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Construction Control by Monitored Geotechnical Instrumentation for New Terminal 46, Port of Seattle

BENGT H. FELLENIUS, ARTHUR J. O'BRIEN, AND FRANK W. PITA

Geotechnical instrumentation was used to monitor and control construction pore pressures and soil movement during major modifications to an existing container terminal (old terminal 46) for the Port of Seattle. There was concern that the construction work, which consisted of dredging, filling, and pile driving, might disturb the confined and sloping (5H:1V) 25-ft-thick loose silt layer beneath the fill at the terminal. Construction control by monitored instrumentation was used because the topographic conditions at the site and the Port's economic and marine design parameters precluded conventional methods of preventing slope failure, such as total excavation of the silt and/or flattening the new fill slope. The instrumentation monitored the behavior of the confined silt layer to ensure that excess pore pressures and soil movements induced by the disturbance of the construction work were within acceptable limits. Two warning levels of observed excess pore pressure were established to control the construction sequence and rate. At the yellow level, extra caution and alertness were imposed. At the red level, construction was halted or relocated. The disturbance caused by dredging and filling operations was small. The disturbance from pile driving was limited to a zone that had a radius smaller than 30 ft. The pile-driving contractor was restricted to driving no more than 3 piles/day within 30 ft of each other. This posed little hardship for the contractor, and the construction was completed successfully.

This paper presents the background and results of the construction-control monitoring program implemented during the construction of new terminal 46, Port of Seattle, Washington. The preliminary design for the new terminal specified that an embankment be built on a confined, sloping layer of loose silt and that, afterward, displacement-type piles be driven through the embankment slope and silt layer into an underlying dense, glacial deposit. There was concern that implementation of these two construction procedures might cause embankment instability.

The preliminary design calculations for new terminal 46 indicated an unacceptably low margin of safety against slope failure if construction procedures caused loss of effective strength in the sloping silt layer. Such loss of strength could occur from increased pore pressures caused by rapid dumping of fill or by pile driving. However, the overall topographic conditions of the site and the marine design parameters were such that conventional solutions, such as complete removal of the silt layer or flattening of the new embankment slope, were not practical. Conventional solutions were also not economical because the cost difference between the use of instrumentation to implement the preliminary design concept and the use of conventional solutions was estimated to be more than $1 million. Therefore, the decision was made to implement the preliminary design with some minor modifications and to monitor the stability of the slope during construction by means of piezometers and slope inclinometers. If any excessive pore pressure or soil movements suggesting imminent risk of failure occurred, the construction would be halted until the risk had subsided.

Proper planning and use of the monitoring program would maintain the risk of embankment failure at an acceptably low level; however, too frequent construction halts and/or relocations could cause costly project delays. Nevertheless, the risk of costly delays was preferred over alternative conventional solutions.

SOUTHEAST HARBOR DEVELOPMENT PROJECT

The Port of Seattle implemented the Southeast Harbor Development Project to improve existing waterfront facilities and to provide new facilities for handling the growing volume of containerized cargo. Phases 1 and 2 of this project, which occurred between old pier 37 and old terminal 46, were completed in 1979. Phase 3, which consisted of a modification and lateral extension of old terminal 46, was completed in 1980. The completed facilities include 86 acres of a container storage and handling area; five container cranes will operate on 2740 ft of the pile-supported apron structure (Figure 1).

During the construction of phases 1, 2, and 3, pier 39 and portions of piers 37, 42, and 43 and old terminal 46 were removed (Figures 1 and 2). An earth-fill embankment was built at the outer edge of the old piers. A container storage area was then constructed by filling between the old piers and the new embankment. A pile-supported apron deck was constructed on the outer slope of the new embankment.

A variety of fill materials was used behind the embankment, including fine-grained organic dredge material from the Duwamish River, demolition rubble, riprap, and gravelly sand. The outer fill slope intersects the natural bottom of Elliott Bay, which descends at a slope of approximately 5H:1V at the site. The slopes were built in water at depths up to 90 ft in phase 1 and to 125 ft in phases 2 and 3.

The construction of new terminal 46 (phase 3)

Figure 1. Site plan, Southeast Harbor Development Project.

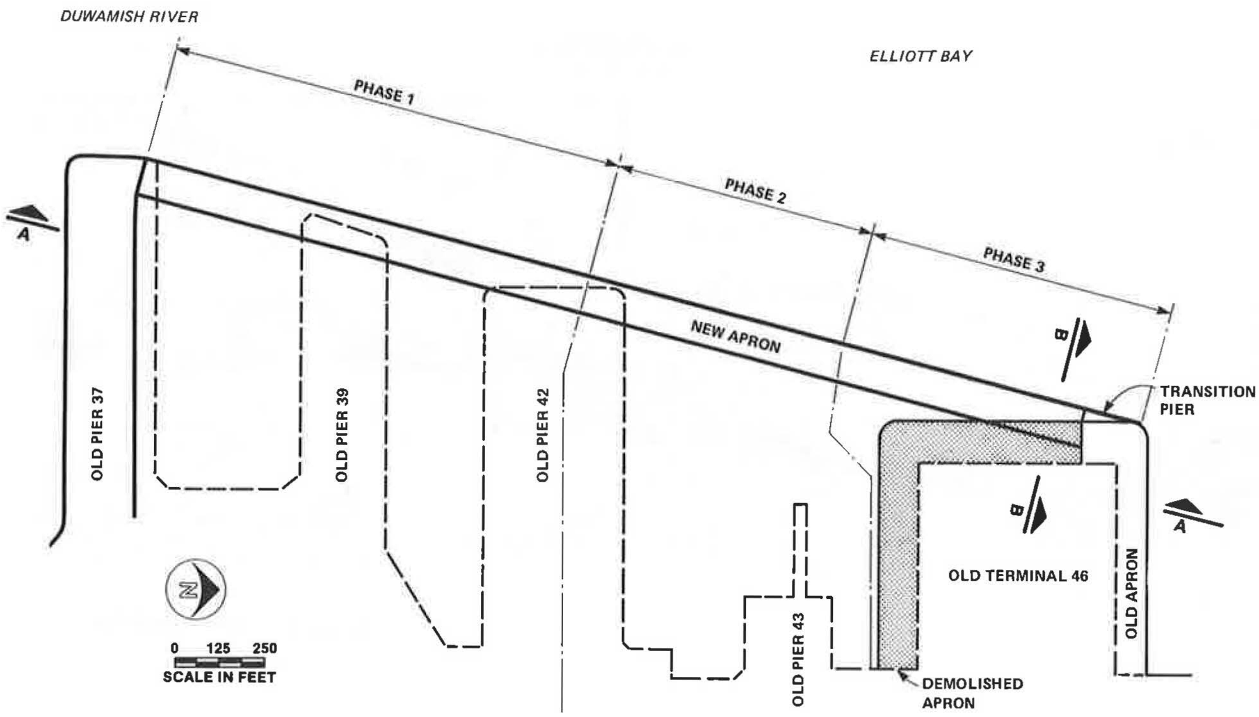
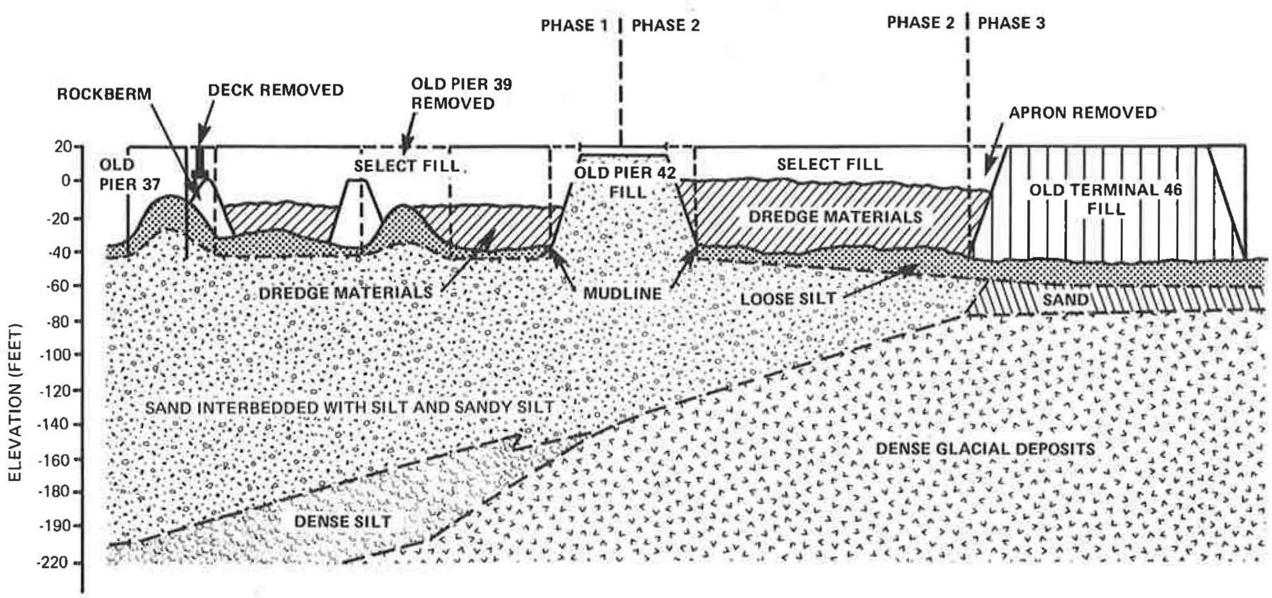


