition, an assumption that is only reasonable when $r_N \geq 2$. For r_N values less than about 2, Iai and Koizumi's diagrams are more conservative than Onoue's.

It is worth noting that the implications of Onoue's (1988) work, or that of several other researchers referenced by Onoue but available in Japanese only (e.g., Iai and Koizumi 1986), do not appear to have been widely referenced in the United States. For example, Onoue (1988) was not referenced while Seed and Booker (1977) were by the following authors: Kramer (1996), Sonu et al. (1993), Baez and Martin (1992, 1993, 1995), Egan et al. (1992), Barksdale and Takefumi (1991). This situation may be partly because gravel drains have not been widely used in the United States while they have been in Japan. For example, a survey in Japan showed that drainage methods were used in 33% of a reported 305 construction projects involving liquefaction remediation from 1985 to 1989 [Tanaka et al. 1991 (in Japanese); as reported by Fuji et al. 1992].

5.4. Effect of Vertical Variations in Drain Permeability

The drainage capacity of gravel drains or stone columns depends on their resistance to vertical flow. Zones of lower permeability along the length of a drainage column will increase its hydraulic resistance to vertical flow, and thus reduce its effectiveness as a drainage element. Lower permeability zones might potentially occur in vibro-replacement stone columns for several reasons, including:

(1) varying degrees of intermixing between the native soil and imported stone;
(2) intermixing of the imported stone with a layer of lower permeability, finer-grained soil within the treatment zone, thereby producing a lower permeability zone in the as-constructed column;
(3) a localized cave-in of the native soil during the vibro-replacement process, resulting in a zone with a higher percentage of native soil and thus lower permeability; and
(4) a higher concentration of fines near the top of the column, such as was qualitatively observed at Mormon Island Auxiliary Dam due to accumulation of sediments from the return water.

The following approach is suggested for approximately evaluating the effect of a lower permeability zone on the drain resistance of an as-constructed drainage column. A drainage column having horizontal layers of different permeabilities might reasonably be represented by an equivalent uniform column of permeability K_W that produces the same resistance to vertical flow over the length of the column. This equivalent system is easily derived by considering a steady seepage condition along the length of the column, and then simply using Darcy's law and continuity of flow to calculate the appropriate value for K_W. This is illustrated in Figure 5-7 where a column having two different permeabilities, K_{W1} and K_{W2}, over lengths of L_1 and L_2, is replaced by an equivalent column having:

$$K_W = (K_{W1}K_{W2}L) / (K_{W1}L_2+K_{W2}L_1)$$

where $L=(L_1+L_2)$. This equivalent value of permeability can now be used to calculate the well resistance factor (L_W) for use with Onoue's (1988) design charts.

An example of the influence of a lower permeability zone, 1 m thick, within a 10 m long drainage column is illustrated in Figure 5-7. For the example set of parameters listed in this Figure, solutions are shown for three cases. First, if there was no low permeability zone (i.e., $K_{W2}=K_{W1}=30$ cm/s, a reasonable value for a clean stone column), the required drain spacing would be 3.2 m. Second, if the low permeability zone had $K_{W2}=1$ cm/s, which is similar to values reported at Redondo Beach and MIA Dam for intermixed native sand and imported stone, then the required drain spacing is reduced to 2.7 m. Third, if the low permeability zone had K_{W2}=0.1 cm/s, only 10 times that of the native soil, then the required drain spacing is reduced to only 1.4 m. This example illustrates the influence that low permeability zones or construction defects can have on the effectiveness of drainage columns.

The simple approach described above provides only an approximate evaluation of how a lower permeability zone may affect a drainage column's effectiveness. For example, the effect of a lower permeability zone is clearly different if it is located near the top of the column than if it is located at the base of the column. Nonetheless, the above approximation is sufficient to demonstrate that lower permeability zones or other construction defects can greatly reduce the effectiveness of drainage columns. Therefore, it is important that quality control procedures for constructing drainage columns be designed to minimize the occurrence of such defects.

5.5. Geosynthetic Drains

Tabata et al. (1993) describe the development of hollow geosynthetic drains as an alternative to conventional gravel drains in Japan. Three different geosynthetic sections were described: drain pipe, grid drain, and spiral drain. Grid drains were shown with dimensions of about 16 cm width and 1-3 cm thickness. Spiral drains were shown with a 9 cm outer diameter. All three drain types consist of a hollow central core with an outer filter layer. Installation methods include pre-boring or press-in methods. The required spacing of geosynthetic drains is smaller than for conventional gravel drains, but installation is faster such that the overall productivity per unit area is reportedly equal to or better than for gravel drains. Design methods for these relatively new geosynthetic drains are not described by Tabata et al. (1993).

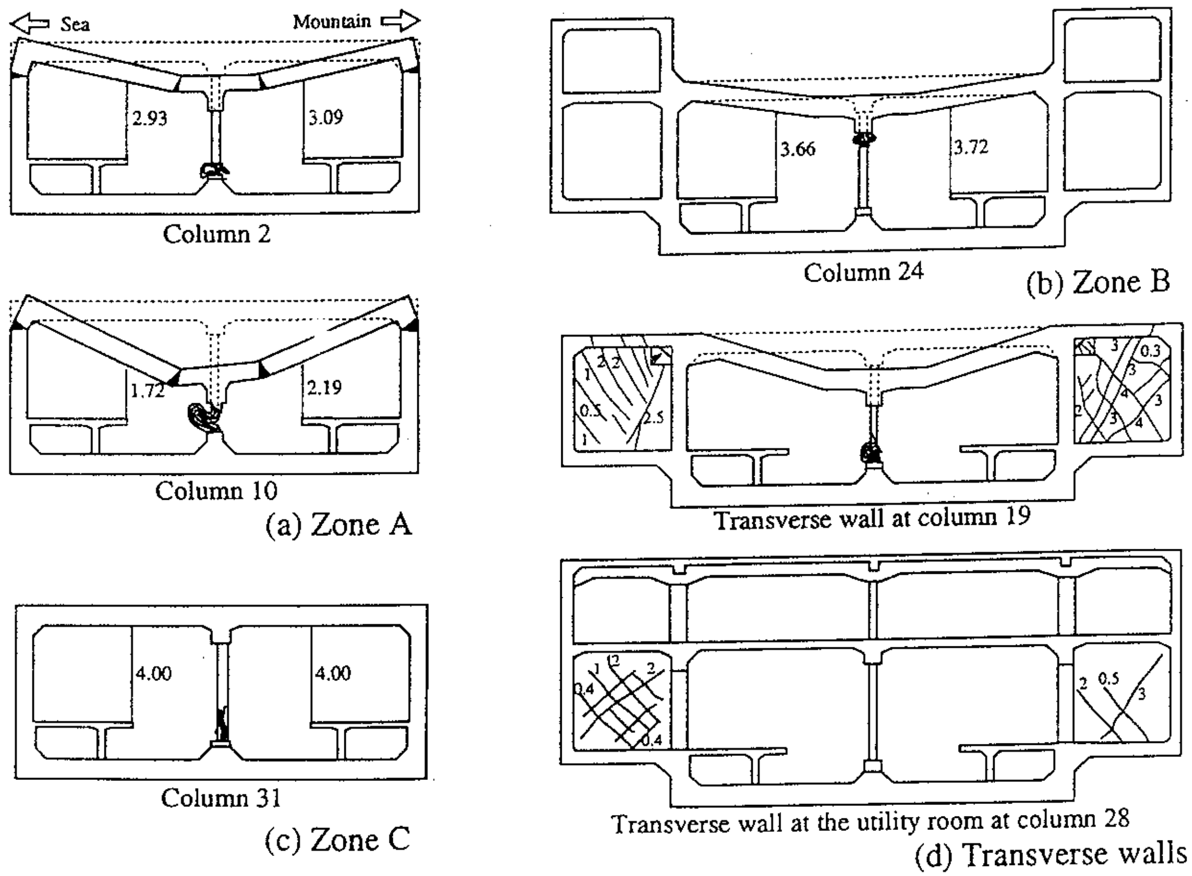
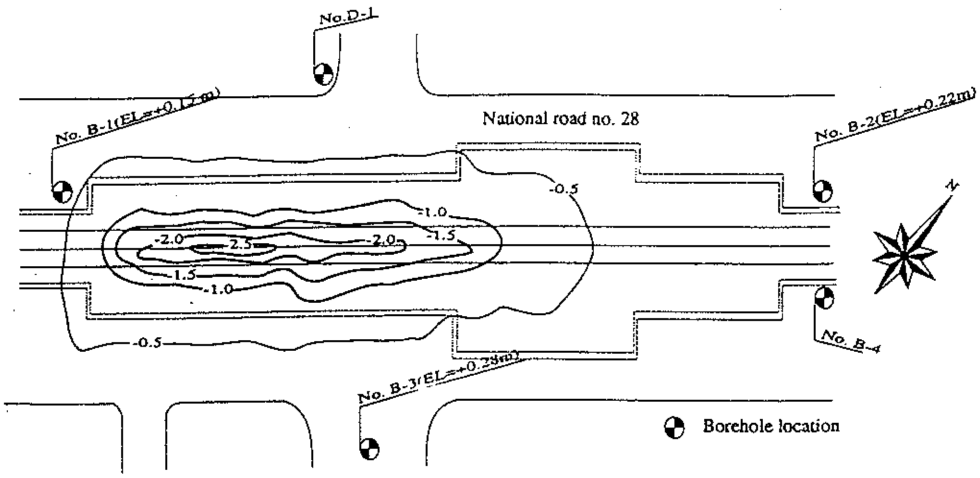
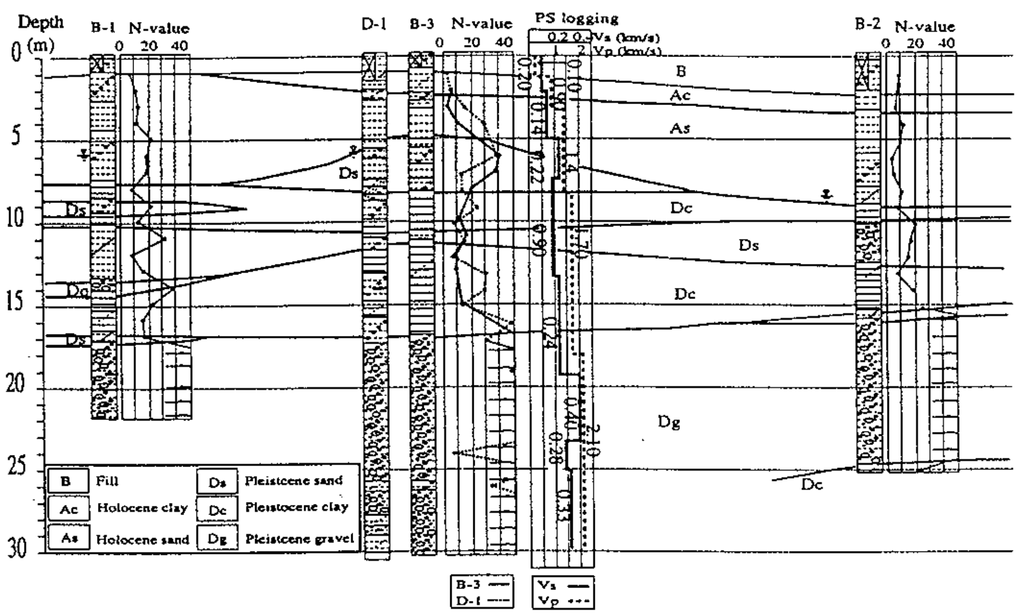


Fig. 5-1. Schematic of Damage Pattern in the Transverse Direction at the Daikai Subway Station (Nakamura et al. 1996)



(a) Plan and contours of the settlement of the ground surface



(b) Soil profiles

Fig. 5-2. Daikai Subway Station: (a) Settlement of Ground Surface; (b) Soil Profile (Nakamura et al. 1996)

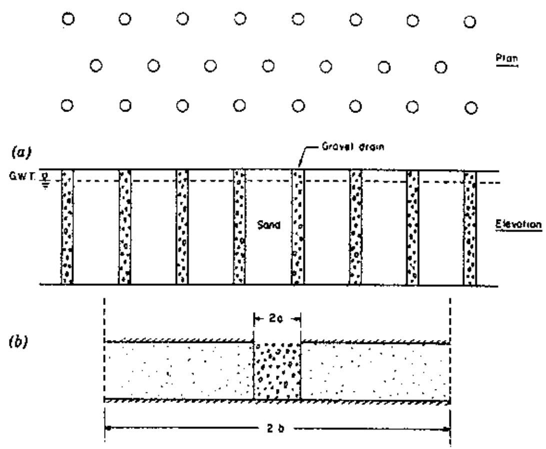


Fig. 5-3. Gravel Drains for Stabilizing Liquefiable Soils: (a) Arrangement of Gravel Drain System; (b) Gravel Drain With Radial Drainage Only (Seed and Booker 1977)

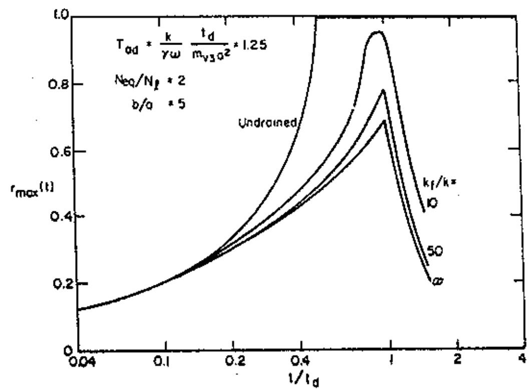


Fig. 5-4. Effect of Permeability of Drain Material on Rate of Pore Pressure Dissipation (Seed and Booker 1977)

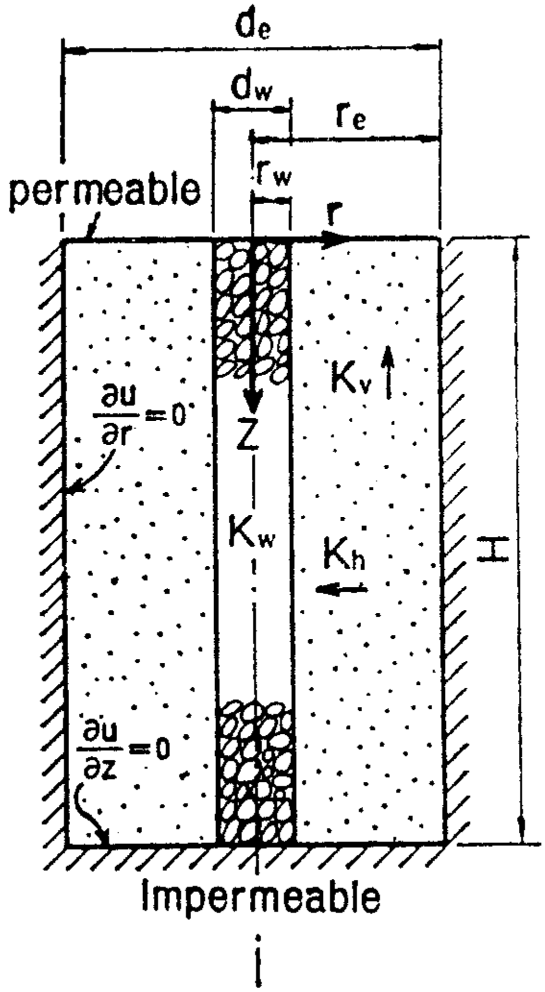


Fig. 5-5. Boundary Conditions for Analysis of Gravel Drain System (Onoue 1988)

