Ground improvement engineering at the Norfolk International Terminal

F Pepe* Parsons Brinkerhoff Quake & Douglas Inc, USA
G A Munfakh, Parsons Brinkerhoff Quake & Douglas Inc, USA
J Jones, Terra Systems, Inc, USA

Abstract

A combination of ground improvement techniques were applied to allow construction of a container terminal in Norfolk, Virginia, USA on land reclaimed with dredged material placed over a layer of soft marine clay. Preloading and wick drains, construction of a stabilizing sand shear key and vibrocompaction of underwater-placed fill were used to satisfy operational, environmental and construction-related requirements. Field trials, verification testing and geotechnical instrumentation were used to evaluate the effectiveness of the ground improvement techniques applied; select treatment patterns and spacings; and assess the impact of construction on adjacent facilities. The increase in strength and relative density of the compacted soil and the ageing phenomenon of vibrocompaction were evaluated through a series of borings and piezocones performed before and after compaction. Substantial economic benefits were accomplished by the combined use of the three ground improvement techniques. Design, construction and performance of the ground improvement methods are discussed in the paper.

Keywords: Ground Improvement, Vibrocompaction, Wick Drains, Marine Construction

1. Introduction

The Norfolk International Terminal (NIT) is a container port facility located on the Elizabeth River in Norfolk, Virginia, U.S.A. The port facility currently handles both domestic and international container ship traffic and is one of the major container ports on the east coast of the U.S.A. Recently, the owner, the Virginia Port Authority (VPA) wanted to expand the port's capacity by constructing a new wharf structure and container laydown area at the north end of the facility. Expanding the north end of the site required the addition of a 460 metre by 36 metre marginal wharf structure to provide five new berths for container ships with approximately 17.5 hectares of laydown and storage area for container handling operations. The facility was built on land which was reclaimed in the 1960's by placing dredged material over a layer of soft marine clay. Since the new facility would place additional loads on the in situ soils, its construction required ground improvement at the site to reduce post construction settlements, and to increase the strength of the underlying soils to the level required to accommodate the loads from the container operations and allow construction of the wharf structure.

Selection of an appropriate ground improvement technique was a challenge. The site was located in an environmentally sensitive area which restricted the ground improvement techniques that could be applied. The environmental and operational constraints at the site dictated the need for multiple types of ground improvement techniques. The combination of techniques used included
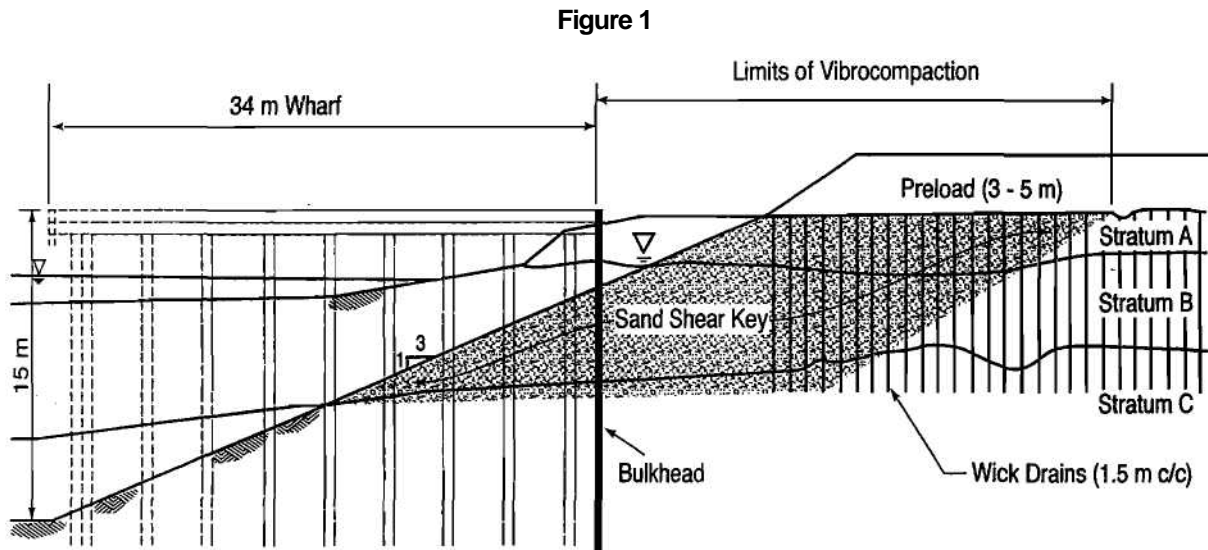
preloading and the use of prefabricated wick drains, construction of a stabilizing sand shear key, and vibrocompaction.

