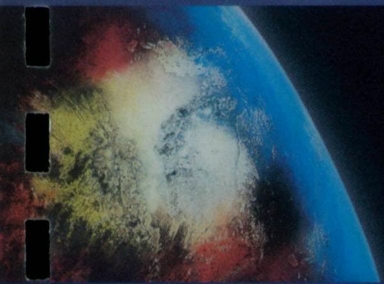

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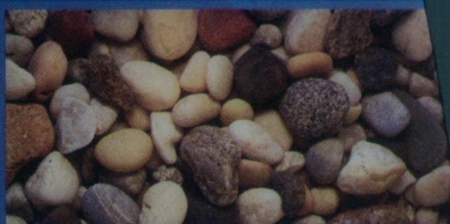

REPORT 7534

Watercare Services Ltd

Huia Water Treatment Plant Rebuild
Preliminary Geotechnical Assessment

Report prepared for:

WATERCARE SERVICES LTD

Report prepared by:

Tonkin & Taylor Ltd

Distribution:

WATERCARE SERVICES LTD	2 copies
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Executive Summary

Tonkin & Taylor was engaged by Watercares Services Ltd (Watercare) to carry out a preliminary appraisal of the geotechnical conditions at the Huia Water Treatment Plant to support the Huia WTP master planning process.

Our assessment has been based on a review of the existing geotechnical ground investigation data collated by others. The key findings are outlined below.

The two key geotechnical risks at the site are settlement of constructed structures and slope instability.

- Settlement
  - The existing infrastructure at the site has performed adequately since construction. Settlement of up to 39mm on shallow spread footings was recorded after construction of the clarifiers.
  - The potential for differential settlement needs to be addressed for areas that have previously accommodated structures (and hence a degree of settlement has already occurred).

We anticipate that for similar or reduced foundation pressures to the existing clarifiers shallow foundation systems could be designed which would accommodate the magnitude of previously measured settlements. Differential settlements could be addressed either by pre-loading areas that have not previously been developed, or by piling structures into the underlying highly weathered Cornwallis Formation.

- Slope instability
  - Preliminary analyses have indicated that a key risk to the site is the potential presence of a continuous shear surface at depth, represented by sheared defects noted in existing borehole logs.
  - While acceptable stability is analysed for static conditions, under the 1:2,500 AEP seismic event required for a Category 4 structure (serving a post disaster function), slope deformations of 100mm to 200mm could be expected.

To confirm or discount the presence of a continuous shear seam at the site, we anticipate further ground investigation would be required, targeting the potential locations for these shear seams developed from present information. If present, laboratory testing of the shear seams should be undertaken to confirm the design strength parameters.

It should be noted that the risks of movement in a 1:2,500 year AEP seismic event need to be taken in context of the rest of the network, in particular the dam and raw water line that supplies the treatment plant. There should be consistency in how these items are treated for seismic design.

We consider that the costs to completely remove the risks of slope movement under a 1:2,500 AEP seismic event would be significant and it may be that innovative structural design may be required to accommodate the calculated 100mm to 200mm slope displacements. If the proposed investigation discounts the continuous shear seam model we would expect acceptable levels of slope stability for design purposes.

Overall we consider that the geotechnical risks that are present at the current Huia Treatment Plant are no worse (and in some cases better) than those faced by similar sites within the foothills of the Waitakere Ranges.

1 Introduction

Tonkin & Taylor was engaged by Watercares Services Ltd (Watercare) to carry out a preliminary appraisal of the geotechnical conditions at the Huia Water Treatment Plant to support the Huia WTP master planning process.

Given that the site provides a significant percentage of the Auckland Regions water supply the risk of ground deformation due to settlement or slope instability during seismic events needs to be assessed.

Our scope of work for the present study was outlined in our letter of engagement dated 08 January 2010 and includes:

- Undertake a review of the collated historical ground investigation data presented in Tower Foundations Reports 5330 and 5331 entitled *Huia Water Treatment Plant - Review of Historical Geotechnical Information (Reference 1).*
- Review of existing geological models and revise these if required.
- Provide a summary report on the expected ground conditions addressing slope instability risk as well as providing preliminary assessments of foundation systems for new structures.

2 Review of Data Sources

The Huia Water Treatment Plant has had a variety of different sub-surface investigation programs undertaken with the collated data stretching as far back as the mid 1980's.

