Seaview Marina, Geotechnical Design of Breakwaters

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1 INTRODUCTION

This paper describes the geotechnical investigation, design and construction monitoring of the breakwaters for the Seaview Marina, Wellington.

The breakwaters are of rubble mound type, 630 m in length and rise up to 8.5 m above seabed. They are founded on 14 m thickness of very soft and soft alluvial silts.

2 BACKGROUND

The Seaview Marina is located in the north east corner of Wellington Harbour (refer to Figure 1). The marina will provide 480 berths and 80 fore/aft moorings for pleasure craft. The site is exposed to waves from the south within the harbour and also from Cook Strait. Design wave heights of 2.5 m were predicted and thus breakwaters were required to protect craft moored in the marina.

Construction of the breakwaters commenced late in 1989 and they were substantially complete by April 1991.

3 GEOLOGY

Basement rock in the Wellington region comprises highly disected greywacke and argillite which has been eroded from a former peneplain and disrupted by faulting. The Seaview Marina is located at the foot of the Hutt Valley which is an alluvial filled valley extending north from Wellington Harbour and bounded on both sides by steep hills of the basement rock. (Refer to Figure 1).

In the lower Hutt Valley the alluvial deposits include a layer of relatively permeable sands and gravels overlain by silts and clays. The sands and gravels outcrop in the bed of the Hutt River part way up the valley and the overlying silts and clays provide confinement downstream resulting in the development of artesian pressures within the sands and gravels (Hutt Valley artesian aquifer).

At the marina site the Hutt Valley artesian aquifer is located approximately 14 m below sea bed level and has a piezometric head of approximately 2 m above mean sea level. The overlying aquiclude comprises soft and very soft silt which extends to the seabed level.

Wellington Harbour Board seabed sounding information dating from 1904 to 1985 indicates continuing deposition of silt. Some 2 m to 3 m of deposition occurred across the site from 1904 to the mid-1960's. This period related to the time of early land development projects within the Hutt River's catchment and the formation of the Seaview reclamations adjacent to the marina site during the 1950's. During the past 20 years further deposition has been minimal (less than 10 mm per year average).

The very soft nature of these recent alluvial deposits was a critical factor in the design of the breakwaters.

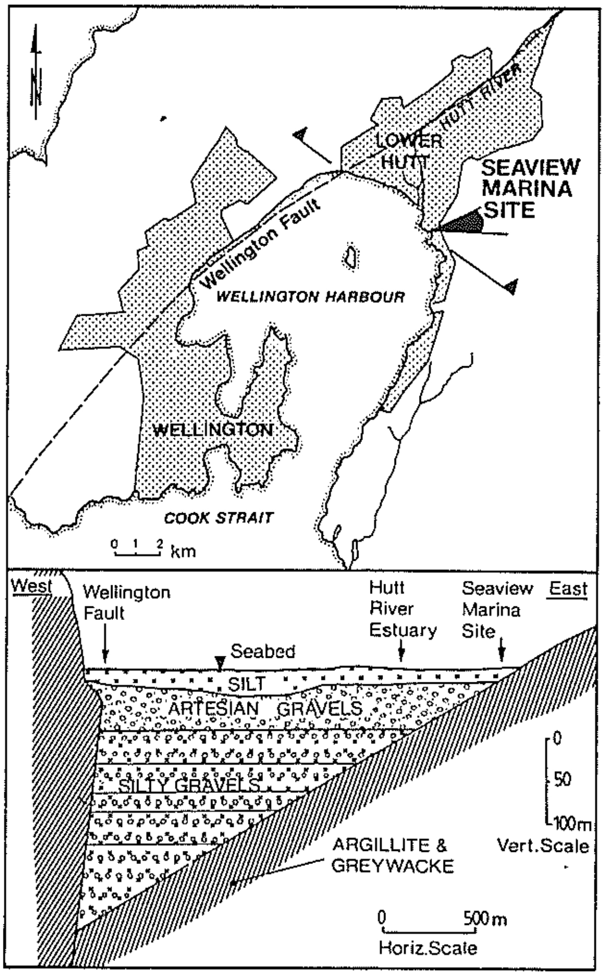

Figure 1: LOCALITY & GEOLOGICAL CROSS SECTION
(After Stevens 1974)

4 SITE INVESTIGATION

4.1 Previous Investigations

Information was available from a number of previous land based investigations and water bores undertaken to the north of the site in the Seaview industrial area, and from a marine based investigation undertaken in 1969 to the south and south west of the site for a wharf development.