This paper discusses the design and construction aspects of the ground improvement techniques used for the project and their effectiveness. It also presents the findings of the field trial program for the vibrocompaction and highlights the key issues that affected the successful implementation of the various techniques.

2. Subsurface Conditions

The NIT site is located within the Atlantic Coastal Plain Physiographic Province. Bedrock is present at depths of greater than 600 metres and is overlain by Upper Cretaceous, Tertiary, Pleistocene and recent sediments. The materials which require ground improvement are the Pleistocene and recent deposits and the dredge spoils that occur in the upper portion of the subsurface profile.

The typical soil profile for the landside and waterside portions of the site are presented in Figure 1.

Figure 1

Concrete Piles (25 - 35 m long) The subsurface conditions on land consist of the following strata

from the ground surface down:

- Stratum A, Fill - This stratum consists of dumped grannular fill which is a mixture of coarse to fine sand. This material was generally placed over the hydraulic dredge spoils. The overall thickness of Stratum A varies from about 3 to 6 metres.

- Stratum B, Recent and Norfolk Deposits - These materials include both naturally deposited soils and materials placed during hydraulic filling operations. The deposits are predominately normally to under consolidated silty clay or clayey silt with occasional pockets of highly fibrous peat and organic silt. The. overall thickness of the Stratum varies from 5 to 13 metres over the site.

- Stratum C, Yorktown Formation - This stratum underlies the Stratum B soils and consists of overconsolidated clayey silt to silty sand. This is the principle bearing stratum for foundations in the Norfolk area.

On the water side of the site, the fine grained Stratum B deposits appear directly below the mudline followed by the Stratum C soils. The underwater thickness of the Stratum B soils varies over the site between 3 and 8 metres.

The design parameters for each Stratum are presented in Table 1.

**Table 1 Design
Soil Parameters**

STRATUM	Unit Weight KN/m^3	Total Strength		Effective Strength		Compression Ratio (CR)	Coefficient of Consolidation, Cv (cmVsec)
		c (KPa)	0 (degrees)	c' (KPa)	0' (degrees)		
A - Fill	17.3	-	-	-	30	-	-
B - Soft Clay	15.7	10-18		-	-	**0.3-0.4**	0.006
C - Yorktown Deposit	19.7	160	34	0	38	-	-

3. Wharf Facility Design

The wharf facility consists of three main components:

1. The wharf structure and operations area
2. The container storage and stacking area (known as the backland area)
3. The dredged ship channel and turning basin

Figure 1 presents a cross section of the wharf structure and operations area with the ground improvement techniques needed to improve the soils. The wharf structure consists of a cast-in-place concrete deck supported on 508mm precast prestressed concrete piles. The piles, which are between 25 and 35 metres long, have a design capacity of between 118 and 150 metric tons. A steel sheet pile bulkhead is tied to the back of the wharf. The purpose of the bulkhead is to minimize the width of the pile supported deck while allowing dredging of the soil beneath the wharf at a slope of 3 horizontal to 1 vertical to the required depth of 15 metres in front of the wharf.

As shown in Figure 1, ground improvement was needed at the site; otherwise: (a) the soft silty clays of Stratum B and the hydraulic fills placed above them could not be dredged to the required 3 horizontal to 1 vertical slope under the wharf with an adequate safety factor, (b) the long term differential settlement of these compressible soils under the operating loads of 24 to 50 kPa would negatively impact the operation of the facility, (c) the long-term settlements would cause severe downdrag forces on the retaining structure and (d) the soft clays behind the wharf would generate large lateral earth pressures on the bulkhead. Since the compressible layer is present throughout the site and extends to depths as large as 13 metres, it was not practical nor economical to remove and replace these materials.