Figure 2. Section A-A of Figure 1.

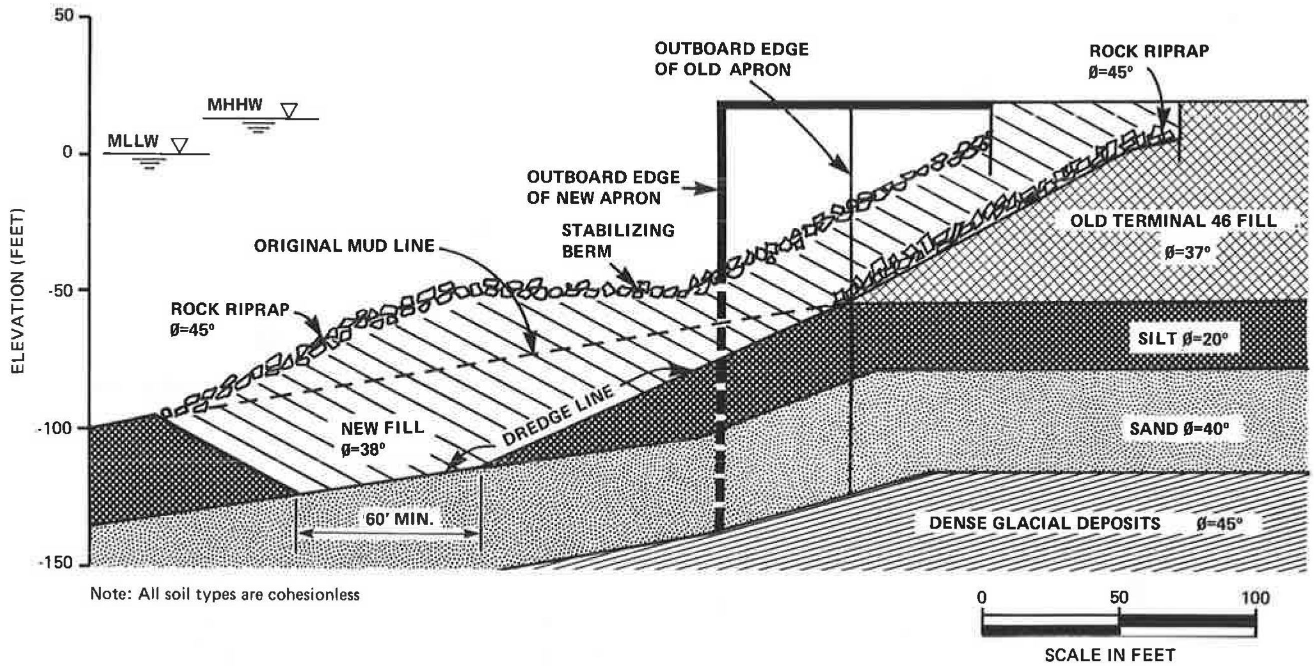


consisted of modifications to old terminal 46 (Figures 1 and 2). The major modifications included demolition of the south apron and approximately 630 ft of the west apron, placement of more fill to extend the embankment to the west, and construction of a new concrete apron that would connect to the previously constructed (phase 2) apron at the south. A transition section was constructed to connect the new and old terminal 46 aprons at the north.

The design criteria for the apron structure and embankment of new terminal 46 were provided by the Port of Seattle. They are summarized below:

1. Apron deck: dead load = 475 lb/ft^2 and live load = 1000 lb/ft^2;
2. Yard, live load = 1000 lb/ft^2;
3. Pseudostatic earthquake loading: seismic coefficient = 10 percent;
4. Piling = 16.5-in octagonal prestressed concrete piles;
5. Embankment slope = 1.75H:1.00V from elevation +7 ft at sheet pile wall to elevation -50 ft at edge of apron deck; and
6. Apron width = 101 ft.

Figure 3. Section B-B of Figure 1, old and new construction.



The material for the new embankment was to be clean, gravelly sand. To maintain sufficient draft at new terminal 46, the mud line at the outboard edge of the new apron had to be no higher than elevation -50 ft. Also, the apron was required to be 101 ft wide. These two conditions imposed an outer slope angle of 1.75H:1.00V. From elevation -50 ft, the embankment and/or the mud-line slope could vary, depending on stability requirements and existing conditions.

SOIL CONDITIONS

A geotechnical investigation that preceded the design was performed in early 1979. It consisted of test pits, test borings, and a static-cone penetrometer test. Disturbed samples were obtained from the test pits and split-spoon samples from the borings. Standard penetration tests (SPTs) and vane shear tests were performed. Shelby tube samples were attempted but not recovered.

Figure 3 (section B-B of Figure 1) presents a simplified vertical section across the site. Very dense glacial deposits underlie dense sand, which is covered by a layer of loose silt sloping toward the bay. The silt varies in depth and forms the base on which fill for old terminal 46 was placed. Ensuring the stability of the new fill with the presence of the loose sloping silt layer became the major concern in the geotechnical design.

In the silt, the sampling spoon and rods advanced ahead of the casing by their own weight, and the SPT values were mostly zero. In some places, however, SPT values as high as 17 were recorded. Also, the static-cone penetrometer showed some values equal to zero in the silt. The maximum cone resistance recorded in the silt was 20 kg/cm^2. The vane shear resistance in the silt was 250 lb/ft^2.

Grain-size analysis of the silt indicated 8-36 percent sand size and 92-64 percent fines (passing sieve No. 200). The clay-sized percentage was less than 10 percent. The organic content was small, about 2 percent.

