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Assessing the Strength of East Coast Bays Formation Soils using in-situ Testing Data

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ABSTRACT

AECOM undertook engineering geological mapping and geotechnical investigations to support resource consent for a proposed subdivision located west of Orewa. The development land consists of a central valley with gentle to moderate sloping upper sections, steep mid sections and moderate toe slopes. The site is underlain by a weathered East Coast Bays Formation (ECBF) sequence of the Waitemata Group, which comprises interbedded very weak sandstone and mudstone units that dip at a low angle northward. Past instability has affected the land and mapped features included head scarps, debris lobes and hummocky ground. Investigations included machine drillholes, machine excavated pits, hand augerholes, cone penetrometer (CPT) and dilatometer testing (DMT). Laboratory testing of select recovered materials was also undertaken, comprising Atterberg Limits, Standard Compaction, Moisture Content and Lime Reactivity tests. Ground conditions encountered in the geotechnical investigation comprise a mantle of weathered, reworked silt and clay to a depth of at least 3 m. Underlying this mantle is a weathered sequence of generally interbedded ECBF sandstone and siltstone. Alluvium/colluvium occurs in the valley floors, which in places are swampy. Geotechnical design parameters required for stability analysis were determined from both measured field vane shear strengths and those derived from CPT and DMT data. CPT empirically derived strengths were calibrated against in-situ vane shear strengths by adjusting the cone factor (N_{kt}) using Geologismiki software CPeTiT until a close fit between the field data was made. This paper presents a review of the investigation results, the geological model for the site and a comparison of measured and derived soil parameters.

Keywords: East Coast Bays Formation, CPT, CPeTiT, cone factor (N_{kt})

1 INTRODUCTION

AECOM was engaged by a developer to provide design services for a proposed residential subdivision located northeast of Orewa, New Zealand (refer Figure 1(a)). As part of the work undertaken AECOM provided a geotechnical assessment of the land to support an application for land use and earthworks consents for the subdivision.

The purpose of this paper is to highlight the risk of using CPT derived soil parameters in East Coast Bays Formation materials without calibration.

The authors note that specific details which could be used to identify the exact site and our client's details have been omitted from this paper at the client's request.

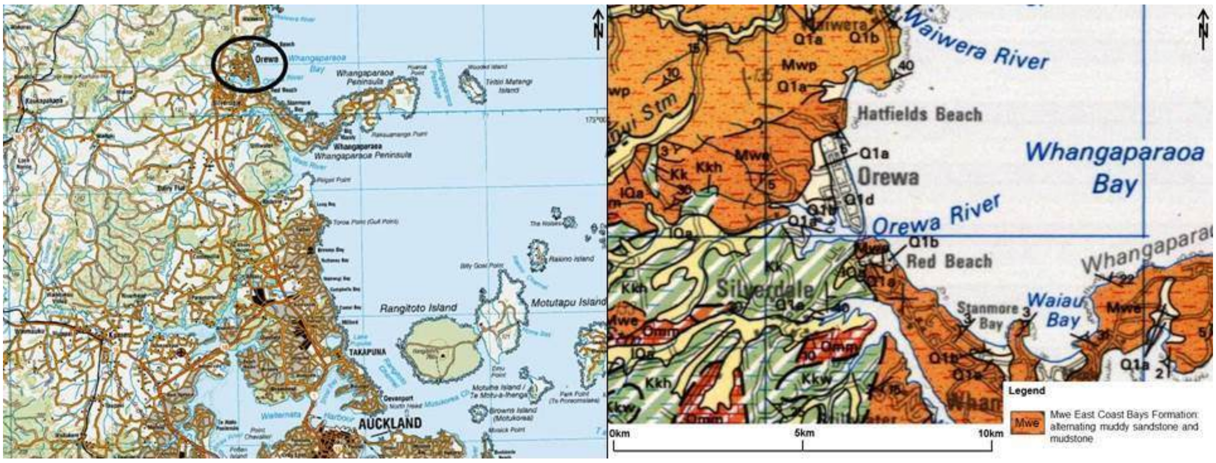
2 LAND DESCRIPTION AND ENGINEERING GEOLOGY

2.1 Description of the land

The development land has an area of approximately 375,000 m^2 and is currently used for pastoral farming. A Significant Ecological Area (SEA) borders the site to the north, with pastoral farming to the west, residential housing to the east and a new residential development to the south.