Information from these previous investigations was reviewed and allowed the upper surface level of Hutt Valley aquifer to be approximately defined across the site.

4.2 Specific Invesitgations

A site investigation was formulated to investigate specifically the overlying soft alluvial silts. The investigation comprised four land based boreholes, nine marine based boreholes, marine based probing and seabed surface sampling.

Drilling was confined to the alluvial silt and stopped well short of the predicted aquifer level. Had it been anticipated to drill near or into the aquifer the controlling authority would have required special techniques including double casing to protect this valuable water resource.

A truck mounted rotary drilling rig was used for both marine and land based drilling. The marine based drilling platform comprised a 17 m by 6 m self propelled barge which was held over the drilling locations with four corner anchors.

With the help of unusually calm November weather the marine based work was successfully and efficiently completed.

Problems were encountered as a result of the casing sinking up to 1.5 m into the seabed under its own weight. This was resolved by welding an 800 mm square flange plate, 500 mm up from the bottom of the casing, to act as a foot.

Insitu testing and sampling included 75 mm insitu shear vane tests, Standard Penetration Tests and 100 mm diameter push tubes. Good recovery was achieved in the silts, however, some sample loss was experienced in interbedded silts and sands encountered closer to shore and at shallower depths. A piston sampler was employed in these materials and recoveries of 90% to 100% were achieved. The boreholes were advanced by washboring between the testing and sampling locations.

Laboratory testing of selected samples included classification tests, consolidation tests and triaxial compression tests.

4.3 Results of the Investigations

The soil profile along the majority of the breakwater length was found to comprise alluvial silt extending from seabed level to 20 m below mean sea level. Seabed level typically varied between 3 m and 6 m below mean sea level. Previous investigations had indicated that the silt was underlain by dense sandy gravels of the Hutt Valley aquifer.

The properties of the alluvial silt, as determined by the insitu and laboratory testing, are summarised by Table 1 and Figure 2. The silt may be described as soft and very soft, with some clay, and with a trace of shell fragments in parts. It is normally consolidated and of moderate to high plasticity. The upper portion of the layer was noted to include a minor fine sand

