The Diversity of the Geology in the Auckland Region and Some of the Associated Challenges for Land Development

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The diversity of the geology within the Auckland region presents a range of challenges for the geotechnical professional. This paper provides an overview of the Auckland geology and presents case studies of three recent land development projects within different geological settings, all of which presented a range of geotechnical problems to be addressed. The three lithologies include very soft organic peats within the alluvial lowlands of South Auckland, the Waitemata Group sedimentary deposits on the North Shore, and the Northland Allochthon/Onerahi Chaos deposits of Silverdale. This paper also discusses examples of appropriate engineering solutions to some of the problems presented by these lithologies.

1.0 INTRODUCTION

The Auckland region has a complex geological history. This history has implications for land development. It is important for geotechnical professionals to have an understanding of the formation and engineering properties of the geology at the location of their project. This paper will discuss some of the challenges that the various geological units in the Auckland area pose for the geotechnical professional. To do this it will discuss the challenges on three recent projects undertaken within different geological settings and their associated engineering solutions.

2.0 GEOLOGY OF THE AUCKLAND REGION

Beneath the varying topography of the Auckland region is a complex geology. This geology is made up of materials deposited during the late Triassic to Holocene times (200 million ears ago to the present) and comprises sediment of differing origins.

From Mesozoic to Cretaceous times the Waipapa Group and Murikiku Terrane ‘greywacke’ was uplifted and deformed. Following this period of uplift and deformation the land was eroded and peat began to form. This peat was later transformed into the Waikato Coal Measures which was further overlain by the Te Kuiti Group (Kermode, 1).

Later the Waitemata Basin began to form. It extended 80km to the north and south of present day Auckland. The basin began to fill up during the early Miocene (24 to 15 million years ago) firstly with materials of the Kawau Subgroup followed by 2km of sediments consisting mostly of mud with occasional large quantities of coarser silt and sand (Kermode, 1). This sequence of sediments became known as the East Coast Bays Formation and forms the vast majority of sea cliffs and rolling hill country in the Auckland area.

While the Waitemata Basin was filling up, several sheets (Northland Allochthon/Onerahi Chaos) of material were emplaced over the newly forming Waitemata Group. This material originated from the north-east of Northland and is intensely deformed (Edbrooke, 2).

Beginning in the Pleistocene (2 million years ago) the Taupo volcanic zone produced sediments that have made a significant contribution to the topography around the Manukau Harbour (Kermode, 1). In between these volcanic sediments peat and organic clays have formed at several levels (Tauranga Group).

Finally in more recent times 48 volcanoes have formed within the Auckland volcanic field producing lava fields, scoria cones, ash mantles and tuff rungs over much of the Auckland area (Kermode, 1).

Figure 1 on the following page shows the simplified stratigraphic relationships between the various geological units in the Auckland area.

3.0 POINT RIDGE RESIDENTIAL SUBDIVISION, ALBANY

The Point Ridge residential subdivision is situated in Albany, a suburb on the North Shore in Auckland. The land gradients on this site generally slope from north to south and the block is underlain by Miocene Age Waitemata Group sedimentary deposits.

These sediments typically weather to residual orange, grey, pink and cream clays and silts. The dark grey transition zone material which comprises very stiff/dense clays, silts, and sands is overlain by the residual overburden. Beneath this transition zone lies bedrock which generally consists of very weak, highly weathered, slightly cemented dark grey sandstones and siltstones.

