Geotechnical Investigation Report
& Retaining Wall Design for
Proposed Dwelling
Waianga Place, Opononi,
Lot 10 Deposited Plan 546644
For Dennis Matene

Haigh Workman reference 21 259

January 2022

Revision History

Revision Nº	Issued By	Description	Date
A	John Power	First Issue	24/01/2022

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Executive Summary

Haigh Workman Ltd. (Haigh Workman) has been commissioned by Dennis Matene (the Client) to undertake a geotechnical investigation and retaining wall design for a proposed development at Lot 10 Waianga Place, Opononi (Lot 10, Deposited Plan 456644). Concept drawings provided by the client indicate a general layout for a proposed dwelling with a future separate garage that is to be on the already prepared cut platform. Based on discussions with the Client, we envisage that the proposed dwelling will comprise a lightweight structure with a suspended timber floor supported on concrete encased pile foundations. It is considered that the future garage will comprise a concrete slab-on-grade type foundation. A level platform has been created for the proposed dwelling location, comprising cut to fill earthworks. Earthworks have been undertaken to create the level building platforms, with an existing cut face above the proposed dwelling up to 2.5 m high (RW01), and the site-won fill material used as a sidling fill downslope of the proposed dwelling to create a larger level area (RW02).

Based on the results of the geotechnical investigation conducted by Haigh Workman and review of published geological maps, it is considered that the natural soils directly underlying the proposed development sites comprise natural soils of the Waitiiti Formation (Mot) of the Otaua Group. Variable depths of non-certified fill material were encountered across and downslope of the proposed building platform.

Two retaining walls are proposed, with RW01 supporting a cut face, and RW02 supporting fill material. RW01 is required to provide ground stabilisation due to the excavations undertaken effectively removing the buttress support and reducing the global stability of the site, i.e., the wall is required to provide a long-term safe building platform. RW02 is recommended to support the sidling fill beneath the proposed dwelling. All retaining wall construction will comprise timber pole retaining walls encased in concrete, with horizontal timber rail lagging to distribute the lateral load to the timber poles. Refer Section 4 for design details.

The natural fine grained clayey soils of the Waitiiti Formation are considered susceptible to swelling and shrinking under seasonal variations of water content. For the purposes of design, the site may be designated as highly reactive (Class H1) in accordance with AS2870:2011. If RW02 is constructed to the design outlined within this report (refer Section 4), then concrete encased timber piled foundations can be designed in accordance with NZS3604:2011, provided the foundations are embedded at least 1.0 m into stiff natural soils, i.e., founded below the non-certified fill material, approximately 3.0 m along the southern side of the dwelling. If RW02 is not constructed, then foundations will need to be designed to take lateral loads imposed from the sidling fill over the top 2.0 m of the foundation pile and subject to specific structural design to size the foundation piles. Foundation design recommendations are provided in Section 5.

We consider the following specific items, but not limited to will need to be addressed prior to and at the time of construction to ensure the foundation soils are consistent with the assumptions made in this geotechnical report:

1. Geotechnical drawing review to confirm the foundation design is as per the geotechnical recommendations and that the location of the proposed dwelling is in accordance with the geotechnical findings.

2. Observe the ground conditions within retaining wall pile holes and foundation excavations prior to pouring of concrete and ensure foundations are founded into stiff natural soils, including dwelling, and retaining wall foundations.

Provision should be allowed for modifying the foundation solution at this time should unforeseen ground conditions be encountered.

1 *Introduction*

1.1 Project Brief and Scope

Haigh Workman Ltd. (Haigh Workman) has been commissioned by Dennis Matene (the Client) to undertake a geotechnical investigation for a proposed development at Waianga Place, Opononi (Lot 10, Deposited Plan 546644). This report presents the information gathered during the site investigations, interpretation of data obtained and site-specific geotechnical recommendations relevant to the site.

The scope of this report encompasses the geotechnical suitability in the context of the proposed development as defined in the Short Form Agreement dated 22 September 2021. This appraisal has been designed to assess the subsoil conditions for foundation design and identify geotechnical constraints for the proposed development.

This report provides the following:

- A summary of the published geology with reference to the geotechnical investigations undertaken.
- Analysis of the data obtained from site investigations and a geological ground model.
- Foundation recommendations.
- Comment on ground stability.
- Design of timber pole retaining walls to support cuts and fills.
- Identification of any additional geotechnical risks and/or hazards.

1.2 Proposed Development

Concept drawings provided by the client indicate a general layout for a proposed dwelling with a future separate garage that is to be located generally centrally on a prepared cut platform. Based on discussions with the client, we envisage that the proposed dwelling will comprise a lightweight structure with a suspended timber floor supported on concrete encased pile foundations. It is considered that the future garage will comprise a concrete slab-on-grade type foundation. A level platform has been created for the proposed dwelling location, comprising cut to fill earthworks. No additional, significant earthworks are anticipated as part of the proposed development other than foundation and retaining wall excavations. Earthworks have been undertaken to create the level building platform, resulting in an unsupported cut face above and to the north of the proposed dwelling, and site-won fill material placed along the southern, downslope edge of the proposed build platform, resulting in a lower, unsupported face.

The report considers the geotechnical aspects of the site with regards to the proposed development with reference to the development locations, (refer to Figure 1 and Appendix A). Should the proposed development vary from the proposals described above and/or be relocated outside of the investigated areas, further investigation and/or amendments to the recommendations made in this report may be required.

1.3 Site Description

The property is legally described as Lot 10 Deposited Plan 546644 with a total land area of 1,288m². The property is generally rectangular in plan shape, elongated east to west across the slope. The site comprises a bare lot with minor grass cover across the property. The proposed dwelling is to be across the central and southern part of the property, with a future proposed garage located towards the northeast corner of the property. The site is located on the southern side of a generally south to southwest facing slope that forms the western end of an east to west trending ridge spur. The natural slope angles across the broader south facing slope prior to recent earthworks is understood to have been in the order of 20° to 28°. At the time of our investigation, the subject site had undergone some modification with the formation of a cut access track that extends from Waianga Place along the southern property boundary to the cut building platform, refer Figure 1 and Appendix A.

The building platform has been developed for the proposed dwelling by means of cut and fill earthworks, with a significant cut along or close to the northern property boundary and the placement of cut material across the southern and western extent of the property to create a level build platform. In addition to the building platform, earthworks comprising cut and fill have been completed to create the existing access track. No retaining structures had been installed at the time of our site investigation.

The cut face along the northern side of the build platform is currently unretained and some degradation of the cut faces due to prolonged exposure has occurred with some small-scale collapse and slope debris observed along the cut face. Similar erosional features and degradation of the fill slope below and to the south of the building platform was also observed. Below and to the south of the property, the ground contour becomes generally gentle with slope angles of between 2° to 5° recorded.

At the time of our investigation, the proposed development footprints were not marked out onsite. Investigation locations have been based on drawings provided by the client.

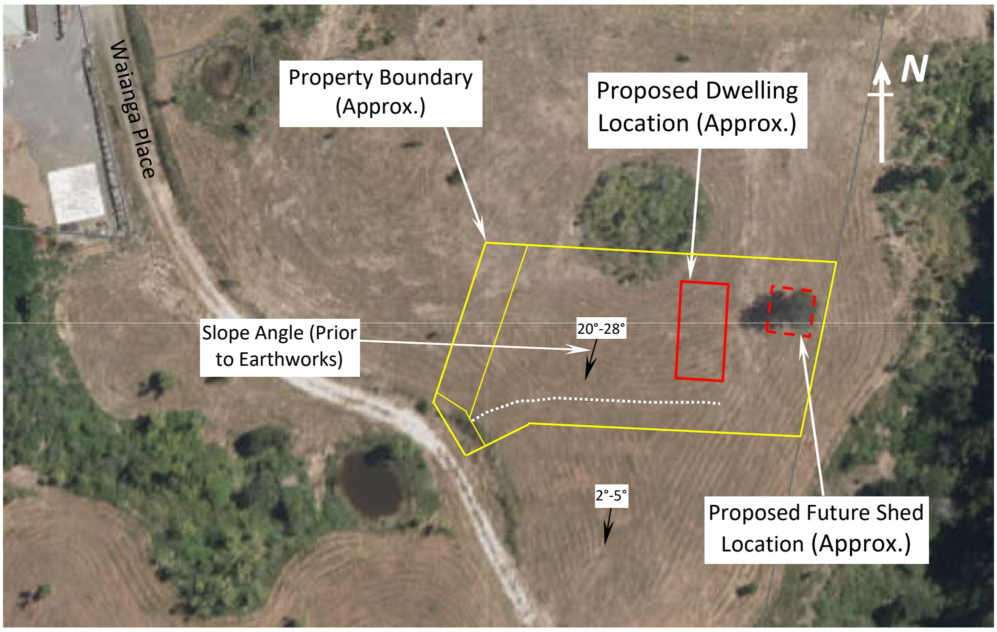

Figure 1 - Property Location

2 Geology

2.1 Published Geology

Sources of Information:

- Institute of Geological & Nuclear Sciences 1:250,000 Geological Map 1, 1996: "*Geology of the Kaitaia*".
- NZMS 290 Sheet O 06/07, 1: 100,000 scale, 1982: *"Waipoua-Aranga" Rock Types.*
- NZMS 290 Sheet O 06/07, 1: 100,000 scale, 1980: *"Waipoua-Aranga" Soil Types.*

The site is within the bounds of the GNS Geological Map 1 "Geology of the Kaitaia area", 1:250,000 scale*. The published geology shows the site to be underlain by Waitiiti Formation (Mot), belonging to the Otaua Group. The Waitiiti Formation is considered to be of Early Miocene, comprising massive to poorly bedded mudstone and muddy sandstone.

An exert of the geological map is shown in Figure 2 below, with geological units presented in Table 1 below.

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* Isaac, M.J. (compiler) 1996. Geology of the Kaitaia area. Institute of Geological and Nuclear Sciences 1:250 000 geological map 1.

Figure 2 - Geological Map Extract

Table 1 - Geological Legend

Symbol	Unit Name	Description
Mot	Waitiiti Formation. (Otaua Group)	Massive to poorly bedded mudstone and muddy sandstone. The Waitiiti Formation is considered to be of Early Miocene age.
Mow	Waiwhatawhata Conglomerate and Otueka Formation. (Otaua Group)	Conglomerate and sandstone derived from the Northland Allochthon. Early Miocene age.

Further reference to the published New Zealand land inventory maps (Ahipara-Herekino), indicates the site is underlain by; *“soils of the rolling and hilly land, imperfectly to very poorly drained – Omanaia clay loam with coarse structural subsoil (ONe)”*. The underlying material weathers to *“soft, reddish-brown clay containing moderately soft cores to depths of 20m”*.

3 Ground Investigations

3.1 Subsurface Investigations

Haigh Workman undertook geotechnical investigations on 16 November 2021. The investigations comprised the drilling of six hand auger boreholes (BH01 to BH06), located across the proposed development area with BH02 to BH04 located across the general area of the proposed dwelling and BH01 located across the general area for the proposed future garage. In addition to the four hand auger boreholes drilled for the proposed dwelling and garage, two boreholes were located and drilled with the intention of identifying the depth of fill material on the site and determine the natural ground conditions below that fill material.

Hand auger boreholes were advanced to a maximum depth of 3.0 metres below ground level (mbgl). A hand shear vane with 19 mm blade was used to measure the Vane Shear Strength of the in-situ material. Vane shear tests were undertaken at regular intervals during the advancement of the hand augers. All shear strengths shown on the appended logs are Vane Shear Strengths in accordance with NZGS; “Test Method for Determining the Vane Shear Strength of a Cohesive Soil using a Handheld Shear Vane”, 2001.

Investigations were logged in accordance with The New Zealand Geotechnical Society, “Guidelines for the Field Classification and Description of Soil and Rock for Engineering Purposes” (2005). Investigation locations are shown on the drawings in Appendix A and investigation hand auger logs are included in Appendix B.

3.2 Ground Conditions

Based on the results of the geotechnical investigation conducted by Haigh Workman and review of published geological maps, it is considered that the natural soils directly underlying the proposed development site comprise natural soils of the Waitiiti Formation (Mot). Soils of the Waitiiti Formation were encountered from surface within BH01 and BH02 with boreholes BH03 to BH06 having variable amounts of non-certified fill material overlying natural soils of the Waitiiti Formation. Borehole BH04 encountered natural soils of the Waitiiti Formation below a layer of non-certified fill material and a thin (0.1m) veneer of buried topsoil.

For the purposes of this report, subsoil conditions on the site have been interpolated between the boreholes and some variation between borehole positions are likely. Detailed logs are presented within Appendix B. Table 2 below, summarises the materials encountered, with depth to base of each unit provided.

Table 2 - Summary of Borehole Results

Borehole Number	Topsoil (mbgl)	Non-certified Fill Material (mbgl)	Tangihua Complex Soils (mbgl)	Soil Moisture and Groundwater Observations
BH01	NE	NA	0.0 to >3.0	Groundwater encountered at 2.8mbgl.
BH02	NE	NA	0.0 to >3.0	Groundwater encountered at 1.9mbgl.
BH03	NE	1.1	1.1 to >3.0	Groundwater encountered at 2.8mbgl.
BH04	0.6 to 0.7	0.0 to 0.6	0.7 to >2.0	Static groundwater not encountered.
BH05	NE	0.0 to 1.6	1.6 to >3.0	
BH06	NE	0.0 to 0.6	0.6 to >2.0	Groundwater encountered at 1.8mbgl.

Note - Depths measured from existing ground surface level.

NE – Not Encountered.

3.2.1 *Topsoil*

A buried topsoil layer was encountered within BH04 only, between 0.6m and 0.7mbgl. No other topsoil layer was encountered within our boreholes. It is considered that any pre-existing topsoil has been removed as part of the cut modifications at the locations of BH01, BH02 and it appears that any topsoil layer has been removed prior to the placement of the non-certified fill material within BH03, BH05 and BH06. The topsoil encountered has generally been described as a firm, brown silt with minor clay.