Ground improvement was also dictated by the short term construction requirements of the project. Although the wharf structure provides the lateral restraint for the bulkhead and the sheet pile bulkhead retain the soil and increase the overall stability, the required dredging would have to be completed prior to construction of the wharf and the bulkhead. Because the in situ shear strength of the fine grained soils is as low as 10 kPa, a very flat dredge slope would be required to maintain a minimum safety factor of 1.1 (construction case). This flat slope would increase the width of the wharf structure and would encroach on existing facilities and a sewer outfall pipe which could not be removed from the site. Ground improvement, was therefore, necessary to increase the shear strength of the soil so that the design slope can be achieved with an adequate safety factor against a slope stability failure during construction.

4. Ground Improvement Design

The ground improvement alternatives considered for the project were:

1. Preloading with prefabricated wick drains,
2. Removal of soft soils and replacement with a granular (sand) shear key,
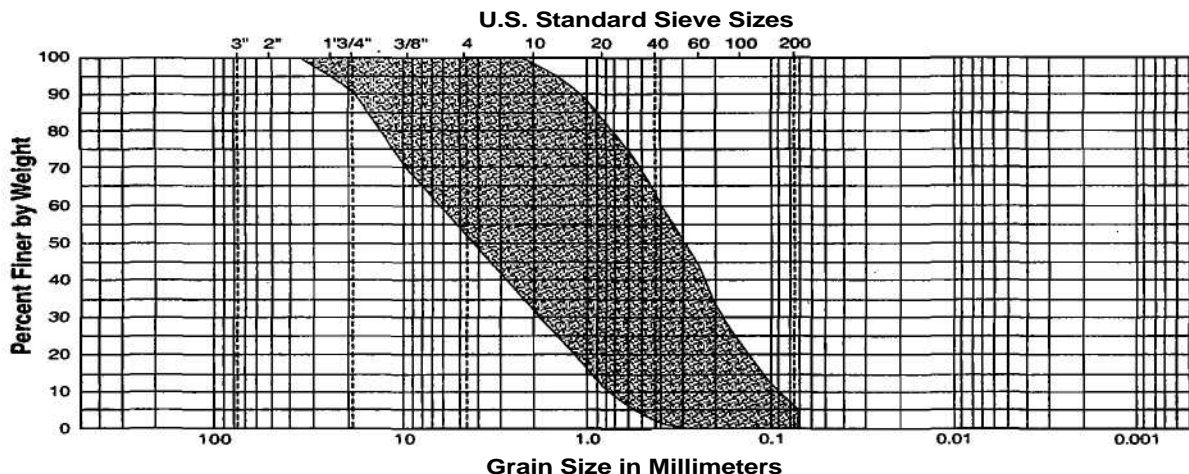3. Vibrocompaction.

Preloading of the soft clays was the most economical alternative. However, due to operational requirements, the position of the wharf structure was set such that a part of the constructed wharf would be over existing land and the other part over water. Since placing fill in the water was

prohibited by environmental restrictions, the surcharge load had to be placed on the land side of the proposed bulkhead at a minimum distance of 12 metres from the shore, dictated by the stability of the existing natural slope along the shore. This preloading configuration, however, was not adequate to achieve the minimum shear strength required for the stability of the dredged slope and the design of the retaining structure. To provide the required factor of safety of 1.3 against slope stability failure, a large enough section of the soft soil at both sides of the sheet pile bulkhead was replaced with sand forming a sand shear key. The width of the required sand shear key at the base was determined to be 18 metres, based on stability analyses.

The sand shear key material was placed in the configuration shown in Figure 1. As the material was placed underwater, it had a low relative density of 10 to 40 percent, as determined by a series of pre-densification borings and piezocone probes. The range of grain size distribution of the sand shear key material is presented in Figure 2. The design of the wharf required this material to have a minimum relative density of 70 percent to achieve the strength needed for the design of the bulkhead and to reduce post construction settlements. To achieve that, the underwater-placed material was densified by vibrocompaction. The design of the vibrocompaction system was based on a field trial performed at the beginning of construction.