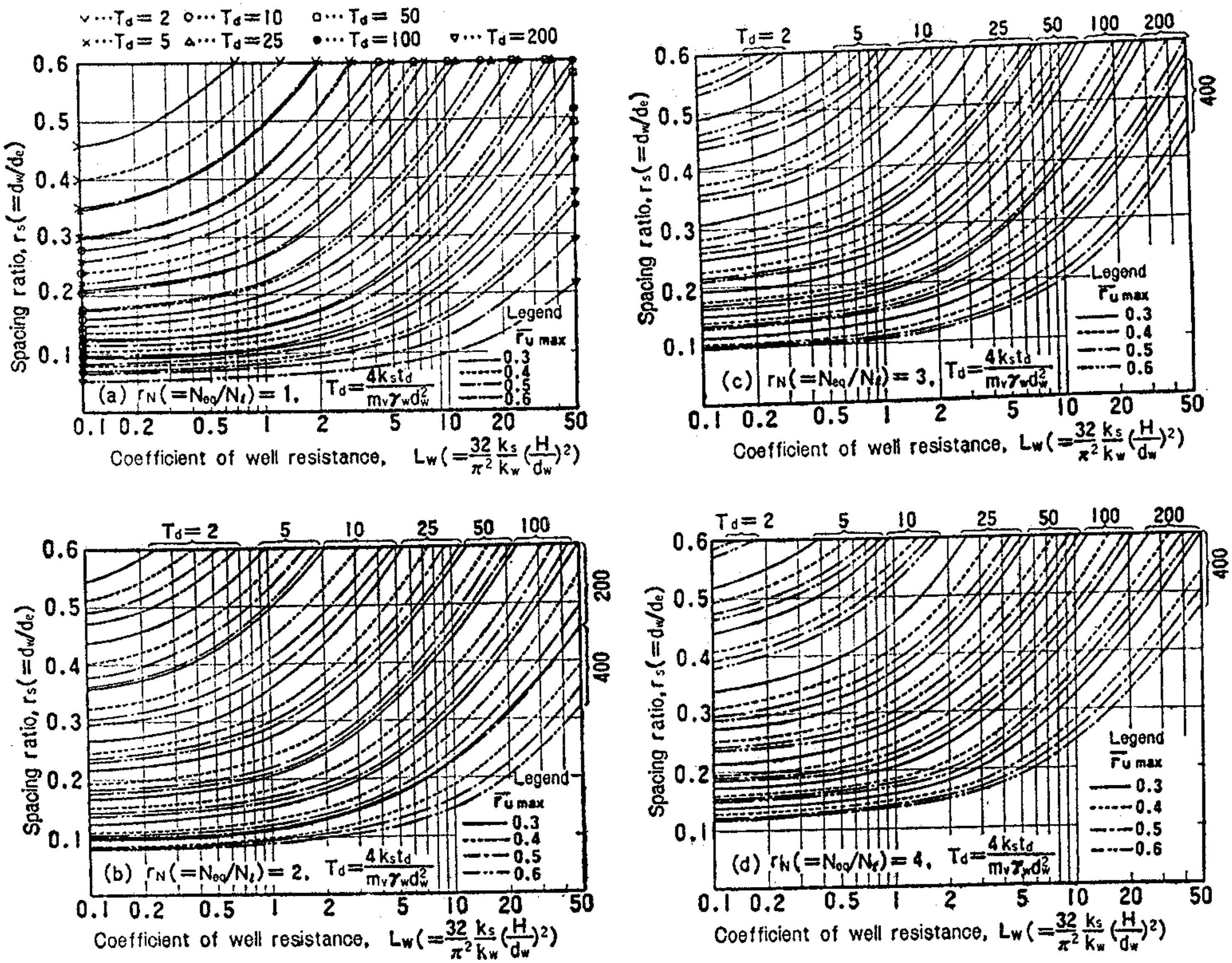


Fig. 5-6. Design Diagrams for Gravel Drain Systems Taking Well Resistance Into Account (Onoue 1988)

Example: Lower Permeability Zone in Drain

Use charts by Onoue (1988)
T_d=50, r_N= 2, $(r_{u,max})_{ave}$<0.6

K_S = 0.01 cm/s
K_{W1} = 30 cm/s, but K_{W2} = ?

For 1-D steady seepage:

$$K_W = \frac{K_{W1}K_{W2}L}{(K_{W1}L_2 + K_{W2}L_1)}$$

K_{W2} (cm/s)	Equivalent K_W (cm/s)	L_W	Required spacing (m)
30=K_{W1}	30	0.3	3.2
1.0	7.7	1.2	2.7
0.1	1.0	9.3	1.4

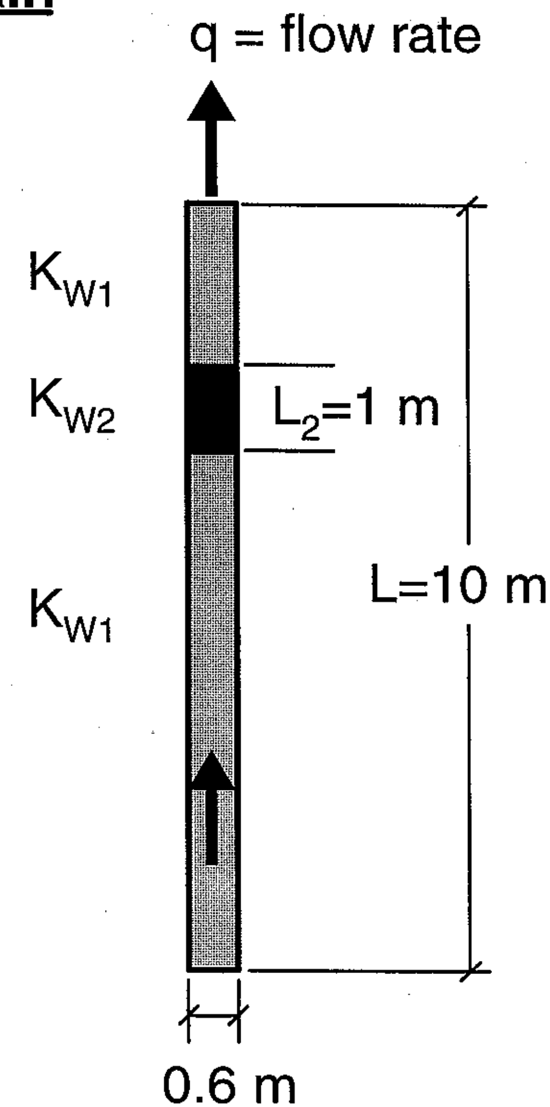


Fig. 5-7. Example of the Effect of Vertical Variations in Drain Permeability

6. CONCLUSIONS

6.1. Stone, Sand, or Gravel Drain Techniques

Earthquake Experience

- There are only two documented case histories involving the earthquake performance of liquefiable sites treated by sand or gravel drains for the primary purpose of dissipating excess pore pressures during shaking. The Jensen Filtration Plant is still under study, but it appears that the questionable performance of the drains at this site may be attributable to the relatively low permeability of the liquefiable silty sands and sandy silts. The good performance of gravel drains at the Port of Kushiro is encouraging, but it is noted that the confining effects of the steel cellular cofferdams may also have contributed to the good behavior.

Design Principles

- The drainage effect of pervious columns (e.g., sand or gravel drains, stone columns) during earthquake shaking is strongly affected by the hydraulic resistance of the column (i.e., drain resistance). Design diagrams by Onoue (1988) incorporate the effects of drain resistance, while the analyses of Seed and Booker (1977) greatly underestimate the range of drain permeabilities for which drain resistance is important.

- The drain resistance of a pervious column will be strongly influenced by any lower permeability portions of the column. Thus, the variation of permeability over the length of a pervious column, including any resistance to seepage at the top of the columns, must be taken into consideration during design and in establishing quality control procedures.

Constructibility

- In evaluating the various methods of constructing drains, it is important to consider the potential for construction defects that may increase the drain resistance. For example, vibro-replacement methods have been observed to mix the imported stone with the native soil, with the degree of mixing apparently increasing with increasing vibrational effort and water jetting. In some situations, this mixing may reduce the drain permeability enough to effectively impede any drainage during earthquake shaking.

6.2. Vibro-Techniques

Earthquake Experience

- Vibro-techniques (vibroflotation, vibro-replacement stone columns, vibrating probes) and sand compaction piles have been effective in mitigating the potential for liquefaction during earthquakes.

- Deformations have been observed near the edges of treatment zones, with the suspected causes being the migration of excess pore pressures, reduction in lateral confinement, or loads imposed by lateral spreading of the surrounding soils.

- Vibroflotation treatment in strips beneath columns and footing beams at the Paper Plant in Hachinohe (Ohsaki 1970) was not effective in protecting the supported structures from excessive deformations.

Design Principles

- SPT based design approaches, which do not account for reinforcing or drainage effects, appear to be reasonably consistent with earthquake experiences. The applicability of the $N_{1\text{-}60}$ versus cyclic strength correlation to soils treated by vibro-techniques is supported by the laboratory/field studies of Tokimatsu et al. (1990).

- The reinforcing effect of stone columns cannot be quantified on the basis of the available case history data. Baez and Martin (1993) present a theoretical method for calculating the reinforcing effect based on a "unit cell" analysis that may be applicable where the treatment area is large relative to the thickness of the treated zone.

- The drainage effect of vibro-replacement stone columns may be largely impeded by mixing of imported stone and native soils. The available data from sites where vibro-replacement stone columns were used for densification purposes suggest that the permeability of an as-constructed column is less, and potentially much less, than 100 times that of the native soil. Furthermore, relatively low permeability intervals may develop in a stone column where it crosses a relatively low-permeability soil layer. Using the design diagrams of Onoue (1988), the resulting drain resistance can be very large in many practical situations.

Constructibility

- The as-constructed gradation of stone column materials have been investigated at three sites (Kings Bay, Mormon Island Auxiliary Dam, Fraser Delta) where the purpose was densification of the native sands. All three investigations showed imported stone had been mixed with the native soil by the vibro-replacement construction process. Gradation tests for Kings Bay and the MIA Dam showed that the column materials consisted of about 20-23% native soil with 77-80% imported stone. At Kings Bay, the use of bottom-feed versus top-feed, or the use of well-graded versus poorly-graded stone, did not seem to affect the final mix of native soils and imported stone in the columns. Mixing of native soil and imported stone can reportedly be minimized if the vibrational effort is minimized as may be the case if the purpose is drainage rather than densification (E. Naesgaard, personal communication).

- Stone columns should be able to penetrate the edges of the magnetite (iron ore) cover over the Alameda tubes. This expectation is based on recent experiences where stone columns have been constructed through soils containing cobble size particles and small rockfill size fills, although it should be noted that water jetting at the tip was generally needed.

- Highly stratified deposits with silt or clay interlayers can be difficult to densify. Furthermore, evaluating the achieved improvement is hampered by the influence of such soft layers on penetration test (SPT or CPT) results.

6.3. In-Ground Walls

Earthquake Experience

- Limited experiences in Kobe (Hamada and Wakamatsu 1996) suggest that in-ground walls can be effective in mitigating damage due to liquefaction during earthquakes. The good behavior observed in some of these case histories, however, may have included contributions from: (i) the liquefiable zones beneath the structures being thinner than in the free-field, and thus producing less settlement or movement, and (ii) the support of the pile foundations beneath the structures involved in most of these case histories.

Design Principles

- In-ground-walls are expected to improve performance of liquefiable soils by (i) reducing the earthquake-induced shear stresses carried by the confined soils, (ii) providing composite shear strength and stiffness to the treatment zone should the confined soils still liquefy, and (iii) providing a barrier to the migration of high excess pore water pressures from the surrounding liquefied soils.

- For the Alameda tubes, in-ground walls could also serve to reduce the tendency for flotation by providing additional ballast or by confining the liquefied soil beneath the tubes. The confinement (or isolation) of the liquefiable soils directly beneath the tubes can be expected to reduce the potential for flotation during an earthquake by impeding the movement of water or liquefied soil from surrounding zones towards the zone directly beneath the tubes. Any vertical movement of the tubes must be accommodated by soil or water moving into the zone beneath the tubes, or else a suction (or reduction in pore pressures) will develop within the confined zone that effectively inhibits any further tube movements. In addition, the potential for post-earthquake settlement of the tubes is minimized by the relatively thin zone of liquefiable soils directly beneath the tubes.

Constructibility

- Quality control procedures for the different methods of constructing in-ground walls (e.g., jet grouting, deep soil mixing) will be made more difficult during construction below water. Alternative quality control procedures, or modifications to current procedures, may need to be developed.

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APPENDIX A

Summary Tables and Figures for Case Histories of Earthquake Performance of Liquefiable Sites Treated by Vibro- or Drain-Techniques

Table 1. Summary of Case Histories

No. (1)	Site (2)	Location (3)	Method of treatment[b] (4)	Earthquake event (5)	Peak accel. (6)	Damage (7)
1	Nippon Oil Co.	Niigata	Vibroflotation	1964 Niigata	0.16 g	None; Minor
2	NTT building	Niigata	Vibroflotation	1964 Niigata	0.16 g	$S_{max}\approx0.5$ m
3	Paper plant: (i) Group I (ii) Group II	Hachinohe	Vibroflotation	1968 Tokachioki	0.225 g	 (i) None. (ii) $S_{max}\approx0.4$ m
4	Group of oil tanks	Ishinomaki Port	Sand compaction piles	1978 Miyagiken-oki	0.18 g[a]	None
5	Med/Dental clinic	Treasure Island, CA	Vibroreplacement stone columns	1989 Loma Prieta	0.16 g	None
6	Building 450	Treasure Island, CA	Sand compaction piles	1989 Loma Prieta	0.16 g	None
7	Facilities 487-489	Treasure Island, CA	Vibrocompaction (vibroflotation)	1989 Loma Prieta	0.16 g	Minor cracking in floor of bldg. 487.
8	Approach to Pier 1	Treasure Island, CA	Vibroreplacement stone columns	1989 Loma Prieta	0.16 g	None
9	Wharves (6 locations)	Port of Kushiro	Gravel drains	1993 Kushiro-Oki	0.47 g	None, ranging to $S_{max}\approx20\text{-}40$ mm
10	Jensen Filtration Plant	Northridge, CA	Sand drains	1994 Northridge	0.98 g	Cracks to 80 mm, offsets to 200 mm.
11	Warehouses (5 buildings)	Port Island, Kobe	Vibro-rod	1995 Hyogo-Ken Nanbu	0.34 g[a]	None, ranging to offsets of 100 mm.
12	Amusement park	Port Island, Kobe	Vibro-rod	1995 Hyogo-Ken Nanbu	0.34 g[a]	None; some cracks to 25 mm and ejecta along south side.
13	Small building	Port Island, Kobe	Vibro-rod	1995 Hyogo-Ken Nanbu	0.34 g[a]	$S_{diff}\approx150$ mm beside building.
14	Rubble mound breakwater	Nishinomiya area	Sand compaction piles	1995 Hyogo-Ken Nanbu		$S_{max}\approx1\text{-}2$ m.