2.2 Geological setting

The published geological map of the area (Figure 1(b)) shows the site is underlain by East Coast Bays Formation (ECBF) of the Waitemata Group, which comprises very weak, interbedded sandstone and mudstone that dip at a low angle northward (Edbrooke 2001). These rocks are typically weathered to depths of about 5 m and commonly have a cover of 1 to 3 m of reworked weathered materials (slope debris), especially on the gentle ridge crests. Alluvium (including swamp deposits) occurs in the valley floors. No active faults are shown to occur in the area (GNS 2016).



(a) (b)

Figure 1. (a) Site location; and (b) Geological map extract (Edbrooke 2001)

2.3 Geomorphology

The dominant topography consists of a north trending horseshoe shaped valley in the centre of the site with gentle to moderate sloping upper sections, steep mid sections and moderate toe slopes (Figure 2). Small areas of flat to gently sloping land occur along the western portion of the site with small pockets around the central valley. The head of the valley is a broad, open basin with a radial pattern of shallowly incised ephemeral watercourses draining to an artificial pond in the valley floor. A central stream drains from the pond to the north. An SEA occupies the northern portion, steeper slopes and tributaries of the central valley.

Past instability has affected the land. Evidence of instability includes head scarps, debris lobes, mid-slope benches and hummocky ground. These instability features generally occur on the slopes between more stable ridgelines. Groundwater seepage and poor surface drainage is associated with some of the instability features. Mapped geomorphic features are shown on Figure 2.

3 GEOTECHNICAL INVESTIGATION

3.1 General

Engineering geological mapping and intrusive investigations were undertaken between August 2014 and October 2015. Investigations included machine drillholes, machine excavated pits, hand augerholes, cone penetrometer tests (CPTs) and dilatometer tests (DMTs).

Seventeen cored machine drillholes were drilled using HQ coring techniques to depths ranging from 15.5 to 24.45 m. Standard Penetration Tests (SPTs) were undertaken generally at 1.5 m centres as drilling progressed. Single standpipe piezometers were installed in all the drillholes and developed using an air compressor.

The first 1.5 m of each machine drillhole was advanced using a hand auger. A series of hand augerholes were also undertaken so that in-situ hand held vane shear strength tests could be undertaken.

Twenty five pits were excavated to depths between 1.5 and 4.2 m by a machine excavator to augment the machine drillholes and provide bulk samples for geotechnical and contamination laboratory testing.

Thirty four CPTs were undertaken using a tracked Pagani portable system to augment the drillhole and pit investigations and target areas requiring substantial engineering design. Two DMTs were undertaken to determine typical soil consolidation properties.

Materials encountered in the drillholes, augerholes and pits were logged on site according to the procedures in the New Zealand Geotechnical Society Field Description of Soil and Rock Guideline (New Zealand Geotechnical Society 2005). Hand held shear vane strength tests were performed where appropriate according to the New Zealand Geotechnical Society, 2001 guideline.

Geotechnical and contamination testing was carried out on a number of samples to determine material properties and potential re-use (such as engineering fill). Geotechnical laboratory testing included Consolidated Undrained Triaxial, Unconfined Compressive Strength, Atterberg Limits, Standard Compaction, Moisture Content and Lime Reactivity tests.