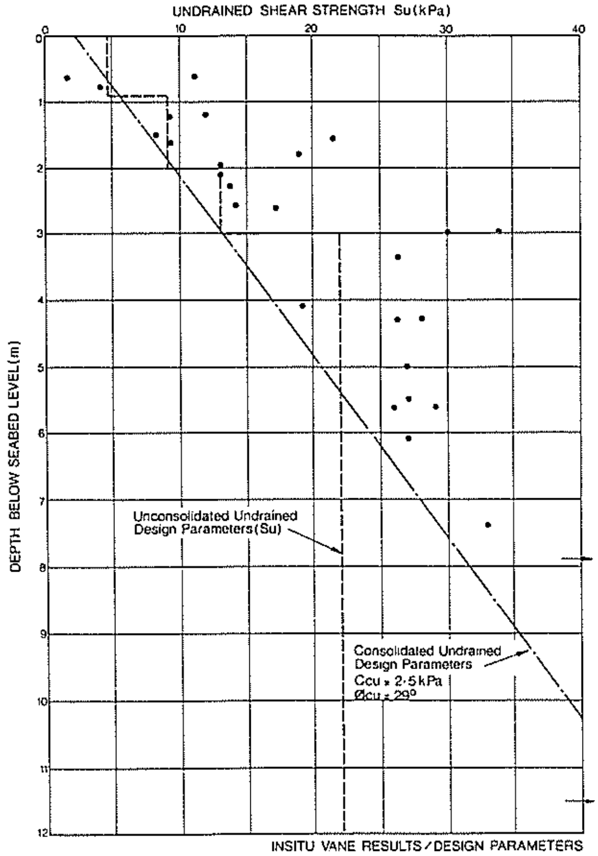

Figure 2 :UNDRAINED SHEAR STRENGTH

TABLE 1: ALLUVIAL SILT TEST RESULTS

Insitu Test Results	Average Laboratory Test Results					
Undrained Shear Strength "S_u" (kPa)	Atterberg Limits (%)	Natural Water Content (%)	Bulk Density (T/m³)	Dry Density (T/m³)	Triaxial Compression c' (kPa) Ø' (Deg)	Consolidation $\frac{Cc}{1+eo}$ C_v (m²/yr)
S_u = 8-30 Refer Figure 2	LL = 67 PL = 38 PI = 29	55	1.60	1.05	C' = 0 Ø' = 37°	Normally Consolidated $\frac{Cc}{1+eo} = 0.18$ $C_v = 10$

content which reduced with depth. The fabric of extruded samples of the silt was studied. There was no evidence of sand beds or partings or other preferential paths for drainage.

Samples tested by triaxial compression were recovered from the upper part of the layer and had sand contents of approximately 20 %. The relatively high internal angles of friction ($\phi' = 37°$) recorded by these tests are attributed to this sand content. In general the silt was found to have only a trace of sand content and thus lower ϕ' values than those recorded above are expected.

5 DESIGN

5.1 Form of the Breakwater

The selected rubble mound form of breakwater (refer to Figure 3) comprises an embankment of low cost weathered greywacke rock fill obtained from local quarries. This core is protected from wave attack by two capping layers of sound rock boulders (armour rock). An intermediate filter layer (filter rock) is provided to avoid piping of the core.

Alternative forms of breakwaters (including floating units, pile screens and vertical seawalls) were discounted in the feasibility stages on technical and economic grounds.

5.2 Design Criteria

For any geotechnical design the aim of the design process is to establish a system that will reduce the risk of failure to an acceptably low value. This is normally done by ensuring that the design has factors of safety against various potential failure modes of greater than appropriate predetermined values.

In the case of the breakwater these risks are due to the possibility of encountering ground conditions weaker than those assumed in design , the risk of extreme environmental conditions (eg severe earthquakes) and due to the limitations of theoretical analysis.

A factor safety of 1.5 (Meyerhof, 1970) is the normal value adopted for long-term earth slopes where failure is not expected to pose a real hazard to human life. A conservative estimate of soil strength parameters is normally taken on the basis of investigation data.

In the short term (i.e. the temporary conditions of high pore pressures and consequently low strengths of the underlying soils that will arise during construction of the breakwater and for a period of months afterwards) a lower factor of safety of 1.3 is generally accepted.

These generally accepted minimum factors of safety of 1.5 for the long term case and 1.3 in the short term have been adopted as the criteria for the geotechnical design of the breakwater. Soil strength parameters based on the insitu shear vane results have been assumed as shown by Figure 2.

5.3 Soil Strength Parameters

5.3.1 Alluvial silts

The rates of displacement within the alluvial silt during a failure are expected to be such that significant pore pressure dissipation during any stability failure would be considered unlikely (Ladd,1991). Undrained conditions would be expected to prevail during a failure and thus undrained soil strength parameters should be applied in the stability analyses.

Three design cases were considered in the design. These design cases and the assumed soil strength parameters are discussed below.

(a) Short Term Unconsolidated Undrained Case

This design case relates to the conditions at the instant the load of the breakwater is imposed on the weak underlying soils, i.e. without dissipation of excess pore water pressures. Undrained soil strength parameters (S_u), as determined directly from the insitu shear vane tests, were assumed. Figure 2 shows the test results and the assumed design parameters.

