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GEOTECHNICAL INVESTIGATIONS ASSOCIATED WITH THE AXIS FERGUSSON EXPANSION

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Abstract:

Ports of Auckland Ltd (POAL) are upgrading Axis Fergusson Terminal to increase container handling capacity in order to meet a projected increase in trade. The upgrade includes construction of a piled wharf 320m long, mooring dolphins, 1000m length of perimeter bunds and a reclamation which will be 9.4 hectares in area and will contain 1.5 million cubic metres of fill.

The existing seabed lies between 1m and 12m below Chart Datum, with the surface of the reclamation at around 5.4m above chart datum, resulting in fill depths of up to 18m. The site is located in Auckland, New Zealand in the Waitemata Harbour which is a drowned river valley system and which has significant depths of weak sediments. The existing terminal is constructed over the top of an old remnant ridge. The proposed reclamation is located to the east of the ridge over a valley with thick layers of weak sediments.

The main geotechnical design issues associated with this development include stability (static and seismic) of the 18m high bunds and reclamation on weak sediments, settlement of the reclamation and bunds, suitability of reclamation fill materials to obtain a reclamation of relatively low compressibility, reuse of marine dredgings, and piling options to support the wharf structure.

Geotechnical investigations have been undertaken at various stages of the project from prefeasibility to detailed design in order to obtain information about the underlying soil conditions. Investigations comprised a series of machine boreholes and Geonor in situ large vane testing undertaken from a barge in the harbour and laboratory testing.

This paper describes the geotechnical design issues associated with the development, the extensive geotechnical investigations to address these design issues, and summarises the results of these investigations.

1 INTRODUCTION

POAL propose to upgrade the capacity of the existing Axis Fergusson Terminal by expanding the reclamation, extending the existing wharf, and constructing a new wharf.

The upgrade is proposed to involve construction of a 320m long wharf (concrete piled flat slab structure) adjacent to the northern edge of the existing reclamation, mooring dolphins, perimeter bunds and a reclamation of approximately 9.4 hectares in area located to the south of the proposed new berth and east of the existing terminal. The reclamation bunds and quay structure will be constructed to a level of about 5.4m above Chart Datum. (+ 5.4CD).

The piled wharf will be required to support container cranes. The reclamation will be used for the storage of containers stacked up to 4 high in the future and will be trafficked by heavy duty vehicles such as straddle carriers.

Dredging of the seabed to the north and east of the existing terminal is proposed to provide the required navigational water depth. Construction of the bunds will also require dredging to remove unsuitable soils from below the bund and allow a keyed toe to be formed. The proposed dredged level along the northern edge of the proposed wharf for the ship berth is -13.5CD with an allowance to dredge to -14.0CD in the future.

2 GEOLOGY

The site is located in Auckland, New Zealand in the Waitemata Harbour (Refer Figure 1).

The Waitemata Harbour is a drowned river valley system. During previous periods of glaciation (i.e. low sea levels) the landform was eroded, forming valleys in the weathered Waitemata Group rocks. As the sea levels subsequently rose, the valleys became submerged and infilled with sediments. These infill materials typically comprise very weak to stiff fine grained materials i.e. sands, silts and clays.

The Geological Map of the Auckland area (Sheet R11, Scale 1:50000) indicates that the underlying rock is the Waitemata Group sandstone and siltstone of East Coast Bays formation. This formation is expected to also contain mudstone with occasional lenses of coarser cemented grit (i.e. "Parnell Grit").

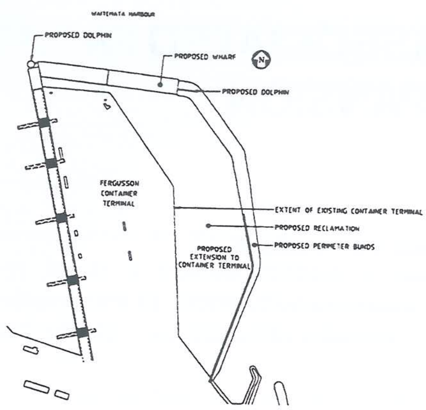

Figure 1 - Site Layout Plan

3 GEOTECHNICAL DESIGN ISSUES

The key geotechnical design issues for this development include:

a) overall stability of the bunds and reclamation;
b) settlement of the bunds and reclamation;
c) suitability of various reclamation fill materials;
d) piling to support the wharf structure; and
e) seismicity

These issues are briefly described in turn below.