[a] See the respective Table for this case history for more detail.

[b] Brief explanations of the different terms are given below.

Vibroflotation (or vibrocompaction): a probe that vibrates laterally due to rotating eccentric weights.

Vibroreplacement stone columns: similar to vibroflotation but with the cavity being infilled with stone or gravel by either a top- or bottom-feed method to produce a coherent column of compacted stone.

Vibro-rod: a vibrating probe that is vibrated vertically, with the vibration applied from above the ground surface; the Vibro-rod, Terraprobe, Vibro-Wing, and Franki Y-probe are variants of the vibrating probe approach.

Sand compaction piles: a closed-end pipe pile is driven to the desired depth, and the resulting hole filled with sand during withdrawal of the casing; may include redriving the casing several times during withdrawal to improve densification (e.g., the vibrocompozer method, common in Japan, involves redriving the casing; redriving was not reported for Building 450 on Treasure Island).

Gravel drains: a casing auger is advanced to the desired depth, gravel is progressively poured in the casing and compacted by a tamper as the casing is slowly withdrawn.

Sand drains at the Jensen Filtration Plant: details not yet released.

Table 2. Nippon Oil Co. in Niigata[a]

Site & locations	Nippon Oil Co. Plant in Niigata, Japan.	
Earthquake	Niigata Earthquake of 1964, M_L=7.3	
Peak accel. at the site	0.16 g recorded in Niigata.	
Facilities	Tank, 44.6 m dia., 13.8 m high, 20 Ml capacity, 95% full during earthquake (with a contact pressure of 120 kpa).	Two tanks, 20 Ml and 30 Ml capacities (contact pressures not reported).
Treatment method	Vibroflotation; 1.5 m triang. spacing, to 5 m depth, 5 m beyond edges.	Vibroflotation; 1.5 m spacing to 5 m depth for 20 Ml tank, 1.4 m spacing to 6 m depth for 30 Ml tank; 5 m beyond edges for both tanks.
Treated areas	Tank settled uniformly 2-3 cm, damage was slight enough that no foundation repair was necessary.	No damage.
Untreated areas	Of 113 tanks, 40 settled more than 10 cm, 11 tilted, and 25 others had some trouble.	
Water table @ 0.4 m. 0-1.8 m: loose sand. 1.8-2.8 m: silty sand. 2.8-4 m: med. gr. sand. 4-7.5 m: coarse sand. >7.5 m not shown.	 N_{1-60}= 2-4 before, 10-17 after.[b] N_{1-60}= 2-6 before, 9-12 after. N_{1-60}= 6-16 before, 29-36 after. N_{1-60}= 9-25 before, 29-40 after. [N_{1-60}= 21-28 at 7-7.5 m before treatment.]	 same.
Insitu test data	Good summary plots of SPT data, but SPT procedures not described.	
Comments	Why did N values increase between 5-7 m deep when treatment was only to 5-6 m deep? The attached liquefaction analyses used either: (i) no surcharge from a tank, or (ii) a 120 kpa surcharge assumed to be constant over the depth of interest. The range of N_{1-60} values for each depth interval represents the middle-two-thirds of the data points for that interval.	
Lessons learned	Treatment was effective in reducing settlements to harmless levels (<2 to 3 cm). Small settlements consistent with a FS_{liq} close to 1.0 in pockets of upper 3 m.	

[a]Watanabe (1966), Yoshimi (1990).
[b]Assumes N_{60}=1.2N, where N are the reported SPT blow counts.

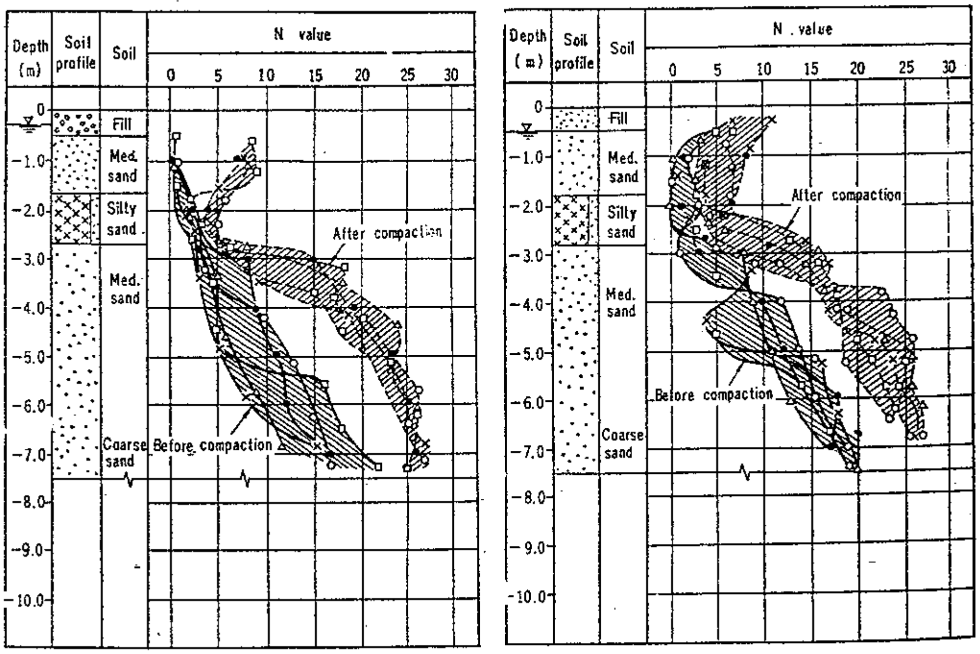

Fig. Oil Tanks at the Nippon Oil Co. in Niigata (Watanabe 1966)

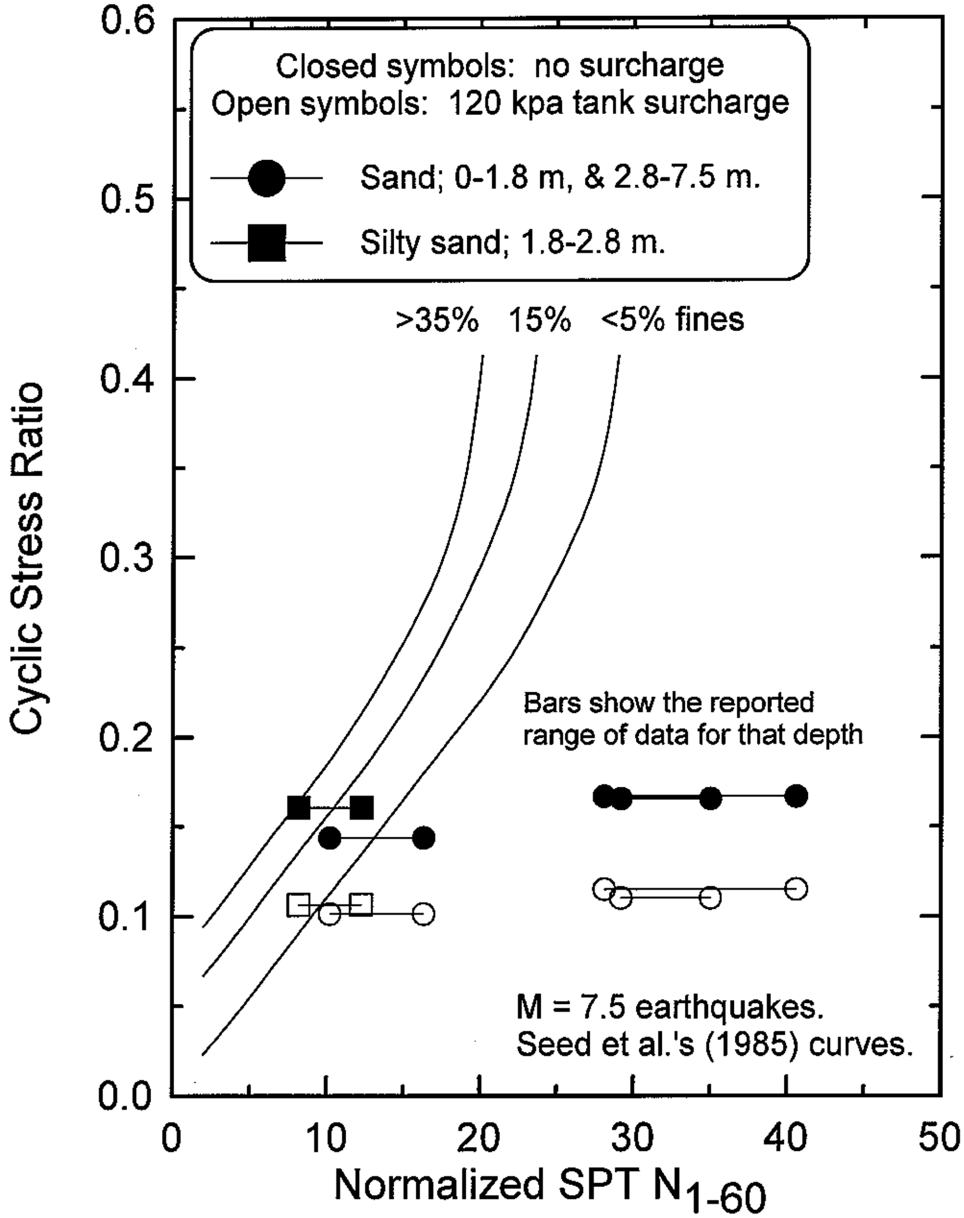

Fig. Liquefaction Analysis for Oil Tanks at the Nippon Oil Co. Plant in Niigata

Nippon64.GR1
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Table 3. Building in Niigata[a]

Site & locations	Nippon Telegraph & Telephone Public Corporation Building, Niigata, Japan.
Earthquake	Niigata Earthquake of 1964, M_L=7.3
Peak accel. at the site	0.16 g recorded in Niigata.
Facilities	Four-story reinforced-concrete building.
Treatment method	Vibroflotation in 1961; spacing unknown, to 7 m depth, 3-4 m beyond edges.
Treated areas	Max. settlement of 50 cm, tilt of 1°.
Untreated areas	Nearby buildings had greater settlements and tilts.
Water table @ 1 m. 0-2 m: loose sand & soft clayey silt. 2-7 m: loose sand. 7-12m: loose sand. 12-20 m: med. dense-dense sand.	 N_{60}= 0-1 before, 9-11 after.[b] $N_{1\text{-}60}$= 8-15 before, 19-46 after. $N_{1\text{-}60}$= 8-14 before. $N_{1\text{-}60}$= 18-32 before.
Insitu test data	Good summary plots of SPT data, but SPT procedures not described.
Comments	The attached liquefaction analyses used a 60 kpa contact pressure for the building, and assumed it spread at a 2:1 (V:H) distribution with depth.
Lessons learned	Large movements consistent with FS_{liq} < 1 and $N_{1\text{-}60}$= 8-14 for 7-12 m depth. Treating the upper 7 m did improve the behavior relative to untreated areas, but not enough to prevent heavy damage. Note: post-earthquake borings in the treatment zone around the building gave N-values lower than from construction, and these lower N-values would give FS_{liq} < 1 in upper 2-7 m.

[a]Watanabe (1966), Yoshimi (1990).
[b]Assumes N_{60}=1.2N, where N are the reported SPT blow counts.

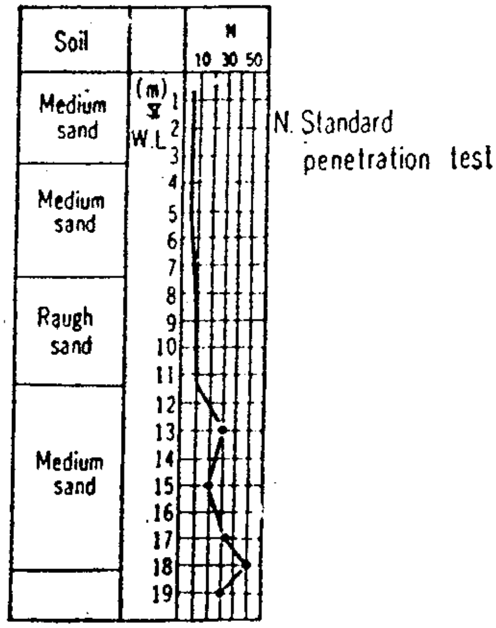

Table 3. Comparison of standard penetration tests before and after compaction (Niigata Telecommunications Division Building)

Depth	N-value before compaction	N-value after compaction	Remarks
1 m	0	7–8	
2 m	0–1	6–13	silty
3 m	7	13–24	
4 m	7	14–21	
5 m	5–10	15–22	
6 m	7	17–22	
7 m	7	14–23	

Fig. Four-Story Building in Niigata (Watanabe 1966)

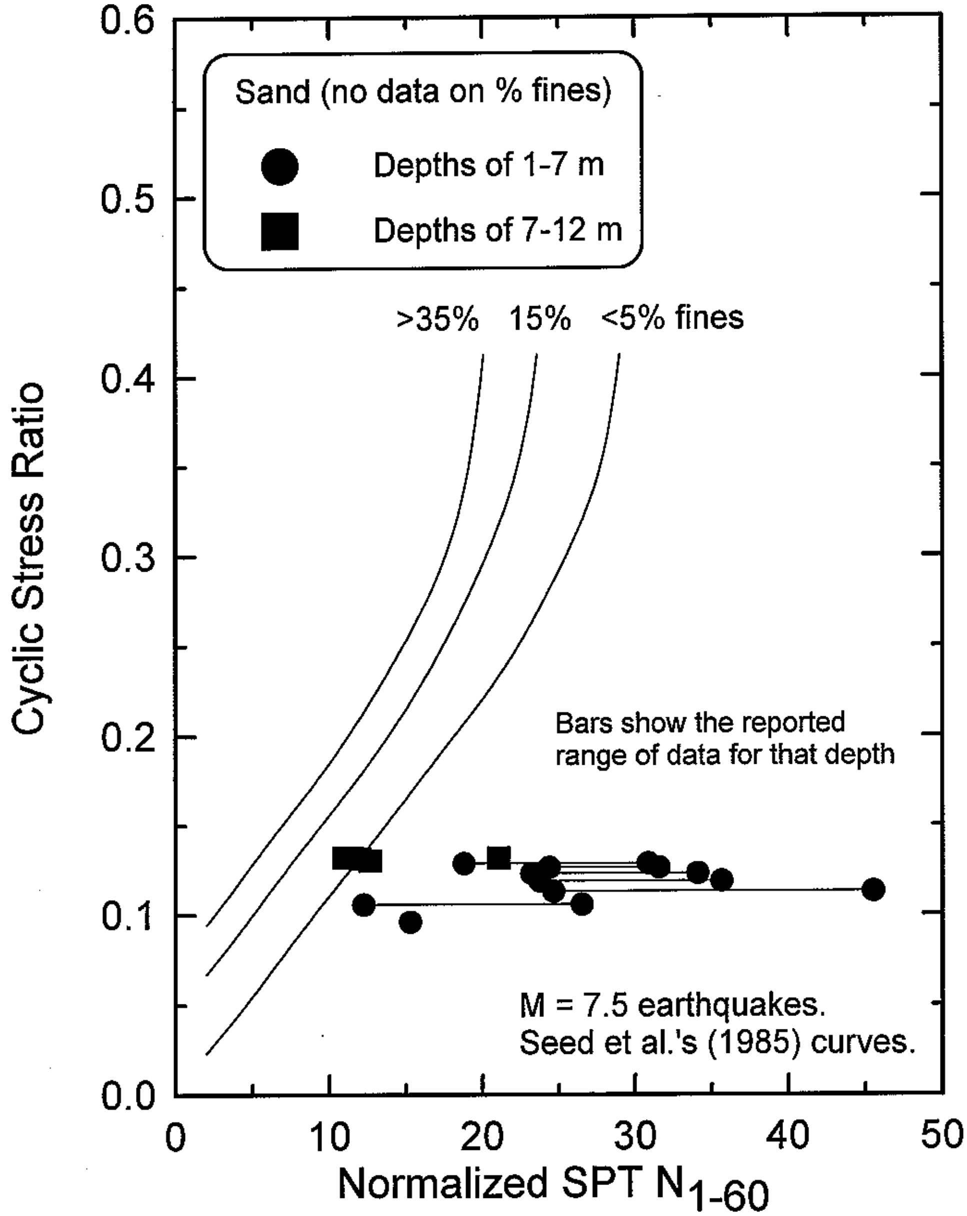

Fig. Liquefaction Analysis for the Four-Story NTT Building at Niigata: Including Weight of the Building in Calculating Cyclic Stress Ratios