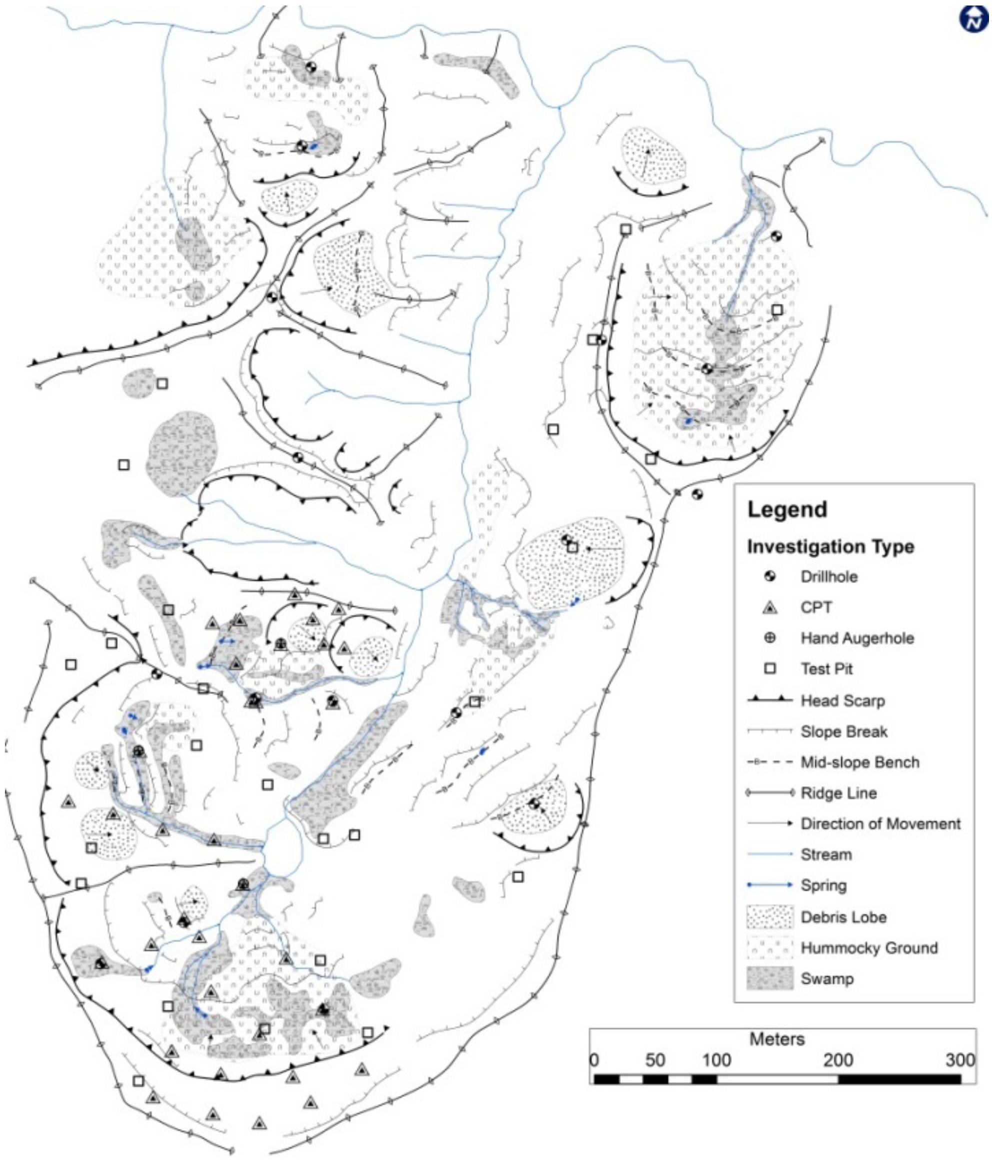


Figure 2. Site geomorphology

4 GEOTECHNICAL ASSESSMENT

4.1 Geological model

Geotechnical investigations and mapping confirmed ECBF underlies the site. Site ground conditions can be summarised as comprising thin topsoil layer (0.1 to 0.4 m) underlain by a mantle of weathered, reworked silt and clay (slope debris) to a depth of up to 5 m. Underlying this mantle is a weathered sequence (completely to slightly weathered) of generally interbedded, sub-horizontal ECBF sandstone and siltstone.

Weathering grades were assigned based on field strength, visual inspection and colour. Reworked completely weathered (RCW) ECBF materials comprise firm to very stiff silt and clay derived from completely weathered siltstone and sandstone. The material has a disturbed homogenous appearance with variable organic content including broken decomposing rootlets. Completely weathered (CW) ECBF materials generally comprise firm to very stiff silt and clay derived from weathered siltstone and sandstone with some relict rock fabric (joints) remaining. Highly weathered (HW) ECBF materials generally comprise interbedded, very stiff silt and clay and loose to medium dense sand derived from weathered siltstone and sandstone with some relict rock fabric (joints) remaining. Moderately weathered (MW) ECBF generally comprises interbedded, very stiff to hard silt and clay and medium dense to dense sand with a rock strength ranging from extremely to very weak. Slightly weathered (SW) ECBF comprises very weak to weak, sub-horizontal, interbedded siltstone and sandstone. No defects with evidence of movement that may indicate deep seated instability was observed in the recovered core in slightly weathered ECBF.

Fill material associated with historic roads, a farm dam and landfill construction were identified to a maximum depth of 1 m in localised areas around the site. Fill generally comprised stiff to very stiff sandy and clayey silt and had a similar appearance to reworked ECBF materials.

Alluvium occurs in the valley floors (central valley) and generally comprises firm to stiff silt and clay with variable decomposing organic material and fibrous peat.

The geological units described above are presented in Table 1. Figure 3 presents a typical engineering geological cross section across one side of the central valley.