*a) **Overall Stability of the Bunds and Reclamation Fill***
The bunds will act as a containment structure to hold the reclamation fill. Two modes of failure will need to be considered in design of the bunds:

- Shallow surface slumping of the bund materials, and
- Deep seated failure through the underlying soils.

The shallow failures are mainly a function of the bund material while the deep seated failures are mainly a function of the strength of the underlying soils. The short term (i.e. during and immediately following construction) stability of the bund is likely to be critical and can be managed by staged construction, limiting surface loadings and sizing of the toe key or improvement of the underlying soils. For design of the bunds it is necessary to define the shear strength of the underlying soils and the depth and extent of very soft soil layers.

*b) **Settlement of the Bunds and Reclamation***
Weak sediments underlying the site will consolidate and contribute to settlement of the reclamation, bunds and structures founded above these materials. Settlement of the reclamation and bund will be due to consolidation of the underlying soils and of the reclamation fill under the weight of the fill and applied (container stacking) loads. Design of the surface gradings, pavements, drainage and other structures will need to take into account predicted settlements. Geotechnical investigations are necessary to define the underlying soil types and consolidation characteristics in order to estimate the magnitude and rate of potential settlements for design.

*c) **Suitability of Various Reclamation Fill Materials***
The majority of the reclamation and bund fill will be placed below water, and therefore will not be compacted. The selected fill materials must however be of low compressibility so as not to add to settlements that will occur due to consolidation of underlying soils and must not be susceptible to liquefaction during earthquake shaking. Selected suitable fill types are likely to be either granular in nature or stabilised mud.

Dredging of marine sediments will be required during construction to remove unsuitable soils and obtain the required berth depth. Material will also be made available for maintenance dredgings around the port. These materials have been found to form a high strength fill of low compressibility when mixed with cement. Cement stabilised marine dredgings (named "mudcrete") has been used successfully as a high strength structural fill material on other local marine projects. The cement has also been shown to "lock up" contaminants within the marine dredgings.

*d) **Piling to Support the Wharf Structure***
Piles will need to support the quay structure, container cranes containers and vehicles. Piling options including both driven and bored piles will be evaluated to obtain an economic solution. While the properties of Waitemata Group rock are relatively well understood, geotechnical investigation is required to determine the location of the rock surface for founding the piles.

*e) **Seismicity***
Seismic hazard is an important aspect of the design of structures in New Zealand which lies on tectonic plate boundaries. For this project seismic effects must be considered in the design of the wharf structure, bunds and reclamation. A site specific seismic hazard assessment has been undertaken to determine the seismic design parameters for this site. Geotechnical investigations were considered necessary to determine the type and strength of the soils to allow an assessment of the performance of these soils under seismic loads and their susceptibility to liquefaction.

A series of geotechnical investigations has been targeted and undertaken to address the various design issues described above. These investigations are described in Section 4.

4 GEOTECHNICAL INVESTIGATIONS

Geotechnical investigations have been undertaken at various stages of the overall development of Fergusson Terminal and in particular to resolve the design issues related to this project. In 1993 preliminary site investigations, comprising machine boreholes, were conducted in the harbour adjacent to the two main container terminals as part of an overall development plan for the wharves and reclamation. In 1997 a more detailed geotechnical investigation comprising machine boreholes, limited large shear vane testing and laboratory testing was undertaken to determine subsoil conditions for designing the extension to the Fergusson Container Terminal and obtaining environmental approvals (i.e. Resource Consents). In 1998 large in situ shear vane testing was carried out along the eastern side of the proposed reclamation, and several machine boreholes were drilled through the existing reclamation for detailed design of the bunds. In situ large shear vane testing was performed to obtain a direct measurement of the shear strength of the weak marine soils for design of the bunds. The locations of the investigations are shown on Figure 2.