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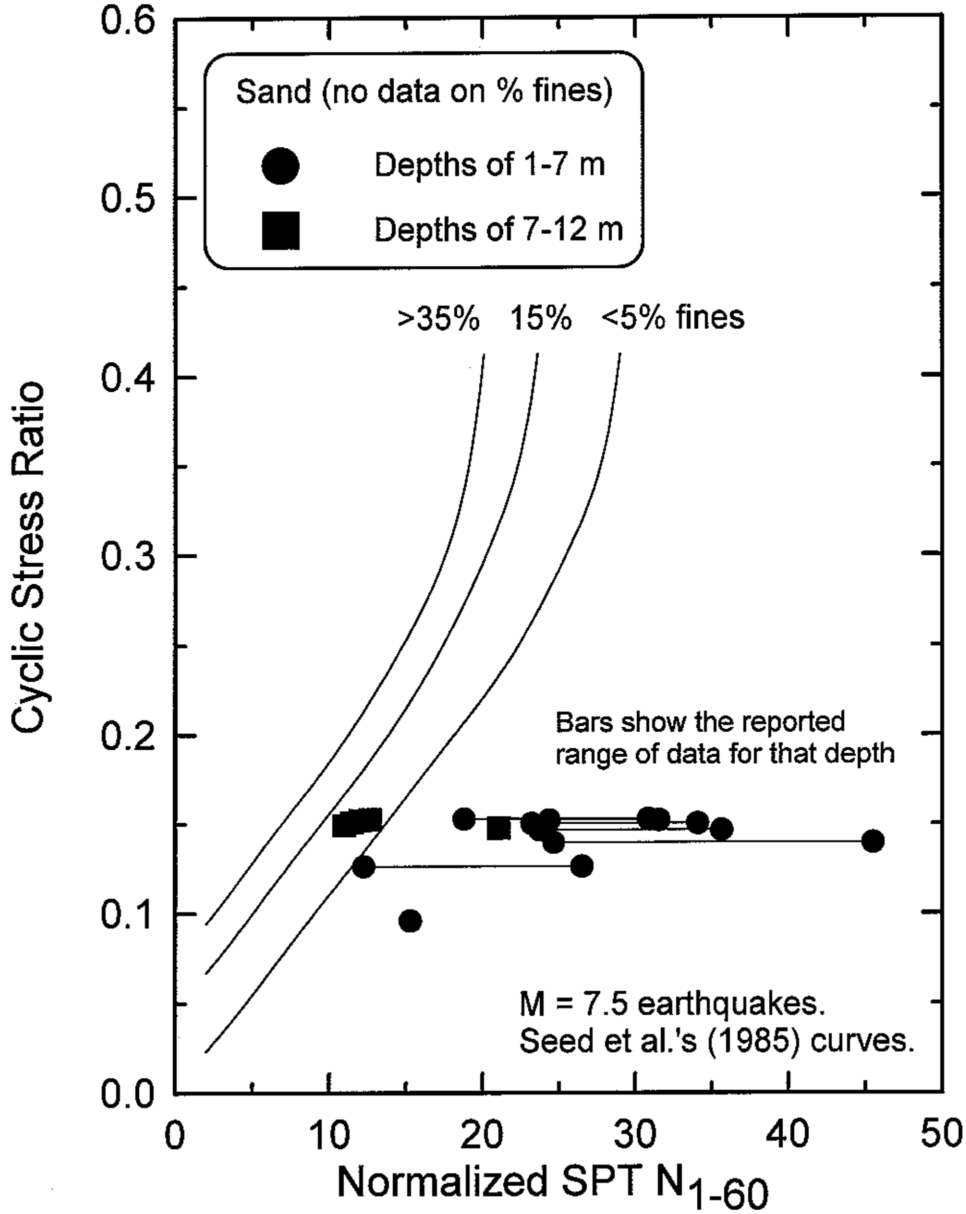

Fig. Liquefaction Analysis for the Four-Story NTT Building at Niigata: Neglecting Weight of the Building in Calculation Cyclic Stress Ratios

NTTBuild.GR1
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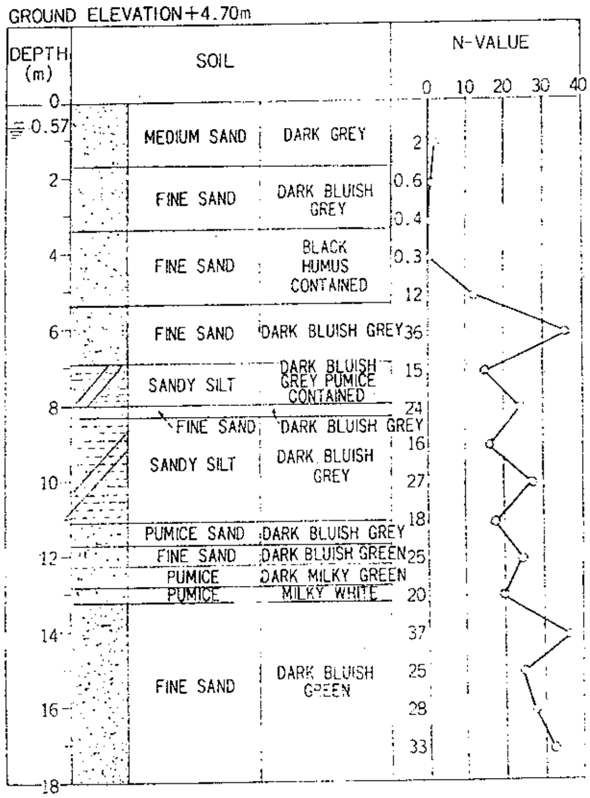
Table 4. Paper Plant in Hachinohe[a]

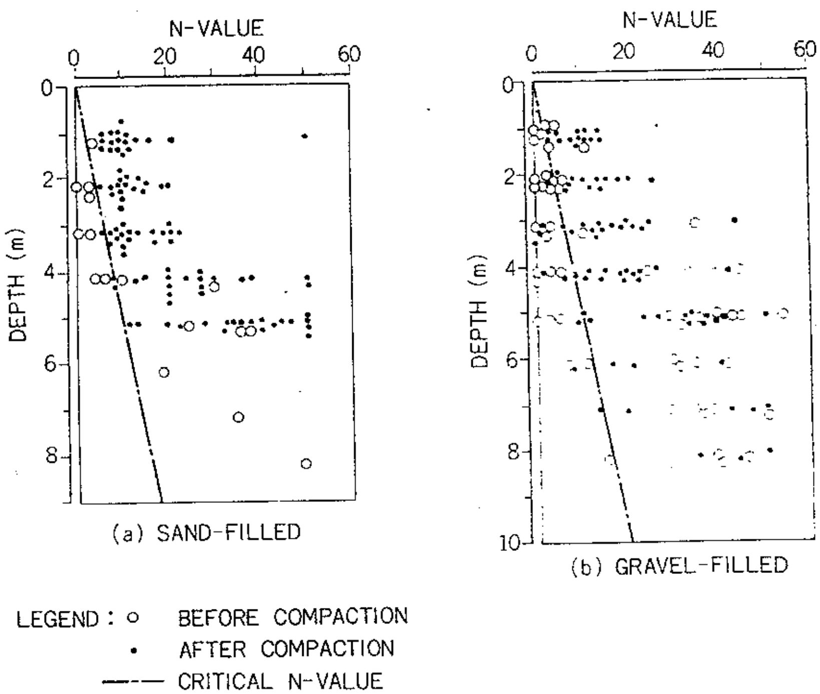
Site & locations	Paper manufacturing plant, Hachinohe, Japan.	
Earthquake	Tokachioki Earthquake of 1968, M_L=7.8	
Peak accel. at the site	0.225 g recorded at the harbor of Hachinohe.	
Facilities	Group I: main industrial buildings.	Group II: secondary buildings, tanks.
Treatment method	Vibroflotation in 1964-66; 1.55m triang. spacing; and buildings on piles.	Vibroflotation in 1964-66; in strips under columns and footing beams; no piles.
Water table @ 1 m. 0-3.7 m: loose backfill sand, 1-10% fines. 3.7-5 m: same as above. >5 m: sand & sandy silt.	 $N_{1\text{-}60}$ = 2-12 before, 15-37 after treatment.[b] $N_{1\text{-}60}$ = 2-17 before, 25-50 after treatment. $N_{1\text{-}60} > \approx 26$ before.	
Treated areas	No visible damage. Floor slabs had differential settlements of 10 mm average, 14 mm maximum.	Slight damage. Slabs settled up to 400 mm. A warehouse moved 400 mm laterally. Differential settlements of 15 mm average, 103 mm maximum.
Untreated areas	Heavy damage, tilting of tanks, etc. Up to 500 mm offsets near nonliquefied zones.	
Insitu test data	Good summary plots of SPT and CPT data, but SPT procedures not described.	
Comments	The attached liquefaction analyses include no surcharge from the buildings. The range of $N_{1\text{-}60}$ values for each depth interval represents the middle-two-thirds of the data points for that interval.	
Lessons learned	Treatment greatly improved behavior relative to untreated areas. Slab settlements <1.4 cm consistent with FS_{liq} >1 over full depth except for at a very small fraction of SPT sampling points. Absence of visible damage to buildings in Group I may be partly due to use of piles. Treatment in strips under columns and footing beams did not prevent large movements.	

[a]Ohsaki (1970).

[b]Assumes N_{60}=1.2N, where N are the reported SPT blow counts.



(a) Example Boring Log



(b) N-Values After Vibroflotation

Fig. Paper Plant in Hachinohe (Ohsaki 1970)

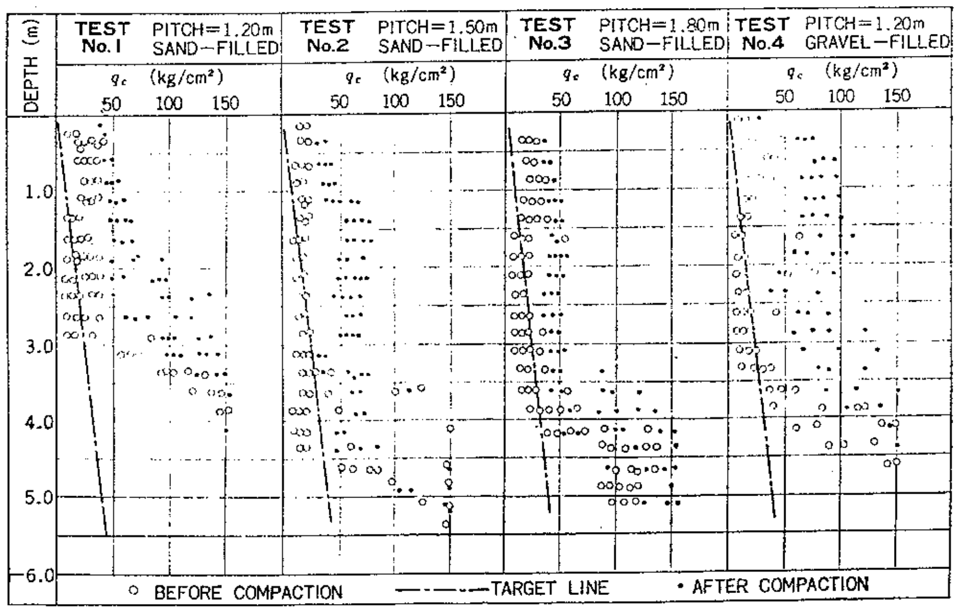


Fig. Results of Vibroflotation Tests (Ohsaki 1970)