Drained, direct shear tests on remolded samples of the silt indicated internal friction angles ranging from 26° through 36°. No cohesion intercept was found. The friction angle increased with the increasing density of the test specimen. The lowest density of the remolded test specimens was considered higher than the lowest in situ values. Based on the results of the field and laboratory testing and on engineering judgment, the design effective friction angle was designated as 20°.

In summary, the silt was found to be nonplastic and loose, and its primary strength of a frictional rather than a cohesive nature. Therefore, it was considered highly susceptible to excess pore pressure.

Based on results of the SPT and cone penetrometer tests, the effective friction angle for the dense sand underlying the silt was estimated at 40°. The effective friction angle of the new fill to be used for the embankment was estimated at 38°.

EMBANKMENT STABILITY

The stability of the embankment during construction (dredging, filling, and pile driving) and after construction (final conditions) was analyzed by using effective stresses. The analyses were made by using both cylindrical rotation slip surfaces (according to a modified Bishop method) and plane slip surfaces (wedge analysis). The cylindrical slip-surfaces analysis resulted in safety factors lower than the plane surfaces. Figure 3 shows the subsurface profile used in the stability analyses. The table below presents the results of the analyses:

Case	Safety Factor
During new embankment construction (after dredging outboard of old embankment)	1.07
Final conditions of new embankment	
No live loads	1.31
1000-lb/ft^2 live load	1.20
Seismic coefficient without stabilizing berm	0.92
Seismic coefficient with stabilizing berm	1.07

The subsurface profile shown in Figure 3 was used to model the most critical section of the embankment. Because the new apron deck joins the existing apron at an angle, the relative locations of the new and old aprons change throughout the site. Figure 3 shows the profile at the intersection between the existing apron and embankment and the new apron and embankment. At the intersection, a limited amount of silt beneath the fill could be dredged without disturbing old terminal 46 fill, and a minimum thickness of new fill could be placed over the silt and still allow for the required draft clearance. Also shown in Figure 3 is a stabilizing berm outboard of the new embankment. The stability analysis indicated that this stabilizing berm must be included to attain acceptable stability for final conditions under earthquake loading.

Early in the design it became obvious that the behavior (pore pressures and lateral movements) of the loose silt layer during construction was critical to embankment stability. Pore-pressure increases during new embankment construction could critically decrease the stability of old terminal 46 fill. Also, the driving of displacement piles through the new embankment and into the loose silt layer would induce excess pore pressures that could critically decrease the safety of the new embankment. In order to proceed with the construction as designed, the decision was made to monitor the behavior of the silt layer and the embankment during the construction by means of geotechnical instrumentation.

To determine the effect of pore-pressure increases on embankment stability, two construction stages were analyzed: (a) building of the new embankment and (b) pile driving through the new embankment. During the analysis, excess pore pressures in the silt layer were imposed, which reduced the previously calculated safety factors.

Two warning levels (yellow and red) were established to help evaluate the observed excess pore pressure during each construction stage. The yellow warning level was defined as the excess pore pressure that decreased the safety factor to 1.0 when using an effective friction angle of 20° for the silt. The red warning level was defined as excess pore pressure that decreased the safety factor to 1.0 when using an effective friction angle of 26° (the lowest laboratory test result) for the silt. The excess pore pressures for these construction stages and warning levels are given in the table below:

	Pore Pressure (psi)	
Construction Stage	Yellow Level	Red Level
During new embankment construction	2.5	4.0
During pile driving	13.0	15.0

When excess pore pressure was below the yellow level, no extra caution was necessary in the construction procedure. Excess pressures between the yellow and red warning levels indicated use of cautionary measures, such as increasing the frequency of the monitoring of the instruments and the rigor of inspection and caution. Pressures above the red warning level required that the construction in that area be halted, or possibly relocated, until the pore pressures dissipated to below the red level.

In addition to pore-pressure measurements, lateral movements were monitored to aid the subjective judgment of the engineers. No specific limits were established for the observed lateral movements. If pore pressures were between the yellow and red levels but lateral movements did not occur, work continued. If lateral movements did occur, procedures for conditions above the red level were warranted, even if the pore pressures stayed between the yellow and red levels.

TYPES AND AMOUNT OF INSTRUMENTATION

Excess pore pressure was considered the most important factor contributing to possible embankment instability. Therefore, the piezometer was the main instrument for construction control. A piezometer, however, monitors the pore pressures at only one point and may not indicate pore pressures over the entire soil mass. For example, local zones of high values might not be representative of the whole, and important areas of excessive pore pressures might not be measured. Therefore, slope inclinometers were installed to provide information on the large-scale effect of construction procedures on the entire soil mass.

The piezometers used for the project were Petur Model P-102 Wellpoint. The inclinometers were Slope Indicator Company Model 50325 Digitilt. The instruments were installed in three phases during construction to accommodate the various conditions at the site. A total of 36 piezometers and 7 inclinometers was used. Figure 4 shows the location of the instruments. Figure 5 shows the approximate depths of the instruments in cross section.

In October 1979, 26 piezometers and 4 slope inclinometers were installed for phase A at old terminal 46 before its demolition. These instruments were used to monitor the embankment during the dredging operation and subsequent filling.

Figure 4. Instrument plan.