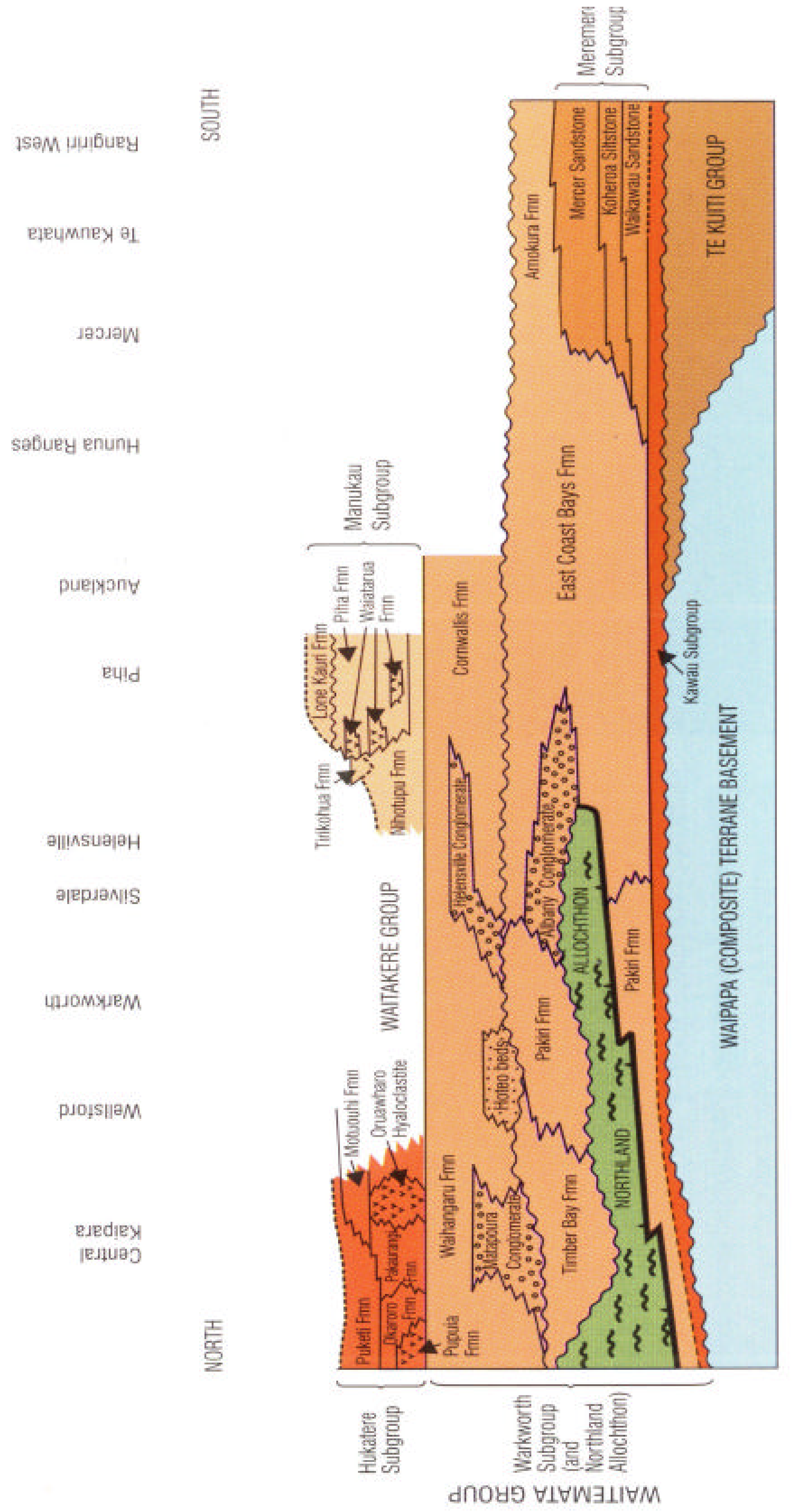

Figure 1: Simplified Stratigraphic Relations Between the Various Geological Units (Edbrooke, 2)

3.1 SITE INVESTIGATION

During preliminary desk top studies a number of instability features were identified. Upon investigation we concluded that the movements were both circular and planar in nature. The circular failures were found to generally lie within the residual overburden materials while the planar failures were found to occur between the residual overburden and transition zone materials at approximately 8 to 10 metres depth.

As part of the investigation a comprehensive series of laboratory tests were undertaken. Tests included Atterburg limits, consolidation, compaction and triaxial testing. From this testing realistic shear strength and settlement characteristics were adopted for stability analyses and settlement calculations.

Some of the challenges encountered at this site were:

- Variable bearing capacity conditions
- Shrinking/ swelling properties
- Shallow and deep seated slope instability
- Settlement

3.2 SHALLOW AND DEEP SEATED SLOPE INSTABILITY

As the site was showing signs of land movement, back analysis was undertaken for an assumed pre-failure profile. This yielded effective stress parameters of c'= 0kPa and Ø'=18 degrees (Bevin, 3) for soft to marginally firm strata directly overlying stiff/ dense transition zone materials.