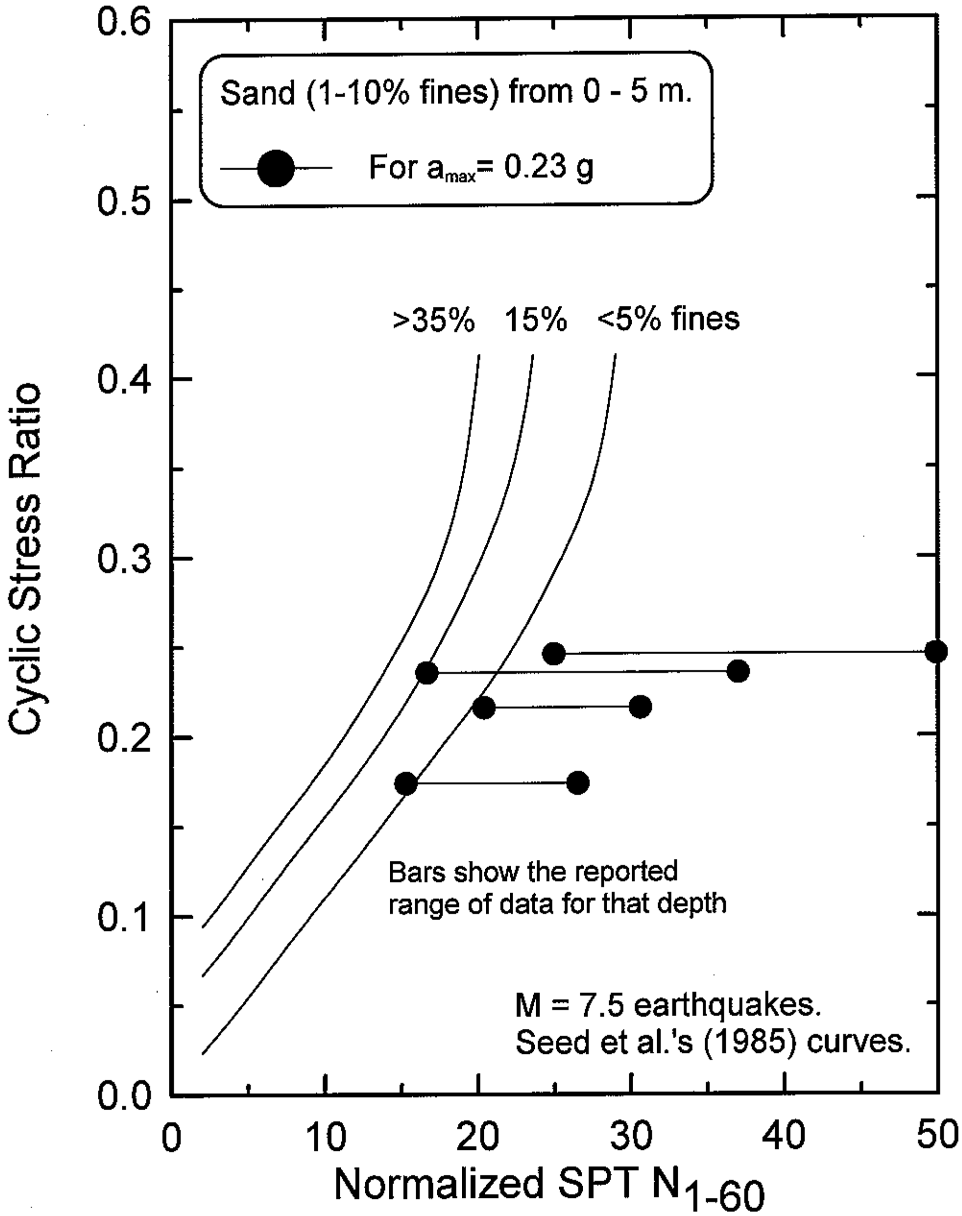


Fig. Liquefaction Analysis for a Paper Plant in Hachinohe

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Table 5. Group of Oil Tanks in Ishinomaki Port[a]

Site & locations	Group of 3 oil tanks in Ishinomaki Port, Japan.
Earthquake	Miyagiken-oki Earthquake of 1978, M=7.4
Peak accel. at the site	0.18 g, estimated by Ishihara et al. (1980). 0.29 g recorded on rock 5 km to the north.
Facilities	Group of 3 oil tanks; largest was 23 m diameter, 15 m high.
Treatment method	Sand compaction piles in 1975; 1.8 m triang. spacing (S), 0.7 m diameter (D), to 16 m depth, 2.8 m beyond tank edges. Note, the ideal replacement ratio was $\varepsilon_{av}=\pi D^2/4S^2 \text{Cos}30° = 14\%$.
Treated areas	No discernible settlement; tanks settled 10 mm/year on average for other reasons.
Untreated areas	Surface cracking and boils.
Water table @ 0.5-1.6 m. 0-16m: loose sand, with silt lenses in the lower half.	In sands: $N_{1\text{-}60}$= 5-15 before, 14-32 after. [b] [In silty layers: $N_{1\text{-}60}$= 2-7 before, 3-16 after, but very limited data.]
Insitu test data	Two SPT profiles each for before and after treatment
Comments	The attached liquefaction analyses used an average contact pressure of 60 kpa, and assumed it was uniform over the depth of interest.
Lessons learned	Treatment was able to prevent deformations while the untreated areas showed surface cracking and boils. Good performance is consistent with $FS_{liq} > 1$ over full depth if a_{max}=0.18 g; if a_{max}=0.29 g, then would expect liquefaction over much of the treated interval.

[a]Ishihara et al. (1980).

[b]Assumes N_{60}=1.2N, where N are the reported SPT blow counts.

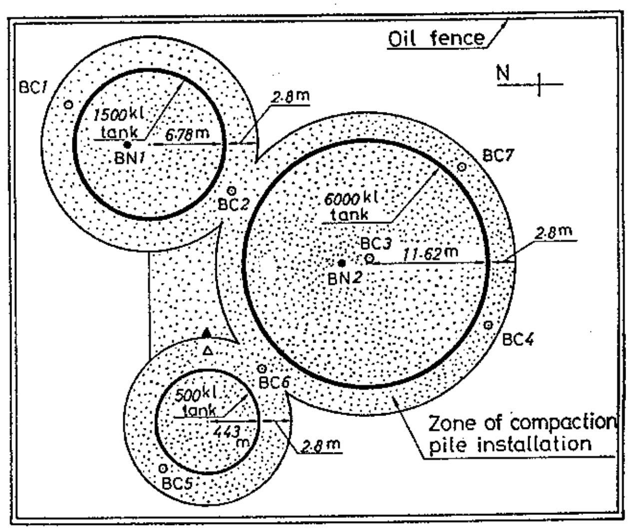

Fig. Zone of compaction pile installation and locations of drilling and sampling

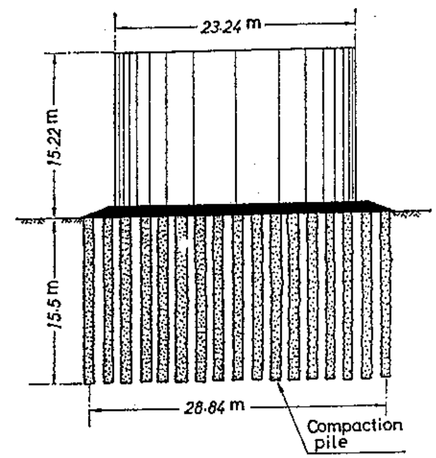

Fig. Extent of compaction pile installation

(Ishihara et al. 1980)

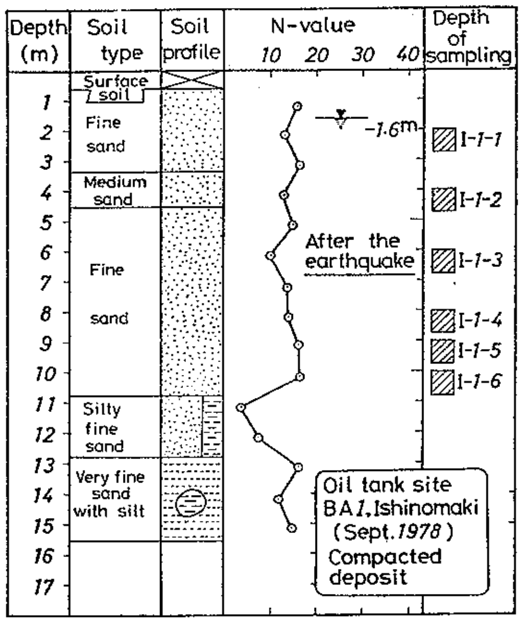

Fig. Standard penetration resistance and depths of Osterberg sampling at the compacted site after the earthquake

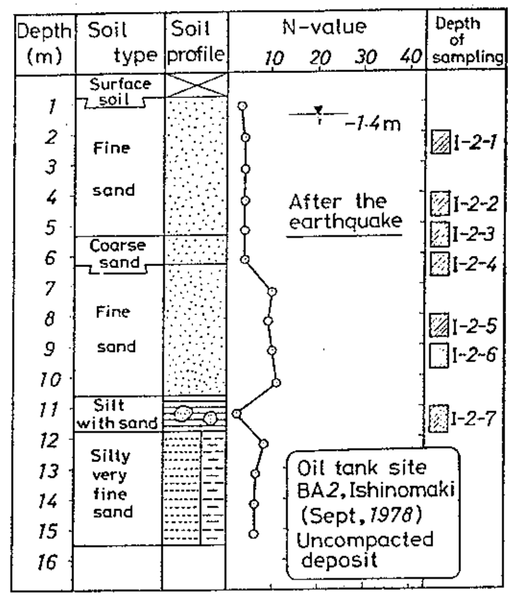

Fig. Standard penetration resistance and depths of Osterberg sampling at the uncompacted site after the earthquake

(Ishihara et al. 1980)

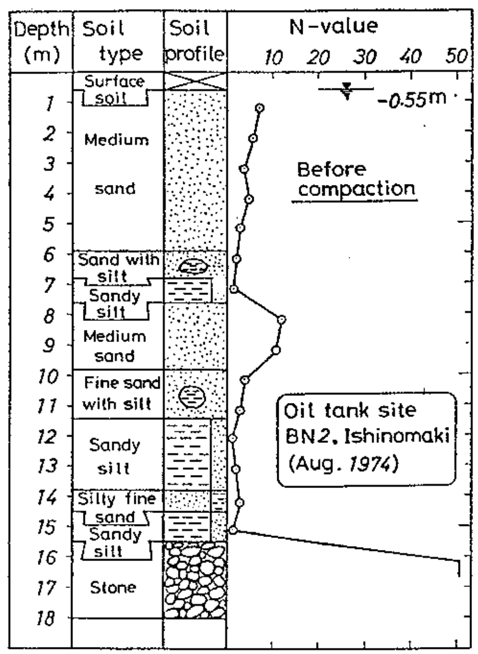

Fig. A soil profile and standard penetration resistance before compaction

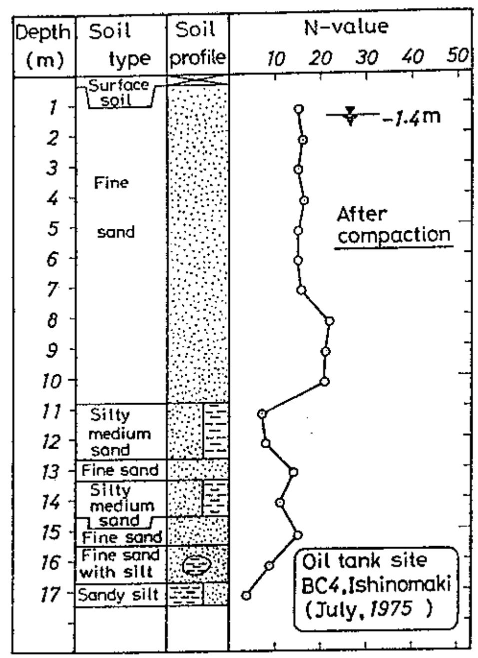

Fig. Standard penetration resistance after compaction

(Ishihara et al. 1980)

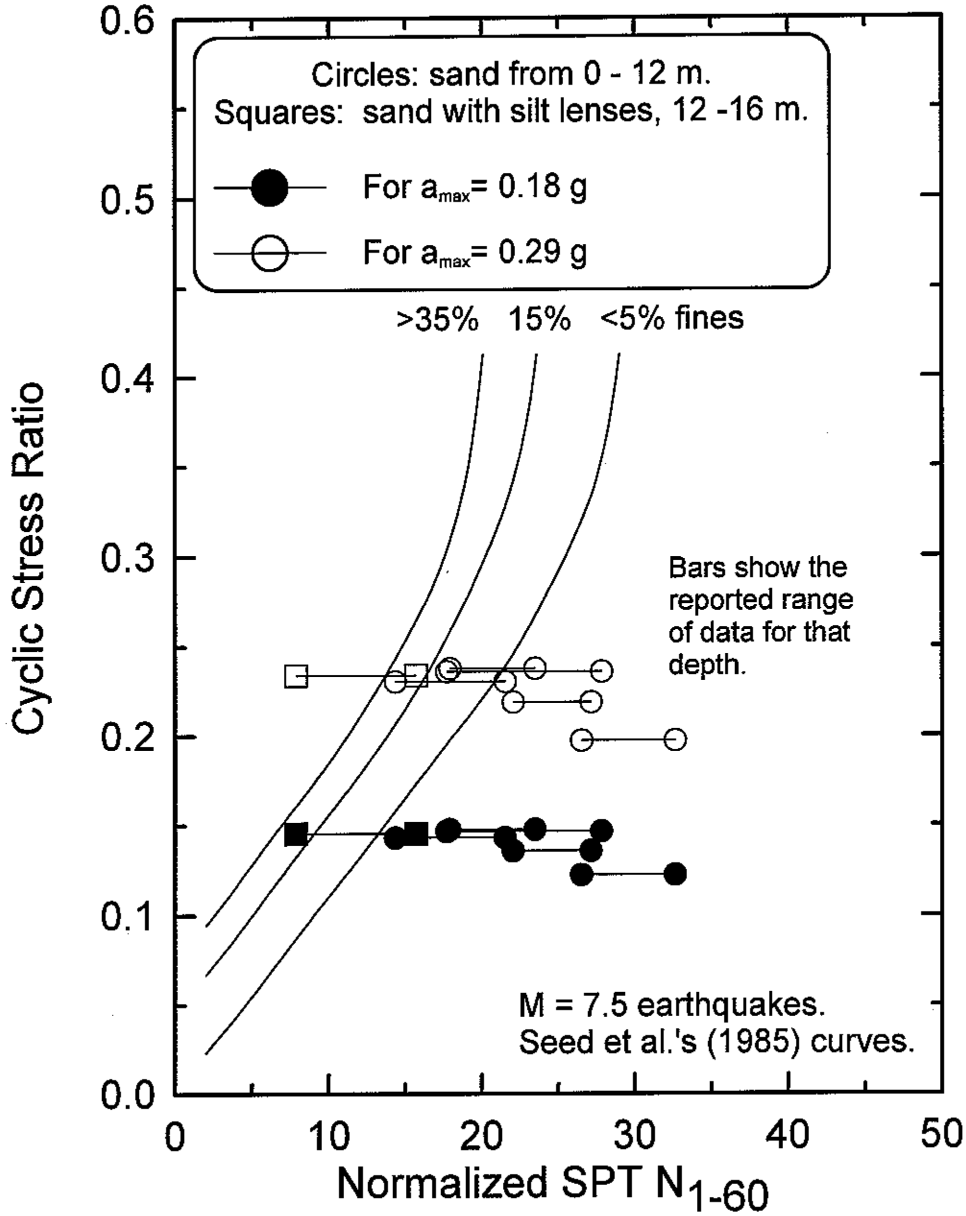

Fig. Liquefaction Analysis for a Group of Three Oil Tanks in the Ishinomaki Port

Ishino78.GR1
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Table 6. Medical/Dental Clinic at Treasure Island[a]

Site & locations	Medical/dental clinic at Treasure Island, California.
Earthquake	Loma Prieta Earthquake of 1989, M_o=7.0
Peak accel. at the site	0.16 g recorded on Treasure Island.
Facilities	Medical/dental buildings in early stages of construction at time of earthquake.
Treatment method	Stone columns in 1989; 3.05 m spacing, to 6.7 m depth, 6.1 m beyond building edges. Trials used 2.44, 2.75, and 3.05 m spacings to 12.2 m depth.
Treated areas	No cracks in footings. Bottom 2.4 m of two 6.7 m deep elevator shafts filled with sand ejecta. Max. differential settlements were 23 mm over 56 m. Max. settlement unknown.
Untreated areas	Surface cracking and boils.
Water table @ 2.1 m. Hydraulic sand fill (<10% fines) to 13 m with some thin clayey silt lenses. 0-3 m: dense sand. 3-6.7 m: loose to med-dense sand. 6.7-12.2 m: sand w/ clay lenses. >12.2 m: Bay Mud.	 n.a. n.a. N=2-5 before treatment, 3-19 after treatment. q_c unchanged by treatment.
Insitu test data	One SPT log given for pre-treatment profile.
Comments	
Lessons learned	Liquefaction over 6.7-12 m depth did not cause significant disruption at the ground surface in the treated area. This illustrates the value of a thick nonliquefiable crust at a level-ground, laterally-confined site.