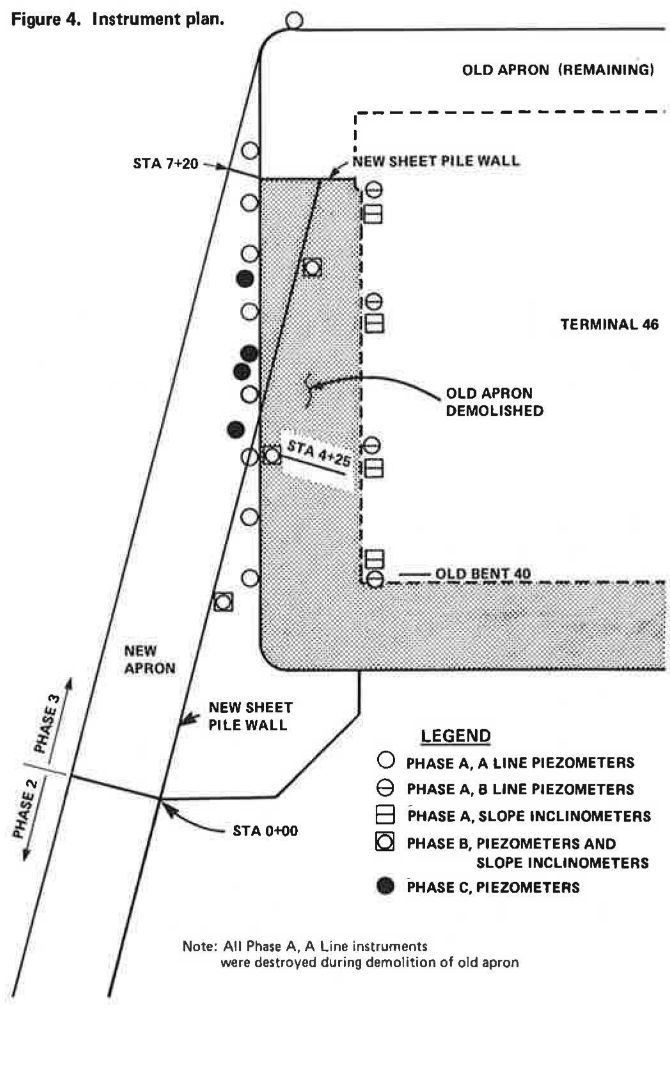
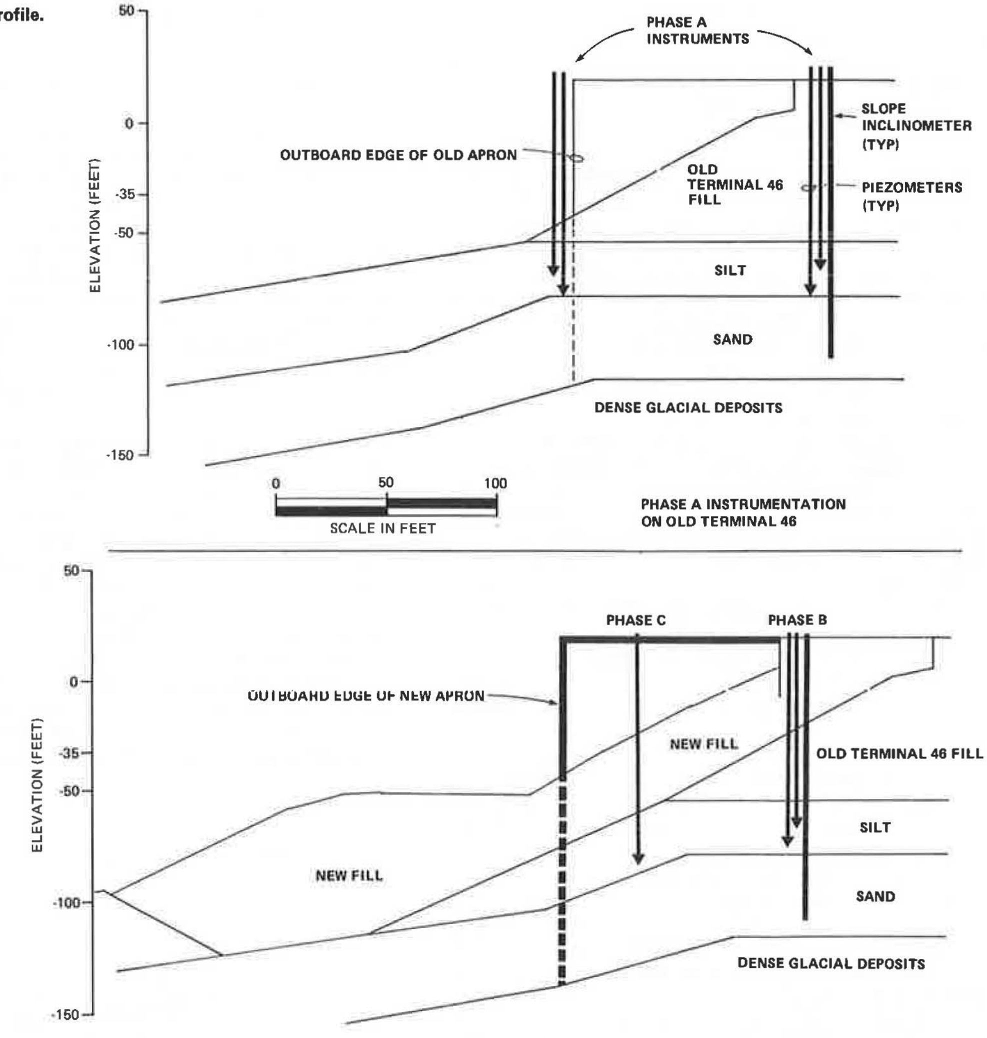


Figure 5. Instrumentation in profile.



Phase B instrumentation, which was installed in March 1980, consisted of six piezometers and three slope inclinometers at three locations behind the new sheet pile wall. These instruments were installed at a distance greater than 30 ft from the nearest pile-driving area to monitor overall stability during pile driving.

Phase C instrumentation was installed in May 1980 and consisted of four piezometers. These instruments were installed adjacent to the pile-driving operation (each within 10 ft of a pile location) to monitor local pore-pressure increases during driving. Figure 6 shows the detailed location of phase C piezometers in relation to pile locations.

PRACTICAL PROBLEMS ASSOCIATED WITH USE OF GEOTECHNICAL INSTRUMENTATION

[Ed. note: This section is a general review of the problems of geotechnical instrumentation, which the authors felt was relevant not only to this project but also to any project in which instrumentation is needed.]

The practical problems of using geotechnical instrumentation must be considered early in the design process. The most important and easily overlooked problems are almost always associated with the people working on a project. The attitudes of owners, contractors, and field staff toward instrumentation are critical to proper operation and protection of the instruments.

Owners often think of instrumentation projects as research projects that have no direct cost benefits. Also, because they have been successful on other jobs without instrumentation, owners do not want to use it on their production-oriented projects. The designer must budget time and money to explain to the owner the technical reasons for, and cost-saving advantages of, instrumentation.

Contractors' opinions of instrumentation are often that it is a nuisance and a hindrance. Many times the contractor is indifferent to protection of the instruments from accidental destruction. Care must be taken to inform the contractor of the purpose and manner of use of the instrumentation to ensure his or her full cooperation and to show that the results can also be a benefit. In addition, strong wording must be included in the contract documents to provide an incentive for protection. Replacement clauses must be enforced from the start of the job. Even with a strongly worded contract, the design must provide for redundant instruments so that, when some of the instruments are destroyed or malfunction, enough remain to do the job.

Finally, the method of data gathering and reporting must be thoroughly planned and tested before the start of the project so that the data can be used quickly and efficiently. It is most important to have field staff who are willing, alert, and competent.

RESULTS OF PORE-PRESSURE MONITORING

Instrumentation Calibration

After each installation phase and before any construction work, the piezometers were monitored to develop initial sets of control data. Each piezometer was monitored hourly over an approximately two-day period so that a normal pressure (in pounds per square inch) versus tide elevation (in feet) curve could be established. The difference between the normal pressure at a given tide elevation and the reading during the construction for the same tide level was considered the excess pressure caused by construction. These data were unique for each piezometer.

Results of Phase A

Only minor increases in pore pressure were observed during the dredging operation (October and November 1979). These small increases did not approach the yellow level. Figure 7 shows the pore pressures observed at old bent 40 (Figure 4) during the filling operation. Filling began at the south end of the new embankment while dredging was being completed at the north.

The filling was accomplished by dumping from a bottom-dump barge at a rate of approximately 2000 tons of fill per dump. When a barge dumped close to

Figure 6. Piezometer locations (phase C).