Table of Soil Properties

Description	c' (kPa)	Ø' (degrees)
Stiff certified clay filling	10	30
Soft to marginally firm strata	0	18
Firm to stiff silty clays, clayey silts and silts	7	28
Very stiff clayey silts, silts and sandy silts	5	28
Bedrock	50	40

Figure 2: Table of Soil Properties (Bevin, 3)

Using the parameters in Figure 2, stability analyses were then performed using the Simplified Bishop and Janbu Methods for circular and planar analyses. As a result of these analyses it was proposed that two shear keys (buttress fills) should be installed on the south facing slope.

The analyses were carried out for normal groundwater conditions, elevated groundwater levels, and extreme groundwater levels. For normal groundwater conditions the assessed factors of safety were in excess of 1.5 while for elevated groundwater conditions factors of safety lay between 1.2 and 1.5, and for extreme groundwater conditions the factors of safety were just greater than unity.

This showed that even with the construction of two shear keys, appropriate groundwater control was also needed in order to maintain factors of safety in excess of 1.5.

The shear keys needed to adequately stabilise the slope were approximately 15 metres wide at the base and needed to be founded on very stiff/dense transition zone and/or bedrock material, below the shear planes. They were oriented in an east-west direction and keyed into competent identified intact ridges at both ends. The lower shear key was situated at the toe of the slope while the upper shear key was situated mid-slope as depicted in figure 3 shown below.

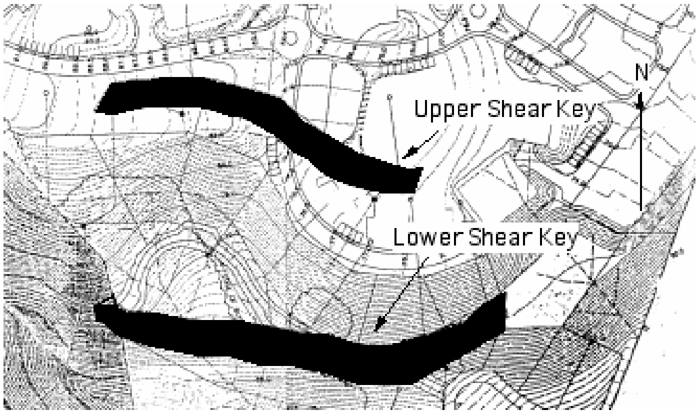

Figure 3: Approximate Shear Key Locations

3.3 SETTLEMENT

Results from consolidation testing showed that likely total settlements beneath the structural fills were of the order of 300mm (Bevin, 4). Although this appeared excessive the results also showed that the settlement would occur relatively rapidly (ie. months) and that t_{90} would be reached before commencement of building construction.

However, of more significance was the potential for differential settlement, particularly in cases where settlement sensitive buildings would be sited partly on fill and partly on cut ground, or where they would be sited entirely on filling whose depth changes significantly across the building platform.

In areas of concern, post construction settlement monitoring was recommended to assist in confirming that t_{90} had been achieved and that any long term settlement would be within code limits. As a further precautionary measure recommendations were also provided in regard to careful detailing of control joints in brittle exterior cladding such as brick veneer, block work, solid and stucco plaster etc.

3.4 SHRINKING/ SWELLING PROPERTIES

A common problem encountered with Waitemata Group materials is their seasonal shrinking and swelling characteristics. To assess this criteria from AS 2870:1996 were used. It provides foundation solutions for five defined site classes. These classes are good ground (A), slight (S), moderate (M), high (H), and extreme (E). AS2870:1996 provides a range of foundation solutions which typically involve combinations of footing depths and widths.

The site was assigned an expansive class of S (Slight) Bevin, 3). An appropriate solution was to found all conventional strip and pad foundations 450mm below finished ground level.

3.5 VARIABLE BEARING CAPACITY

Generally residual Waitemata Group sediments are of sufficient strength to provide a geotechnical ultimate bearing capacity of 300 kPa, as required for NZS 3604:1999, "Timber Framed Buildings". However areas exhibiting a reduced bearing capacity are often encountered. Several areas of soft natural ground were found at this site and accordingly a reduced geotechnical ultimate bearing capacity of 210 kPa was recommended.