A minimum factor of safety of 1.3 was the design criteria.

(b) Short Term Partially Consolidated Undrained Case

This design case relates to the conditions during construction and for a short period after. Ten percent of the excess pore water pressures induced by the weight of the breakwater were assumed to have dissipated immediately. This is considered to be a conservative estimate of the dissipation which will occur during construction. Partially consolidated undrained soil strength parameters (C_u) as determined by the relationship $C_u = C_{cu} + \sigma'_v \operatorname{Tan} \phi_{cu}$ were assumed. σ'_v is the effective overburden at the level of the potential failure surface following the partial consolidation. The consolidated undrained parameters C_{cu} and ϕ_{cu} were determined on the basis of the insitu shear vane results and the calculated effective

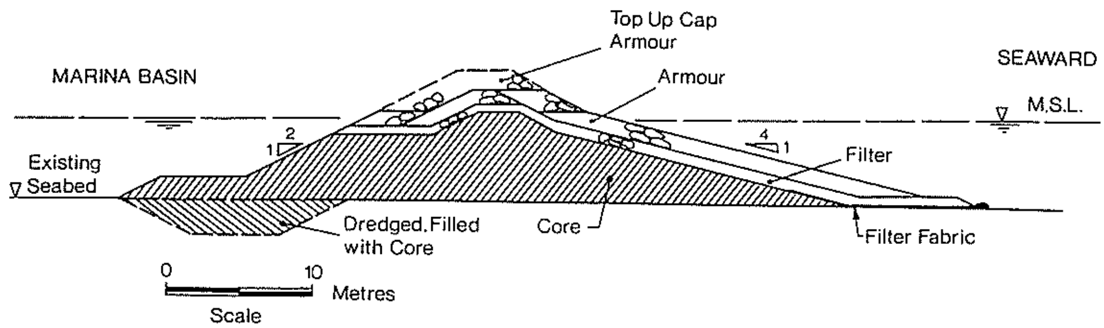

Figure 3: BREAKWATER CROSS SECTION

overburden at the level of each test. These design parameters are shown on Figure 2.

A minimum factor of safety of 1.3 was adopted.

(c) Long Term Consolidated Undrained Case

This design case relates to the conditions once dissipation of excess pore water pressures is virtually complete. These conditions were estimated to occur some two years after the completion of construction. The consolidated undrained design parameters were applied and a factor of safety of 1.5 or greater was considered acceptable.

The factor of safety requirement of 1.3 for the short term unconsolidated and partially consolidated undrained cases are somewhat conservative when compared to the actual situation resulting from construction. For the design, the breakwater is assumed to be constructed virtually instantaneously, whereas in reality construction work will take several weeks to complete any individual breakwater section.

5.3.2 Breakwater construction materials

The construction materials of filter and armour rock and to a lesser extent the core are free draining materials. Drained soil strength parameters of $C' = O$ and $\phi' = 35°$ and effective stress analyses were assumed for these materials.

5.4 Stability Analysis

Analyses were carried out using the computer programme SLIPSYST which is based on the theory of Bishop's simplified method of slices. This programme has the facility to search for the critical circular failure surface. Figure 4 shows these critical failure surfaces with associated computed factors of safety for the selected breakwater profile and the design cases discussed above. These critical failure surfaces are discussed further in Section 5.7.

During analysis each profile was modified with the aim of producing an economic design while meeting the selected factor of safety requirements.

5.5 Options Considered

A variety of Breakwater profiles and construction technique options were investigated to determine the most satisfactory and cost effective solution.

Initially a profile comprising a simple continuous batter was considered. A batter slope of 1 vertical to 4 horizontal was found to meet the design criteria. Profiles with a single bench and locally steeper batter slopes were considered. The benched profiles offered reduced core volumes but the locally steep batter slopes required larger armour units to provide protection from wave attack. Due to the high cost of the armour, particularly for larger size units, and the relatively low cost of core, the overall cost of the benched profiles were estimated to be greater than that for the continuous batter.