3.2.2 *Non-certified Fill Material*

Non-certified fill material was encountered within BH03, BH04, BH05 and BH06 to depths of between 0.6m to 1.6mbgl. It is considered that the non-certified fill material encountered comprises cut material as part of the site earthworks that have been undertaken during development of the building platform. Based on our site observations, we understand that the cut fill material has been placed over the natural ground contour during development of the cut platform, creating a wedge of fill material on the downslope, southern side of the proposed build platform with a secondary, smaller wedge of fill material along the southern edge of the access track. Our investigations indicate that the fill material within BH04 has been placed over the original topsoil surface and the underlying Waitiiti Formation soils.

The non-certified fill material has been described as light grey, light brown, light orange, brown and orangish brown intermixed. The fill material is further described as comprising a clayey silt, silty clay and clay that is generally firm to very stiff, dry to moist and of having medium to high plasticity. Natural soils of the Waitiiti Formation were encountered below the non-certified fill material and buried topsoil layer.

3.2.3 *Waitiiti Formation Soils*

The natural soils encountered have been interpreted as soils belonging to the Waitiiti Formation and were encountered within all six boreholes. The natural soils are generally described as being stiff to very stiff variable silts, clayey silts, silty clays and clays with trace fine gravel content in parts.

The soils were variably coloured from light grey, light orange, brownish orange and brownish grey, streaked and mottled orange, brown, grey, dark orange and light bluish grey. Soils were further described as being generally moist to wet and of having medium to high plasticity. Vane shear strength tests ranged from 45kPa to 226kPa, indicative of firm to very stiff soils. However, all but one vane shear test, returned results greater than 68kPa, indicating generally stiff to very stiff soils. Recorded vane shear strengths are shown on the appended borehole logs.

3.2.4 *Groundwater*

Groundwater was encountered within boreholes BH01, BH02, BH03 and BH06, measured at depths of between 1.8mbgl and 2.8mbgl. Groundwater levels were measured shortly after drilling was completed, it is considered that groundwater levels will not have had sufficient time to reach equilibrium, i.e., the fine-grained clayey soils have low permeability, and it may take several days for the groundwater level to equalise. Measured groundwater levels can be expected to change after the effects of ground investigations have dissipated, i.e., if water is perched within the upper clay layers, then water levels may reduce over time. At the time of drilling the boreholes, the surface conditions were generally dry. However, some minor surface water seepage was observed on the slopes above the subject property. Groundwater levels can and do fluctuate and higher groundwater levels may be encountered following periods of prolonged or heavy rainfall.

4 *Geotechnical Assessment*

4.1 Geotechnical Design Parameters

Geotechnical design parameters recommended in this report are based on in-situ test results and local knowledge of similar soils. Refer Table 3 below for soil parameters adopted within this report.

Table 3 - Geotechnical Design Parameters

Soil Unit	Bulk Unit Weight γ (kN/m^3)	Undrained Shear Strength S_u (kPa)	Effective Cohesion c' (kPa)	Effective Friction Angle φ' (degrees)	Young's Modulus, E (MPa)
Non-certified clay fill	17	40	2	25	10
Very stiff residual soils	18	80	5	29	25
Completely weathered Otaua Group	18	100	5	30	50

4.2 Slope Stability Assessment

4.2.1 *General*

During our site walkover survey, no evidence of global slope instability was identified across the proposed development area, i.e., the flat building platform. However, the existing cut face that extends along the northern side of the proposed building platform is currently unretained and some degradation/sloughing of the cut face due to prolonged exposure has occurred with some small-scale collapse and slope debris at the toe of the cut face. An area of slumping and collapse of the cut face of 4m wide (approx.) was observed near the northwest end of the cut face. It is considered that the observed collapse is likely to be due to oversaturation of the cut face, with collapse debris being generally wet to saturated at the time of our observations. Evidence of surface water flow across parts of the site were observed with some generally, shallow erosional features observed. Based on the current configuration of the slopes to the north of the subject site, it is considered that surface water is being shed from the slopes to the north, across the subject site, resulting in most of the observed erosional features. Stormwater management and drainage will be required to control overland stormwater flows from the slopes to the north.

Similar erosional features of the unretained fill batter below the proposed building platform was also observed. Both the cut face and the fill batter will require retaining walls to ensure the long-term stability of the cut and fill faces, as detailed within Section 5.

The broader slopes to the north of the subject site have undergone some modification to the original site contour, with some earthworks, vegetation removal and recontouring of the broader northern slopes. No obvious signs of slope instability were observed across the broader modified slopes to the north. However, it is anticipated that continued degradation and erosion of the exposed soils across the slopes will continue without stormwater management and disposal being considered.

In general, and ignoring any modified ground (i.e., unsupported cut faces and fill batters) around the proposed development area, it is considered that the natural slope angle within the Waitiiti Formation soils is in the order of 20° to 28° and are generally suitable for development. The earthworks undertaken recently have resulted in a cut face, removing some buttress effect of the northern slopes, reducing the overall global stability, site specific stability modelling has been undertaken, which has resulted in ground stabilisation works being required.

4.2.2 *Geological Ground Model*

A geological ground model has been developed based on the site investigation data. The ground surface was determined by tape and clinometer survey, undertaken as part of our site investigations in conjunction with topographical survey data.

The purpose of developing the geological ground model was to assist with site development plans and retaining wall design. Sections A-A' and B-B' were developed for site assessment purposes, refer Appendix A.

4.2.3 *Seismic Design*

Anticipated peak ground accelerations have been estimated assuming Site Class C, as per NZS 1170.5. The seismic coefficients for geotechnical design are based on the NZTA Bridge Manual SP/M/022 (NZBM) and NZS1170. Accordingly, the peak ground acceleration for pseudo-static horizontal seismic analysis is 0.1 g for a 150-year return period event.

4.2.4 *Stability Analysis*

Slope stability analyses were undertaken using computer software by Rocscience, Slide2 (Version 9.012). Groundwater level has been assessed using a groundwater piezometric surface based on the measurements taken during the site investigations. Groundwater level has been assumed as approximately 1.0 m higher to represent normal groundwater conditions during winter. The nature of the site, having high groundwater under normal conditions, results in a minimal difference between normal and extreme groundwater conditions, resulting in the normal groundwater conditions being taken as the critical case for design and remediation. Selected outputs are presented in Appendix D. The criteria adopted for assessing the global stability is outlined in Table 4 below.

4.2.5 *Modelling Philosophy*

The model was developed with the proposed concept as discussed with the Client. A back analysis was undertaken of the pre-existing site conditions, prior to the cut excavation being undertaken. The purpose of the back analysis was to develop a baseline for the site prior to the site excavations being undertaken and to determine if ground stabilisation works are required. The design factors of safety required post development are provided in Table 4.

Table 4 - Design Factors of Safety (FOS)

Load Case	Design Factor of Safety	Groundwater Conditions
Static – proposed development	≥ 1.5	Highest credible groundwater level – steady state seepage
Static, elevated groundwater	≥ 1.3	Elevated groundwater level
Seismic, 0.1 g (150yr return period)	≥ 1.1	Highest credible groundwater level

4.2.6 *Stability Analysis Results*

Slope stability results are presented in Table 5 below. The earthworks undertaken have reduced the global stability of the site and as a result ground stabilisation is required, this has been modelled as a timber pole retaining structure.

Table 5 - Stability Results

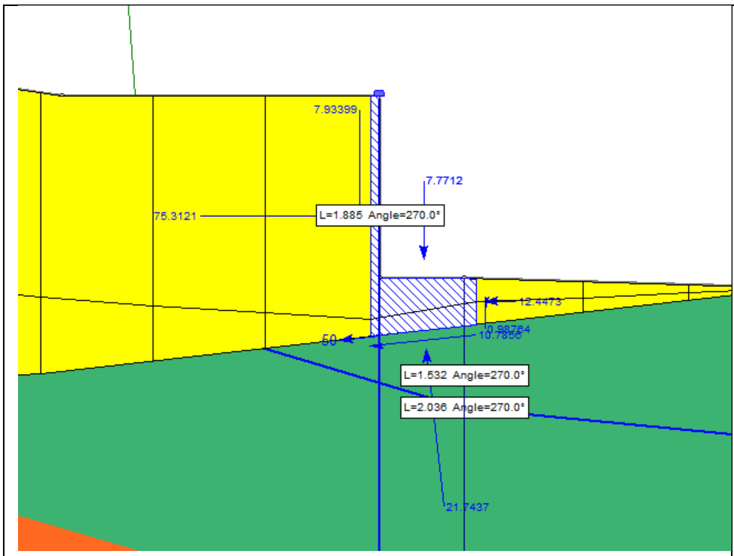
Section I.D.	Scenario	Result	Required	Outcome	Notes
01	Pre-existing site conditions	1.43	1.50	NOT OK	Pre-existing condition has been undertaken to assess the global stability and to check if the earthworks have worsened the site. This site condition is a baseline value only.
	Existing site conditions	1.37	1.50	NOT OK	The excavations have resulted in a reduction in the global stability due to loss of buttress effect. Stabilisation required.
	Ground stabilisation – retaining wall model.	1.51	1.50	OK	Retaining wall – 50 kN shear capacity. Slice height = 1.9 m (factor of safety of 1.5 required intersecting the pile).
	Ground stabilisation – retaining wall model. Seismic, 0.1 g	1.19	1.10	OK	Static case governs design.

The stability results show acceptable factors of safety are achieved at the proposed dwelling location under the conditions analysed, provided ground stabilisation is undertaken on the northern cut face slope, refer Section 4.3.

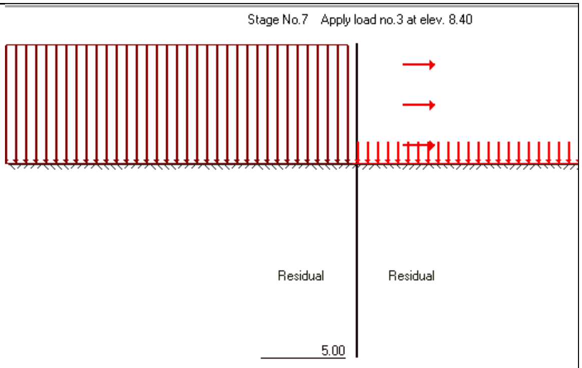
4.3 Timber Pole Wall Design

A retaining wall is required to support the existing cut face near the northern boundary (RW01), and a second wall is recommended to support the fill slope created (RW02). Geotechnical design parameters presented in Table 3 have been adopted in design. The wall is approximately 35 m in length (subject to final arrangement of the dwelling and other structures), with a maximum retained height of 1.5 m.

RW01 was designed using the slice forces taken from within the stability analysis. The forces on the slice were used to represent horizontal loads within Wallap. Staging within Wallap included removal of soil on both sides of the wall, applying a surcharge on the active side of the wall that is representative the existing soil condition and horizontal loads taken from the stability model (this was undertaken to not double up the loads using active earth pressure theory). Figure 3 shows how the loads were computed and applied under static conditions, the same approach was taken under seismic conditions, i.e., loads acting on the wall under 0.1g are taken from the stability assessment.



Horizontal force = 75 – 12 - 11 = 52 kN with a factor of safety = 1.51 [Design pile shear force of 50 kN applied in stability software].



Horizontal loads added into Wallap – triangular distribution to give a maximum shear force of 50 kN (static).

Figure 3 - Retaining wall loads

Design actions, deflections and length of embedment were derived from the analysis undertaken using Wallap (Version 6.06) using the subgrade reaction model with active and passive limit pressures calculated using wedge stability analysis.

Moment actions and shear forces have been taken from the analysis and used for design of the timber poles for, resulting in a design bending moment of 55 kNm, design shear force of 60 kN, and displacement of 56 mm (taking into consideration spacing of the poles) for RW01. RW02 has adopted conventional earth pressure theory as it is not subject to global instability.

The retaining wall is composed of high-density timber poles encased in 20 MPa of concrete with grade G8 timber horizontal planks, or rough sawn 150 x 50 mm H4 treated lagging. A summary of the analysis is presented in Table 6. Lagging details are provided on the typical sections within Appendix A.

Table 6 - Retaining Wall Design Summary

Wall Properties	RW01	RW02
Maximum Height (H)	1.2	1.5 / 1.0
Pole Spacing (c/c)	1.2 m	1.2
Pole type	High Density, 325 mm SED, H5 treated	High Density, 200 mm SED, H5 treated
Embedment Length (L)	3.5 m	2.5 m / 1.5 m
Total Pile Length (H + L)	5.0 m	4.0 m / 1.0 m

Encasement	0.6 m bored pile, encased in 20 MPa concrete	0.45 m bored pile, encased in 20 MPa concrete
Timber Lagging rails (from top of wall) – spanning continuously across a minimum of 2 poles.	refer drawings	

4.3.1 *Safety in Design*

A safety in design register has been prepared and should be updated during construction when required.

Table 7 - Safety in Design Risk Register

Issue	Risk	Proposed mitigation measure
Excavations	Collapse of material and potential to strike people	All earthworks to be staged where possible and cuts to remain open for the smallest possible duration. No one to work immediately adjacent to the cut or during poor weather conditions.
Open auger holes	Falling from height	No holes to remain open overnight. No one allowed to walk around the construction site, other than those who understand site hazards. Holes should be backfilled with concrete as soon as possible.
Lifting timber poles and putting into ground	Falling from height (heavy)	Lifting gear (straps and chains) to be in good condition and certified if required.
Groundwater	If encountered, groundwater will make constructability difficult	We expect holes to remain free of groundwater in the short term. Holes not to remain open overnight and should be backfilled as soon as possible with concrete. Pumping may be required.

5 *Foundation Recommendations*

5.1 General

At the time of writing, no concept drawings for the proposed development were available. However, based on discussions with the client, we understand that the proposed development will comprise the construction of a single level, light weight dwelling with a suspended timber floor supported on concrete encased timber pile foundations. The proposed future garage will be a slab-on-grade type construction.

Earthworks to prepare the sites have already been undertaken, and no further signification earthworks will be required. Based on our findings, the proposed dwelling location will be founded partially on fill material and foundations will need to penetrate the natural stiff soils underlying the fill. We consider the stiff natural soils suitable for supporting foundations subject to ground verification during construction. Non-certified fill was encountered up to 1.7 m deep in BH02, therefore foundation embedment can be expected to be near 3.0 m below existing ground level when founded over the sidling fill area.