Although the presence of the sand shear key ensured the stability of the dredged slope, preloading of the land side was still needed to provide the minimum shear strength needed for safe and economical excavation for constructing the sand shear key itself within the geometric constraints of the site. Preloading was also needed in the operations laydown area and backland area to reduce post construction settlement of the soft clays in those areas, including those left in place beneath the back slope of the sand shear key. To achieve the anticipated long-term settlement criteria of the compressible soils under the operating loads within the maximum preconsolidation period available for the project, prefabricated wick drains were specified in conjunction with preloading to accelerate consolidation.

Figure 2

U.S. Standard Sieve Sizes

Grain Size in Millimeters

The preloading and wick drain configurations are shown on Figure 1. A 3 to 5 metre high surcharge was needed with wick drains installed at 1.5 m centre-to-centre in a triangular array. The preloading and vertical drain program was designed so that the shear strength of the soft clay would be increased from 10 kPa to 24 kPa, and that over 90 percent of the long-term settlement would be completed within the 9 month construction period. The time-rate of settlement of the soil was calculated using a vertical drain analysis which takes into account vertical and radial drainage[1].

5. Construction Sequence

The sequence of construction established for the wharf structure was as follows: (1) install wick drains and preload the site for a period of 6 to 9 months using the configuration shown on Figure 1, monitor the consolidation and strength gain in the silty clay layer; (2) after 90 percent consolidation is completed, remove the surcharge and dredge to the design slope; (3) excavate on both sides of the sheet pile bulkhead location in the configuration of the sand shear key, and backfill the excavated area with clean sand; (4) drive the bearing piles of the wharf structure and construct its deck; (5) drive the

sheet pile bulkhead but delay its final connection to the back of the deck (6) backfill behind the sheet pile bulkhead; (7) perform vibrocompaction of the sand shear key materials behind the bulkhead; (8) connect the bulkhead to the deck and pave behind the wharf.

6. Construction Aspects and Performance Verification

The successful application of ground improvement depends to a great degree on the soil's response to the applied technique, and on the means and methods of construction employed by the contractor. Since there is always a certain level of uncertainty as to the success of a given treatment, a performance verification program is usually implemented during construction. Performance verification was particularly important at the NIT site since the construction was implemented in stages, with each stage dependent on the successful application of ground improvement in the previous stage. The construction aspects of the ground improvement techniques applied and their monitored performance are discussed in the following sections.
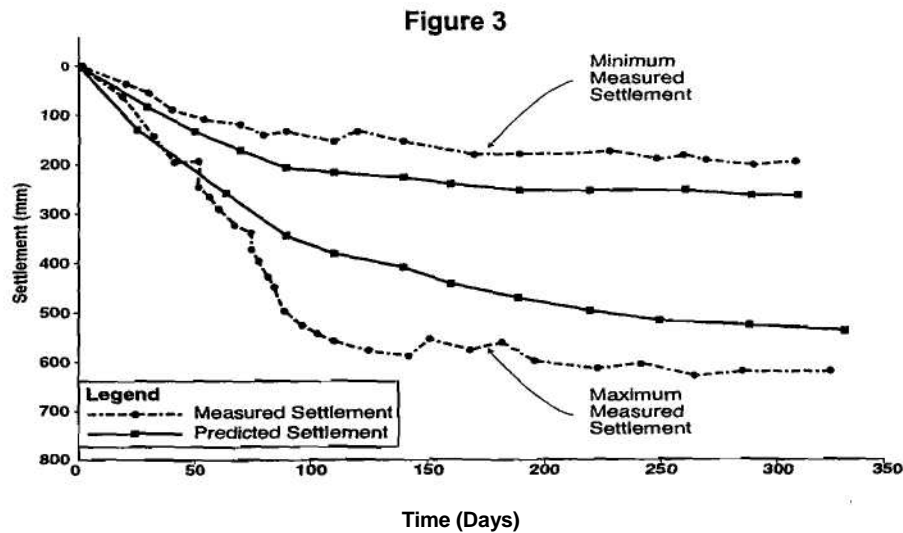
6.1 Preloading

Preloading with and without wick drains was the main technique used for the site. Approximately 75 percent of the 17.5 hectare site was preloaded with 3 to 5 metres of surcharge fill. Over 640,000 linear metres of wick drains were installed at the site. The length of the drains varied between 10 and 17 m. The wick drain brand used on the project was Alidrain, Type ST. The drain is 100 mm wide and approximately 3 mm thick and consists of a studded or grooved plastic water channel surrounded by a geotextile filter jacket. The wick drain installation rates averaged approximately 5,000 linear metres per day per wick drain installation rig.