The investigations have typically been carried out over the water from a barge, and as such were limited to periods of calm water. At times the drilling operation had to contend with water currents of 2 knots when drilling was undertaken in the main harbour channel. The machine boreholes were advanced using open barrel sampling in soil and triple tube coring in the Waitemata Group rock. Undisturbed samples were taken in tubes as required for selected laboratory testing, and standard penetrometer testing was undertaken at 1.5m centres through the length of the borehole. Pilcon vane shear strength tests were undertaken on core in the open barrel. Large shear vane test holes were carried out in sections at 50m centres along the length of the bund. Each section comprised 3 test holes each around 25m apart across the width of the bund. Shear vane tests were undertaken at 1m intervals in each test hole until a shear vane strength of 80kPa or greater was recorded. A Geonor vane was used with a diameter of around 50mm and a height of 100mm. The purpose of this testing was to obtain a profile of the shear strength of the soil in sections across the width of the bund and with depth.

Laboratory testing was undertaken on a range of soil samples obtained during the investigation. The laboratory testing comprised soil classification tests such as natural moisture content and Atterberg limits, gradings, consolidation tests, consolidated undrained triaxial tests and unconfined compressive strength tests.

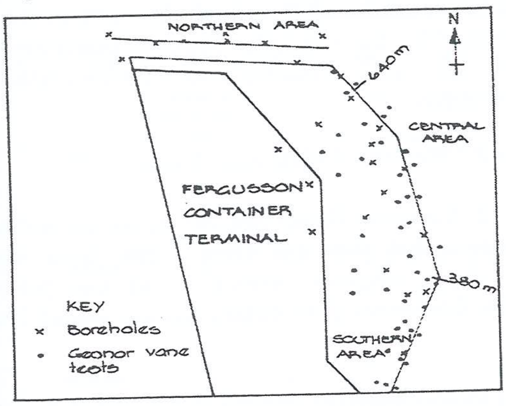

Figure 2 - Location of Investigations

5 SOIL PROFILE

In summary, the foundation conditions encountered during these investigations typically comprised weak marine sediments overlying residual soils of the Waitemata Group (i.e. sands, silts and clays) grading to interbedded layers of Waitemata Group sandstone and siltstone. Figures 7 and 8 show schematically the soil profile encountered along the northern and eastern edges.

The results of these investigations indicated that Fergusson Terminal was built along the top of a ridge and that the sandstone/siltstone surface generally dipped gently northwards. A gully has been identified along the eastern side of Fergusson Terminal. Contours of the sandstone/siltstone surface have been interpolated from historical and recent site investigation results and are shown on Figure 3.

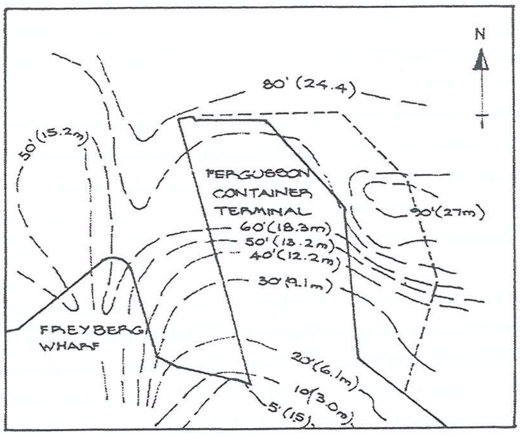

Figure 3 : Contours of Waitemata Group Rock

The relatively weak sediments have been divided into two categories: very weak Holcene sediments (i.e. deposited the last 10,000 years), overlying stronger Pleistocene sediments (i.e. deposited within the last 1.6 million years). The categorisation between the two layers was assessed based on the shear strength of the soil, the natural moisture content and the plastic and liquid limits.

The in situ large shear vane testing along the eastern bund indicated three distinct areas of relatively consistent soil shear strength profile.

At the Southern end (Chainage 0 to 380m) the strength profile typically recorded was around 3-5m of very soft Holocene sediments with shear strengths of 2-10kPa overlying stiff Pleistocene sediments and residual (weathered) Waitemata group soils with shear strengths around 80kPa.

Soils underlying the Central section (Chainage 380 to 640m) of the bund typically comprised 2-5m very soft Holocene sediments (as above) overlying firm Pleistocene sediments with shear strengths of around 30 to 50kPa. These in turn were all underlain by stiffer weathered Waitemata group soils.

Towards to Northern end, around 1-2m of very soft Holocene sediments was encountered overlying stiff Pleistocene sediments and weathered Waitemata group (i.e. soils with shear strength 80kPa+).