5.2 Shrink Swell Soil Characteristics

The natural, fine grained clayey soils of the Waitiiti Formation are considered susceptible to swelling and shrinking under seasonal variations of water content. For the purposes of design, the site may be designated as highly reactive (Class H1) in accordance with AS2870:2011. Where encountered, the foundations will need to penetrate through the non-certified fill material and buried topsoil to be founded within the stiff, natural Waitiiti Formation soils.

5.3 Seismic Hazard

The site comprises fine grained residual clay soils and is considered too plastic to liquefy. The site conditions have been assessed to be consistent with seismic subsoil Class C (shallow soil site) in accordance with NZS1170.5. The underlying soils were fine-grained clayey soils and are considered as non-liquefiable.

5.4 Foundation

Ground investigations at the site identified that the ground across the proposed dwelling area does not meet the definition of good ground as defined in NZS3604:2011 and B1/AS1, due to the presence expansive soils and non-certified fill material. We recommend that the dwelling foundations comprise concrete encased piled foundations that are taken to found into stiff natural soils below the non-certified fill material.

We have recommended a retaining wall be constructed to support the sidling fill (RW02). This wall is recommended to provide buttress support of the fill and prevent the fill from sliding. If this wall is constructed to the design outlined within this report, then concrete encased timber piled foundations can be designed in accordance with NZS3604:2011, provided the foundations are embedded at least 1.0 m into stiff natural soils, i.e., founded below the non-certified fill material, approximately 3.0 m along the southern side of the dwelling.

If RW02 is not constructed, then foundations will need to be designed to take lateral loads imposed from the sidling fill over the top 2.0 m of the foundation pile and subject to specific structural design to size the foundation piles. The following geotechnical parameters can be adopted in design, embedment length to be calculated using moment equilibrium methods and B1/VM4 for strength reduction factors and load factors:

- Effective cohesion to be ignored, c’ = 0 kPa
- Effective friction angle, φ’ = 25 degrees
- At rest earth pressure coefficient (over depth of fill, approx. 2.0m thick), k_0 = 0.6
- Passive earth pressure coefficient (embedded into stiff natural soils), k_p = 3.5

- Minimum embedment depth into stiff natural soils = 2.0 m (4.0 m below existing ground level).
- Short term loading, e.g., wind or seismic loading, refer Table 3.

The proposed future garage location can adopt a concrete slab on grade foundation. Foundations can be designed in accordance with B1/AS1 for Class H_1 soils, or specific design by a Chartered Professional Engineer (Structural) adopting the recommendations within AS2870 with the updated return periods outlined within B1/AS1.

A geotechnical drawing review will be required to confirm the foundation recommendations have been followed. Foundations conditions will be subject to site verification and approval by Chartered Professional Engineer (geotechnical) during construction. Foundations can be designed using an ultimate bearing capacity of 300kPa and a geotechnical strength reduction factor of 0.5.

- Ultimate bearing capacity in natural soils for shallow foundations – 300kPa.
- Ultimate shaft resistance for deep foundations into stiff natural soils (embedment length greater than 5x width – piles to be designed as friction piles only) – 30 kPa
- Geotechnical strength reduction factor – 0.5 for shallow foundations, 0.45 for piled foundations.
- Soil expansivity class – Class H_1 (highly reactive soils).

6 Construction

6.1 Earthworks

Earthworks across the site have already been undertaken, comprising excavations to create a level building platform area for the proposed dwelling and future garage, with the site-won cut material used as a sidling fill. The fill material was not supervised by an Engineer and is considered unsuitable for foundations. Retaining walls are proposed to support the cut and fill faces, which will provide some confinement to the sidling fill and make the building suitable for piled foundations. No further earthworks are proposed, other than backfilling behind retaining walls, e.g., drainage gravel.

6.2 Filling

We recommend that filling be avoided due to the existing cut faces, sloping ground and the potential for fill to negatively impact stability of the site. Should any further filling be proposed, then we recommend that slope stability and settlement analysis be undertaken prior to the placement of any future proposed fill. No filling around the foundation piles should be undertaken as this could result in negative skin friction/down drag on the foundation piles. Further advice should be sought if additional filling is required

6.3 Services

At the time of writing, no known underground services cross beneath the proposed development area. We recommend that any new services are accurately located on site and the depth to invert be determined prior to the commencement of foundation excavations.

6.4 Stormwater Disposal

All stormwater is to be diverted away from any proposed building platform and any steep slopes to avoid over saturation of the subsoils and to maintain stability across the site. All stormwater overflow drainages should be channelled away from the development platform and discharged in a controlled and dispersive manner.

6.5 Planned Vegetation

The foundation designer and architect must consider the proximity of trees when preparing designs as trees can exacerbate the normal seasonal variation of soil moisture levels and associated with that, the vertical and horizontal movement of the founding soils. Further, mechanical interference with foundations by tree roots should be considered.

6.6 Geotechnical Review

We recommend that the consent drawings are submitted for review to either ourselves, or another professional geotechnical engineer who is familiar with the contents of this report, prior to submission to Council for Building Consent. We recommend this review is carried out to check the compatibility of the design with the recommendations given within this report.

6.7 Construction Monitoring

Specific engineering inspections of retaining walls, building platform preparation and/or foundation construction with certification by a Producer Statement, PS4, are often required by Council and outlined in the Building Consent. These observations are generally required to ensure that the foundation soils exposed at the time of construction are consistent with the assumptions made in this geotechnical report.

We consider the following specific items, but not limited to will need to be addressed prior to and at the time of construction to ensure the foundation soils are consistent with the assumptions made in this geotechnical report:

1. Geotechnical drawing review to confirm the foundation design is as per the geotechnical recommendations and that the location of the proposed dwelling (and future garage if applicable) is in accordance with the geotechnical findings.
2. Observe the ground conditions within retaining wall pile holes and foundation excavations prior to pouring of concrete and ensure foundations are founded into stiff natural soils, including dwelling, and retaining wall foundations.

Provision should be allowed for modifying the foundation solution at this time should unforeseen ground conditions be encountered.

We can carry out the engineering inspections and provide the PS4 documentation if required. Should any required inspections not be completed, then any required PS4 documentation may not be obtained for the work which may result in a Code Compliance Certificate being withheld. We recommend that all required inspections as stated on the Building Consent inspections are undertaken by a Chartered Professional Engineer (CPEng) with the relevant practice field. Prior notification of at least 48 hours ahead of any site inspection is appreciated.

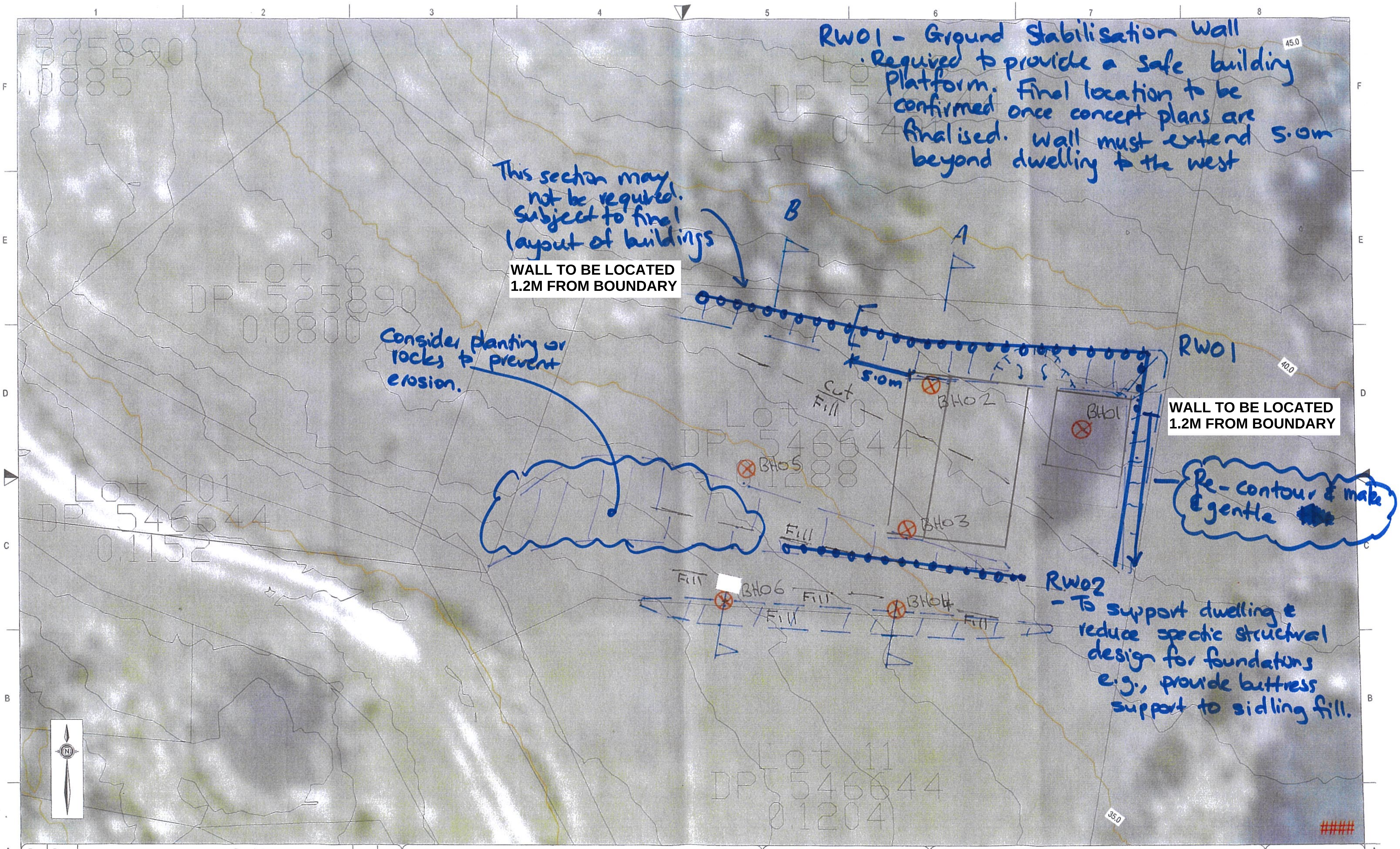
7 Limitations

This report has been prepared for the use of Dennis Matene with respect to the brief outlined to us. This report is to be used by our Client and their Consultants and may be relied upon when considering geotechnical advice. Furthermore, this report may be utilised in the preparation of building and/or resource consent applications with local authorities. The information and opinions contained within this report shall not be used in other context for any other purpose without prior review and agreement by Haigh Workman Ltd.

The recommendations given in this report are based on site data from discrete locations. Inferences about the subsoil conditions away from the test locations have been made but cannot be guaranteed. We have inferred an appropriate geotechnical model that can be applied for our analyses. However, variations in ground conditions from those described in this report could exist across the site. Should conditions encountered differ to those outlined in this report we ask that we be given the opportunity to review the continued applicability of our recommendations.

Appendix A – Drawings

Drawing No.	Title
21 259/01	Site Features, Borehole and Cross Section Locations
21 259/02	Geological Cross Section A-A’ & B-B’
21 259/03	Typical Timber Pole Wall Details (RW01 and RW02)



Rev	Date	Description	By	Checked
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DWG **SITE INVESTIGATION PLAN**

A3 SCALE 1:250

Date **20/01/2022**

Drawn **JP** Checked **WT** Approved **JP**

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Project **WAIANGA PLACE, OPONONI (LOT 10, DP 546644**

Client **DENNIS MATENE**

Project No. **21 259**

RC no. ####

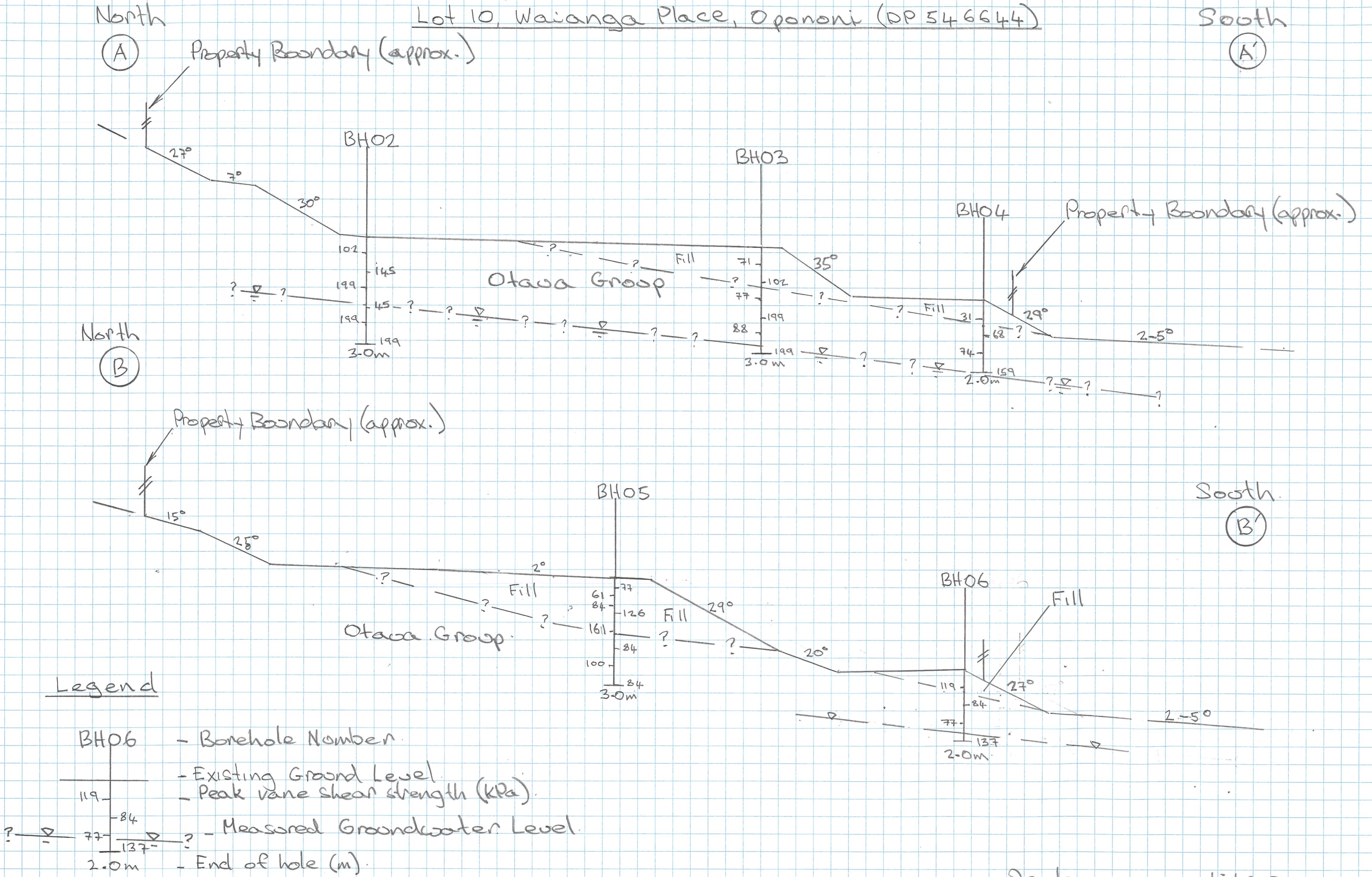
Stage ####

Dwg No. **G01**

Sheet No. **01 OF 03**

Geological Cross Sections A-A' and B-B'