The key ground investigation data includes:

- Investigation boreholes by the Auckland Regional Authority in 1988 (BB series)
- Investigation boreholes undertaken by Beca in 2005 (BBH series)
- Investigation boreholes undertaken by Tonkin & Taylor in 2005 (AH and BH series)
- Investigation boreholes undertaken by Geolab in 2008 (MB series)
- Investigation boreholes undertaken by Ormiston Associates in 2008 (MH Series)

This information has been collated by others and a copy of the borehole location plan has been incorporated in Appendix A.

3 Ground and Groundwater Conditions

3.1 Geotechnical Units

The borehole data indicates that the site is generally underlain by the following geotechnical units.

3.1.1 Fill

Placed fill has been located to the south of much of the existing infrastructure and has been used to develop the historical sludge ponds (subsequently infilled in the last few years). The fill material comprises reworked site won materials consisting of clayey or sandy silts, which are yellow brown to greenish grey brown, with fragments of fine gravel or coarse sand. This material is typically firm to stiff with shear strengths ranging from 29/15 kPa to 130/41kPa (peak/residual) and SPT N values ranging between 0 and 10 blows for 300mm. Towards the base of this unit, thin layers of organic rich (fibrous) silts are present indicating that scarification of the original ground surfaced prior to fill placement was of variable quality or not carried out at all. The fill materials are generally between 3 and 6m thick.

3.1.2 Colluvium

Colluvial materials are present at the site both at ground surface on the upper parts of the slope adjacent to Woodlands Park Road and underlying the fill to the south of the existing infrastructure.

The colluvium comprises silts with some clay and sand (fine to coarse) reworked from the underlying Cornwallis Formation and possibly from the bluffs of Nihotupu Formation materials to the north of the site. The colluvium is typically light brown grey mottled orange in places with rare organic inclusions. Shear strengths range between 27/6 kPa and >200 kPa, but typically fall within the range of 70-140kPa (stiff to very stiff), while SPT N values of 3 to 11 were recorded for this material.

Reference 1 has reviewed strengths of Colluvial soils based on consolidated undrained triaxial test results. The raw data from all of the test results was combined and an overall strength envelope assessed. The assessed results from that study have been reviewed and are considered appropriate, and the lower bound Mohr Coulomb effective stress strengths of c' = 0 and phi' = 32° are recommended for this material.

3.1.3 Completely/Highly Weathered Cornwallis Formation

This material consists of fine to coarse dense sand or hard silt with some fine to medium grained rounded gravel in places. It is typically light to dark orange brown having weathered insitu from the underlying Cornwallis Formation rock mass.

There is typically a marked increase in SPT N values within this material compared with those recorded within the colluvium above, ranging between approximately 15 and 30 blows per 300mm. In some thin zones highly weathered sandstone rock with SPT N values greater than 50 blows per 300mm recorded, but these are outliers to the general trend of strengths in this material.

Laminations of silt within the coarse sand can often be observed and are indicative of relict sedimentary structure within the soil mass.

3.1.4 Softened (MW) Cornwallis Formation

Towards the base of the slope beneath the old sludge ponds and underlying the CW/HW Cornwallis Formation a 10m thickness of softened Cornwallis Formation materials are present. These materials maintain the colour (greenish grey and pink speckled) and sedimentary structure of the underlying rock mass but SPT N values indicate that this unit is either poorly cemented or has been moderately weathered (MW) and softened from a true rock mass with results ranging between 25 and 45 blows per 300mm.

It is typically recovered as a greenish grey speckled pink and white, very dense, fine to coarse grained silty sand.

3.1.5 Slightly Weathered Cornwallis Formation

The underlying rock mass comprises fine to medium grading to fine to coarse sandstone with occasional thin beds of siltstone. The rock mass is typically greenish grey with pink white speckling. Field estimates of intact rock strengths range between very weak and weak, and the rock mass is typically slightly weathered.

SPT N values recorded in this material are all in excess of 50 blows per 300mm.

3.2 Geological Structure

Previous mapping undertaken by others (reference 2) indicates that the general trend of bedding within the Waima Catchment (within which the Treatment Plant is located) is for moderate bedding dips (5 to 10 degrees) towards the northwest. This regional trend is evident in the borehole core where sub-horizontal to gently inclined bedding is apparent in weathered and unweathered Cornwallis Formation rock mass at depth.

Jointing in the underlying rock mass is typically very steeply inclined to moderately inclined with joint spacing ranging between closely and widely spaced. The joint surfaces are typically clean in unweathered core or have a silt coating where evident in weathered core.