Boreholes drilled in the eastern side of the reclamation indicated an extensive gully infilled with very weak Holocene sediments within the Central bund section. The Geonor vane shear strength testing confirmed the presence of this infilled gully.
Three machine boreholes were also drilled through the

reclamation along the existing eastern edge. The location of these are shown on Figure 2. Soils encountered in these boreholes typically comprised stiff clayey silt and medium dense sand. In situ testing indicated Pilcon vane shear strengths of 80 to 120kPa and standard penetrometer tests recorded typically 11 to 50 blows per 300mm in the Pleistocene sediments below the old rock bund.

The soil types encountered are described below. A summary of the soil test results is shown on Table 1.0.

(a) Layer A : Sediments

The sediments encountered in the machine boreholes typically comprised recent very weak Holocene sediments overlying older slightly over-consolidated Pleistocene sediments.

(i) Layer A1: Holocene Sediments

The very weak Holocene sediments encountered were typically around 0.5-2m thick along the Northern edge of the proposed reclamation increasing to around 5m thick beneath the Central eastern side. In situ testing indicated relatively consistent strengths with large vane shear strengths of typically around 2 to 10kPa and Pilcon shear strength measurements of 4 to 30kPa. Figure 4 shows a general increase in shear strength with depth.

It is noted that the Pilcon vane tests carried out in the open barrel measured higher soil shear strengths than the large in situ vane testing. This is considered to be due to the soil sample becoming compressed in the open barrel during sampling. Also the Pilcon vane is rotated faster than the large Geonor vane, resulting in a faster rate of shearing and hence higher apparent soil shear strengths.

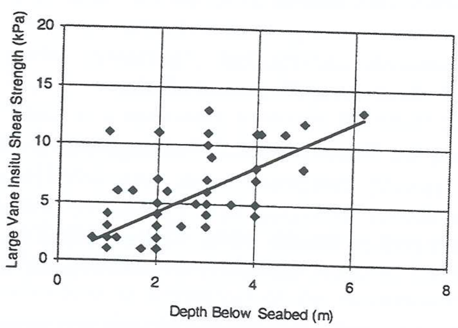

Figure 4 : Shear Strength vs Depth Below Seabed
HOLOCENE LAYER

Atterberg limit tests on samples of these soils indicate that the material is highly plastic and has a natural water content close to the liquid limit. Soil particle size gradings carried out on these soils indicate that the soils are typically silty clays/clayey silts.

(ii) Layer A2: Pleistocene Sediments

In-situ strength testing and laboratory consolidation tests of the Pleistocene sediments indicate that these deposits are overconsolidated. Pilcon vane shear strengths measured in this material typically varied between 30 and 100kPa and standard penetrometer tests recorded typically 4 to 20 blows/300mm penetration. Large vane shear strengths recorded in these soils to the east of the existing reclamation varied between 20 and 80kPa.

Figure 5 shows that generally the shear strength of the Pleistocene sediments also increases with depth below seabed.

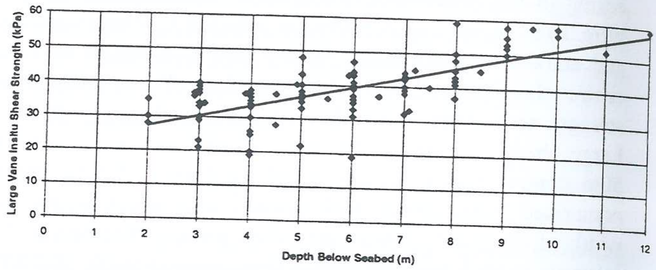

Figure 5 : Shear Strength vs Depth Below Seabed
PLEISTOCENE LAYER

Atterberg limit tests on these Pleistocene materials indicates this material is also moderately to highly plastic. The natural moisture content of these soils is typically closer to the plastic limit. Soil particle size gradings carried out on these soils indicates that the soils are typically sandy or clayey silts.

A hard lens of material was located in boreholes located along the alignment of the northern bund between -12 and -16 CD. Standard penetrometer tests recorded typically 40+ blows/300mm penetration.

(b) Layer B : Weathered Waitemata Group

The weathered Waitemata Group typically comprises stiff to very stiff interbedded silts and clays. This layer was encountered in the boreholes between 6 and 14m below seabed (10 and 24m below chart datum) and was around 2 to 5m thick.