4.0 KINGSWAY SCHOOL, SILVERDALE

As part of site development Kingsway School have added playing fields and several blocks of classrooms. The land gradients within the site are typically gentle with some steep areas along the southern boundary of the site. The site is underlain by Onerahi Chaos/ Northland Allochthon materials.

This lithological unit is highly deformed and is generally comprised of crushed limestone, mudstone, conglomerate, and siltstone within a matrix of mudstone, siltstone, and sandstone (Bevin, 5).

Generally overlying this shattered rockmass is a cap of residual clay soils which acts as a semi-impermeable layer limiting the amount of rainfall infiltration. This layer also helps to confine the underlying rockmass.

Onerahi Chaos materials can be unstable at slope angles as low as 5 to 10° (Rafferty, 6). It weathers rapidly and swells due to stress relaxation when exposed.

4.1 SITE INVESTIGATION

The site investigation for this development found that the semi-impermeable layer of clay was between 1.5 and 2.0 metres thick. This layer was underlain by the shattered Onerahi Chaos rockmass.

Some of the challenges identified on this site were:

1. Rockmass Degradation
2. Soil Creep and Slope Instability at Relatively Shallow Angles
3. Significant Shrinking/ Swelling Properties

4.2 ROCKMASS DEGRADATION

As part of earthworks for the proposed development the semi-impermeable clay cap was removed. This exposed the underlying Onerahi Chaos rockmass at several places. In order to maintain a semi-impermeable layer above the rockmass it was decided that the best course of action was to undercut the rockmass at these locations by 250mm and to replace it with compacted clay (Lander, 7). This served to limit rockmass degradation and reduce swelling due to stress relief. Figure 4 below shows the cut that would expose the Onerahi Chaos.

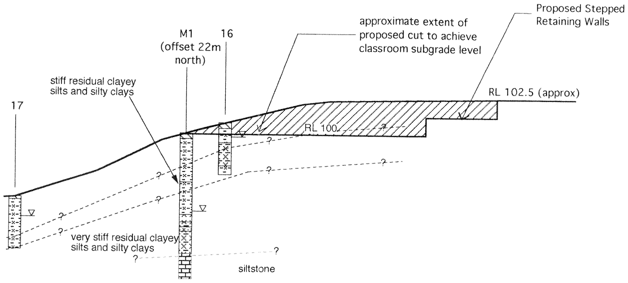

Figure 4: Cross-Section Showing the Cut Exposing the Onerahi Chaos (Lander, 7)

4.3 SOIL CREEP AND SLOPE INSTABILITY

Onerahi Chaos materials can be unstable at slopes 10° (Rafferty, 6). The majority of land gradients on this site were gentle, however the localised gradients adjacent to the southern boundary were as steep as 36°. Accordingly the new classroom block was given a generous set-back from areas having such gradients.

In addition to this, the leading edge foundations were piled. Piles were bored and concreted to depths of 4.5 metres and they were designed for a 2 metre deep creep zone (Lander, 7). This allowed for any long-term creep between the semi-impermeable clay layer and the underlying Onerahi Chaos rockmass.

4.4 SHRINKING/SWELLING

The residual Onerahi Chaos soils tend to be expansive. At this site the expansive class assigned was M (Moderate). Accordingly all conventional strip footings were constructed 600mm below finished level (Lander, 7).

5.0 ADDISON BLOCK RESIDENTIAL DEVELOPMENT, TAKANINI

This development comprises a medium density residential subdivision. The site is generally flat and is contained within the South Auckland Lowlands. This area is underlain by organic/peaty clays of the Tauranga Group.

The Tauranga Group alluvial sediments are typically soft and are among the most recent to be deposited in the Auckland Region. These materials typically include peats, organic clays, and lenses of pumiceous material. Groundwater levels are generally close to the surface.

5.1 SITE INVESTIGATION

The site investigation found that organic/peaty clays were present beneath a 1.0 to 1.5 metre deep crust of firm organic material to depths beyond 10 metres. The groundwater level typically lay between 1.2 to 1.5 metres depth (Melville-Smith, 8).

The materials underling the site pose several challenges:

7.0 Low Bearing Capacity and California Bearing Ratio (CBR)
8.0 Potential for Large Total and Differential Settlements

5.2 LOW BEARING CAPACITY AND CBR

Peats and organic clays generally have a low bearing capacity and CBR. The field investigation showed that an ultimate geotechnical bearing capacity of 180 kPa and an equivalent CBR of 4 was available (Melville-Smith, 8).