Defects that show signs of historical movement were encountered at 8.5m in borehole BBH3 (slickensided, smooth, planar sandstone interface), at 6.7m in borehole BBH2 (gently inclined, slickensided {waxy} brown organic silt lamination, broken and offset 5mm vertically), at 13.6m and 14.8m in BBH1 (polished and slickensided defects dipping 45 and 30 degrees respectively) and at 10.8m in BBH 5 (50mm of firm dark green grey silt with polished clasts of siltstone gravels included).

These logged zones of potential historical movement have been noted on the cross section developed in drawing 27064-01. The potential for a continuous shear surface made up of these intersections in borehole core is a key risk to the site and is addressed further in Section 4 below.

3.3 Groundwater

While piezometers have been installed in many of the historical boreholes drilled at the site, there is a scarcity of good groundwater monitoring information in the records that Tower Foundations have been able to collate from the Watercare Archives.

Groundwater information is typically limited to a single reading undertaken within a week of completion of the piezometers installation and noted on the borehole logs.

These results indicate groundwater levels at approximately 5m from ground surface in piezometers in colluvium soils in the upper slopes, increasing to with 3m of ground surface on the lower slopes adjacent to the old sludge ponds.

Piezometer screens in the underlying highly weathered Cornwallis Formation return depths to ground water of between 8m on the upper slopes reducing to 4m on the lower slopes adjacent to the old sludge ponds.

Without any additional information, these generalised groundwater profiles have been taken as design piezometric surfaces for this preliminary geotechnical assessment and are illustrated on Drawing 27064-01. This is a conservative assumption that could be modified as additional data becomes available.

Overall, we would expect a downward flow gradient within these materials with 2-4m of groundwater pressure at any location above the regional hydrostatic groundwater profile at depth. This needs to be verified by further monitoring.

3.4 Interpreted Geological Model

Our geological model for the site is illustrated in drawing 27064-01. The site is covered by a upper layer of colluvium between 5 and 7m thick, overlain on the lower slopes by placed fill, which is up to 5m thick.

The completely weathered (cw) to highly weathered (hw) Cornwallis Formation underlies the Colluvium and is between 7 and 10m thick. The potential for a continuous shear surface within this unit needs to be considered on the basis of borehole intersections outlined in Section 3.2 above.

The CW to HW Cornwallis Formation directly overlies slightly weathered Cornwallis Formation on the upper slopes, while on the lower slopes beneath the old sludge ponds an 8 to 10m thickness of Softened (moderately weathered) Cornwallis Formation lies between the completely weathered and slightly weathered Cornwallis Formation.

4 Geotechnical Design Considerations

Recommendations and opinions contained in this report are based on data from boreholes and geological mapping undertaken predominantly by others. The nature and continuity of subsoil away from the investigations are inferred but it must be appreciated that actual conditions could vary from the assumed model.

4.1 Slope Stability

4.1.1 General

The site has been significantly modified over the last 40 years with numerous areas of cut and fill and changes to site topography and any potential geomorphic evidence of historical slope instability has long since been remove.

While the steep slopes on the ridge to the North may be indicative of large ancient deep seated slope instability, this would encompass the entirety of the Little Muddy Creek Catchment. This hypothesis is not supported by the observed geological data from borehole logs within the subject site. Colluvium present on the site is expected to be derived from more localised slope failures above (to the north of) the site.

Overall slope angles are 10°, made up of:

- Approx 10° slopes extending onto the site from above Woodlands Park Road upon which the clarifiers have been constructed,
- Approx 5° lower slopes developed during construction of the old sludge ponds
- Approx 18-20° slopes into the gully that traverses obliquely across the southern boundary.

We note that borehole BBH2 drilled by Beca in 2005 included the installation of an inclinometer to a depth of 39m to monitor potential deep seated ground movements. Unfortunately the collated data (reference 1) does not include any information on the results of monitoring of this inclinometer.

We have been advised that for the purpose of design Watercare has categorised the treatment plant as an importance level 4 structure in accordance with AS/NZS 1170:0 2002. An importance level 4 structure indicates a seismic event with a 1:2,500 annual probability of exceedence. For design, peak ground accelerations of 0.30g have been calculated been in accordance with AS/NZS 11705:2004. While not specifically a geotechnical standard this peak ground acceleration has been adopted for slope stability design.