Lot 10, Waianga Place, Opononi (DP 546644)



Legend

BH06 - Borehole Number

- Existing Ground Level

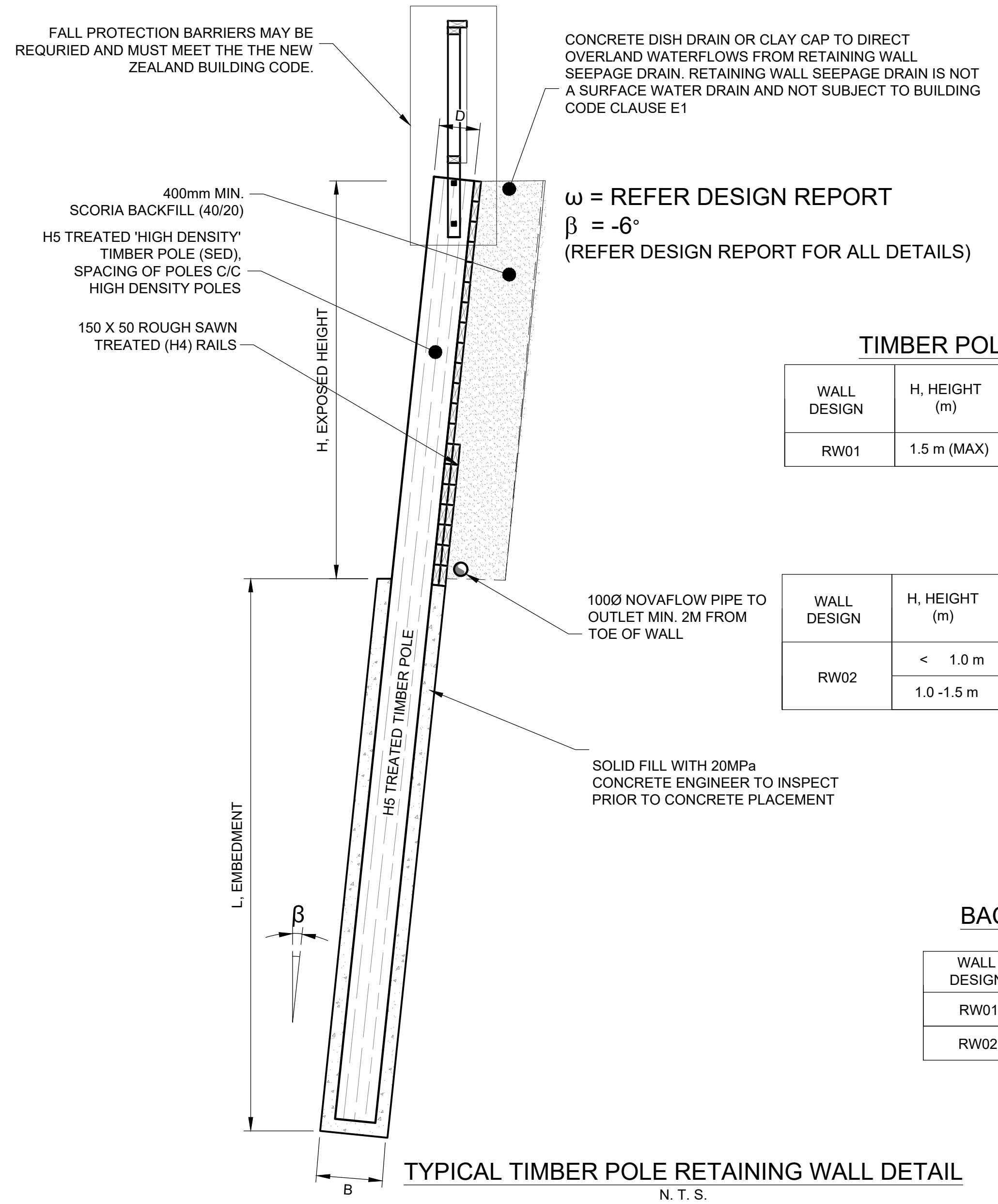
119 - 84 - Peak vane shear strength (kPa)

? ▽ ? - Measured Groundwater Level

2.0m - End of hole (m)

Scale. 1:100

Drawing No. 21 259/02



TYPICAL TIMBER POLE RETAINING WALL DETAIL

N. T. S.

NOTE:

1. TO BE CONSTRUCTED IN ACCORDANCE WITH B1 GUIDANCE ON BARRIER DESIGN (MARCH 2012).

TIMBER POLE RETAINING WALL DETAILS

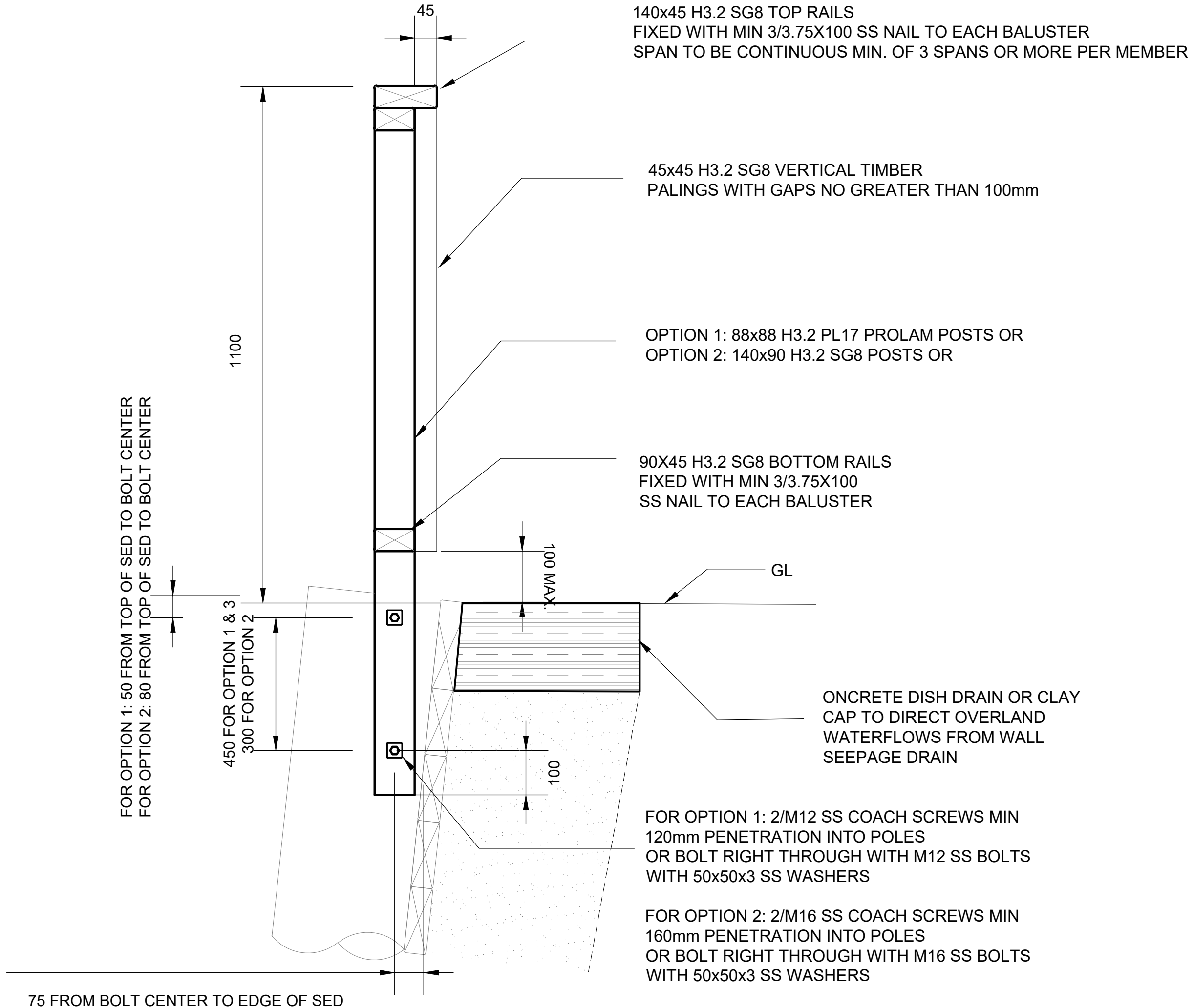
WALL DESIGN	H, HEIGHT (m)	D (SED) (mm)	SPACING OF POLES c/c (m)	L, EMBEDMENT (m)	B, BORED Ø (mm)
RW01	1.5 m (MAX)	325	1.20	3.50	600

WALL DESIGN	H, HEIGHT (m)	D (SED) (mm)	SPACING OF POLES c/c (m)	L, EMBEDMENT (m)	B, BORED Ø (mm)
RW02	< 1.0 m	200	1.20	1.50	450
	1.0 -1.5 m	200	1.20	2.50	450

BACKING BOARD DETAILS

(m FROM TOP OF WALL)

WALL DESIGN	SINGLE RAILS	DOUBLE RAILS
RW01	0.00 - 0.75 m	>0.75 m
RW02	0.00 - 1.05 m	>1.05 m



BARRIER DETAIL

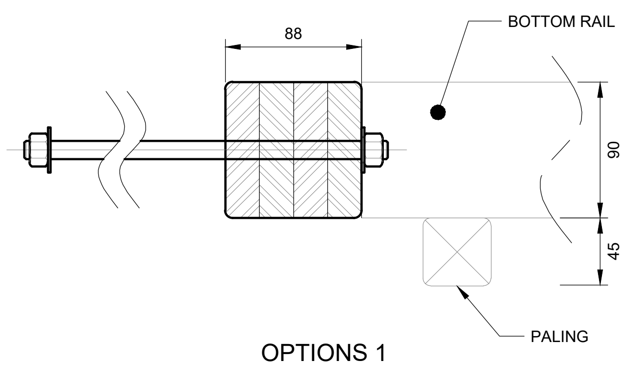
N. T. S.

TIMBER SPECIFICATION NOTES:

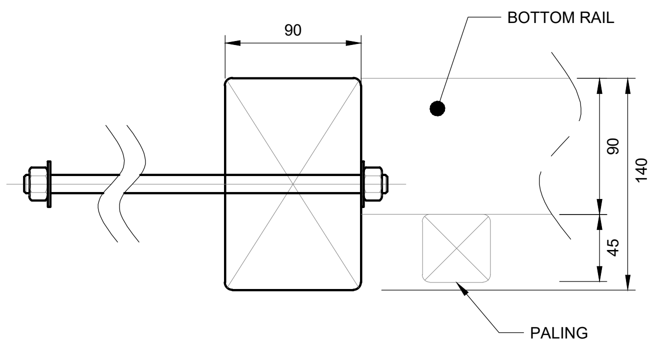
1. CAUTION:
   DEEP EXCAVATIONS UNSUPPORTED DURING CONSTRUCTION MAY BE HAZARDOUS PARTICULARLY WHEN WORKING IN CONFINED SPACES. THE DEPARTMENT OF LABOUR'S EXCAVATION GUIDE GIVES RECOMMENDED SAFETY PROCEDURES FOR SUCH SITUATIONS. THE EXCAVATION AND EARTHWORKS CONTRACTOR SHALL TAKE ALL NECESSARY PRECAUTIONS TO PROTECT ADEQUATELY ALL PERSONS AND PROPRETY LIABLE TO THE BE AFFECTED BY THE EXCAVATION AND EARTHWORKS OPERATIONS.
2. MATERIALS:
   TIMBER:
   TIMBER POLES SHALL BE PEELED RADIATA PINE LOGS COMPLYING WITH THE REQUIREMENT OF NZS 3605 "LOAD BEARING ROUND TIMBER PILES AND POLES" TREATED TO TPA COMMODITY SPECIFICATION H5. ALL TIMBER POLES TO BE HIGH DENSITY.
   DIMENSIONS OF POLES ARE SPECIFIED AS MINIMUM SMALL END DIAMETERS.
   ACTUAL DIAMETERS WILL BE GREATER DUE TO TAPER AND TIMBER GRADING. SAWN TIMBER IN GROUND CONTACT SHALL BE RADIATA PILE TREATED TO SPECIFICATION H4.
   ALL TIMBER SHALL HAVE TPA IDENTIFICATION BRANDS VISIBLE WHEN DELIVERED TO THE SITE AND SHALL BE PROTECTED AGAINST DAMAGE DURING STORAGE AND HANDLING
3. CONCRETE:
   CONCRETE FOR FOUNDATION BACKFILL SHALL BE ORDINARY GRADE CONCRETE COMPLYING WITH NZS 3109 'SPECIFICATION FOR CONCRETE', AND WITH A 28-DAY STRENGTH OF 20 MPa.
   CONCRETE SHALL BE PLACED UNDER AND AROUND POLES AND WELL COMPACTED BY VIBRATING. POLES SHALL BE TEMPORARILY PROPPED AND PROTECTED AGAINST DISTURBANCE FOR AT LEAST TWO DAYS AFTER PLACEMENT OF CONCRETE
4. EXCAVATION:
   EXCAVATION IN STAGES TO ALLOW FOR TEMPORARY SUPPORT DURING CONSTRUCTION IS REQUIRED. NO MORE THAN 3.0m OF UNSUPPORTED SLOPE SHALL EXIST AT ANY ONE TIME.
   EXCAVATION FOR POLES SHALL BE TAKEN OUT BY AUGERING TO THE DIMENSIONS DETAILED, WITH ALL SURPLUS SOIL BEING DISPOSED OF AWAY FROM THE SITE.
   ALLOWANCE SHALL BE MADE IN POSITIONING AUGERED HOLES FOR THE SLOPE OF THE WALL AND FOR CONCRETE TO SURROUND THE POLES. DRIVING OF POLES IS NOT ACCEPTABLE AS AN ALTERNATIVE TO AUGERING. POLES SHALL BE INSTALLED AS SOON AS POSSIBLE AFTER EXCAVATION. EXCAVATIONS FOR POLES SHALL BE FREE OF WATER AND LOOSE MATERIAL BEFORE CONCRETING. IF NECESSARY THE CONTRACTOR SHALL ALLOW FOR HANDCLEANING AND PUMPING OF EXCAVATION.
5. INSTALLATION:
   DRIVING OF POLES IS NOT ACCEPTABLE AS AN ALTERNATIVE TO AUGERING. FIXING OF HORIZONTAL TIMBERS TO POLES SHALL UTILISE GALVANISED NAILS AS DETAILED. TIMBERS SHALL BE LAYED IN POSITION COMMENCING AT THE BOTTOM OF THE WALL WITH JOINTS BETWEEN TIMBERS STAGGERED BETWEEN THE POLES BY USE OF SHORT TIMBERS AT ENDS OF ALTERNATIVE ROWS. IF CUTTING IS NECESSARY THE EXPOSED SURFACES SHALL BE FLOODED WITH A COPPER NAPTHENATE TYPE OF WOOD PRESERVATIVE.
   CARE SHALL BE TAKEN IN SELECTING AND LAYING HORIZONTAL TIMBERS TO MAINTAIN THE SPECIFIED MINIMUM THICKNESS OF TIMBERS, PARTICULARLY NEAR THE BASE OF THE WALL AND TO ACHIEVE NEAT STRAIGHT LINES AT THE TOP OF THE WALL.
6. BACKFILLING:
   A PERFORATED OR OPEN JOINTED SUBSOIL DRAIN SHALL BE LAID AND SURROUNDED IN FINE GRANULAR MATERIAL WITH THE INVERT BELOW LOWER GROUND LEVELS AND LED TO A FREE OUTLET AT A POINT OF SAFE DISCHARGE.
   REMAINING BACKFILL TO WITHIN 300mm OF FINISHED SURFACE LEVEL SHALL BE DRAINED COMPACTED GRANULAR FILL NOT LARGER THAN 65mm DIMENSIONS (eg RUN OF PIT SCORIA OR SIMILAR).
   THE FINISHED SURFACE OF BACKFILL SHALL BE SEALED AGAINST ENTRY OF SURFACE WATER WITH A LAYER OF TOPSOIL, CLAY OR CONCRETE.

IMPORTANT NOTE:

1. PROLAM GRADE: PL17 H3.2 88x88
   RADIATA PINE: SG8 H3.2 140x90
2. BOTTOM RAIL, PALING AND POST CONFIGURATION SHOULD BE AS PER DETAIL SHOWN BELOW
3. SED POLE NOT SHOWN FOR CLARITY
4. PROLAM LAMINATION MUST BE PERPENDICULAR TO BOLT



OPTIONS 1



OPTION 2

Issue	Date	Revision
A	21/01/2022	GEOTECHNICAL INVESTIGATION AND RW DESIGN

DWG
TYPICAL TIMBER POLE WALL DETAILS
RW01 AND RW02

Scale N. T. S.
Date 21/01/2022
Drawn WT
Checked JP
Approved JP
File S:\CLIENTS\DENNIS MATENE\JOBS\21 259 - WAIANGA PLACE, OPONONI (LOT 10 DP 546644)\ENGINEERING\DRAWINGS\21 259_RW01-02.DWG

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Project **WAIANGA PLACE, OPONONI**
Client **DENNIS MATENE (LOT 10, DP 546644)**
Project No. 21 259
RC no.

DWG No. **G03**
Sheet No. **03** of **03**

Plotted By Wayne Thorburn at 21/01/2022 2:47:04 PM

Appendix B – Hand Auger Borehole Logs

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Borehole Log - BH01

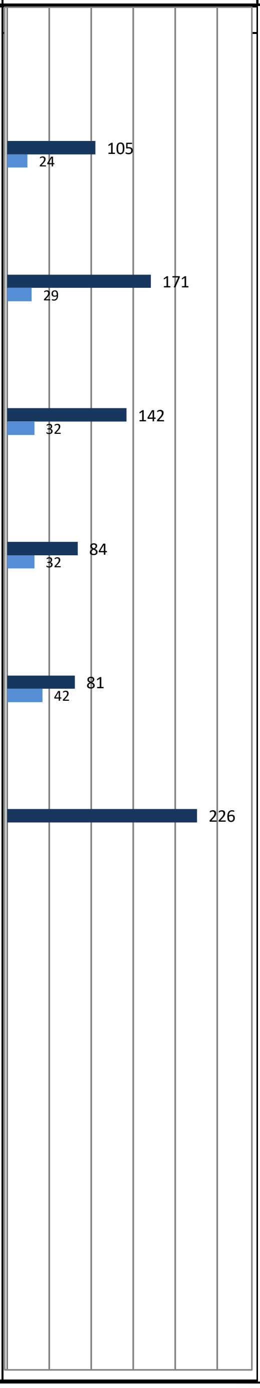
Hole Location: Refer Site Plan

JOB No. 21 259

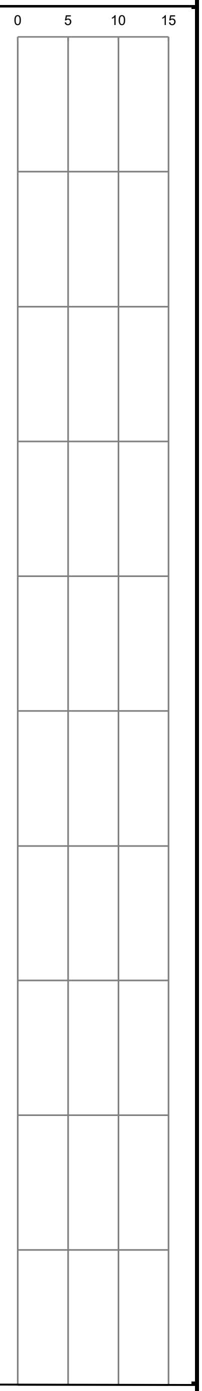
CLIENT:	Dennis Matene	**SITE:**			
Date Started:	16/11/2021	**DRILLING METHOD:**	Hand Auger	**LOGGED BY:**	JC
Date Completed:	16/11/2021	**HOLE DIAMETER (mm)**	50mm	**CHECKED BY:**	JP

Soil Description Based on NZGS Logging Guidelines 2005	Depth (m)	Geology	Water Level	Sensitivity
Silty **CLAY**, some light grey clay seams; orangish brown and light grey. Stiff, moist, highly plastic. [Waitiiti Formation]	0.0	WAITIITI FORMATION		4
From 0.5m: Becomes brownish orange with light grey clay seams.	0.5			6
Clayey **SILT**, trace light grey clay seams; brownish grey, mottled orangish brown and light grey. Very stiff, moist, medium plasticity.	1.0			4
Silty **CLAY**, trace gravel; brownish grey. Stiff, wet, high plasticity. Gravel: medium grained, weakly cemented. From 1.8m: Groundwater seepage.	1.5			3
Clayey **SILT**, trace gravel; light grey. Stiff, moist, medium plasticity. Gravel: fine to medium grained, weakly cemented.	2.0			2
At 2.8m: Becomes very stiff to hard.	2.5		Groundwater Encountered at 2.8m.	
End of Hole at 3.0m (Target Depth)	3.0			
	3.5			
	4.0			
	4.5			

Vane Shear and Remoulded Vane Shear Strengths (kPa)



Scala Penetrometer (blows/100mm)



LEGEND

▨ **TOPSOIL** 🟨 **CLAY** ▨ **SILT** ▨ **SAND** ▨ **GRAVEL** ▨ **FILL**

Corrected shear vane reading	▬
Remoulded shear vane reading	▬
Scala Penetrometer	●

Note: UTP = Unable to penetrate. T.S. = Topsoil.
Hand Held Shear Vane S/N: DR2278. Groundwater encountered at 2.8m at completion of drilling.
Scala penetrometer testing not undertaken.

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Borehole Log - BH02

Hole Location: Refer Site Plan

JOB No. 21 259

CLIENT: Dennis Matene
Date Started: 16/11/2021
Date Completed: 16/11/2021
SITE: Waianga Place, Opononi, (Lot 10 Deposited Plan 546644)
DRILLING METHOD: Hand Auger
HOLE DIAMETER (mm) 50mm
LOGGED BY: JP
CHECKED BY: JC

Soil Description

Based on NZGS Logging Guidelines 2005

Clayey **SILT**; light brownish orange, streaked light grey. Stiff, moist, medium plasticity. [Waitiiti Formation]

Silty **CLAY**; light grey and light orange, streaked orange. Very stiff, moist, medium plasticity.

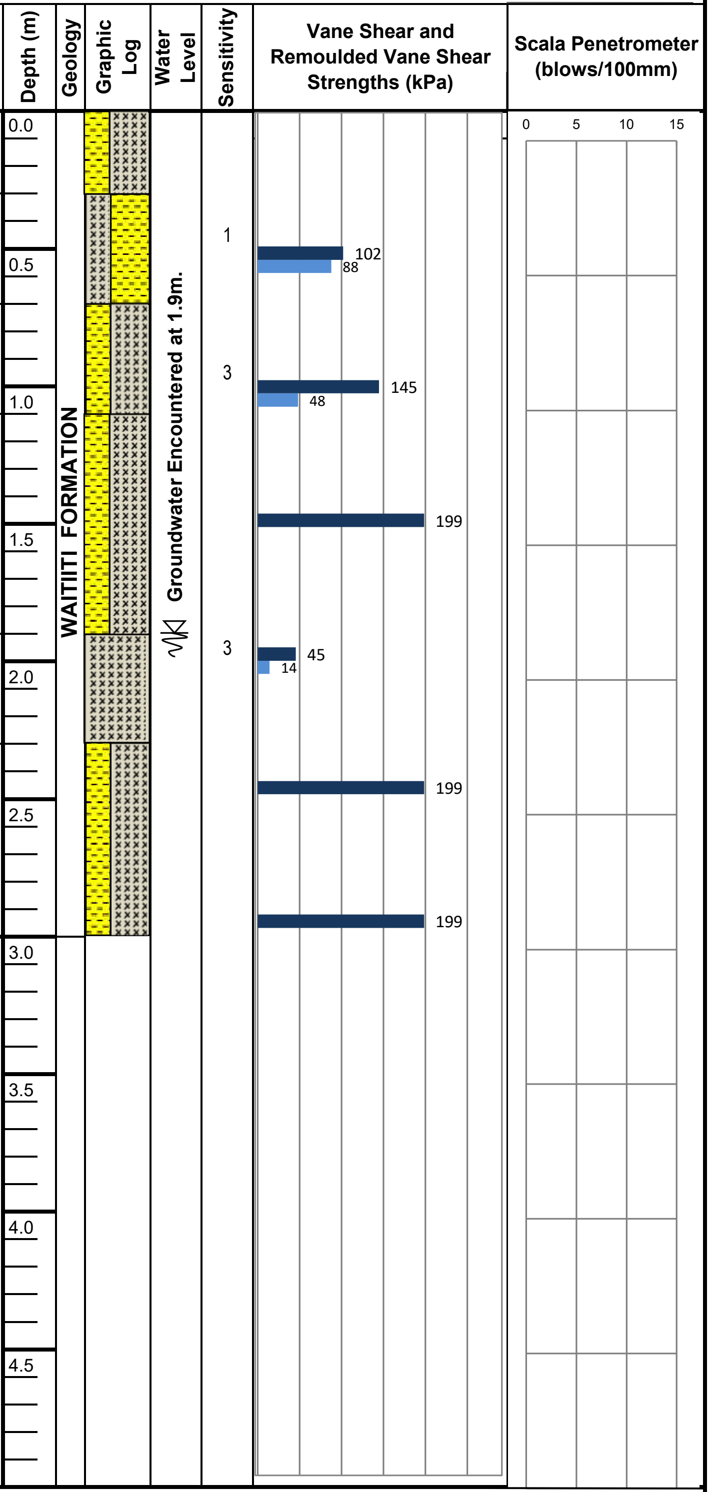
Clayey **SILT**; light brownish orange, streaked orange and grey. Very stiff, moist, medium plasticity.

Clayey **SILT**, minor light bluish grey clay seams; light brownish orange, streaked orange and light grey. Very stiff, moist, medium plasticity.

SILT, some clay; brownish orange, mottled dark orange, streaked grey. Very stiff, moist to wet, low to medium plasticity.
From 2.0m: Becomes Firm to stiff, wet.

Clayey **SILT**; brownish grey, mottled orange. Very stiff, moist, medium plasticity.
From 2.5m: Becomes grey to bluish grey, mottled orange, streaked dark orange.

End of Hole at 3.0m (Target Depth)



LEGEND

TOPSOIL | CLAY | SILT | SAND | GRAVEL | FILL

Corrected shear vane reading ▬
Remoulded shear vane reading ▬
Scala Penetrometer ●

Note: UTP = Unable to penetrate. T.S. = Topsoil.
Hand Held Shear Vane S/N: 2220. Groundwater encountered at 1.9m at completion of drilling.
Scala penetrometer testing not undertaken.