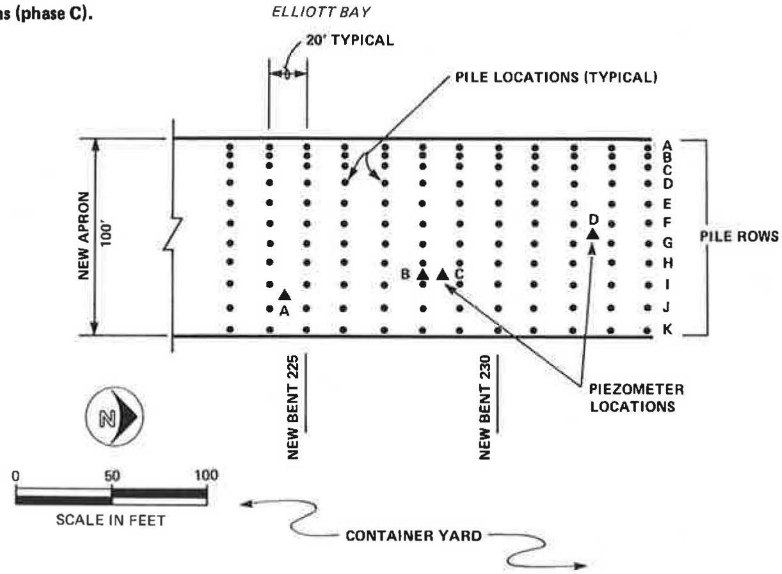
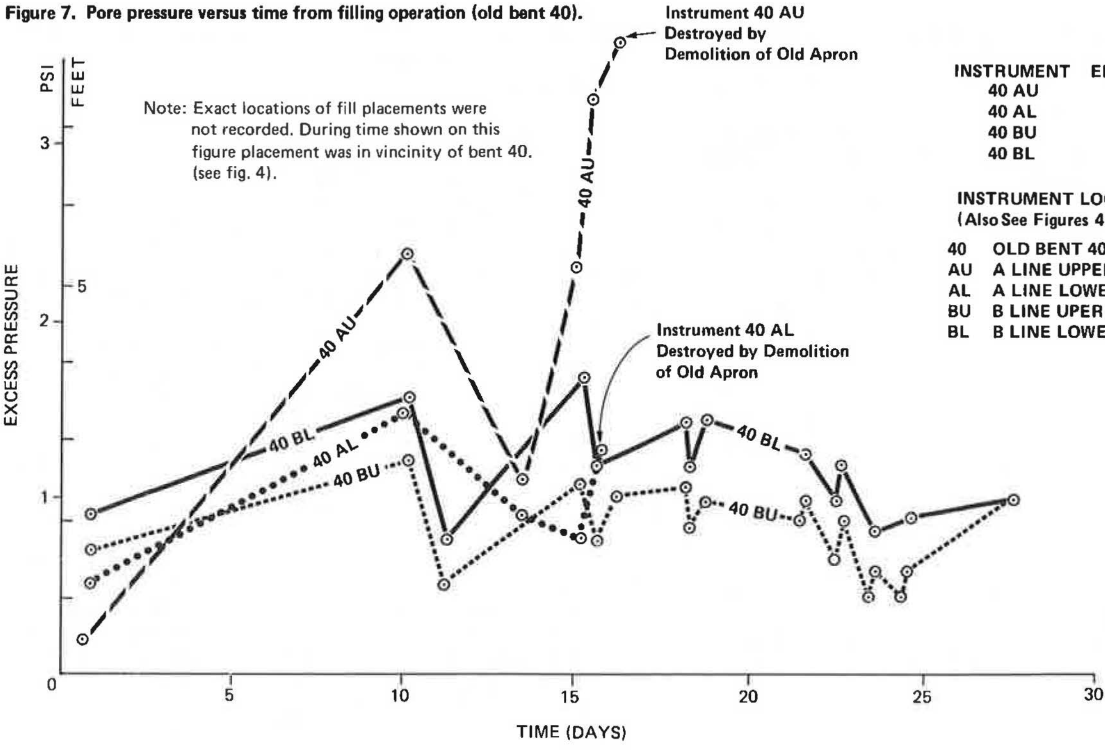


Figure 7. Pore pressure versus time from filling operation (old bent 40).

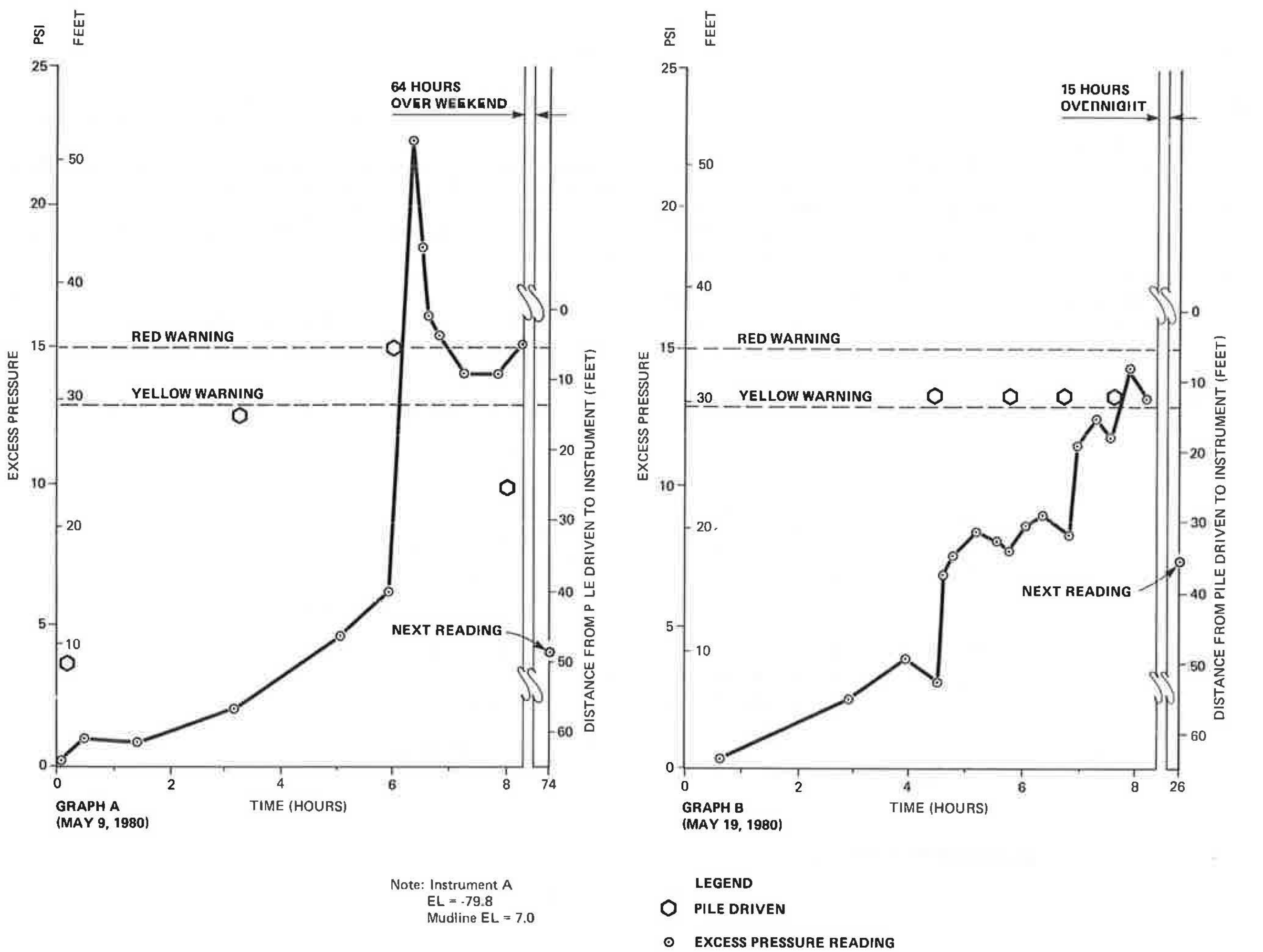


INSTRUMENT	ELEVATION	MIDLINE ELEVATION
40 AU	-65.3	-43.2
40 AL	-72.7	-43.2
40 BU	-55.8	19.0
40 BL	-58.9	19.0

INSTRUMENT LOCATION LEGEND
(Also See Figures 4 & 5)

40 OLD BENT 40
AU A LINE UPPER LOCATION
AL A LINE LOWER LOCATION
BU B LINE UPER UPPER LOCATION
BL B LINE LOWER LOCATION

Figure 8. Instrument A pore-pressure increase during pile driving (phase C).



a piezometer, the pressures typically rose and then dissipated during the next 1-2 h if no additional dumps were made in the vicinity. Piezometer 40AU on day 14 (Figure 7) showed the accumulated effect of several successive dumps close to its location. Coincidentally, this piezometer was destroyed shortly after this reading.