[a]Mitchell and Wentz (1991).

Table 7. Office Building No. 450 at Treasure Island[a]

Site & locations	Office Building No. 450 at Treasure Island, California.
Earthquake	Loma Prieta Earthquake of 1989, M_o=7.0
Peak accel. at the site	0.16 g recorded on Treasure Island.
Facilities	Two 3-story buildings on shallow footings.
Treatment method	Sand compaction piles (no vibration) in 1967; 0.36 m diameter (D) sand piles, 1.22 m triang. spacing (S) under footings and 1.53 m spacing under slabs, to 9.2 m depth, and 3.1 m beyond building edges. Note, the ideal replacement ratio, $\varepsilon_{av}=\pi D^2/4S^2\text{Cos}30°$, was 3.4% @ S=1.83 m and 7.8% @ S=1.22 m.
Treated areas	No visible building damage. No measurements of settlement.
Untreated areas	Surface cracking, localized settlement, and sand boils.
Water table @ 1.8 m. Hydraulic sand fill (<15% fines) to 9.2 m, with some thin clayey silt lenses. 0-3 m: dense sand. 3-5 m: med-dense sand. 5-9.2 m: loose sand. 9.2-11.6 m: med.-dense sand. >11.6 m: Bay Mud.	Data only available for test sections: negligible improvement in N-values for 1.83 & 2.13 m spacings. No data for 1.53 m spacing (ideal ε_{av}=5.0%). N_{1-60}> 35 before, > 40 after treatment @ 1.22 m spacing.[b] N_{1-60}= 16-24 before, 20-34 after treatment @ 1.22 m spacing. N_{1-60}= 3-11 before, 6-12 after treatment @ 1.22 m spacing.
Insitu test data	Summary SPT plots from test sections with 0.91, 1.22, 1.83, and 2.13 m spacings, having 2, 1, 1, and 3 post-treatment borings, respectively. No data for 1.53 m spacing.
Comments	The attached liquefaction analyses include no surcharge from the building.
Lessons learned	Treatment improved the behavior of the site relative to the surrounding untreated areas. SPT data suggest a FS_{liq} < 1 over some of the 5-9.2 m treated interval. Good performance of the site may be partly attributable to the 5-m-thick nonliquefiable crust at this level-ground, laterally-confined site.

[a]Mitchell and Wentz (1991), and Basore and Boitano (1969).
[b]Drilling procedures not given. Used a 2" Mod. California sampler after a comparison to the standard SPT sampler showed no difference. Assumes N_{60}=N where N is the reported SPT blow count.

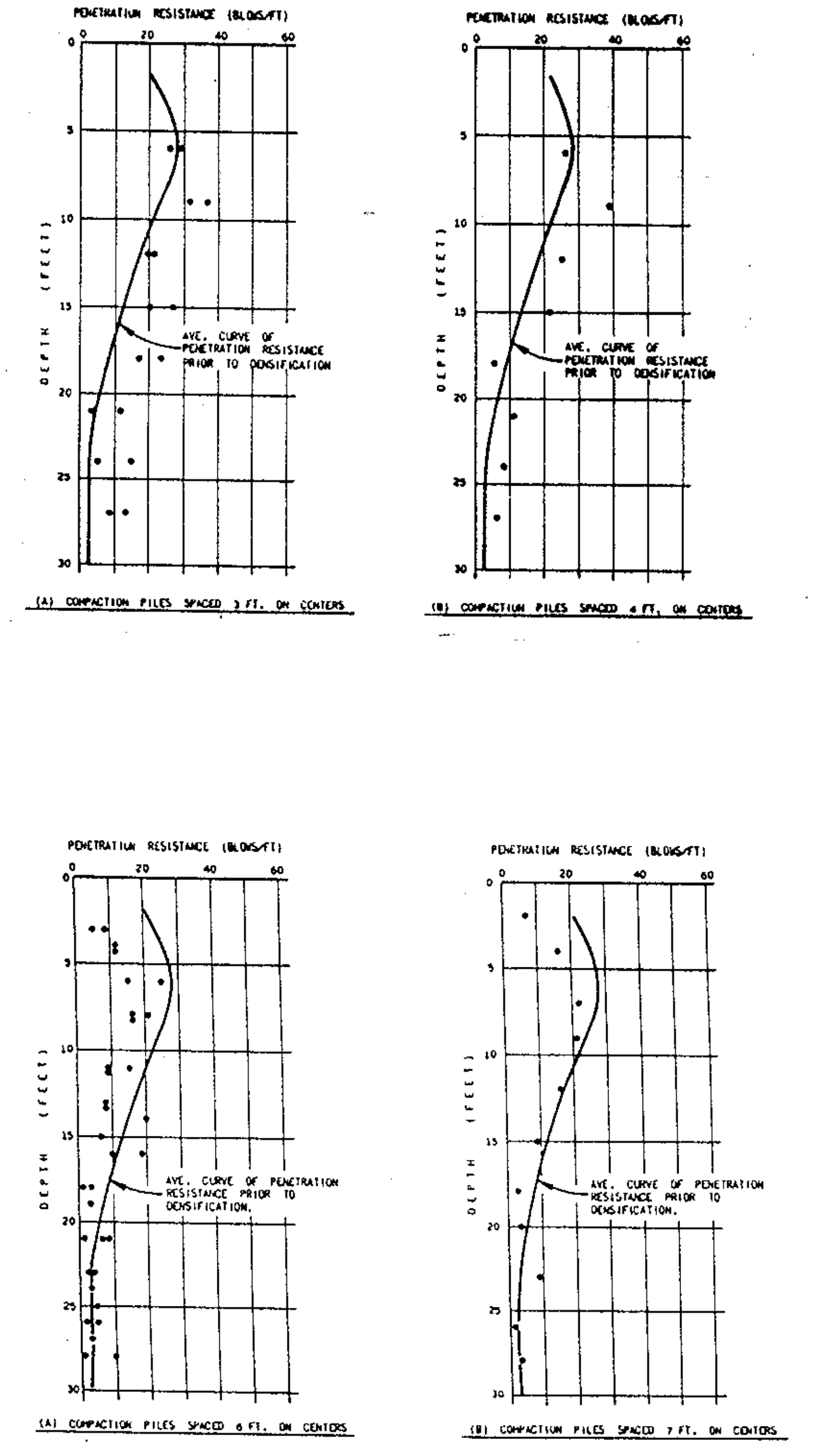

Fig. N-Values at Office Building 450 on Treasure Island (Basore and Boitana 1969)

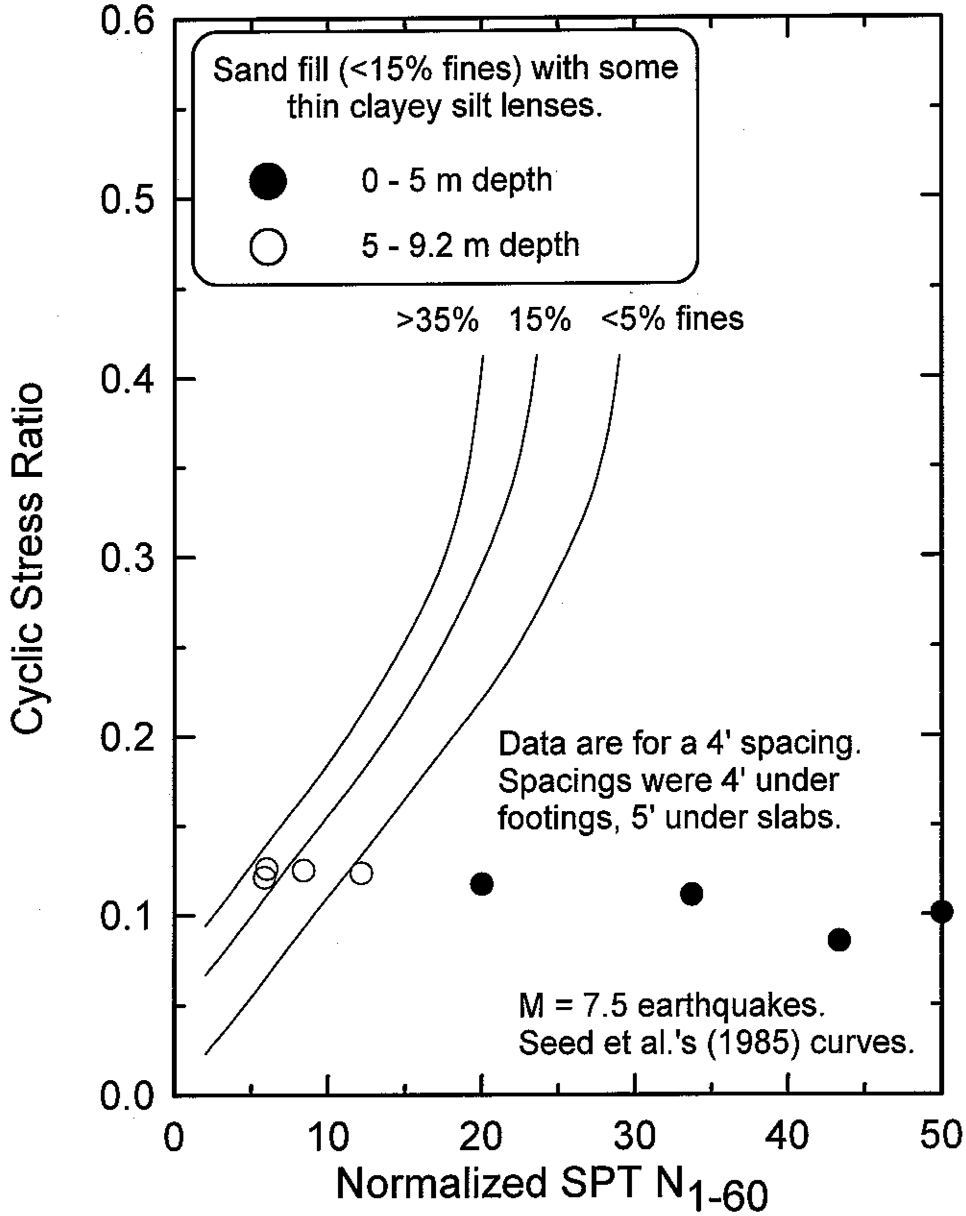

Fig. Liquefaction Analysis for Office Building 450 on Treasure Island in the 1989 Loma Prieta Earthquake

BLDG-450.GR1
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Table 8. Facilities 487-489 at Treasure Island[a]

Site & locations	Facilities 487-489 at Treasure Island, California.
Earthquake	Loma Prieta Earthquake of 1989, M_o=7.0
Peak accel. at the site	0.16 g recorded on Treasure Island.
Facilities	3-story buildings, all loads on bearing walls.
Treatment method	Vibrocompaction (vibroflotation) in 1972; 1.98 m triang. spacing, to 9.2 m depth, and 3.1 m beyond building edges.
Treated areas	Some minor cracking in floor of building 487 caused by differential settlement (magnitude not known). No damage to buildings 488, 489.
Untreated areas	Surface cracking, settlement, and boils.
Water table @ 1.7 m. Hydraulic sand fill to 7-10 m. Very loose to med.-dense, <12% fines typ., thin soft silt/clay lenses below 4.5 m.	No SPT data given.
Insitu test data	None published.
Comments	
Lessons learned	Good performance may be partly attributable to the 4.5 m thick nonliquefied crust at this level-ground, laterally-confined site.

[a]Mitchell and Wentz (1991).

Table 9. Approach Area of Pier 1 at Treasure Island[a]

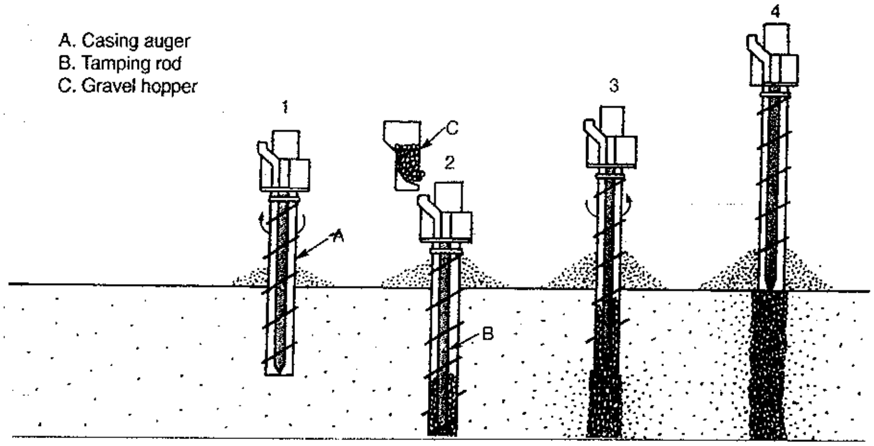
Site & locations	Approach area at Pier 1 at Treasure Island, California.
Earthquake	Loma Prieta Earthquake of 1989, M_o=7.0
Peak accel. at the site	0.16 g recorded on Treasure Island.
Facilities	About 30 m of shoreline at the pier.
Treatment method	Vibro-replacement stone columns in 1985; to 12.2 m depth; spacing, extent, and size unknown.
Treated areas	No visible movements.
Untreated areas	Boils and sinkholes.
Water table @ 0.8 m. Hydraulic sand fill to 13 m; <10% fines typ.; upper 6 m is loose to med.-dense with lenses of clay. Lower 2 m is silty sand and sandy silt.	No SPT data given. Minimum relative density specification was met, but it is not clear if this was based on a volume calculation or insitu testing. Lower 2 m did not meet specifications.
Insitu test data	None published.
Comments	
Lessons learned	Absence of visible movements does not preclude the possibility of small movements having occurred. Treatment reduce the deformations.

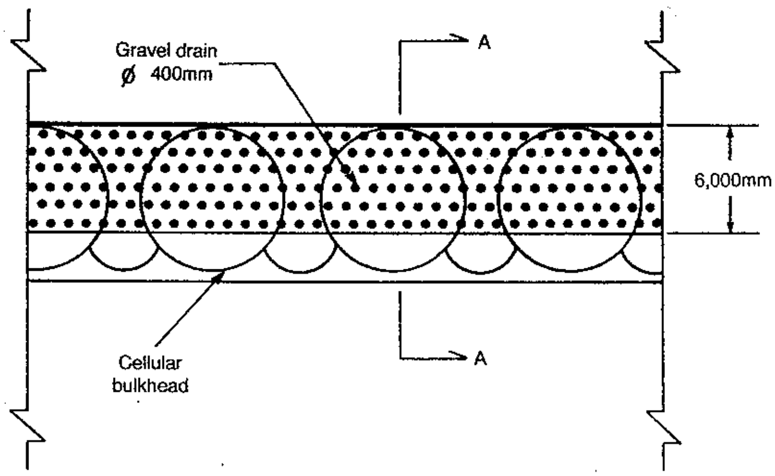
[a]Mitchell and Wentz (1991).