4.1.2 Material Strength Parameters

Material parameters for preliminary analyses have been selected on the basis of the material descriptions in borehole core, in-situ testing (Shear Vanes and SPT's) and laboratory testing where available. The assessed material parameters for the geotechnical units outlined in Section 3.1 above are outlined in Table 1.

Table 1: Assessed Preliminary Material Parameters

Geotechnical Unit	Bulk Density (kN/m^3)	Effective Cohesion (kPa)	Effective Friction Angle (°)
Fill	17.5	0	28
Colluvium	17.5	0	32
CW/HW Cornwallis Formation	18	0	35
Shear Surface	17.5	0	17
Softened Cornwallis Formation	18	5	35
SW Cornwallis Formation*	20	30	38

* modelled as impenetrable bedrock for the purposes of preliminary analyses

4.1.3 Results of Preliminary Modelling

Modelling has been undertaken using limit equilibrium methods automated by use of Geostudio (Slope W) 2007 software. Analyses have been undertaken on the basis of the geological model outlined in Section 3.4 and utilising the groundwater model outlined in Section 3.3 with applied material parameters from Table 1 above.

Two modelling cases have been considered:

- Case A: No continuous shear surface
- Case B: Continuous shear surface as indicated in drawing 27064-01.

Our preliminary analyses indicate that generally acceptable levels of stability are expected for case A under both static and seismic conditions. For Case B, static conditions are expected to result in acceptable levels of stability.

For Case B under seismic loading, slope movements are expected if a continuous shear surface connects the shears encountered in borehole core as described in section 3.2 above.

A comparison of yield acceleration (Seismic Load for FOS = 1) vs peak ground acceleration indicates a ratio of approximately 0.5. When compared with charts from Makdisi and Seed (reference 3), slope displacement for the 1:2,500 AEP seismic event of between 100 and 200mm may be expected.

4.2 Foundations

4.2.1 General

Initial infrastructure was constructed in 1926, with the majority of the remaining filter block and clarifiers constructed in the Early 1970's. It is our understanding that the existing infrastructure has been supported on shallow spread footings.

Information found in the Watercare archives and collated in reference 1 indicates that settlement monitoring of the clarifiers (constructed in 1973) indicated a maximum settlement of 39mm with a maximum differential of 30mm.

The clarifiers and other pieces of infrastructure have performed acceptably since construction and the levels of settlement experienced by the clarifiers are within typically accepted design tolerances.

4.2.2 Shallow Foundation Systems

On the basis of the historical settlement monitoring data we expect that structures with foundation pressures similar to those of the existing infrastructure should perform adequately with respect to bearing and settlement if constructed on shallow foundation systems.

However, while tolerable settlements are expected for foundation pressures similar to those applied by previous structures, additional settlements may arise around new structures in the event that higher foundation pressures are to be designed for.

Additionally, pre-consolidation of the soils beneath the footprint of the existing structures may result in reduced settlement magnitudes in these locations for any new structures as consolidation settlement has already occurred in these locations. However, high differential settlements may then be encountered where a new structure is to be partially constructed across both the footprint of the pre-existing structures and ground that has not previously accommodated a structure. This issue could be addressed by developing the new infrastructure entirely within the current structure footprints or by pre-loading those areas that have not previously been built upon.

4.2.3 Piled Foundation Systems

Pile foundations would be an appropriate alternative to support the design foundation pressures by transferring them to the top of the underlying completely to highly weathered Cornwallis Formation, which is expected to be present at depths ranging between 5-7m on the upper slopes and 10-12m on the lower slopes where the sludge ponds have been historically located.

We anticipate that piled foundations may be appropriate if high foundation pressures are anticipated. Piles may also be considered if structures are to be constructed out over areas of the backfill sludge lagoons or where there is concern over differential settlements where new structures are to span the footprints of existing structures.

5 Discussion

In general the collation of all of the archive investigation data for the site provides a good spatial distribution and there is a substantial degree of lithological information for the site. In addition there is a reasonable database of laboratory testing data to allow estimates of soil parameters and settlement to be undertaken as the design progresses.

However, despite a significant number of piezometers having been installed at the site, there is a shortage of groundwater information, with only a single reading in most boreholes. This lack of data makes interpretation of design groundwater pressures problematic and consideration should be given to undertaking a medium term groundwater monitoring programme to allow a more accurate assessment to be made. This could be undertaken by either manual reading or automated by use of groundwater level loggers (divers).