All observations indicated that the induced pore pressures were below the yellow warning level. By mid-December, the remaining phase A piezometers were destroyed during demolition of the old apron, thereby preventing additional monitoring of the dredging and filling operations.

Results of Phase B

With few exceptions, pore pressures observed in phase B piezometers were below the yellow warning level. Detailed data from these instruments have not been included in this paper.

Results of Phase C

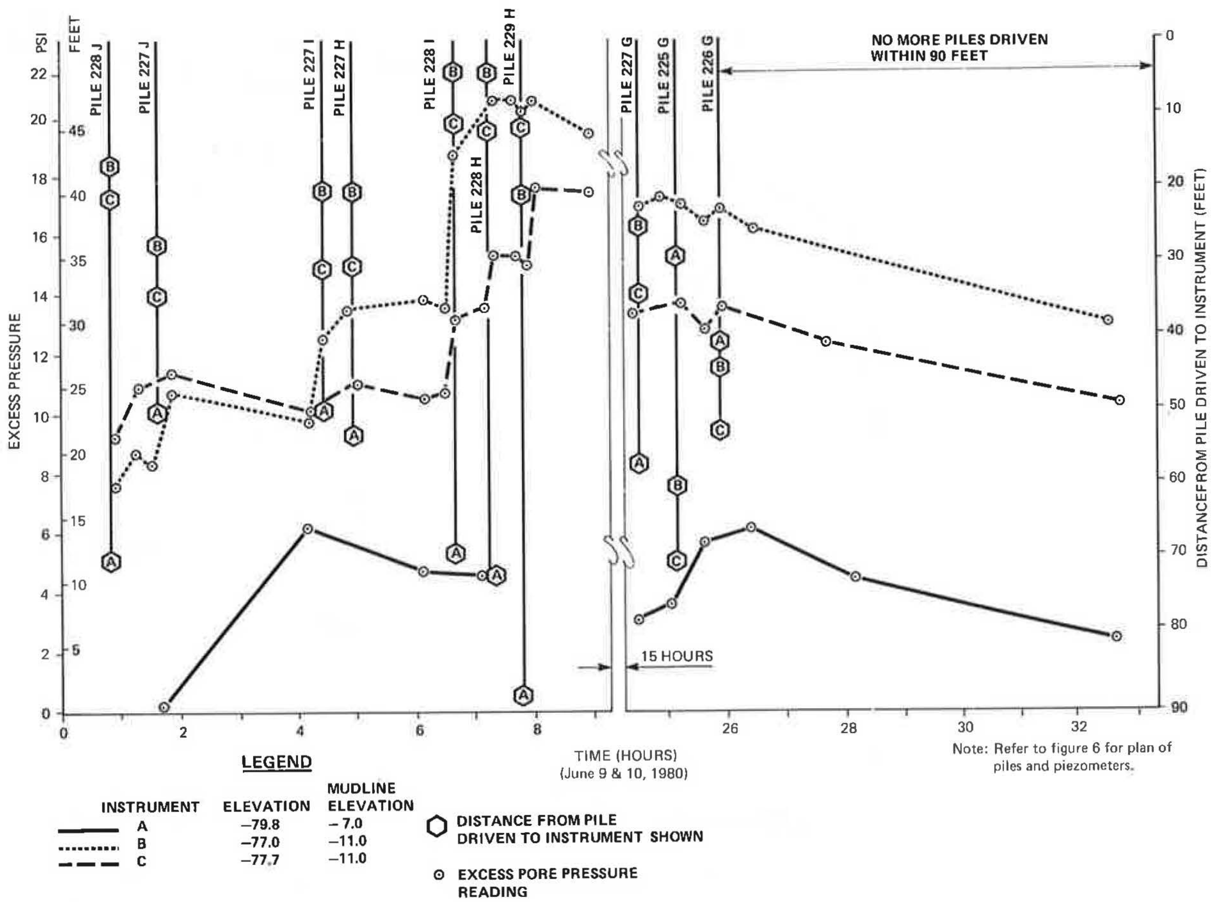
Figure 8 presents pore-pressure data taken from piezometer A of phase C during the pile-driving operations. Two separate sets of data are shown on each graph to give the relation among increase in excess pressure, distance from driven pile to piezometer location, and time. The first set of data (connected by the line) shows excess pore pressure (left ordinate) as a function of time (abscissa). The second set of data (hexagons) shows the distance from the driven pile (right ordinate) as a function of time.

Graph A of Figure 8 shows the relation between pile-driving distance and pore pressure. Piles driven more than 50 ft away did not significantly affect the pressures. However, as pile driving moved to within 15 ft, pressure increases were noticeable, and a pile driven within 5 ft caused sharp increases. Pore pressures dissipated rapidly after the sharp increase and increased again as the final pile for the day was driven 25 ft away. Dissipation occurred when no piles were driven nearby.

Graph B of Figure 8 shows the results of driving four successive piles within 12 ft of piezometer A. Results were cumulative, in that each pile caused an increase in pressure followed by a slight dissipation before the driving of the next pile. Each additional pile caused the same effect, which resulted in pressures above the yellow level; however, dissipation occurred overnight.

Figure 9 presents data similar to Figure 8 for three piezometers of phase C. At this time, the contractor was driving 7-8 piles/day (one shift per day). The data indicate that piles driven 20-30 ft from the instruments increased the pore pressures to the 10- to 12-psi range. When pile driving came to within 10 ft of a piezometer, the pressures increased significantly and entered the red level.

Figure 9. Pore pressures during pile driving (phase C).



The pressure decreased more slowly than during the initial pile-driving observations (Figure 8).

RESULTS OF SLOPE INCLINOMETER MONITORING

The results of the phase A inclinometer monitoring indicated no significant slope movement during the dredging and filling operations; however, some interesting results were recorded in phase B.

Figure 10 presents data from one typical slope inclinometer from phase B at station 4+25. No deep stability problems were observed in the deep silt layer, as shown by the small size and slow rate of movement. In the upper 20-40 ft of fill, large horizontal movements (about 5 in) and significant acceleration of movements were observed during pile driving within 50-60 ft of the inclinometer casing. After pile driving had moved away from the vicinity of the instrument location, the rate of movement decreased.

DISCUSSION OF RESULTS

The dredging work did not cause any significant increase in pore pressures nor any appreciable soil movements. All of the dredging work, therefore, was performed without changes in the construction techniques.

During filling, the observations indicated no instability except when several dumps were concentrated in one area (Figure 7). As a consequence, the continued dumping of fill was distributed over a larger area to keep the pore-pressure increases low.