Table 10. Wharf at the Port of Kushiro[a]

Site & locations	Wharf at the Port of Kushiro, Japan.
Earthquake	Kushiro-Oki in 1993, M_{JMA}=7.8[b]
Peak accel. at the site	0.469 g recorded within the Port.
Facilities	Wharf constructed with cellular cofferdams. Typical drawing shows 9.2 m diameter, 9 m high, steel cellular bulkheads. Treatment at six locations, for a total shoreline length of 600 m.
Treatment method	Gravel drains in 1990; 1.2 m triang. spacing, 0.40 m diameter, to 6 m depth, over a 6 m wide zone mostly inside the cellular bulkheads.
Treated areas	Fisherman's wharf (120 m length) had no visible damage or movements. Another wharf had ground slumping of 20-40 mm (worst of all treated areas).
Untreated areas	Sand boils, and up to 0.6 m slump behind one caisson wall. Pavements outside treated areas at Fisherman's Wharf suffered severe damage.
Sand (dredged fill), about 6 m deep.	At the wharf with the greatest settlements, the "soil was somewhat loose, with N values less than 8."
Insitu test data	None published.
Comments	Need to obtain insitu test data, and address the role of the cellular cofferdams in restricting movements.
Lessons learned	Treatment appears to have been effective in preventing significant movements. Steel caissons may have helped, although they did not preclude settlement or movement at other locations.

[a]Sonu et al. (1993).
[b]Also experienced the Hokkaido-Toho-Oki (M_{JMA}=8.1) earthquake, with a peak acceleration of 0.07 g at the site.





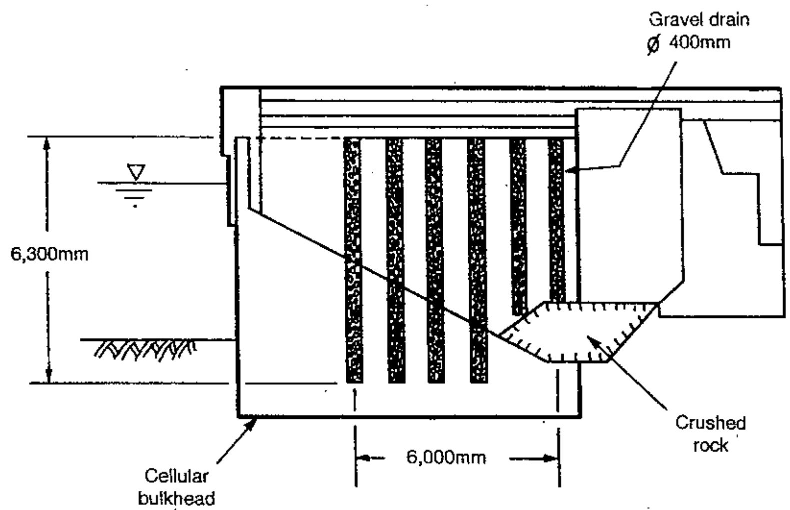
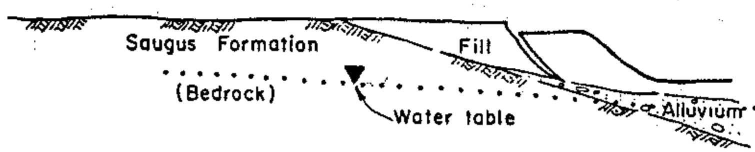


Fig. Gravel Drains at Port of Kushiro (Sonu et al. 1993)

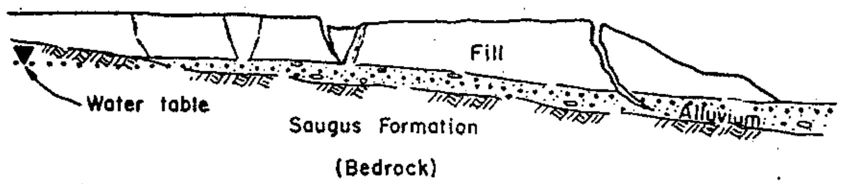
Table 11. Jensen Filtration Plant in Northridge[a]

Site & locations	Jensen Filtration Plant in Northridge, California.
Earthquake	Northridge Earthquake of 1994, M_o=6.7
Peak accel. at the site	0.98 g.
Facilities	Main control building.
Treatment method	Sand drains in 1986. Insitu buttress alongside existing building. 1.83 m triang. spacing, 0.46 m diameter, to 21-27 m depth. No compaction of the sand drains was specified.
Treated areas	No visible damage to building. Cracking to 8 cm with offsets of 20 cm within the treated areas adjacent to the building.
Untreated areas	
Typ. 15 m of compacted fill over a 1.5-7.6 m thick silty sand/sandy silt layer (typ. >50% fines). Water table near top of the liquefiable layer.	$N_{1\text{-}60}$=10-15 in liquefiable layer before treatment; no published data after treatment, but no improvement would be expected since the sand drains were not compacted.
Insitu test data	None published.
Comments	Woodward-Clyde still studying this site, and expect to release their report in the next few months.
Lessons learned	Sand drain buttress did not prevent movements in treated area. Movements were less than experienced in 1971 San Fernando Earthquake. Poor performance is attributable to the high percentage of fines in the target layer which likely prohibited the dissipation of excess pore pressures during the earthquake.

[a]Metropolitan Water District (personal communication), Dixon and Burke (1977), Youd (1971).



NORTHEAST SLIDE AREA



SOUTHEAST SLIDE AREA

SCHEMATIC CROSS-SECTION OF GROUND MOVEMENTS AT JENSEN FILTRATION PLANT IN 1971 (Dixon and Burke 1977)

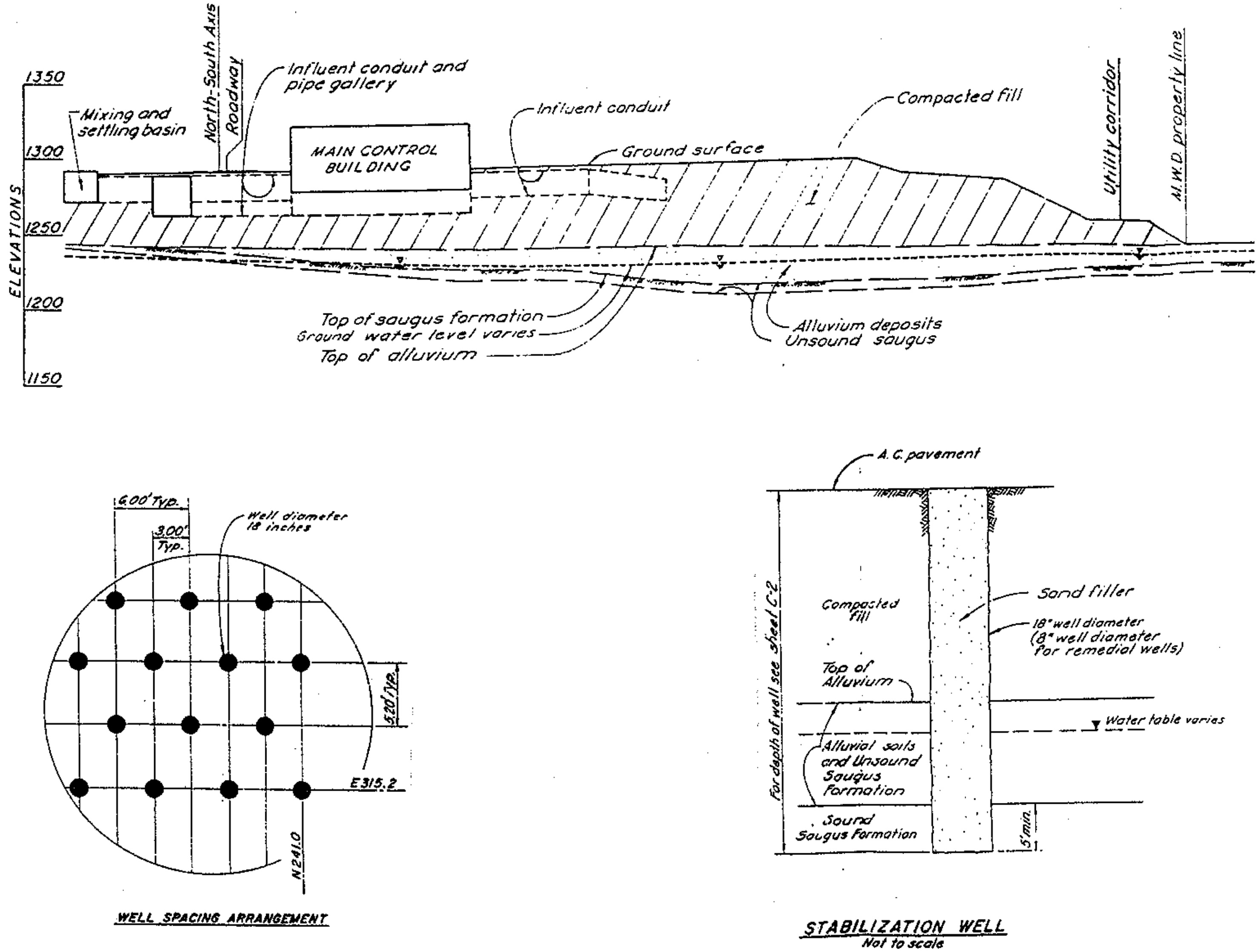
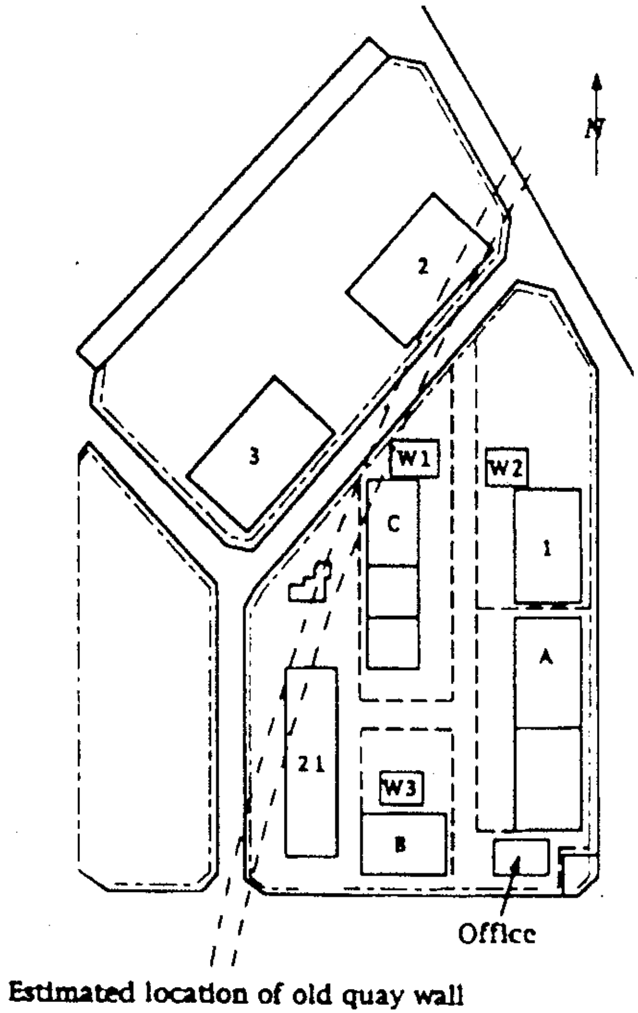


Fig. Sand Drains at the Jensen Filtration Plant (MWD 1986)

Table 12. Warehouse Facility on Port Island[a]

Site & locations	Warehouse facility on Port Island, Kobe, Japan.
Earthquake	Hyogo-ken Nanbu (Kobe) Earthquake of 1995, M_o=6.9
Peak accel. at the site	0.34 g measured at an untreated corner of the site that liquefied; treated areas may have been shaken harder.
Facilities	Five main warehouses on shallow foundations. One office building on piles. Several temporary warehouse structures.
Treatment method	Vibro-rod in 1977; 2.4 m triang. spacing, to 16 m depth, 2-5 m beyond edges. Perimeter strip of gravel compaction piles, 0.6 m diameter, 2.0 m spacing, 2 m wide strip.
Treated areas	No visible structural damage to any buildings; all were in use after earthquake. Warehouses A, B & 1: mostly no visible damage; some minor separations in slabs up to 10 mm. Warehouse C: cracks and separations up to 20-100 mm in slabs. Warehouse 21: slabs removed in north end; cracks and differential settlements of 50 mm in south end.
Untreated areas	Outside the facility: boils, cracks, and settlements up to 500 mm; warehouses on surrounding lots were badly damaged. In-between buildings: boils, and cracks to 100 mm.
Water table at 4 m depth.. 0-16 m: silty gravely sand (fill), 15-30% fines typ.	N_{1-60}= 9-14 before, 34-40 after treatment.[b]
Insitu test data	1 SPT log before and after treatment.
Comments	Improvement may have been hindered under Warehouses C & 21 due to an old sea-wall; this needs to be clarified. The attached liquefaction analyses include no surcharge from the buildings.
Lessons learned	Treatment greatly improved behavior, and kept movements small enough to only damage the slabs in two of the five warehouses. Good behavior is consistent with the one SPT log showing $FS_{liq} >> 1$ in the treated area. The slab damage may be due to the influence of the surrounding liquefied soils since the 2-5 m wide treatment zone beyond the building edges is only 0.1-0.3 times the thickness of the liquefiable fill.

[a]Yasuda et al. (1996), EERC (1995).
[b]Assumes N_{60}=1.2N, where N are the reported SPT blow counts.