No data is available for the inclinometer installed in BBH2. This type of monitoring data is useful in calibrating stability models and if available should be requested from the consultant that undertook the work. The existing data should then be supplemented by an additional reading to compare with previous trends.

The potential for a low strength shear seam to be present within the completely weathered Cornwallis Formation cannot be discounted and should be actively investigated to verify or disprove this hypothesis. This is the single biggest geotechnical risk to the Water Treatment Plant rebuild as if it is present, analyses indicate slope movements of 100 to 200mm could occur for a 1:2,500 AEP seismic event. However, no movement is expected for lesser events such as the 1:500 AEP SLS2 event.

This risk can be mitigated in design by targeted borehole investigation to confirm or discount this interpretation. If such a seam exists, then appropriate laboratory testing to assess its material strengths should be undertaken during the investigation. We anticipate that this investigation would require 3-4 boreholes to approximately 20m depth. Laboratory testing of the strength of any intact shear seams encountered should also be carried out. We understand that consideration is being given to using land on the north side of Woodlands Park Road (the extent of current data); this area could be incorporated in any additional ground investigation and the data used to extend the geological model into this area. If there is no historical data in this area from the recently constructed tank to the North of the WTP then an allowance for a further 2 boreholes in this area should be made.

If this risk is confirmed then remedial actions will be required to ensure that the plant functions as intended during extreme events. Stabilisation of the site is unlikely to be economic due to the size of the potential slope movements. However, functionality of the structure to an appropriate level may be possible with careful structural design.

It should be noted that the risks of movement in a 1:2,500 year AEP seismic event need to be taken in context of the rest of the network, in particular the dam and raw water line that supplies the treatment plant. There should be consistency in how these items are treated for seismic design.

Overall we consider that the geotechnical risks that are present at the current Huia Treatment Plant are no worse (and in some cases better) than those faced by similar sites within the foothills of the Waitakere Ranges. Our preliminary assessment supports the most recent assessment (reference 2) with respect to expected settlements and large scale slope instability, although our interpretative geological model indicates slightly thicker colluvium at the site than previously suggested.

6 Conclusions & Recommendations

Tonkin & Taylor were engaged by Watercares Services Ltd to carry out a preliminary appraisal of the geotechnical conditions at the Huia Water Treatment Plant. The plant is due for a significant overhaul in the next two to five years with a mixture of new build and refurbishment and upgrade of existing infrastructure planned.

We have reviewed the collated archive geotechnical information provided to us (reference 1) and make the following general conclusions.

- The geological model outlined in drawing 27064-01 has been developed from the existing data. It comprises fill on the lower slopes, overlying up to 8m of Colluvium, in turn overlying completely to highly weathered Cornwallis Formation which has soil strengths. Depths to the underlying slightly weathered Cornwallis Formation rock mass range between 15m (upper slopes) and 27.5m (lower slopes).
- In some boreholes slickensided clay seams have been identified in the historical logging. The potential for these defect intersections in borehole core to represent a continuous shear surface has been considered.
- Groundwater information is sparse, but broadly indicates an upper piezometric surface in the Colluvium within 3-4m of ground surface and a lower piezometric surface within the CW/HW Cornwallis Formation around 8m below ground surface. We would expect a downward flow gradient within these materials with 2m to 4m of groundwater pressure at any location above the regional hydrostatic groundwater level at depth. This needs to be verified by further monitoring.

- Slope stability analyses indicate that the key geotechnical risk to the site is the potential for a continuous shear surface at depth. Under the 1:2,500 AEP seismic event, slope movements of between 100mm and 200mm would be expected to occur along a feature of this nature.
- The existing infrastructure at the site has performed adequately since much of it was constructed on shallow spread footings in the early 1970's. Settlement of up to 39mm was recorded during monitoring of the clarifiers after construction, and we anticipate that shallow foundation systems for the new build could be designed to accommodate this level of settlement for similar or reduced foundation pressures.
- The issue of differential settlement needs to be considered for the new builds. While shallow footings may be suitable, consideration needs to be given to the pre-consolidation that has occurred beneath existing infrastructure, compared to areas not previously constructed on. Where a new structure is to span across both areas, high differentials may result. This could be addressed by the placement of a pre-load prior to construction in areas not previously constructed upon, or alternatively piled foundation systems extending into the underlying CW/HW Cornwallis Formation may be considered.