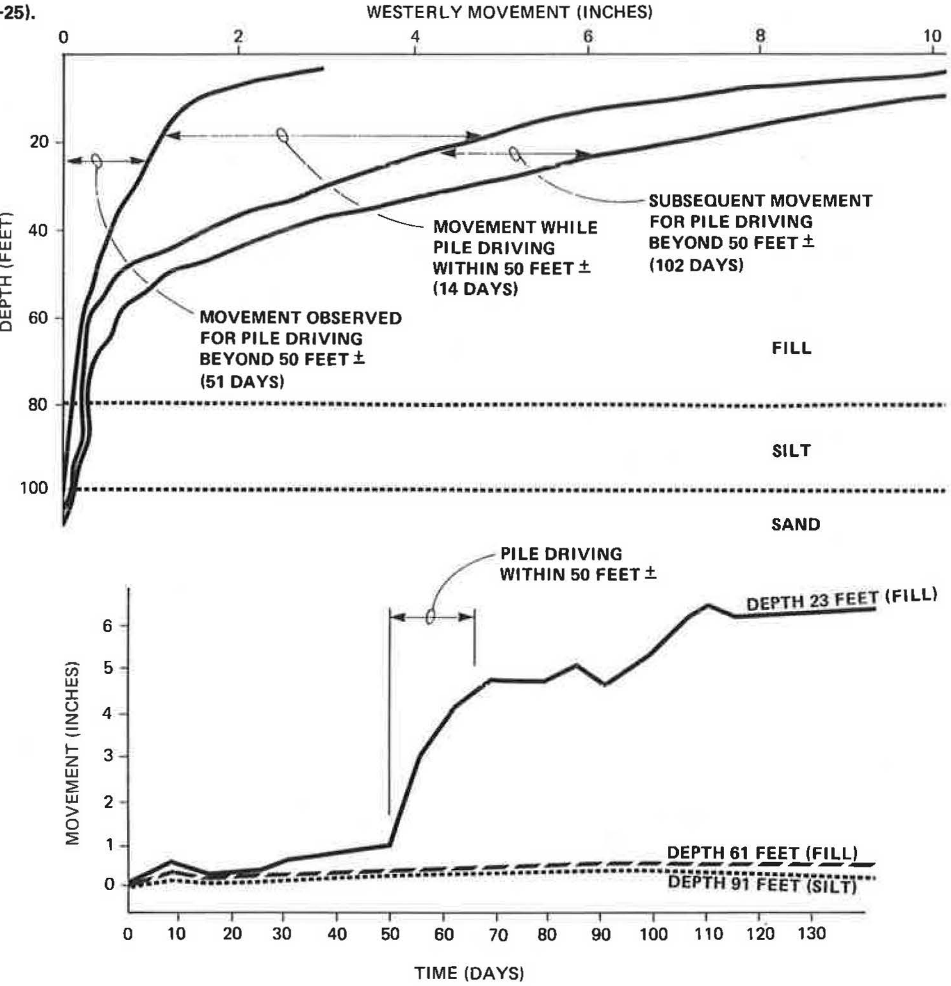
During pile driving, phase B piezometers, which were located more than 30 ft from the nearest pile location, registered only occasional pore pressures above the yellow warning level. However, the phase C piezometers, which were located near the pile locations, registered noticeable increases when piles were driven within a distance of 15 ft (11 pile diameters) of the instruments (Figures 8 and 9). When piles were driven within a distance of 12 ft (8.7 pile diameters), the accumulated pore pressures in the silt rose above the yellow warning level. Driving within a 10-ft distance (7.3 pile diameters) caused pore pressures to rise above the red warning level.

The indication in phase B piezometers that the effect of pile driving in the silt was local and did not extend beyond 30 ft (21.8 pile diameters) was confirmed by the phase B inclinometer observations, which showed only small movements in the silt layer (Figure 10).

The horizontal movements in the new fill shown in Figure 10 were considered a result of compaction of the fill from the pile-driving vibrations. The movements, although large, were not considered to indicate instability of the embankment and confirm the densification effect of driving displacement piles into loose granular materials.

Because induced pore pressures were relatively local and dissipated rapidly, and because no slope movements were observed, the pile-driving work suffered only minor disruptions. The results indicated that no more than three piles were to be driven within 30 ft of each other in a 24-h period. This

Figure 10. Slope indicator movement (station 4+25).



proved to be no hardship on the contractor and caused the pore pressures to remain below the yellow warning level for the remainder of the pile-driving work.

CONCLUSIONS

The construction-control program enabled phase 3 of the port development (new terminal 46) to be designed and built for costs comparable with those for phases 1 and 2. Close monitoring of the silt layer allowed implementation of a design that had factors of safety during construction that would have been unsatisfactory without the use of instrumentation data to control the construction sequence.

ACKNOWLEDGMENT

The project was designed by CH2M Hill and Bengt Fellenius under contract to the Port of Seattle: Vern Ljungren, chief engineer; Eric Soderquist, project manager; and Dean Poole, chief resident inspector. The piling and general contractor was Manson Construction, Seattle, Washington: Gus Lorenz, superintendent.

Discussion

Philip Keene

Fellenius, O'Brien, and Pita are to be congratulated on their clear description of a difficult project. By using modern geotechnical techniques and seasoned judgment, they successfully completed this project, which involved loose inorganic silt, and saved the owner about $1 million. Of special note for the reader is the section on practical problems associated with geotechnical instrumentation; the warnings in this section are a valuable part of the paper.

The most critical feature of the project was the control of pile driving to avoid widespread temporary liquefaction that results in a slide in the silt. Temporary liquefaction of fine-grained soils due to pile driving can be a difficult phenomenon, particularly when the piles are closely spaced (as for a large bridge abutment). As there appear to be rather few case histories of this in the literature, I will describe two cases from my experience when I was head of the geotechnical division of the Connecticut Department of Transportation. The projects were built in 1953 and 1958 and involved cast-in-place piles driven in fine-grained soils. In both cases, temporary liquefaction was generated by pile driving; it took approximately 10-20 days after driving was finished for the liquefaction to be dissipated.

The earlier project, a Farmington River bridge in Simsbury, is briefly described in Keene (1). The soil under the abutments is brown silt, 160 ft thick, and has approximately 0.50-in clay layers every foot. It has a natural water content of 35-40 percent and an N-value of 3 or 4 in the SPT. The design called for 35-ton cast-in-place piles in Monotube shells that were 70 ft long; spacing of piles was 4 ft on centers (front to rear) and 3-6 ft

Figure 11. Simsbury: two pile load tests.

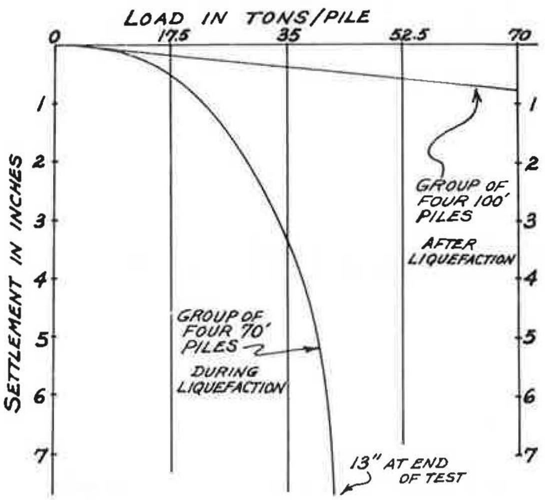
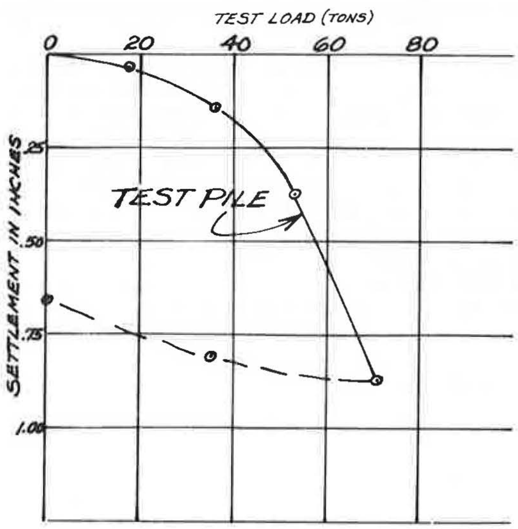
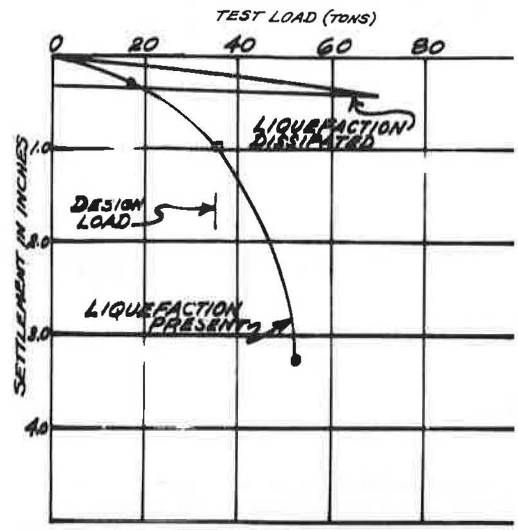