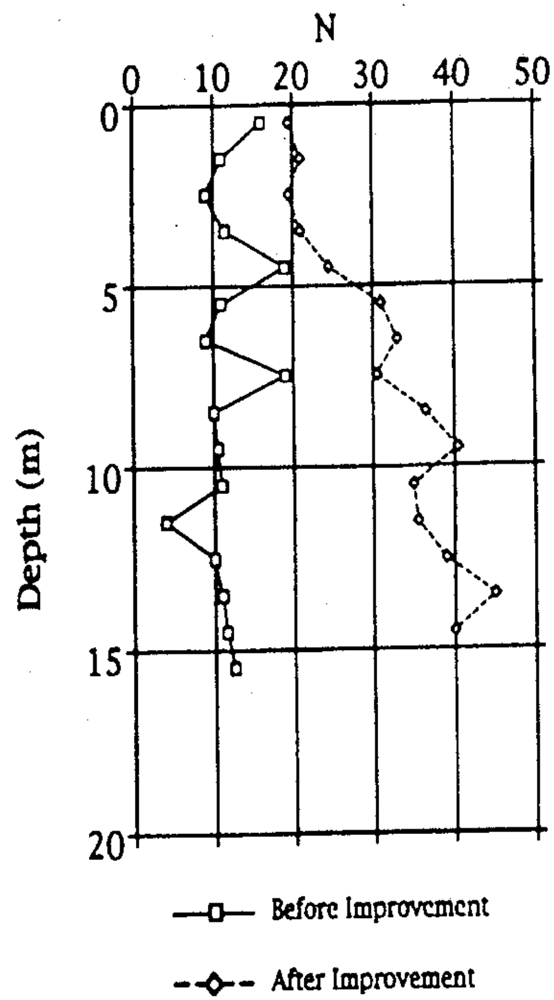


Fig. Warehouse Facility on Port Island (EERC 1995)

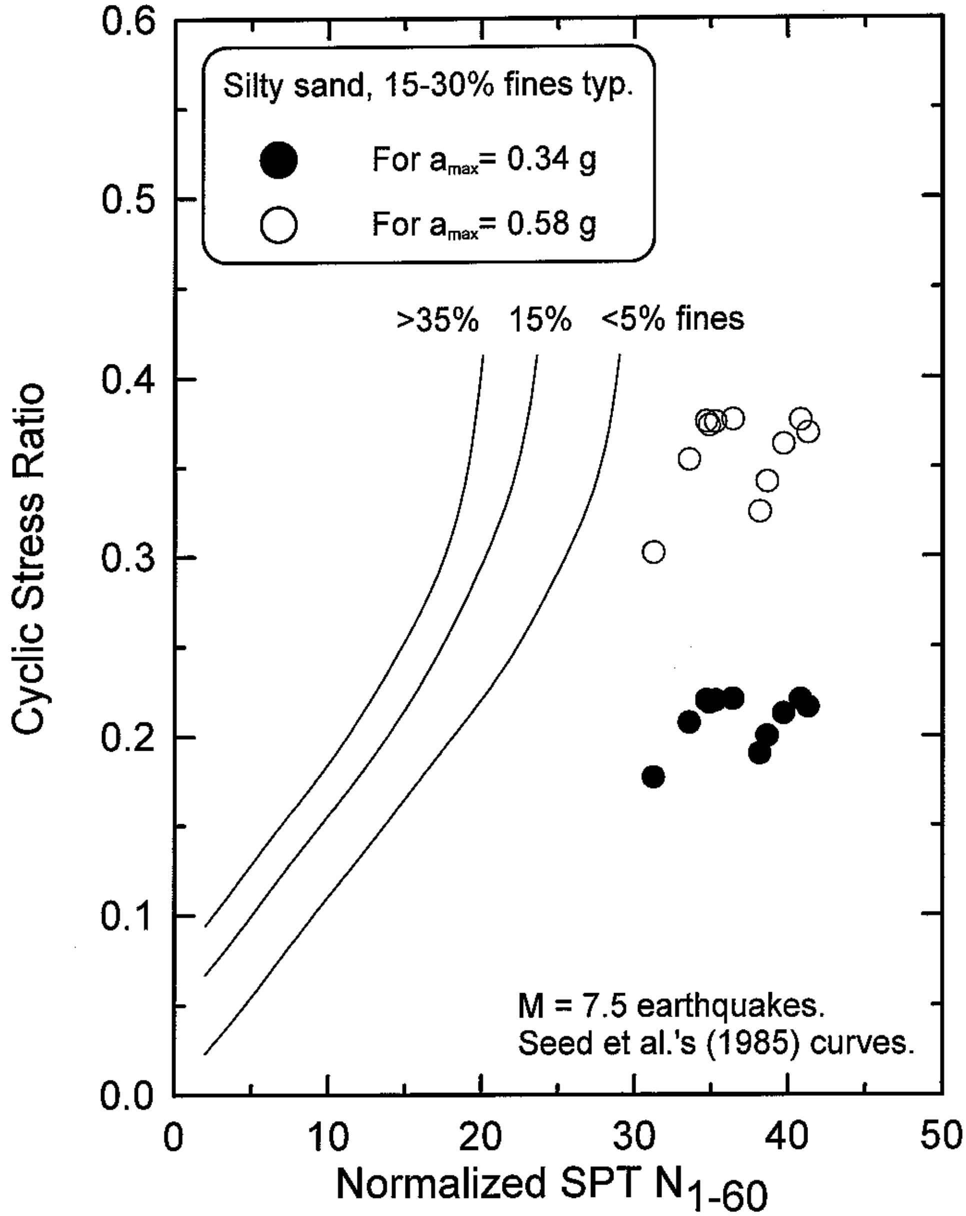


Fig. Liquefaction Analysis for the Warehouse Facility on Port Island in the 1995 Kobe Earthquake

WAREHOUS.GR1
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Table 13. Amusement Park on Port Island[a]

Site & locations	Portopialand, amusement park, on Port Island, Kobe, Japan.
Earthquake	Hyogo-ken Nanbu (Kobe) Earthquake of 1995, M_o=6.9
Peak accel. at the site	0.34 g measured at a liquefied site on the other side of Port Island; treated areas may have been shaken harder.
Facilities	Amusement rides (large ferris wheel, roller coaster, etc.) on shallow footings well-tied together with grade beams. Small buildings on shallow footings.
Treatment method	Vibro-rod in 1979; 2.6 m square spacing, to 19 m depth, lateral extent unknown.
Treated areas	No cracks or boils over majority of site; some cracks to 25 mm and ejecta along south side. No visible damage to any structures.
Untreated areas	Outside the facility: boils, cracks, and settlements up to 500 mm; however, settlements were amazingly uniform over large areas.
Water table at 4 m depth. 0-19 m: silty gravely sand (fill), 15-30% fines typ.	$N_{1\text{-}60}$= 8-13 before; 15-38 (23 average) in-between rod locations and 17-30 (23 average) at rod locations after treatment.[b]
Insitu test data	1 SPT log before, and 2 SPT logs after treatment (1 in-between rod locations, and 1 at a rod location).
Comments	Need more data on the insitu testing, extent of treatment, and settlements. The attached liquefaction analyses include no surcharge from the buildings.
Lessons learned	Treatment greatly improved behavior of the site. Good performance is consistent with the $FS_{liq} > 1$ throughout the fill, except perhaps in isolated pockets. A stiff upper crust, well-designed shallow foundations, and uniform settlements could have contributed to the good performance. Liquefaction evidence along south side appears to have been within a distance equal to about 1/2 the thickness of liquefiable soil from the treatment boundary.

[a]Yasuda et al. (1996), EERC (1995).
[b]Assumes N_{60}=1.2N, where N are the reported SPT blow counts.

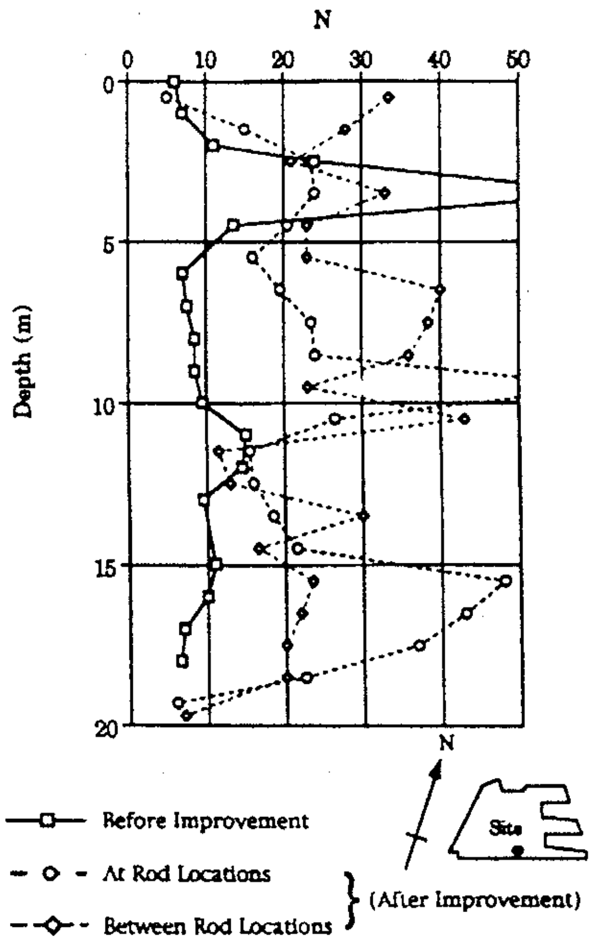

Fig. Amusement Park on Port Island (EERC 1995)

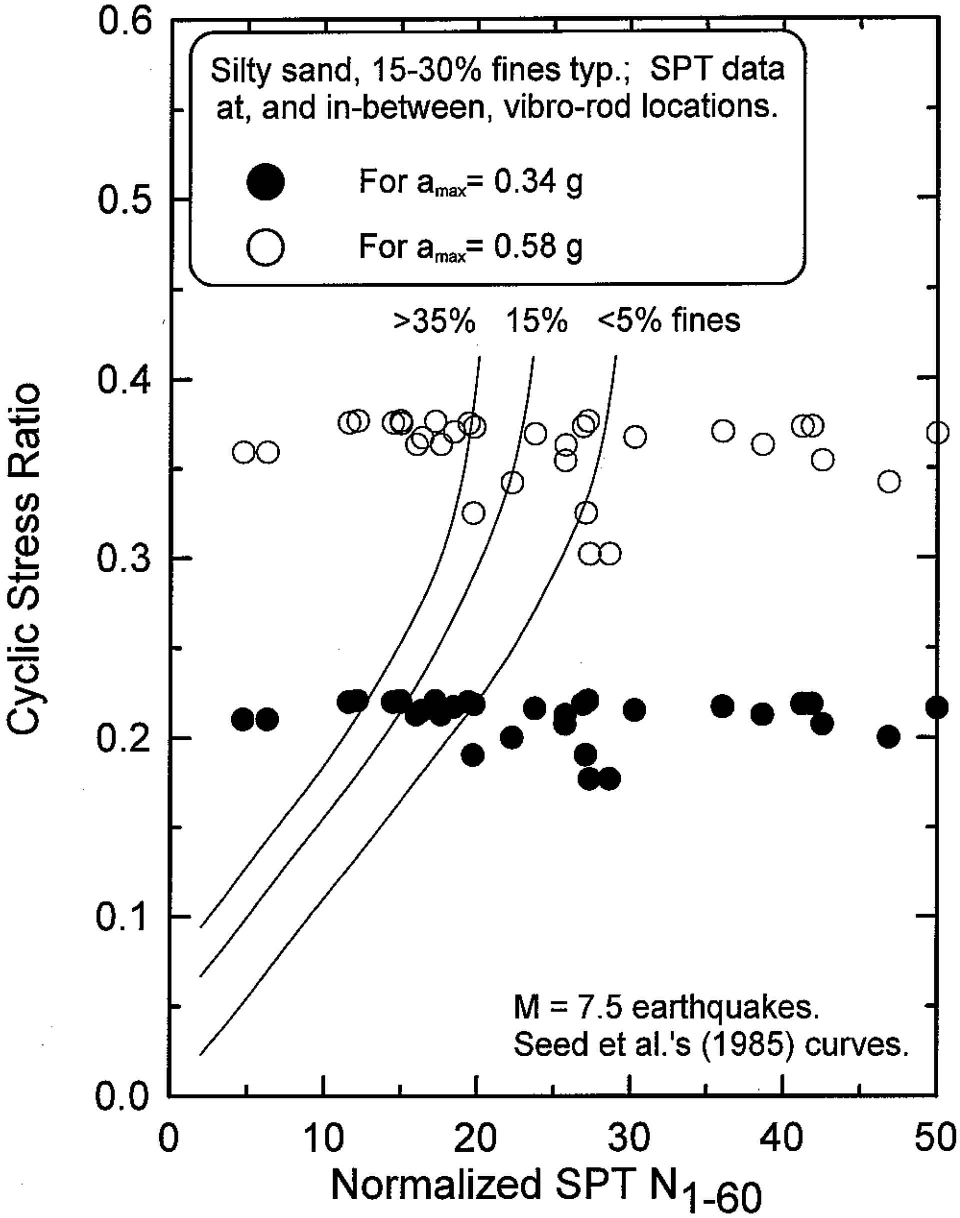

Fig. Liquefaction Analysis for Portopialand on Port Island in the 1995 Kobe Earthquake

PORTOPIA.GR1
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Table 14. Small Building on Piles on Port Island[a]

Site & locations	Small building on piles on Port Island, Kobe, Japan.
Earthquake	Hyogo-ken Nanbu (Kobe) Earthquake of 1995, M_o=6.9
Peak accel. at the site	0.34 g measured at a liquefied site on the other side of Port Island; treated areas may have been shaken harder.
Facilities	Small 3-story (check) building on piles.
Treatment method	Vibro-rod in 1981; 10 m beyond building edges; spacing, and depth uncertain.
Treated areas	No structural damage; differential settlements of 150 mm beside building.
Untreated areas	Differential settlements of 0.5-1.0 m beside other pile-supported buildings.
Silty gravely sand (fill); expected depth of 12-20 m. Water table at 4 m depth.	No insitu test data published.
Insitu test data	None published.
Comments	Need more insitu test data, and details of treatment.
Lessons learned	Treatment improved behavior around the building, but the soils either still liquefied or were influenced by surrounding untreated areas.

[a]Yasuda et al. (1996), EERC (1995).

Table 15. Rubble-Mound Breakwater in Kobe Area[a]

Site & locations	Rubble-mound breakwater in the Nishinomiya area, near Kobe, Japan.
Earthquake	Hyogo-ken Nanbu (Kobe) Earthquake of 1995, M_o=6.9
Peak accel. at the site	
Facilities	Reinforced-concrete breakwater supported on a rubble-mound in about 7 m deep water.
Treatment method	Sand compaction piles; 2 m diameter, 2.1 m spacing, treatment over the base-width of the rubble mound.
Treated areas	Breakwater settled 1-2 m.
Untreated areas	Sand boils at the toe of the mound.
Sand fill, 2-3 m thick, over 10 m of soft clay.	No insitu test data published.
Insitu test data	None published.
Comments	Need to obtain insitu test data, and to address the roles of the soft clay and rubble-mound.
Lessons learned	

[a]Yasuda et al. (1996), EERC (1995).

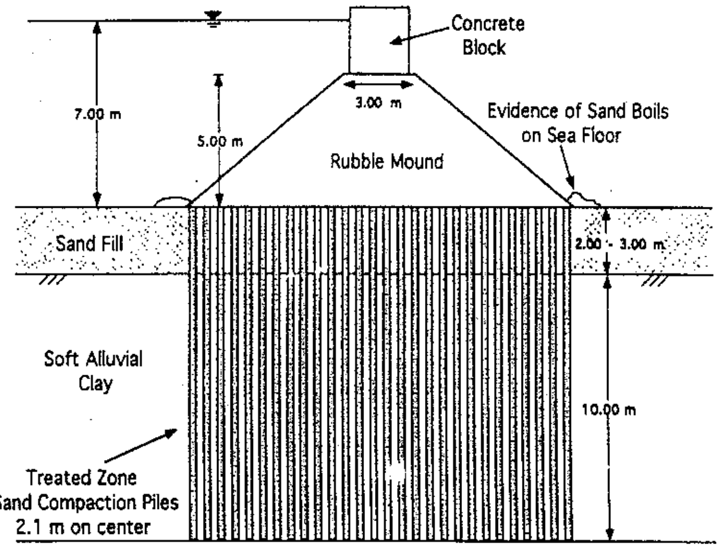

Fig. Rubble-Mound Breakwater in Nishinomiya Area (EERC 1995)