The preload program was monitored with a comprehensive series of instrumentation. The main goals of the field instrumentation were to; (a) monitor the actual performance of the surcharge fill and subsoils to verify design assumptions, (b) provide warning signals for impending stability problems during filling, and (c) determine the actual time for surcharge removal.

In general, the instruments were grouped together and placed at different locations throughout the site. Within the fill area and along its centerline, they included settlement platforms, vertical extensometers and piezometers to evaluate the degree of consolidation and the resulting increase in the effective stress and the shear strength of the soil. The groups along the slopes were similarly used to evaluate consolidation and stress increase, but also to monitor the stability of the fill slopes. These groups consisted typically of settlement platforms, inclinometers, and vertical extensometers. The instrumentation data were collected and reported on a weekly basis during fill placement, and monthly thereafter.

Figure 3 illustrates time-settlement plots for the maximum and minimum settlements predicted during the design and those measured at some monitoring points in the wharf area. As shown, the measured settlement at the site was variable and did not quite agree with the predicted values. The difference is likely the result of the considerable variation in the thickness of the compressible soil throughout the site and that some undrained deformations that took place during the surcharge placement but were not accounted for in the settlement calculations. The actual settlement rates were also higher than those predicted, particularly for the first 150 days. This may be due to the presence of sand and silt lenses in the clay that may have facilitated drainage and/or the c_h/c_v value used in the analysis was too conservative. The inclinometers and piezometers monitoring the slopes did not show any sign of instability during preloading.

In addition to the geotechnical instrumentation and monitoring program, six borings were taken after the preload was in place in the wharf operations area to further evaluate the degree of consolidation and shear strength increase in the soft cohesive soils. A shear strength increase from about 10 kPa to 24 kPa was needed to ensure slope stability prior to dredging. The results of laboratory tests on undisturbed soil samples indicated undrained shear strength between 24 kPa and 75 kPa, thus, exceeding the required value of 24 kPa. The consolidation tests also indicated that the material was consolidated to more than the required preconsolidation stress. The results of the geotechnical instrumentation and the post preloading borings, therefore, verified that the preloading was effective and the ground improvement program achieved its objectives.

Figure 3

6.2 Sand shear key

To determine the exact limits of the unsuitable material to be removed for construction of the sand shear key, a program of borings and probes was undertaken at the site prior to dredging. This program consisted of ten borings and a series of 46 drive probes performed on a rectangular grid pattern. The drive probes consisted of a standard size split spoon sampler fitted with a cap at the tip to prevent the entrance of soil. The blows to drive the sampler with a 63 kilogram hammer dropping 760 mm were recorded for each 300 mm of penetration. The drive probes were driven approximately 17 metres apart and some were located adjacent to boring and cone penetration test (CPT) locations for correlation. In this way, the bottom of the shear key was established prior to dredging. Additional probes and small clam shell samples were also taken as the filling progressed to verify that the soft compressible materials were removed and not entrapped under or within the fill. The fill was placed with a clam shell bucket on the firm bottom of the sand shear key to force any remaining unsuitable soil away from the wharf and the bulkhead.