The low bearing capacity and relatively consistent round conditions made the site suitable for raft/ pod type foundations. The raft/ pod type foundations were suitable because the loads are spread over an increased area therefore reducing the pressure applied to the subgrade.

Along with the implications for the construction of housing, the low bearing capacity and CBR presented additional problems for road pavement construction and for the installation of service lines.

The solution that was adopted for the placement of the service lines was to undercut 300mm below the pipe bedding and replace with hardfill/sand wrapped in filter cloth as shown below in figure 5. The pipe bedding was one sixth of the pipe diameter.

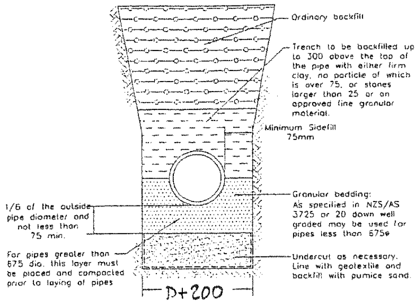

Figure 5: Typical Service Line Construction Detail

For pavement construction the initial solutions involved subgrade stabilisation using lime. Lime testing showed an increase in CBR from 4 to 5 with 6 kilograms of lime used per square metre per 250mm depth (Melville-Smith, 8).

This solution was then discarded in favour of an increased pavement depth. The pavement depth was increased to 400mm with a further undercut of 300mm to be replaced with sand (Melville-Smith, 9). At the time of writing this paper a proposal to use geogrid reinforcing beneath the road pavement was being considered. The aim of this is to reduce the depth of undercut required.

5.3 POTENTIAL FOR LARGE TOTAL AND DIFFERENTIAL SETTLEMENTS

The groundwater level on this block was relatively high. Due to the presence of compressible peats it was imperative that the groundwater level be maintained at its pre-development levels to reduce settlement that may occur.

To help achieve this the subsoil drains along the accessways were designed to serve the dual purpose of protecting the road subbase and allowing the groundwater to recharge the peaty soils.

Because water flows from high to low head, it was important that the groundwater flow was unobstructed. If the backfilling of service lines was compacted to a higher standard than the insitu material the groundwater flow could be obstructed. This higher density material could act as a dam causing the water table on the low head side of the service line to drop causing a change in effective stress resulting in settlement.

6.0 CONCLUSIONS

The three case studies that have been presented in this paper illustrate the variation in geological conditions in the Auckland Region. They also show that the geological conditions and associated challenges present in the Auckland Region require differing engineering solutions for the problems that are encountered at a specific site. In effect, an engineering solution that may be appropriate for a particular site may not be appropriate for another.

Therefore it is important for the geotechnical professional to have a sound knowledge of the geological conditions present at their project location. This knowledge combined with the data obtained during the site investigation should enable the geotechnical professional to a provide an appropriate engineering solution to a site specific problem.

7.0 REFERENCES

1. KERMODE, L.O., “Geology of the Auckland Urban Area”, Institute of Geological and Nuclear Sciences, 1992, p8

2. EDBROOKE, S.W. (Compiler), “Geology of the Auckland Area”, Institute of Geological and Nuclear Sciences, 2001, p19, 20, 23

3. BEVIN, J., “Supplementary Geotechnical Report for Gills Road Residential Subdivision”, September 2000, p6, 7, 11

4. BEVIN, J., “Geotechnical Investigation Report for Gills Road Residential Subdivision”, February 2000, p26

5. BEVIN, J., “Geotechnical Investigation Report for Proposed Extensions at Kingsway School”, October 1998, p4

6. RAFFERTY, W.J., “Pre-Quarternary Geology of Southern Northland”, August 2001, p29

7. LANDER, S.G., “Foundation Investigation Report for Proposed Building Developments at Kingsway School”, January 2003, p10, 12, 31

8. MELVILLE-SMITH, R.W., “Geotechnical Investigation Report for Addison Residential Development Stage 1A”, December 2002, p8, 10

9. MELVILLE-SMITH, R.W., “Geotechnical Completion Report for Addison Residential development State 1A”, November 2003, p7