While the geotechnical issues are not insignificant, they are similar to those faced by many sites in the Waitakere Ranges foothills, and need to be considered in the context of the remainder of the network which is in similar (or more challenging) terrain.

We make the following recommendations to progress the design work once capital expenditure funding has been secured:

i. A geotechnical site investigation programme should be undertaken to target potential locations of continuous shear seams, to verify or discount their presence. This would encompass between 3-4 boreholes to approximately 20m depth. If present an allowance should be made for laboratory testing of the slickensided defect surfaces encountered and if required an allowance for an additional 2 boreholes should be made for to extend the geological model to the North of Woodlands Park Road For your information and guidance only, we anticipate the costs of such an investigation, analysis and reporting package to be in the range of $40,000 to $45,000 (ex gst).

ii. A request should be made to Beca to source any data from the installation and subsequent monitoring of the inclinometer that has been installed in borehole BBH2. Once any baseline data has been reviewed then this inclinometer should be re-read and the results compared with previous monitoring data. For guidance only and assuming a baseline reading has already been undertaken, the cost to re-read this inclinometer and provide data analysis would be in the vicinity of $1,000 (ex gst).

iii. The groundwater database needs to be improved significantly. Several piezometers are already in place, but further monitoring needs to be undertaken prior to the bulk of the design being undertaken. This may be carried out by manual reading of water levels, or by installing groundwater divers to provide a continuous record of groundwater levels which can be downloaded as necessary. For your guidance only, installing three divers to automatically record groundwater levels for a 12 month period would cost in the vicinity of $5,000 (ex gst).

7 Applicability

This report has been prepared for the benefit of Watercare Services Ltd with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

The information used to base this reporting on has been collated by others and has been used in good faith. While every attempt has been made to verify the data we cannot guarantee the accuracy of data supplied by other parties.

During excavation and construction the site should be examined by an engineer or engineering geologist competent to judge whether the exposed subsoils are compatible with the inferred conditions on which this report has been based. We would be pleased to provide this service to you and believe your project would benefit from such continuity. However, it is important that we be contacted if there is any variation in subsoil conditions from those described in the report.

Tonkin & Taylor LTD

Environmental and Engineering Consultants

Report prepared by:	Authorised for Tonkin & Taylor by:
....	....
Cameron Lines	Chris Bauld
Engineering Geologist	Project Director

CJL

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8 References

i. Huia Water Treatment Plant – Review of Historical Geotechnical Information Volume 1 and Volume 2 (refs 5330 and 5331). Tower Foundations Ltd, October 2008. Ref JN004/08.

ii. Geological Map, Titirangi No 3 Water Reservoir, Ormiston Associates Ltd, September 2008, Drawing 2113/2950-1

iii. Makdisi, F. I., and Seed, H. B. 1978. "Simplified Procedure for Estimating Dam and Embankment Earthquake Induced Deformations," *Journal of the GeotechnicalEngineering Division*, ASCE, Vol 104, No. GT7, pp 849-867.

Appendix A: Borehole Investigation Plan

ISSUE	DATE	AMENDMENT	BY	APPD.		BY	DATE
–	7:08	FOR CLIENT CONSIDERATION	G.B.	T.C.	DESIGNED	T.C.	7-08
					DES. CHECKED	J.P.	7-08
					DRAWN	G.B.	7-08
					DWG. CHECKED	I.M.	7-08
					PROJECT LEADER	T.C.	7-08
					AM APPROVED	A.S.	7-08