Table 1. Summary ground conditions and in-situ testing

Depth of Unit (m)	Geotechnical Unit						
	Alluvium	**Fill**	**RCW ECBF[a]**	**CW ECBF**	**HW ECBF**	**MW ECBF**	**SW ECBF**
Minimum	0.0	0.0	0.0	0.2	1.0	3.3	4.8
Maximum	1.5	0.0	5.0	10.5	18.4	20.1	24.5+
Corrected vane shear strengths Peak/remoulded (ave.)	83 / 37	105 / 46	113 / 57	131 / 68	140 / 55	N/A	N/A
SPT N Value Average (range)	N/A	N/A	2 (0-5)	4 (0-10)	12 (0-25)	30-50+	50+

[a] Topsoil included in the reworked ECBF unit
[b] Deepest investigation

Piezometers were installed in all of the drillholes. Measured water level depths range from 1.81 m for holes located at the base of the valley to 11.26 m for holes located on ridge tops.

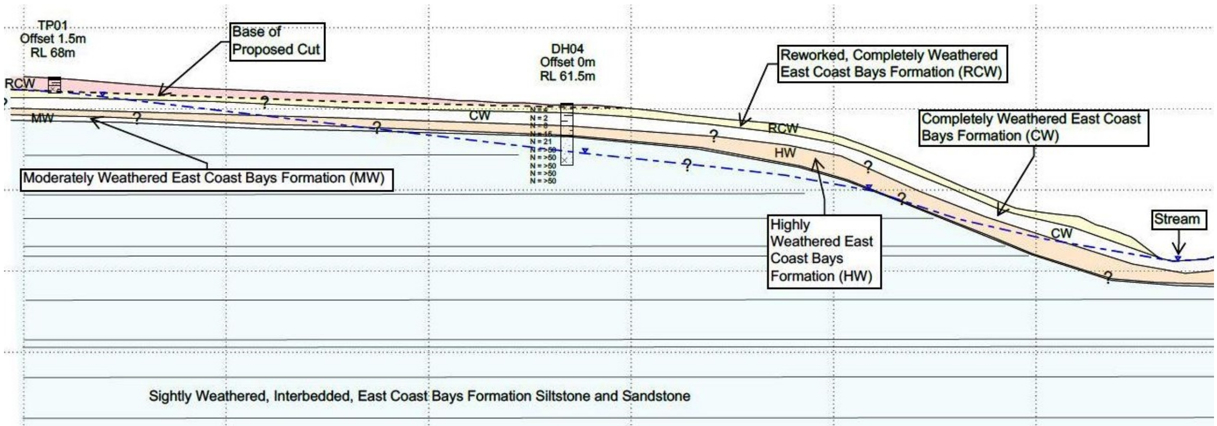


Figure 3. Typical engineering geological section across the site

4.2 Geotechnical Design Parameters

Geotechnical design parameters required for stability analysis and geotechnical design (subdivision infrastructure including retaining walls, roads, drainage etc.) were determined from measured in-situ strengths, strengths derived from CPT and DMT data, geotechnical laboratory testing and experience with similar materials in this area. Table 2 presents geotechnical design parameters used in stability analysis and design.

Table 2. Geotechnical design parameters

Geotechnical Unit	Unit Weight, γ (kN/m^3)	Total Stress	Effective Stress		Young's Modulus, E' (MPa)	Poisson's Ratio, ν
		Shear Strength S_u (kPa)	Cohesion c' (kPa)	Friction angle, ϕ (degrees)		
Alluvium	16	40	3	25	10	0.3
RCW & CW ECBF	17	30 to 60	5	28	10 to 20	0.3
HW ECBF	17.5	100	7	30	40	0.3
MW ECBF	18.5	150	10	32	70	0.2
SW ECBF	22	500	50	35	200	0.2

4.3 CPT undrained shear strength calibration

In order to utilise the extensive CPT data (34 CPTs and 2 DMTs), calibration with the undrained soil shear strengths (S_u) derived from the CPTs is required. The principal reason is that the CPT results were noticeably lower than hand held vane results from hand augerholes and would have led to overly conservative design. This calibration involved the adjustment of the CPT cone factor N_{kt} until the CPT derived shear strength values approximately matched corrected field hand held vane shear strength measurements.

CPT derived soil shear strength is controlled by various factors such as cone factor, soil plasticity, soil sensitivity and soil stress history. These factors are incorporated into empirical equations used to estimate shear strength for the respective test methods, e.g. cone factor N_{kt} is used for the CPT method.

For CPTs, N_{kt} typically ranges from 10 to 18, with an average of 14. N_{kt} tends to increase with increasing plasticity and decrease with increasing soil sensitivity (Robertson and Cabal 2015).

The following relationship is used for calculating shear strength from CPT data:

$$S_u = q_t – \sigma_v / N_{kt} \qquad (1)$$

Where q_t is corrected cone resistance, σ_v is total overburden stress and N_{kt} is the cone factor.

Due to the broad spread of cone factor values i.e. 10 to 18, calibration of N_{kt} is undertaken. The calibration process is carried out using Geologismiki software CPeTiT. Equation (1) is built into the software program and automatically calculates soil shear strength using N_{kt} = 14 as a default value.