The continuous 1 vertical to 4 horizontal batter slope offered an economic solution for the seaward batter. However, this profile was undesirable on the internal batter because it encroached on useable water space. Alternatives were considered with the aim of steepening the batter slope. Ground improvements in the form of vertical wick drains coupled with staged construction offered a feasible solution, as did geogrid reinforcement of the base of the breakwater. However the cost of these techniques proved to be prohibitive. A profile including a trench backfilled with core fill at the toe of a 1 vertical to 2 horizontal batter was then examined (refer to Figure 3). This profile offered a cost effective solution and was selected for the internal batter.

5.6 Consolidation Settlement

Settlement due to consolidation of the 14 m thick alluvial silt layer was calculated on the basis of the parameters determined by the site investigation and reported in Table 1.

These calculated consolidation settlements were 1.5 metres where the proposed breakwater is at its maximum height of 8.5 metres above seabed (mean water depth 6.5 metres) and 800 mm where the breakwater stands 5.0 metres above seabed (mean water depth 3.0 metres). Seventy percent of this consolidation was predicted to be complete within approximately 2 years.

To compensate for the consolidation settlements, it is proposed to provide a cap of further armour material on top of the breakwater 18 months to 2 years after its initial construction. This armour cap detail is shown on Figure 3.

5.7 Detailed Stability Analysis of the Selected Profile

Figure 4 shows the critical circular failure surfaces, as determined by application of the SLIPSYST computer programme for the selected breakwater profile.

For the initial construction case factors of safety of greater than 1.3 were calculated assuming the unconsolidated and partially consolidated undrained conditions.

At the time the top up cap of armour material is placed the excess pore water pressures generated in the silt by the loads imposed during initial construction have been assumed to have dissipated to a degree of 75 % (at the level of the critical failure surfaces). The placing of the cap can be expected to generate additional excess pore water pressures. Figure 4 (top up stage) indicates calculated factors of safety under this assumed excess pore water pressure, immediately following the placing of the top up cap. Undrained strengths expected with these partially consolidated conditions were assumed. It can be seen that factors of safety of 1.9 have been calculated.

In the long term the excess pore water pressures will dissipate increasing the factor of safety.

6 CONSTRUCTION

6.1 Construction Materials

Quarry strippings from the local quarries provided suitable low cost core rock comprising moderately to highly weathered, highly fractured Greywacke. These quarries were also able to supply filter rock to meet the quality and grading requirements. However a suitable local supply of armour rock was not found. The tectonic history of the Wellington region has left the local rock highly fractured making it difficult to supply the required 500 kg and 950 kg sound rock units. Consequently marble armour was obtained from Collingwood (north western corner of the South Island) and barged a distance of 200 km across Cook Strait.