6.3 Vibrocompaction

Vibrocompaction was required if the sand shear key material exhibited lower relative density than the 70 percent value specified in the contract documents. The relative density of the underwater-placed material was determined by a series of 15 borings and 15 piezocone soundings. The relative density was determined from correlations with SPT-N values given by Gibbs and Holtz as referenced by Lacroix and Horn [2], and with CPT correlations developed by Robertson and Campanella [3]. The results of the investigation indicated considerable variations in the relative density ranging from about 10 to 70 percent with most values less than 40 percent. Therefore, vibrocompaction was needed to achieve the performance criteria of 70 percent relative density.
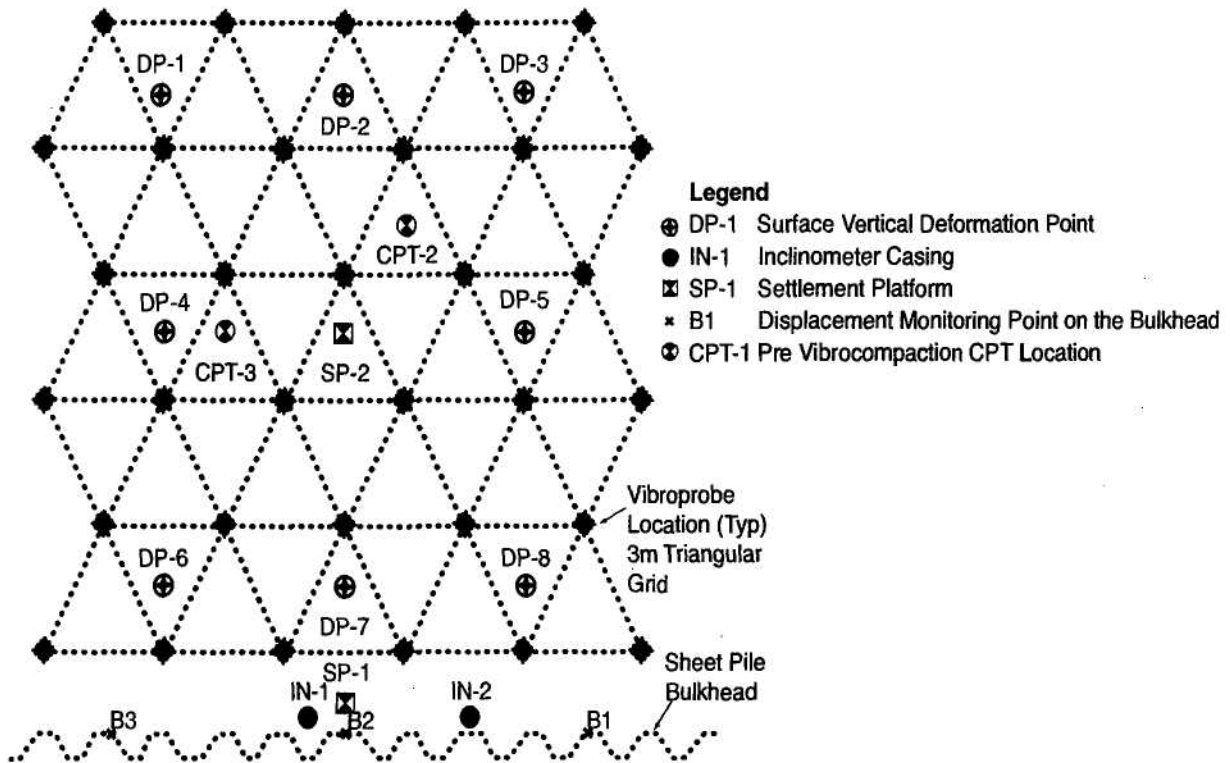
The vibrocompaction subcontractor was given the responsibility of selecting the methods, equipment and vibroprobe pattern and spacing to achieve the required degree of densification. In order to evaluate the proposed procedures and vibroprobe arrangements, the specifications required that a field trial be performed. The field trial was developed to answer the following questions:

1. Can the proposed methods and equipment achieve the required relative density?
2. What spacing and pattern of the vibroprobes are the most efficient and economical?
3. How close to the back of the bulkhead can vibrocompaction be performed without adversely impacting the bulkhead and the wharf structure?

The field trial included patterns with at least 20 probes at 3 or 4 metre spacing. The instrumentation included settlement points to evaluate the amount of settlement occurring and inclinometers to evaluate lateral movement of the soil and its impact on the bulkhead. In addition, piezocone soundings and borings were taken before and after vibrocompaction to evaluate the increase in relative density. The layout of the field trials and the associated instrumentation and piezocone soundings are presented in Figure 4.

Vibrocompaction was performed with 0.4 metre diameter electric vibroprobes. The free-hanging amplitude of the 100 hp vibrating machine was approximately 15 mm, operating at a frequency of 1,800 rpm. The vibroprobe was inserted into the ground using both water jetting and vibration. After the vibrator reached the required depth of improvement, sand was added from the ground surface and the vibrator was extracted at a rate of approximately 1 metre every 1.5 minutes.