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Borehole Log - BH03

Hole Location: Refer Site Plan

JOB No. 21 259

CLIENT: Dennis Matene
Date Started: 16/11/2021
Date Completed: 16/11/2021
SITE: Waianga Place, Opononi, (Lot 10 Deposited Plan 546644)
DRILLING METHOD: Hand Auger
HOLE DIAMETER (mm) 50mm
LOGGED BY: JP
CHECKED BY: JC

Soil Description

Based on NZGS Logging Guidelines 2005

Clayey **SILT**; light grey and light orange, streaked brownish grey, brown and orange. Stiff, moist, medium plasticity. [Fill]

From 0.3m: Becomes light grey, brownish grey, light brown and light orange intermixed, mottled orange and grey.

From 0.7m: Becomes stiff to very stiff.

From 0.9m: Becomes greyish brown, streaked brownish grey and orange. Trace fibrous organics [slight organic odour].

Silty **CLAY**; light grey and light orange, streaked orange. Very stiff, moist to wet, medium to high plasticity. [Waitiiti Formation]

Clayey **SILT**; light grey and light orange, streaked orange. Very stiff, moist, medium plasticity.

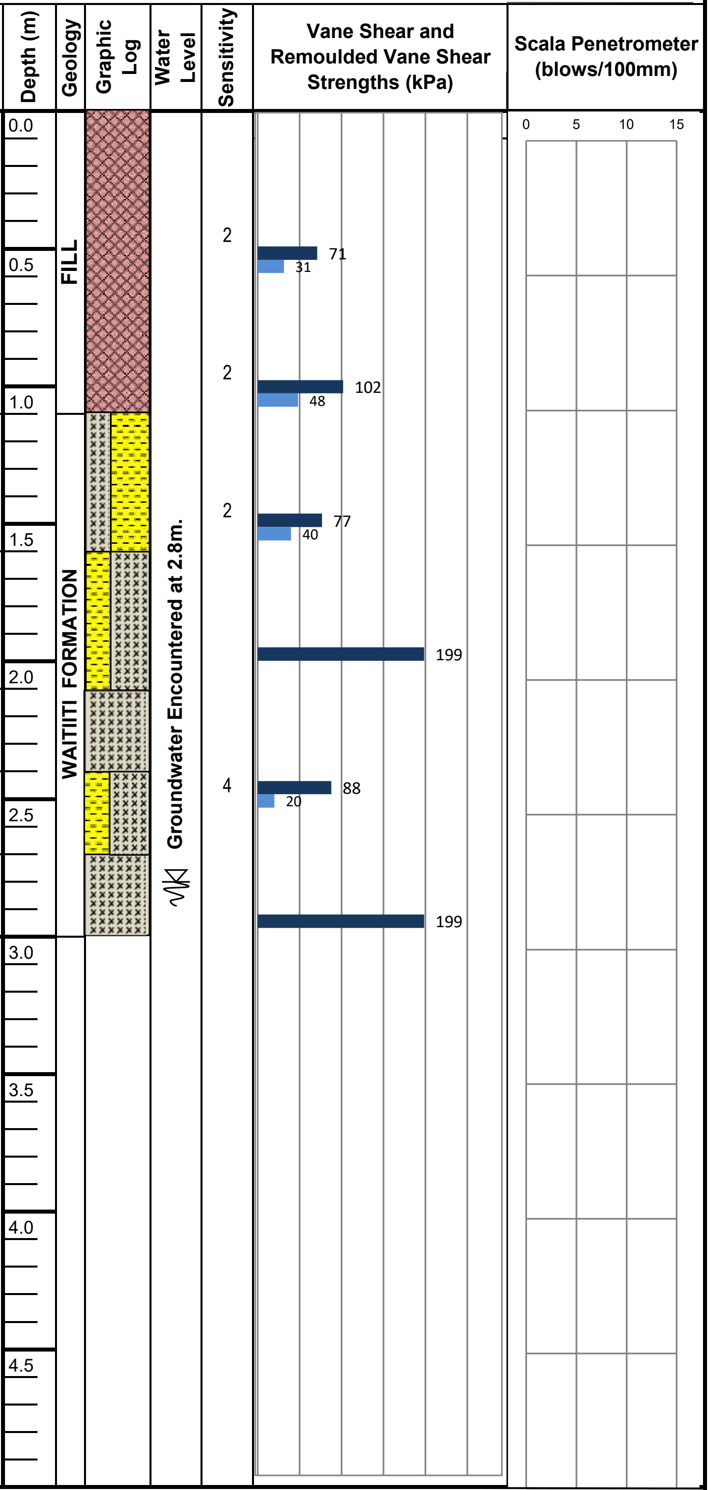
From 1.9m: Becomes light bluish grey to grey, mottled orange, streaked dark orange.

SILT, some clay, trace fine gravel; light brownish orange and light grey, mottled dark orange. Very stiff, moist, low to medium plasticity.

Clayey **SILT**; light brownish grey and light bluish grey, streaked dark orange. Very stiff, moist to wet, medium plasticity.
From 2.6m: Becomes moist to wet.

SILT, some clay; light brownish grey and light orange, streaked brown and orange. Very stiff, moist, low to medium plasticity.

End of Hole at 3.0m (Target Depth)



LEGEND

TOPSOIL CLAY SILT SAND GRAVEL FILL

Corrected shear vane reading	▬
Remoulded shear vane reading	▬
Scala Penetrometer	●

Note: UTP = Unable to penetrate. T.S. = Topsoil.
Hand Held Shear Vane S/N: 2220. Groundwater encountered at 2.8m at completion of drilling.
Scala penetrometer testing not undertaken.

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Borehole Log - BH04

Hole Location: Refer Site Plan

JOB No. 21 259

CLIENT:	Dennis Matene	**SITE:**	Waianga Place, Opononi, (Lot 10 Deposited Plan 546644)
Date Started:	16/11/2021	**DRILLING METHOD:**	Hand Auger
Date Completed:	16/11/2021	**HOLE DIAMETER (mm)**	50mm
		LOGGED BY:	JP
		CHECKED BY:	WT

Soil Description

Based on NZGS Logging Guidelines 2005

Clayey **SILT**, trace fine gravel; light orange, light grey and brown intermixed. Stiff, dry to moist, low to medium plasticity. [Fill]

Silty **CLAY**; light orange and grey, streaked orange. Firm to stiff, moist, medium plasticity. Trace fibrous organics.

SILT, minor clay; brown, mottled orange. Firm, moist, low plasticity. [B.T.S.]

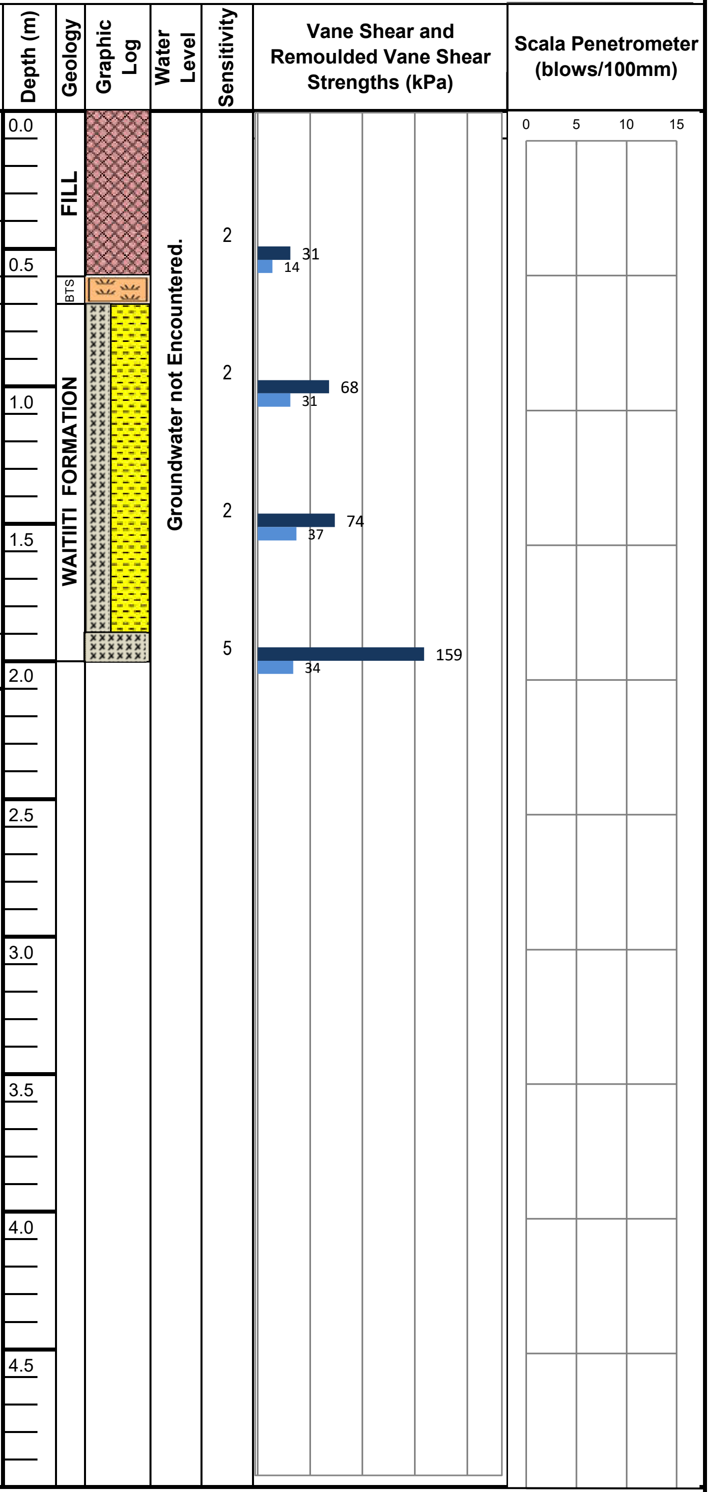
Silty **CLAY**; light brownish grey and light orange. Stiff, moist, medium to high plasticity. [Waitiiti Formation]

From 0.9m: Becomes light grey and light orange, streaked orange.

From 1.3m: Becomes light whitish orange, streaked orange. Stiff to very stiff.

SILT, some clay; light orange, streaked dark orange. Very stiff, dry to moist, low to medium plasticity.

End of Hole at 2.0m (Target Depth)



LEGEND

TOPSOIL	**CLAY**	**SILT**	**SAND**	**GRAVEL**	**FILL**

Corrected shear vane reading	▬
Remoulded shear vane reading	▬
Scala Penetrometer	●

Note: UTP = Unable to penetrate. B.T.S. = Buried Topsoil.
Hand Held Shear Vane S/N: 2220. Groundwater encountered not encountered at completion of drilling.
Scala penetrometer testing not undertaken.

Borehole Log - BH05

Hole Location: Refer Site Plan

JOB No. 21 259

CLIENT:	Dennis Matene	**SITE:**	Waianga Place, Opononi, (Lot 10 Deposited Plan 546644)		
Date Started:	16/11/2021	**DRILLING METHOD:**	Hand Auger	**LOGGED BY:**	JC
Date Completed:	16/11/2021	**HOLE DIAMETER (mm)**	50mm	**CHECKED BY:**	JP

Soil Description

Based on NZGS Logging Guidelines 2005

Clayey **SILT**; brown. Firm to stiff, moist, low plasticity. [Topsoil/Fill]

CLAY; orangish brown, mixed light grey and dark brown. Stiff, moist, high plasticity. [Fill]

Silty **CLAY**; orangish brown, mottled dark greyish brown. Very stiff, moist, high plasticity.

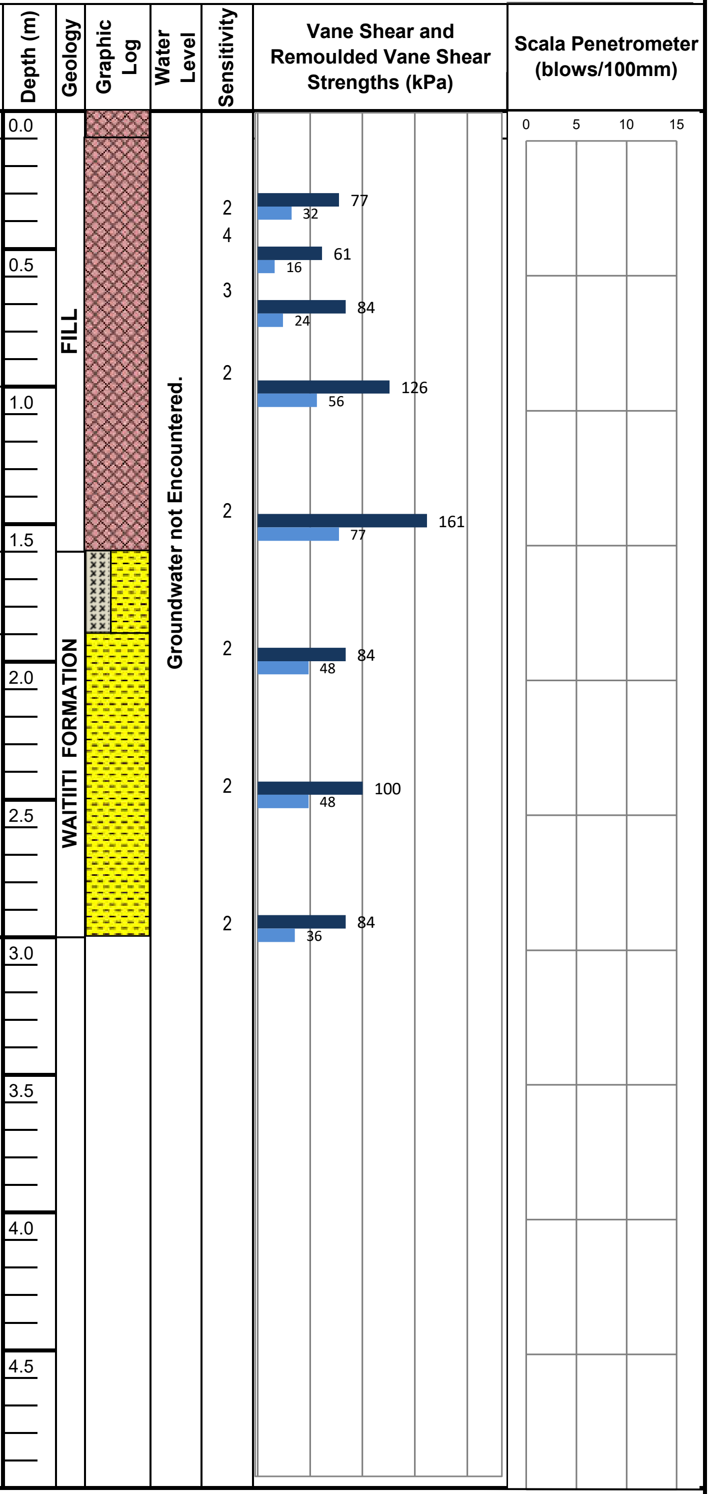
Silty **CLAY**; dark greyish brown, speckled orangish brown. Stiff, moist, medium plasticity. [Waitiiti Formation]

At 1.7m: Becomes orangish brown and dark grey.