In situ testing indicated that the soils are stiff to very stiff with standard penetrometer test results of between 12 and 41 blows per 300mm penetration and Pilcon vane shear strengths between 80 and 130kPa. Figure 6 shows a plot of normal stress versus shear stress from results of triaxial tests on the Pleistocene and weathered Waitemata Group soils collected during various investigations for development of the Fergusson Container Terminal. The line on this graph indicates this soil has a cohesion (c') of around 5kPa and an angle of friction (ϕ') of around 35º.

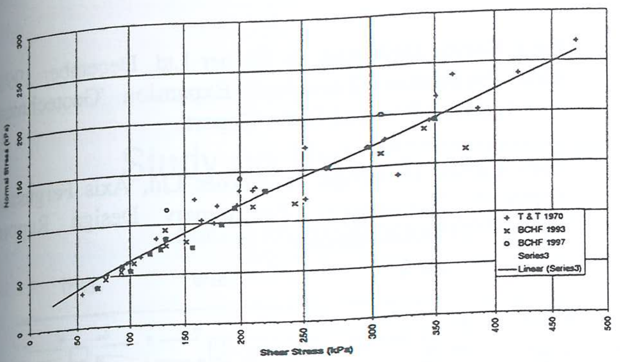

Figure 6 : Normal Stress vs Shear Stress

(c) *Layer C : Waitemata Group Sandstone and Siltstone*

This layer generally comprises interbedded extremely weak to very weak (in terms of rock strength) grey fractured sandstone and siltstone. Figure 3 shows approximate contours of the surface of sandstone/ siltstone. Along the alignment of the proposed piled quay structure (ie northern edge) sandstone/siltstone was encountered in the boreholes between -22 CD and -29CD (ie 10 to 17m below seabed).

Along the eastern edge sandstone/siltstone was encountered between -11 and -26CD. The depth to sandstone encountered in the boreholes indicates that the sandstone/ siltstone surface slopes from the southern end of the reclamation towards the north. A gully was located at around distance 500m, to the east of the existing terminal.

Standard penetrometer tests produced penetrations of between 95 and 290mm for 50 blows ie N = 50 - 150 blows/300mm. This layer is considered suitable for pile foundations.

6 CONCLUSION

In order to resolve the design issues described in Section 3 geotechnical investigations comprising the drilling of machine boreholes, large vane in situ shear strength testing and laboratory testing have been undertaken.

Boreholes were drilled to obtain samples of the soils for logging, laboratory testing and determining the depth to Waitemata Group rock. In situ strength testing such as standard penetrometer tests and large vane shear tests down the borehole and Pilcon vane tests on soil samples in the open barrel provided information to allow a general assessment of the soil conditions across the site. The laboratory testing has provided additional information on the nature of the soils (ie. particle size gradings for a liquefaction assessment) and geotechnical parameters for an assessment of consolidation settlements and shear strength parameters from triaxial tests for stability analysis.

The boreholes revealed significant depths of weak soils across the site. The assessment of the shear strength of these soils is considered critical to the stability of the bunds. In situ large shear vane testing was considered the most appropriate method of determining the actual shear strength of the underlying soils and this testing was undertaken as described earlier. Indirect measurement of the shear strength of very weak soils using Pilcon vane in a sample can provide misleading results due to sample disturbance and rate effects of testing. Direct measurement of soil shear strength in situ is considered necessary, particularly where very weak soils are present and where the actual shear strength of the soil is critical to the design of the structure.

Staging of the geotechnical investigations has allowed the appropriate testing at each stage by focusing on resolving each of the geotechnical design issues, as the various stages of investigations revealed the critical soil conditions.