Figure 4

The results of the field trials indicated that the proposed vibrocompaction scheme could achieve the desired degree of compaction with either spacing. The post vibrocompact design indicated that the initial relative density of less than 40 percent increased to greater than 70 percent after vibrocompaction. At certain depths, where the soil has high silt or fines content, the piezocone results did not show significant improvement. However, these soil pockets were limited in extent and were not anticipated to impact the overall performance of the backfill.

The settlement readings obtained during the field trials were also used to verify that the required increase in relative density was achieved after vibrocompaction. Estimates of the initial void ratio of the soil determined from the pre-vibrocompaction borings and piezocone soundings were used in a conventional settlement analysis to calculate the required decrease in void ratio and corresponding settlement needed to achieve the specified relative density of 70 percent. Based on this approach, it was estimated that the settlement required to achieve the relative density of 70 percent was about 460 mm. The settlement readings indicated that the ground surface at the 3 and 4 metre spaced probes had settled approximately 615 and 540 mm respectively. Therefore, the results of the settlement readings also support the conclusion that the vibrocompaction achieved its objective. Based on these results, the Contractor elected to use a 3 metre spacing of the vibroprobes for the production treatment.

The inclinometer data measured near the bulkhead indicated that if the vibroprobes were kept more than 3 metres from the inclinometers, the horizontal deflection was less than 10 mm. As the probe got within 2 metres of the inclinometer the deformations increased significantly to about 40 to 50 mm. Therefore, it was concluded that a minimum distance of 3 metres should be maintained between the vibrocompaction probes and the bulkhead.

The vibrocompaction for the project included the treatment of approximately 145,000 m^3 of materials with probe depths of 8 to 14 metres. The production phase lasted three months. Verification testing was performed throughout this period by conducting piezocone soundings in treated areas a minimum of 14 days after treatment. The results of these soundings indicated successful treatment of the soil in most cases. The ageing phenomenon (Mitchell and Solymar, 1984) which signifies an increase in strength and deformation modulus with time after vibrocompaction was investigated by testing the densified soil immediately after compaction and retesting the same area after approximately 20 days. The test results showed an increase in the relative density from 15 to 30 percent which illustrates that the ageing phenomenon is valid. The retesting indicated that the material had achieved the required degree of densification after a waiting period of 20 days.

7. Conclusions

1. The ground improvement techniques applied at the site were successful in achieving their objectives. The preloading increased the shear strength of the soft clay by 2.5 times, and the wick drains allowed the 90 percent consolidation of the site to be completed within nine months. The sand shear key allowed dredging of the slope safely under the bulkhead, and vibrocompaction of the underwater-placed fill increased its relative density from 40 to over 70 percent which, in turn, improved the stability of the wharf structure and reduced post construction settlements.
2. Operational, environmental and construction-related factors necessitated the use of a combination of ground improvement techniques.
3. Field trials, verification testing and geotechnical instrumentation facilitated the evaluation of   . the effectiveness of the ground improvement techniques used, and selection of the most appropriate vibrocompaction probes pattern and spacing.
4. A vibroprobe spacing of 3 metres applied on a triangular pattern was sufficient to achieve the required 70 percent relative density.
5. A minimum distance of 3 metres was needed between the vibroprobe and any existing structure to safeguard against any negative effect from the compaction process on the structure.
6. The ageing phenomenon increased the relative density of the compacted soil by an average of about 20 percent.
7. The combined use of the ground improvement techniques provided substantial economical benefits for the project.

References

[1] Hansbo, S. (1979) Consolidation of clay by band-shaped prefabricated drains, Ground
    Engineering, 12,5. [2] Lacroix, Y. and Horn, H.M. (1973) Direct Determination and Indirect Evaluation of Relative
    Density and Earthwork Construction Projects, ASTM, STP 523 [3] Robertson, P.K. and Campanella, R.G. (1983) Interpretation of Cone Penetration Tests, Part
    1: Sand, Canadian Geotechnical Journal, Vol. 20, No. 4. [4] Mitchell, J.K. and Solymar, Z.V. (1984) Time dependent strength gain in freshly deposited or
    densified sand, Journal of Geotechnical Engineering Division, ASCE, 110, No. 11.