Figure 12. Danbury: load test on test pile.



laterally. The plans called for a pile load test on a group of four piles--to be performed after the 25 adjacent piles were driven--to determine their effect on the test. Each load increment (17.5 tons/pile) was to be held for 48 h. The test began 10 days after these 29 piles had been driven; the test piles settled an average of 13 in at 44 tons/pile (Figure 11). Two days after the test was stopped, 30-ft extensions were added to the four test piles at the insistence of the construction engineers. These four piles were then redriven, but the piles had firmed up so much that it required about 130 blows on each of the extended piles to drive the first foot and, after some jetting along the sides of the piles, from 60 to 80 blows/ft to the new penetration of 100 ft. A subsequent load test showed 0.75-in settlement at 280 tons, which was twice the design load (Figure 11). Final pile lengths were made 90 ft. Movements of the abutments were monitored for 18 months and showed settlements of 0.5 in or less. It appears from the above that liquefaction was dissipated at about 17 days after

Figure 13. Danbury: load tests on two separate production piles.



the piles were originally driven, although it may have been hastened around the four test piles by the effect of the first load test.

The other project is in Danbury, Connecticut. A four-lane expressway (later a part of I-84) goes over Tamarack Avenue, a local two-lane street. At the east abutment, four 60-ft test borings that were made under the direction of the consulting (contracting) engineers described the soil as fine sand, trace of silt. The SPT gave N-values of 14 to 20. The 35-ton cast-in-place concrete piles, which were 30 ft long, were designed with spacing similar to those in the project described above. The total for the abutment was 185 piles.

When work began, the contractor chose Raymond standard step-taper piles and drove a 30-ft test pile at each corner of this abutment. The softest pile, driven to 29 blows/ft under a Vulcan 65C hammer (39-ton formula value), was then load tested to 70 tons (Figure 12). It experienced 0.87- and 0.65-in gross and net settlement, respectively. Consequently, 30-ft piles were ordered.

Very soon after production pile driving began, blow counts became very low--about 8-12 blows/ft at the 30-ft depth. Then 40-ft lengths were tried for a few piles; but there was no significant improvement in driving resistance; thus, the rest of the piles were made 30 ft long. Nineteen days after all piles for this abutment were driven, a load test on one of the softest piles (driven five weeks earlier) was performed, with distressing results of more than 3-in settlement at 52 tons (Figure 13). It should be noted that the remaining piles at the far end of the other abutment, which were about 130 ft from the load test, were being driven before and after this load test. At this time, it was discovered that the consulting engineers had made no laboratory tests on the test boring samples. Therefore, grain-size analyses were immediately made of the samples, and it was found that the fine sand, trace of silt had 10-40 percent silt, averaging about 17 percent.

Finally, five days after the distressing load test, all pile driving was completed; six days after that a final load test was made on a different but soft pile. This last test (Figure 13) showed a settlement of only 0.17 in at the 35-ton design load. A review of the dates indicates that liquefaction of the east abutment piles had dissipated about four weeks after all east abutment piles had been driven or about one week after the last of the west abutment piles, which were 130 ft away, had been driven. Settlement points established when the footing was poured showed final settlement of the east abutment was less than 0.25 in.

REFERENCE

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Composite Piles with Precast Enlarged Bases Driven for Fuel Oil Tank Foundations

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Deep foundations were required for the support of six large fuel oil storage tanks in Queens, New York. A system of 150-ton-capacity composite piles with precast enlarged bases (TPT piles) was selected for the job. An extensive load test program, which included testing of a dogleg pile, was conducted to establish the criteria for pile installation. A variety of installation procedures was required to overcome difficulties in penetrating cumbersome overburden materials to reach the bearing stratum. Hydrostatic loading of the completed tanks showed settlements of less than 0.25 in.

The Power Authority of New York State recently constructed six 6 000 000-gal fuel oil storage tanks for the Astoria Generating Station No. 6 in Queens County, New York City. These tanks each measured 160 ft in diameter by 40 ft high. A fuel oil auxiliary building was also built at this time. All of these structures were designed to be supported on 150-ton-capacity piles. Figure 1 shows the job layout.

CONSIDERATIONS IN SELECTION OF PILE TYPE

The Stone and Webster Engineering Corporation of New York City was the project administrator for the Power Authority and supervised all of the foundation work for the job. They did an extensive subsurface and foundation study to determine the most appropriate support system for this work. Much of the site had been filled in over a number of years. Old drawings were retrieved that showed the location and construction of timber piers and bulkheads that were no longer visibly in evidence. It had been assumed that the remnants of these structures were buried under the fill. This fill contained wood, cinders, and boulders that extended to depths of up to 30-35 ft. Some preliminary excavation at the site in connection with other work revealed the presence of large areas of "subway rock"; i.e., large blocks of mica schist and granite in sizes up to 5 yd^3 that, in all likelihood, were dumped during the construction of the New York City subways.

The soil profile below the fill was not uniform. In general, it consisted of a layer of soft river silt, varved lenses of silt and sand, dense cemented sands and boulders (hardpan), weathered mica schist, and, finally, bedrock that consisted of mica schist and granite that had recoveries of about 40-60 percent in the upper 5 ft. The water table, which was influenced by the tide variations in the adjoining East River, varied between elevation 0 and +5.

The total maximum load on each tank mat was of the order of 30 000 tons distributed over an area of approximately 20 000 ft^2, or 1.5 $tons/ft^2$. For less onerous soil profiles, the tank slab design would have governed the pile design capacity. Usually this would indicate the use of low-capacity piles spaced closely together, such as 30-ton piles at 4-6 in on center. However, the cost of installing any type of pile through the rough fill material and compressible soils into acceptable bearing soils mandated the selection of a high-capacity pile in order to limit the total number of such units.

A composite pile that had an enlarged base and a capacity of 150 tons was selected to be driven into the dense sand and glacial till below the poorer soils. The enlarged base was needed to develop this high capacity. H-beams or closed-end pipe piles would have to be driven to bedrock to satisfy this design load. The stem of the composite pile was a corrugated shell filled with plain 5000-psi concrete by using type 2 cement for compatibility with the groundwater that had a high salinity because of the adjoining estuary. H-beams would have required an

Figure 1. Test pile location plan.