TABLE 1 - SUMMARY OF SOIL PROFILE AND LABORATORY TEST RESULTS

Layer	Description	Depth below seabed to top of layer (m)	Thickness (m)	SPT N (Blows/ 300mm)	Pilcon Vane Shear Strength (kPa)	Geonor Vane Strength (kPa)	Atterberg Limits		Natural Moisture Content (%)	Grading			Consolidation		Triaxial CU		Bulk Density t/m³	UCS (MPa)
							LL (%)	PL (%)		Sand (%)	Silt (%)	Clay (%)	$\frac{C_c}{1+e_o}$ (%)	Pc' (kPa)	c' (kPa)	Ø' deg		
A1	Holocene Sediments	0	0.5-5	0-1	4-30	2-10	82 to 93 Avg =88	27 to 33 Avg =30	77 to 98 Avg =82	1 to 36 Avg =10	36 to 66 Avg =52	33 to 51 Avg =38	23 24 25	70 50 70	12	24	1.48 to 1.56 Avg =1.52	
A2	Pleistocene Sediments	0-12	2-14	3-50 typically 15	30-130 Typically 70	20-80	49 to 98 Avg =76	23 to 45 Avg =30	32 to 72 Avg =48	2 to 63 Avg =21	28 to 64 Avg =44	9 to 64 Avg =35	15 23 26	180 200 250	25	24	1.52 to 1.78 Avg =1.65	
B	Weathered Waitemata Group	6-14	2-5	12-41 typically 30	80-130 typically 100	80+							10 14 17	150 300 350	25 12 50	30 33 34		
C	Waitemata Group Sandstone & Siltstone	8-15	-	50+	-	-												1020 to 4000

Note: (i) LL = Liquid Limit; PI = Plasticity Index; Pc' = Preconsolidation Pressure; $\frac{C_c}{1+e_o}$ = Compressibility; Ø' = Effective Angle of Friction; c' = Effective Cohesion; CU = Consolidated Undrained with Pore Pressure Measurement; UCS = Unconfined Compressive Strength

7 ACKNOWLEDGMENTS

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8 REFERENCES

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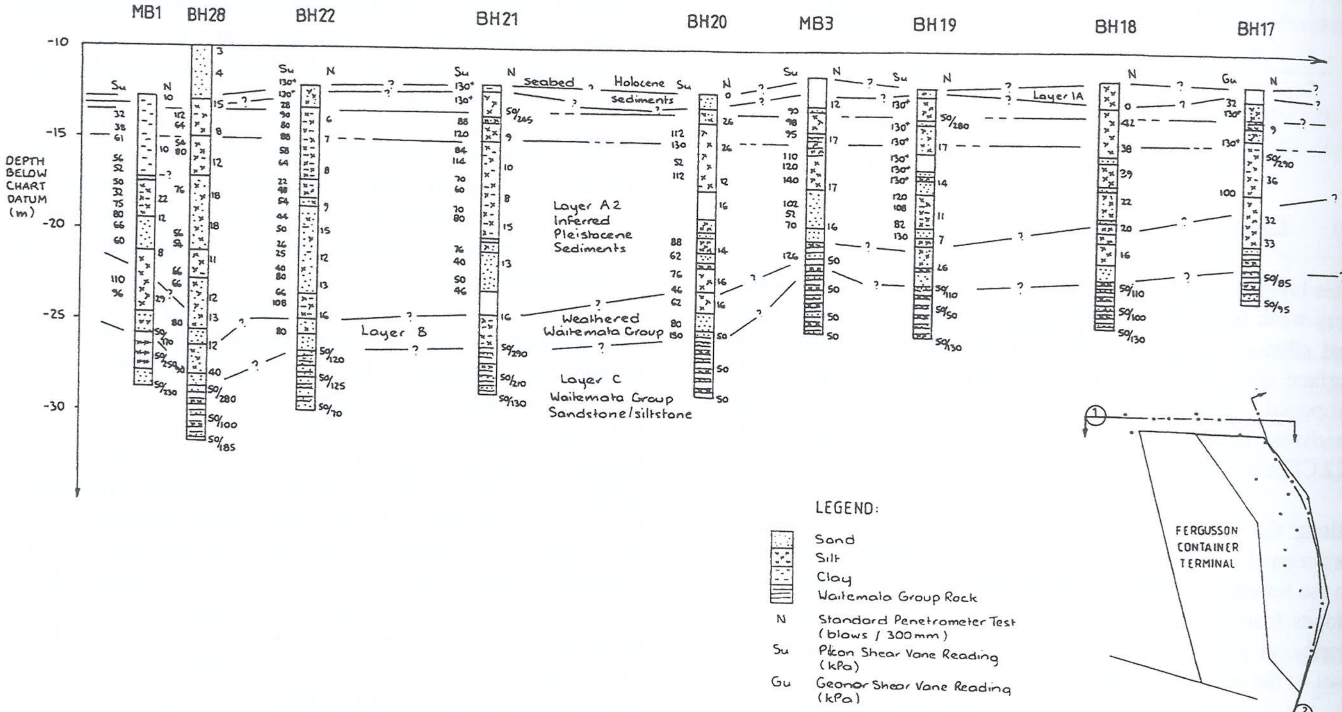