CLAY; orangish brown and light grey. Stiff, moist to wet, high plasticity.

From 2.7m: Trace fine gravel, weakly cemented.

End of Hole at 3.0m (Target Depth)



LEGEND

▨ **TOPSOIL** 🟨 **CLAY** ▨ **SILT** ▨ **SAND** ▨ **GRAVEL** ▨ **FILL**

Corrected shear vane reading	▬
Remoulded shear vane reading	▬
Scala Penetrometer	●

Note: UTP = Unable to penetrate. T.S. = Topsoil.

Hand Held Shear Vane S/N: DR2278. Groundwater not encountered at completion of drilling.

Scala penetrometer testing not undertaken.

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Borehole Log - BH06

Hole Location: Refer Site Plan

JOB No. 21 259

CLIENT:	Dennis Matene	**SITE:**	Waianga Place, Opononi, (Lot 10 Deposited Plan 546644)		
Date Started:	16/11/2021	**DRILLING METHOD:**	Hand Auger	**LOGGED BY:**	JC
Date Completed:	16/11/2021	**HOLE DIAMETER (mm)**	50mm	**CHECKED BY:**	JP

Soil Description

Based on NZGS Logging Guidelines 2005

Silty **CLAY**; orangish brown, mixed dark brown. Very stiff, dry, high plasticity. [Fill]

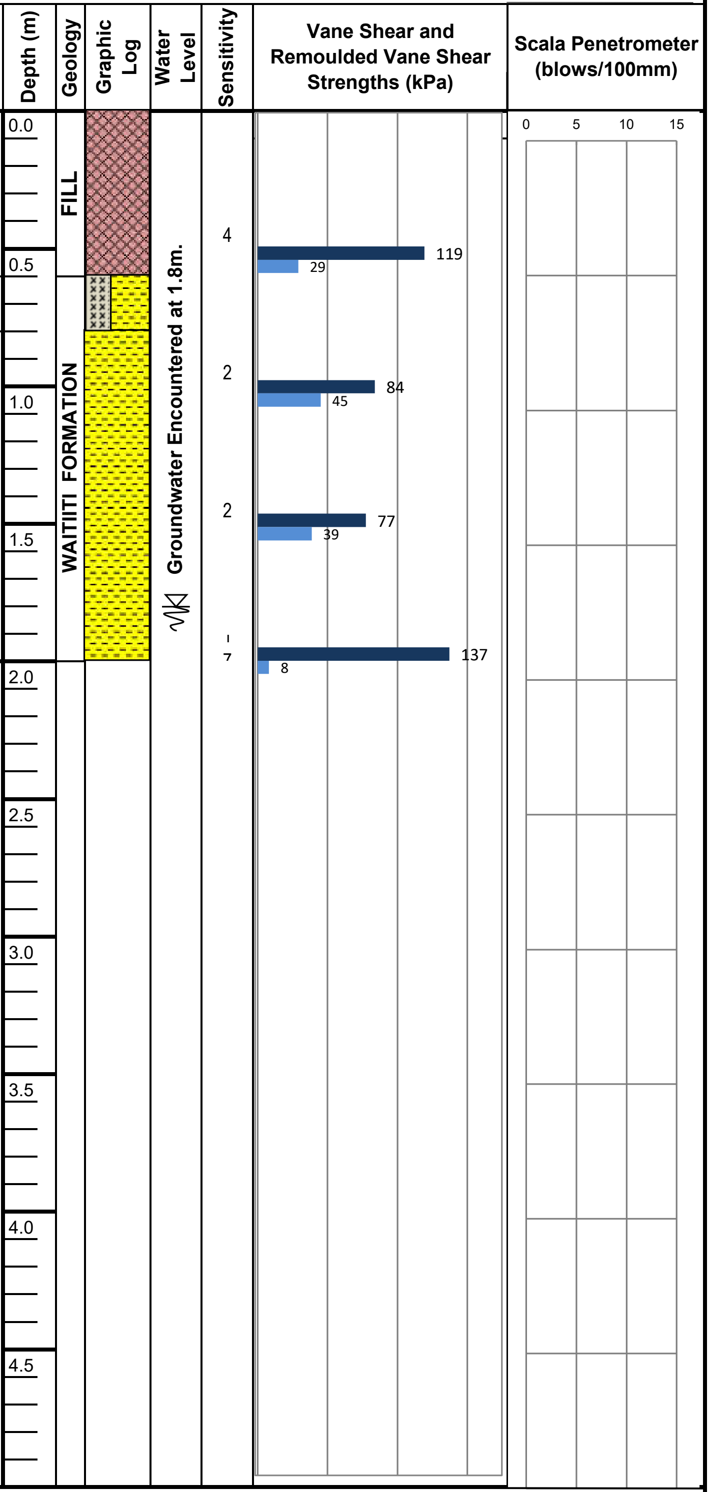
Silty **CLAY**; dark greyish brown, speckled orangish brown. Stiff to very stiff, moist to wet, high plasticity. [Waitiiti Formation]

CLAY; light grey, streaked dark grey and dark orangish brown. Stiff, wet, high plasticity.

At 1.0m: Becomes grey, streaked orangish brown.

At 1.9m: Some greyish brown silt lenses.

End of Hole at 2.0m (Target Depth)



LEGEND

▨ **TOPSOIL** ▨ **CLAY** ▨ **SILT** ▨ **SAND** ▨ **GRAVEL** ▨ **FILL**

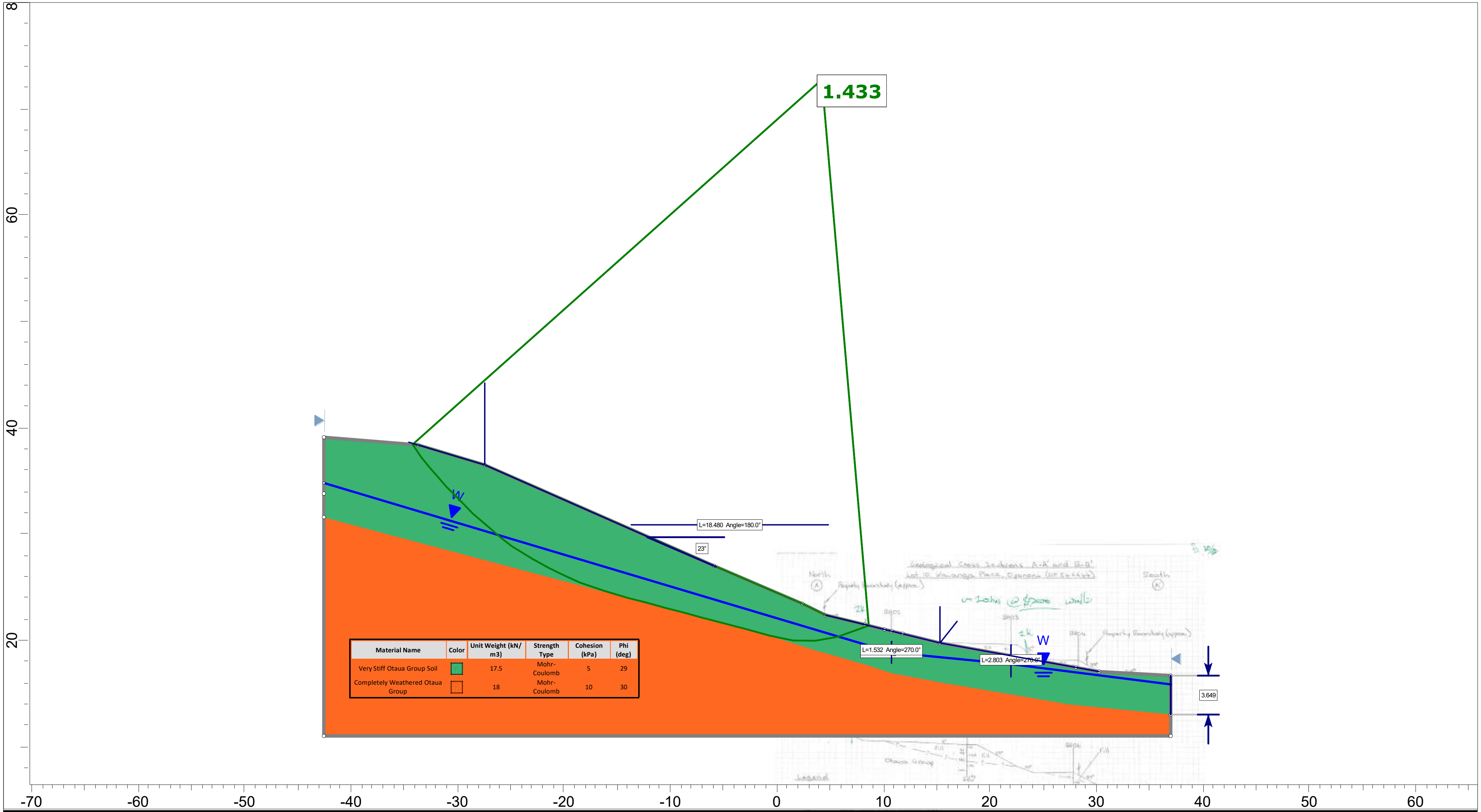
Corrected shear vane reading	▬
Remoulded shear vane reading	▬
Scala Penetrometer	●

Note: UTP = Unable to penetrate. T.S. = Topsoil.
Hand Held Shear Vane S/N: DR1617. Groundwater encountered at 1.9m at completion of drilling.
Scala penetrometer testing not undertaken.

Appendix C – Retaining Wall Calculations

Stability Modelling

Wallap Retaining wall analysis and calculations

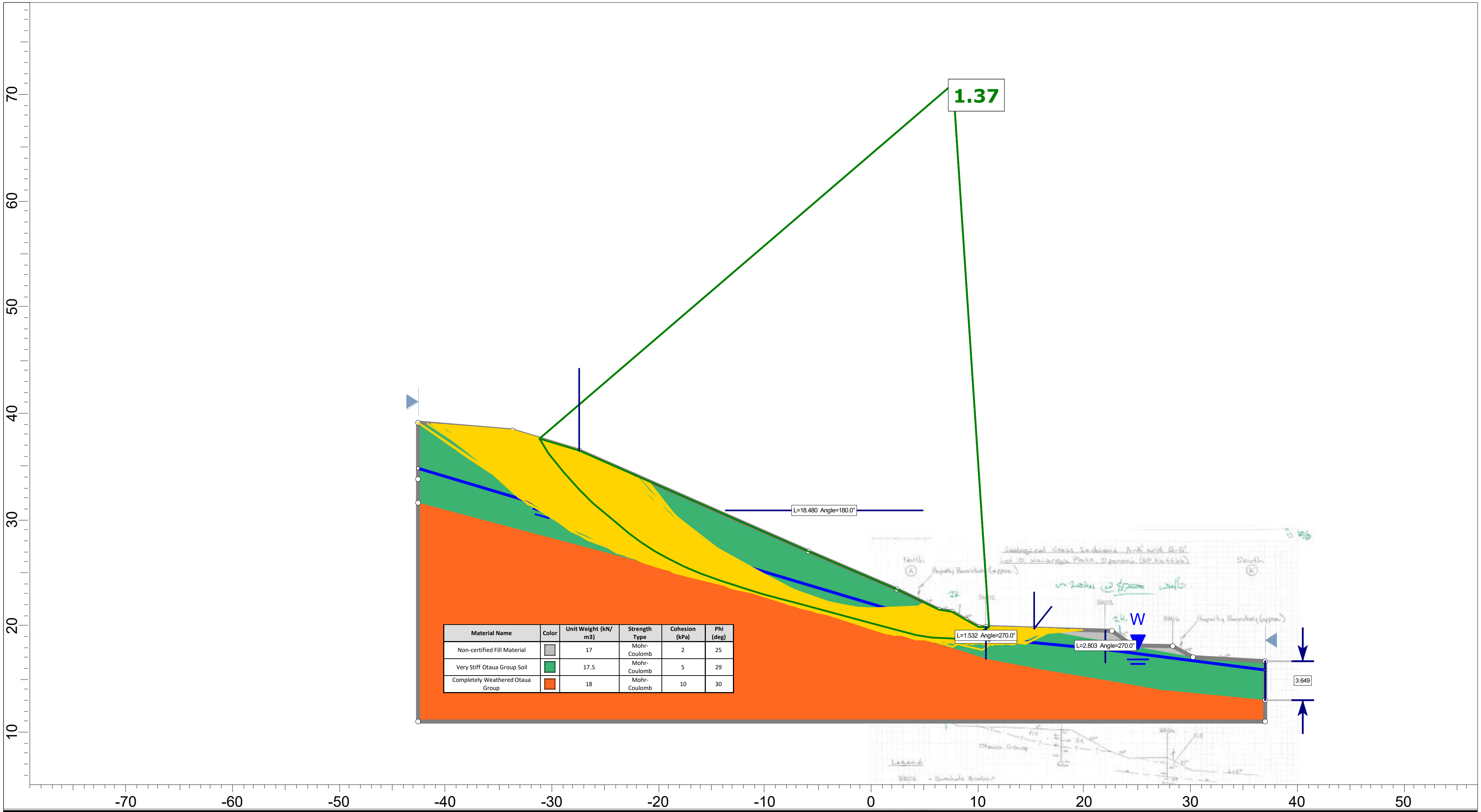


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Very Stiff Otaua Group Soil	🟩	17.5	Mohr-Coulomb	5	29
Completely Weathered Otaua Group	🟧	18	Mohr-Coulomb	10	30

rocscience

SLIDEINTERPRET 9.012

Project	21 259 - Waianga Place Opononi		
Group	Group 1	*Scenario*	Master Scenario
Drawn By	J. Power	*Company*	Haigh Workman Ltd
Date	05/12/2021	*File Name*	01 Non-circ - pre-Existing - Assumed GW.slmd