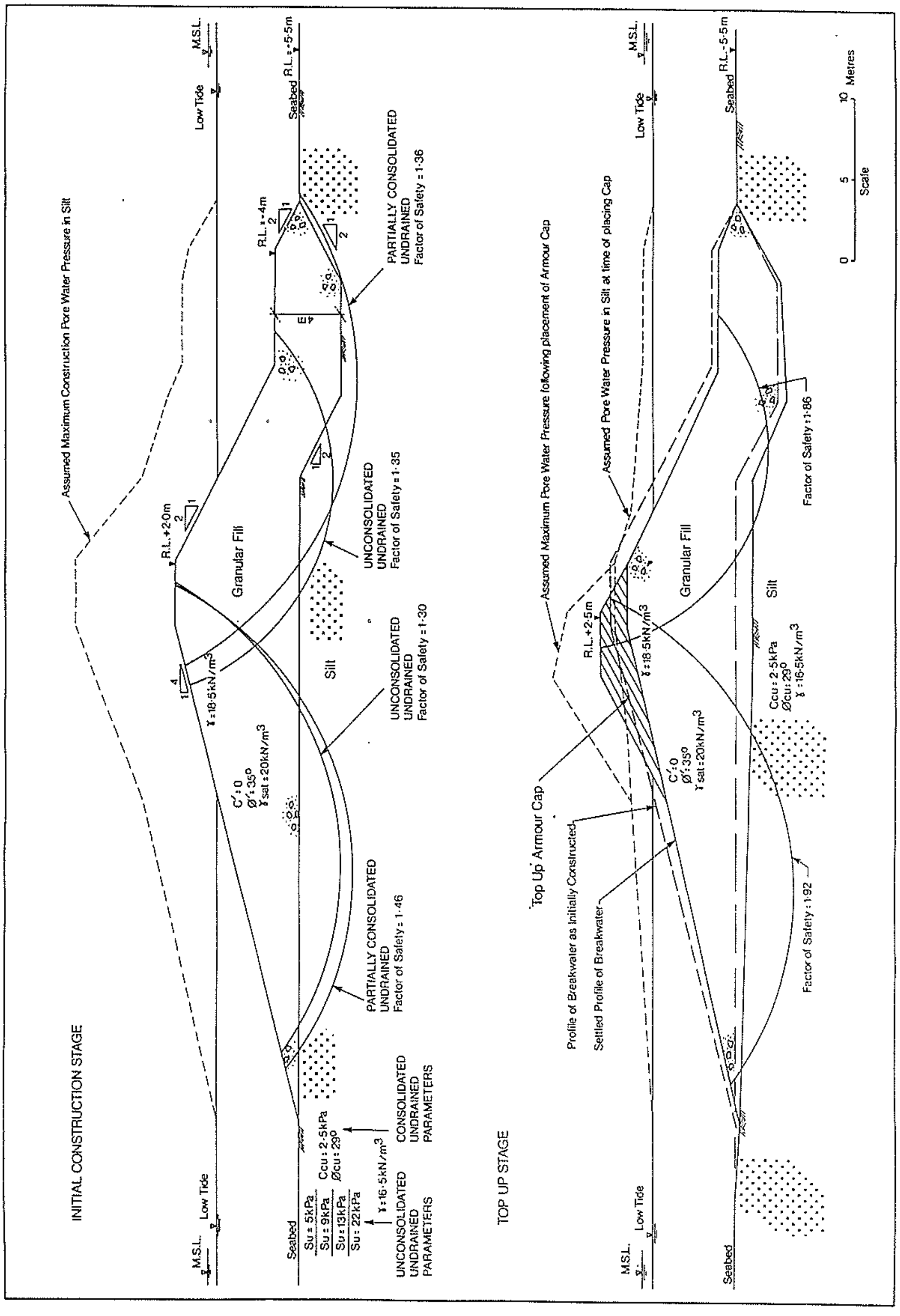

Figure 4 : STABILITY ANALYSIS

6.2 Construction Stability

The initial lift of core rock placement was limited to 1.5 m in height and subsequent lifts to 2 m in height to assure stable profiles during construction. This required marine based construction including bottom dumping barges.

6.3 Settlement and Pore Pressure Monitoring

Six settlement plates and pneumatic piezometers were installed along the length of the breakwaters. A settlement plate comprised a length of 100 mm diameter thick walled steel pipe with a 1.5 m x 1.5 m steel plate welded to the pipe 3 m from one end. The 3 m end of the pipe was pushed into the seabed until the steel plate was level with the surface. The other end of the pipe extended above water level and was monitored for changes in level during and after construction. The pneumatic piezometers were installed through the steel pipe down to a mid height level in the layer of alluvial silt.

The prime purpose of the monitoring was to provide information on the rate and amount of consolidation settlements. This information was required to confirm the appropriate time for placing the top up cap. Sufficient consolidation of the alluvial silt is required before the cap can be placed to ensure the stability of the breakwater during the top up construction work and to ensure that remaining settlements can be adequately predicted and the thickness of the cap can be set to provide a breakwater crest at the required level of 2 m above mean sea level after consolidation has occurred.