Figure 7 : Cross Section 1, Soil Profile Along Northern Edge

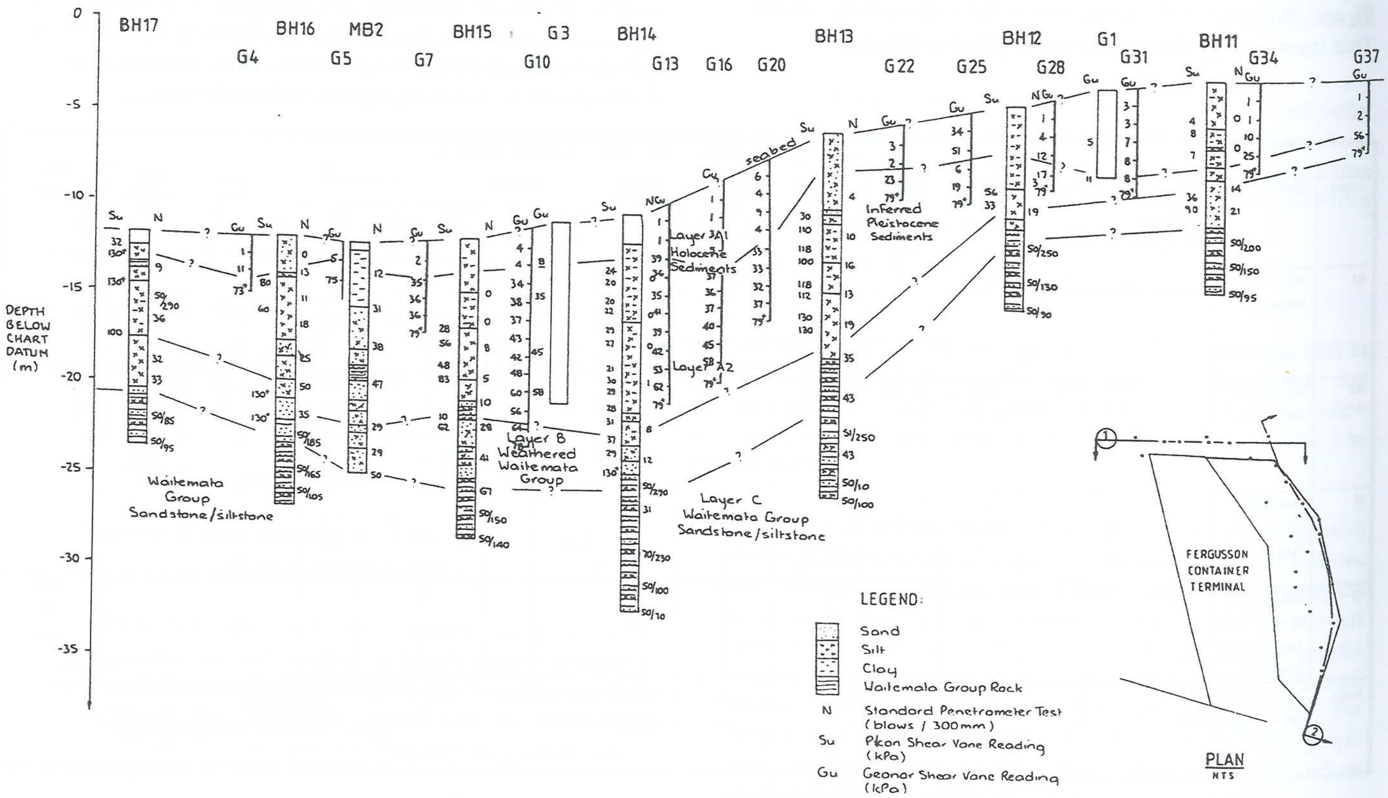

Figure 8 : Cross Section 2, Soil Profile Along Eastern Edge