OPERATIONS

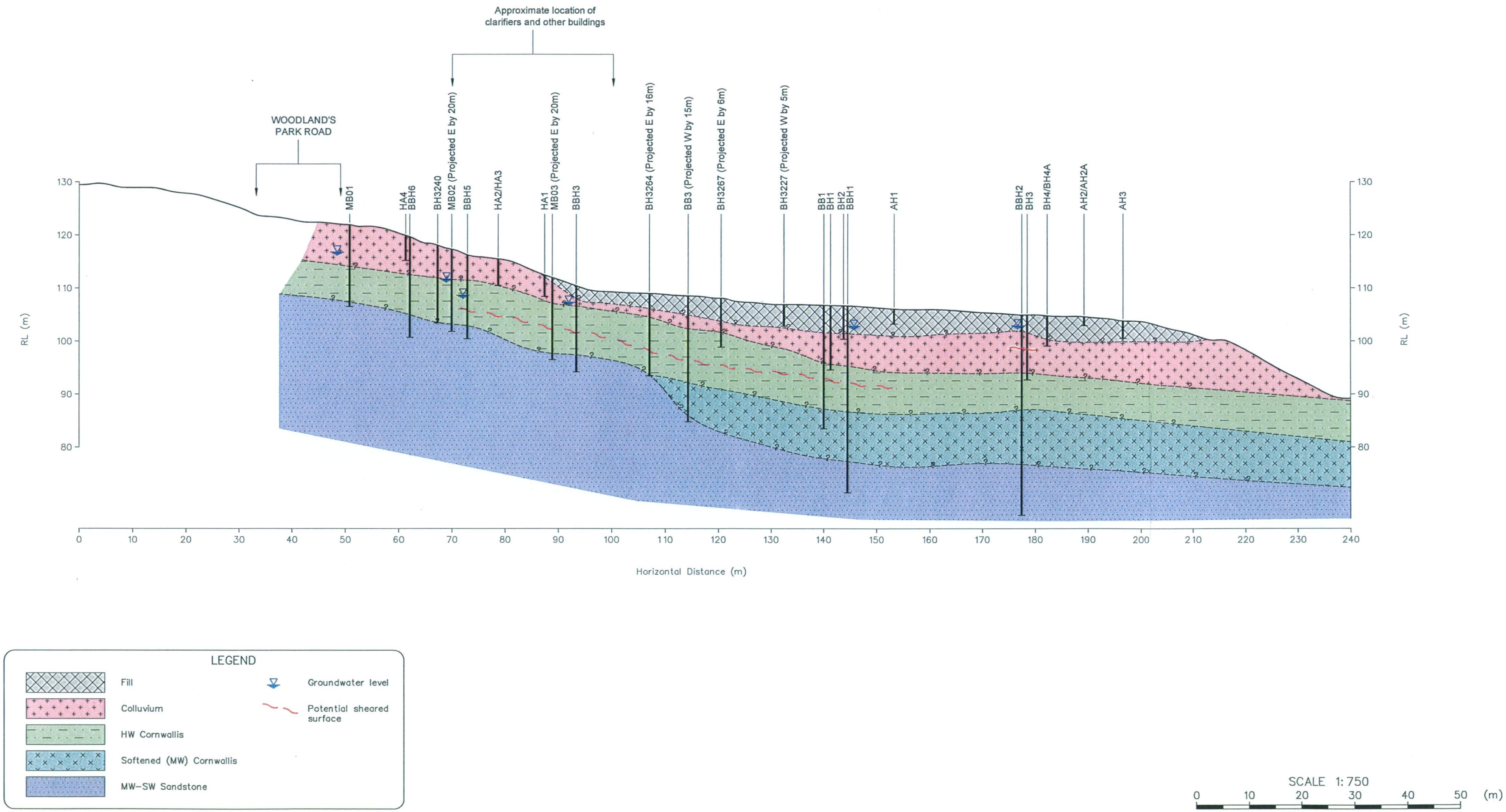
ASSET MANAGEMENT

water Care services limited

PROPOSED TITIRANGI RESERVOIR No3
RESERVOIR SITING – SCHEME 11
BOREHOLE LOCATION PLAN – SHEET 2

CAD FILE 2005523.017	DATE 9-09-08
ORIGINAL SCALE A1 1:500 (A1)	CONTRACT No. –
DRAWING No. 2005523 .017	ISSUE –

Appendix B: Geological Section - Drawing 27064-01



DRAWING STATUS: **PRELIMINARY DRAFT**

			DESIGNED :	CJL	Jan. 10	NOTES :
			DRAWN :	AGI	Jan. 10	1. All dimensions are in millimetres unless noted otherwise.
			DESIGN CHECKED :			2. Collar positions of old ARA boreholes not certain.
			DRAFTING CHECKED :			
			CADFILE : \\27064-01.dwg			
			APPROVED : NOT FOR CONSTRUCTION This drawing is not to be used for construction purposes unless signed as approved			
0 First Issue						
REVISION DESCRIPTION	BY	DATE	COPYRIGHT ON THIS DRAWING IS RESERVED			REFERENCE :

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CLIENT, PROJECT
WATERCARE SERVICES LTD
HUIA WATER TREATMENT PLANT

TITLE
PROPOSED INFRASTRUCTURE UPGRADE
Geological Cross Section A–A'

SCALES (AT A3 SIZE)	DWG. No.	REV.
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