Shear strength values estimated by CPT and measured by hand held shear vane values are plotted side by side in CPeTiT. Based on plot results (Figure 4) it is observed that, for this site, the direct in-situ shear vane test measurements produce values of S_u higher than CPT derived values using the default N_{kt} = 14.

Calibration of N_{kt} is undertaken by trial and observation. The value of N_{kt} is adjusted until a best fit with the field shear vane results is achieved. It is reported that generally soil shear strength estimation based on field shear vanes and DMTs is more accurate and dependable for design (Marchetti et al. 2001) compared to other methods such as CPT, SPT.

From this assessment an N_{kt} value of 10 is found to have the best fit with measured field shear strengths (Figure 4). This is at the lower limit of the published range, i.e. 10 to 18.

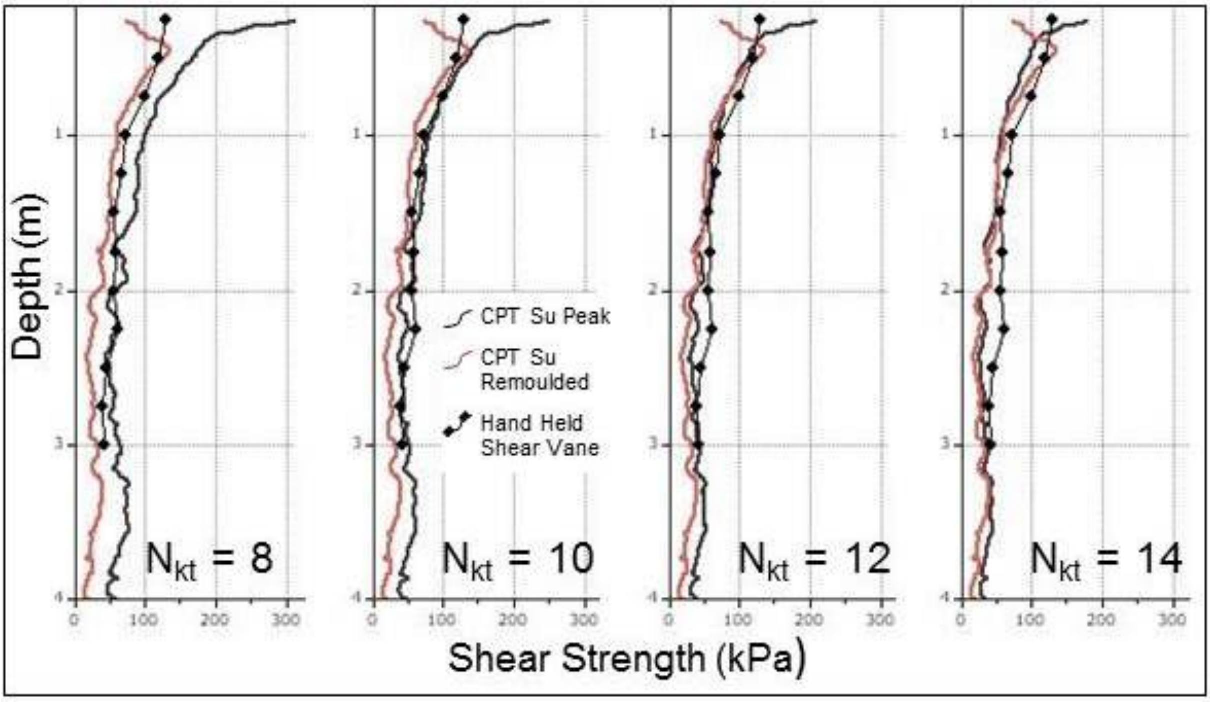


Figure 4. CPT cone factor, N_{kt} calibration with in-situ shear vane tests (N_{kt}=10 shows the best fit)

5 CONCLUSION

Ground conditions at the site comprise a mantle of reworked weathered material to at least 3 m depth. Underlying this mantle is a weathered sequence of interbedded East Coast Bays Formation sandstone and siltstone.

For this site it was found that direct in-situ shear vane measurements were higher than those predicted by CPT analysis using a default cone factor, N_{kt} of 14. Shear strength values derived from CPT were found to have a close fit with measured field vane values in weathered East Coast Bays Formation materials when using a cone factor, N_{kt} of 10.

Due to repeatability, relatively quick and portable testing, and low cost CPTs are deemed to be effective at determining shear strength over a large site provided calibration against in-situ testing is undertaken and appropriate cone factors are applied.

6 ACKNOWLEDGEMENTS

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