Consolidation settlement is following that predicted very closely both with respect to magnitude and rate (refer to Figure 5). Immediate settlements which occured in addition to the consolidation settlement have been deducted in the presentation of this data. The monitored pore pressures have been erratic.

The accurate prediction of the rate of consolidation settlement (and hence pore pressure dissipation) has provided confidence as to the stability of the embankments.

7 CONCLUSIONS

The recently constructed Seaview Marina breakwaters comprise rock fill embankments rising up to 8.5 m above seabed level founded on a 14 m thick layer of very soft and soft normally consolidated alluvial silt. The design of the breakwaters was based on traditional slip circle analyses and undrained soil strength parameters determined by insitu shear vane tests. Consolidation settlement of the breakwaters is closely following that predicted from one dimensional consolidation theory. Monitored staged construction was successfully applied.

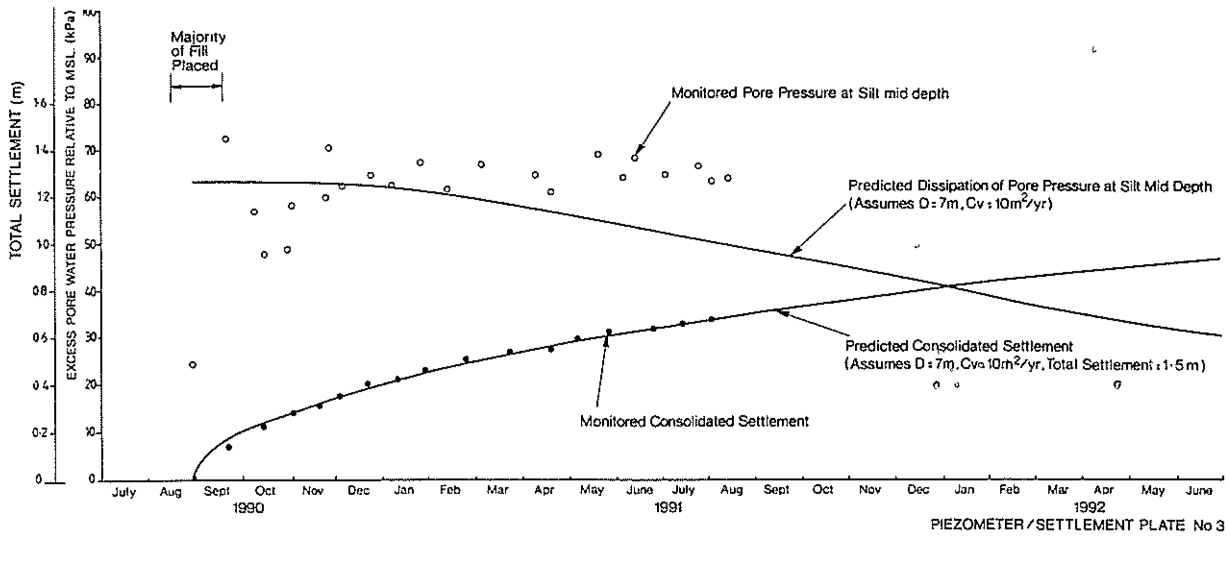

Figure 5: CONSOLIDATION SETTLEMENT & PORE PRESSURE MONITORING

ACKNOWLEDGEMENTS

The Seaview Marina project was initially undertaken by the Wellington Harbour Board. The Lower Hutt City Council took over the project after the Harbour Board was dissolved in 1989. The author acknowledges and thanks Lower Hutt City Council for approval to publish this paper.

Beca Carter Hollings & Ferner Ltd have been the principal consultants through all stages of planning, investigation, design and construction of the marina. Works Construction Ltd have been the main contractor for construction of the breakwaters and reclamation areas.

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