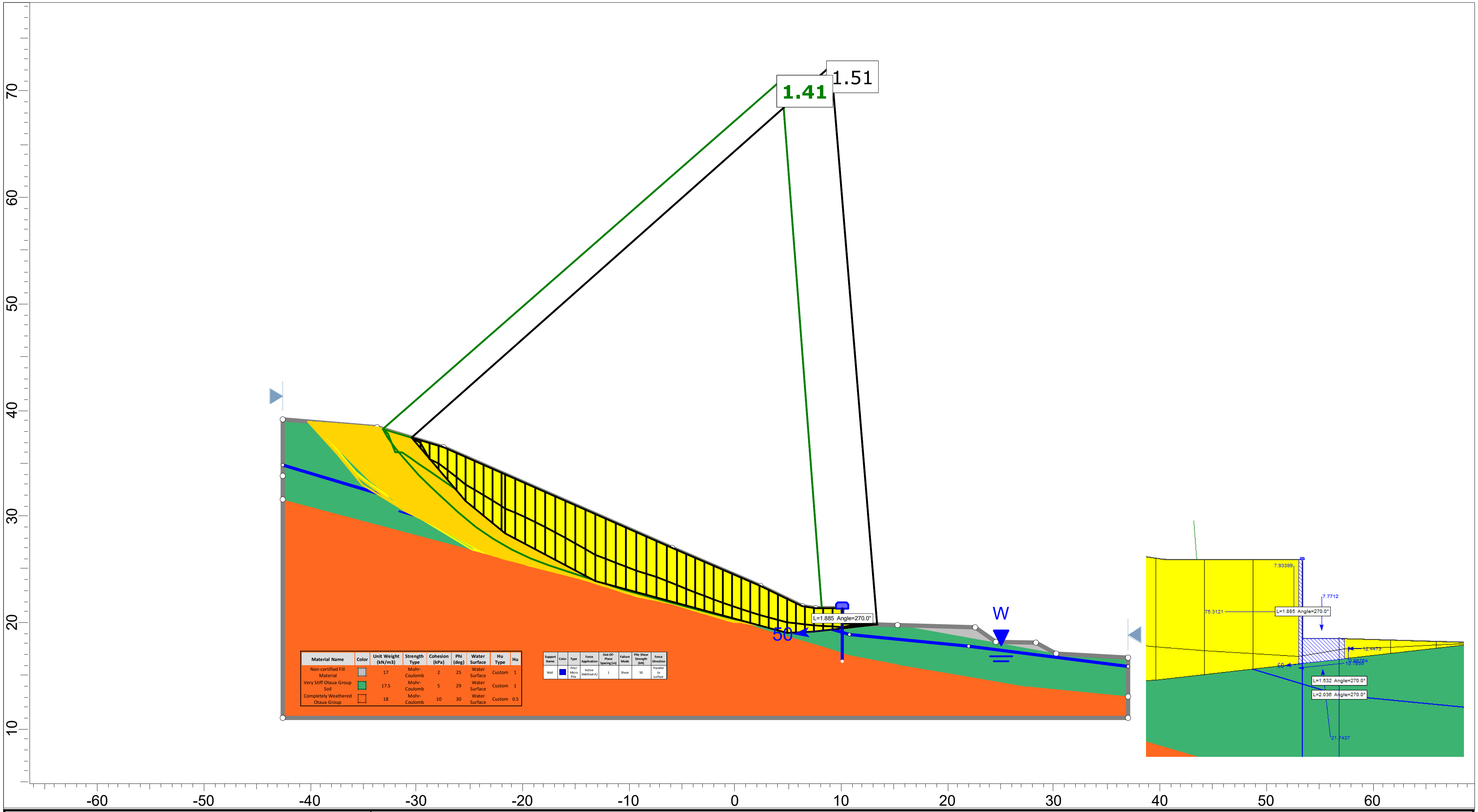


Material Name	Color	Unit Weight (kN/ m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Non-certified Fill Material	⬜	17	Mohr-Coulomb	2	25
Very Stiff Otaua Group Soil	🟩	17.5	Mohr-Coulomb	5	29
Completely Weathered Otaua Group	🟧	18	Mohr-Coulomb	10	30

rocscience

SLIDEINTERPRET 9.012

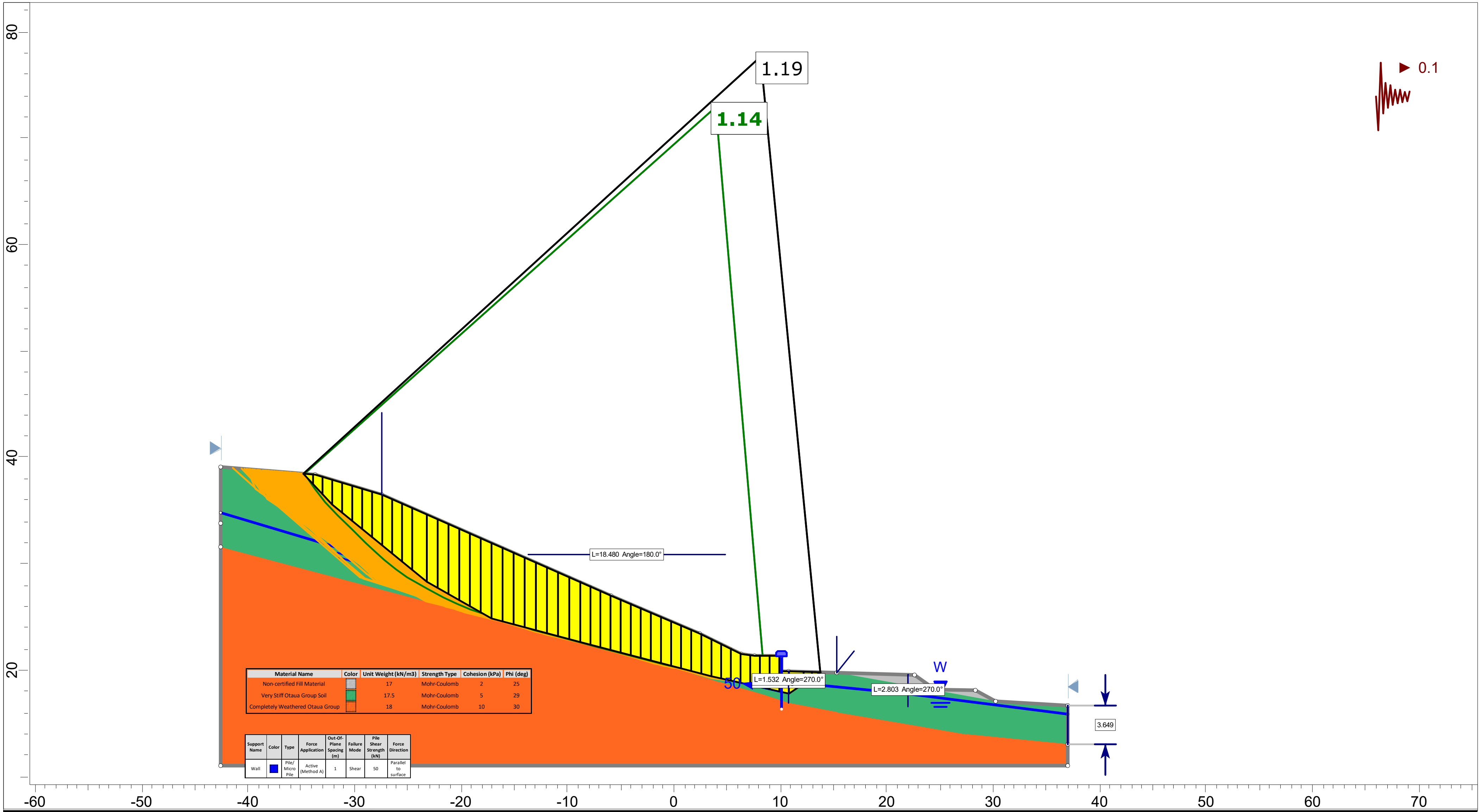
Project	21 259 - Waianga Place Opononi		
Group	Group 1	*Scenario*	Master Scenario
Drawn By	J. Power	*Company*	Haigh Workman Ltd
Date	05/12/2021	*File Name*	01 Non-circ - Existing - Assumed GW.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Hu Type	Hu
Non-certified Fill Material	▨	17	Mohr-Coulomb	2	25	Water Surface	Custom	1
Very Stiff Otaua Group Soil	🟩	17.5	Mohr-Coulomb	5	29	Water Surface	Custom	1
Completely Weathered Otaua Group	🟧	18	Mohr-Coulomb	10	30	Water Surface	Custom	0.5

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (m)	Failure Mode	Pile Shear Strength (kN)	Force Direction
Wall	🟦	Pile/ Micro Pile	Active (Method A)	1	Shear	50	Parallel to surface

rocscience	*Project* 21 259 - Waianga Place Opononi		
	Group Group 1	*Scenario* Master Scenario	
	Drawn By J. Power	*Company* Haigh Workman Ltd	
SLIDEINTERPRET 9.012	*Date* 05/12/2021	*File Name* 01 Non-circ - Ground Stabilisation Wall.slmd	



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Non-certified Fill Material	⬜	17	Mohr-Coulomb	2	25
Very Stiff Otaua Group Soil	🟩	17.5	Mohr-Coulomb	5	29
Completely Weathered Otaua Group	🟧	18	Mohr-Coulomb	10	30

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (m)	Failure Mode	Pile Shear Strength (kN)	Force Direction
Wall	🟦	Pile/ Micro Pile	Active (Method A)	1	Shear	50	Parallel to surface

Project	21 259 - Waianga Place Opononi		
Group	Group 1	*Scenario*	Master Scenario
Drawn By	J. Power	*Company*	Haigh Workman Ltd
Date	05/12/2021	*File Name*	03 Non-circ - Ground Stabilisation Wall(seismic).slmd

SLIDEINTERPRET 9.012

```
                                                       Units: kN,m
INPUT DATA

SOIL PROFILE
Stratum   Elevation of    ------------------ Soil types -------------------
  no.    top of stratum    Left side                 Right side
   1          10.00       1  Residual                1  Residual
   2           4.50       2  rock                    2  rock

SOIL PROPERTIES
                     Bulk    Young's   At rest  Consol  Active  Passive
-- Soil type --  density  Modulus    coeff.  state.  limit    limit   Cohesion
No. Description   kN/m3  Eh,kN/m2      Ko     NC/OC    Ka       Kp      kN/m2
  (Datum elev.)           (dEh/dy ) (dKo/dy) (  Nu ) ( Kac ) (  Kpc ) ( dc/dy )
 1  Residual      18.00     25000    0.500     OC    0.298    3.675     5.000d
                                             (0.350) (1.267) ( 4.825)
 2  rock          18.00     50000    0.500     OC    0.285    3.878     5.000d
                                             (0.350) (1.238) ( 4.985)
 3  HARDFILL      20.00     50000    0.426     OC    0.229    5.153
                                             (0.350) (0.000) ( 0.000)

Additional soil parameters associated with Ka and Kp
                          --- parameters for Ka ---  --- parameters for Kp ---
                            Soil      Wall    Back-    Soil      Wall    Back-
------- Soil type ------- friction  adhesion  fill   friction  adhesion  fill
No. Description             angle    coeff.   angle    angle    coeff.   angle
 1  Residual                29.00    0.633    0.00     29.00    0.304    0.00
 2  rock                    30.00    0.631    0.00     30.00    0.302    0.00
 3  HARDFILL                35.00    0.616    0.00     35.00    0.292    0.00

GROUND WATER CONDITIONS
 Density of water = 9.810 kN/m3
                                   Left side     Right side
 Initial water table elevation        0.00            0.00

 Automatic water pressure balancing at toe of wall :  No


WALL PROPERTIES
                        Type of structure = Soldier Pile Wall
                       Soldier Pile width =  0.60 m
                     Soldier Pile spacing =  1.20 m
              Passive mobilisation factor =  3.00
                    Elevation of toe of wall =  5.00
             Maximum finite element length =  0.30 m
                    Youngs modulus of wall E = 8.7000E+06 kN/m2
               Moment of inertia of wall I = 4.5640E-04 m4/m run
                                           = 5.4768E-04 m4 per pile
                                       E.I = 3970.7 kN.m2/m run
                     Yield Moment of wall = Not defined

HORIZONTAL and MOMENT LOADS/RESTRAINTS
 Load            Horizontal    Moment      Moment     Partial
  no.  Elevation     load        load     restraint   factor
                  kN/m run   kN.m/m run  kN.m/m/rad  (Category)
   1      9.70       1.400      0            0         N/A
   2      9.05       13.60      0            0         N/A
   3      8.40       35.00      0            0         N/A
```

SURCHARGE LOADS

```
Surch         Distance   Length    Width        Surcharge      Equiv. Partial
-arge           from    parallel  perpend. -----  kN/m2  -----  soil  factor/
 no.   Elev.    wall    to wall   to wall  Near edge  Far edge  type  Category
  1     7.50   -1.00(R)   20.00     20.00      0.00   -180.00     1     N/A
  2     8.10   -0.00(R)   20.00     14.00      7.20     =         0     N/A
  3     8.10    0.00(L)   20.00     20.00     34.20     =         1     N/A

    Note: L = Left side,  R = Right side
          A trapezoidal surcharge is defined by two values:
          N = at edge near to wall,  F = at edge far from wall
```

CONSTRUCTION STAGES

```
Construction   Stage description
  stage no.    --------------------------------------------------------
      1        Excavate to elevation 8.10 on RIGHT side
      2        Excavate to elevation 8.10 on LEFT side
      3        Apply surcharge no.3 at elevation 8.10
      4        Apply surcharge no.2 at elevation 8.10
               No analysis at this stage
      5        Apply load no.1 at elevation 9.70
      6        Apply load no.2 at elevation 9.05
      7        Apply load no.3 at elevation 8.40
```

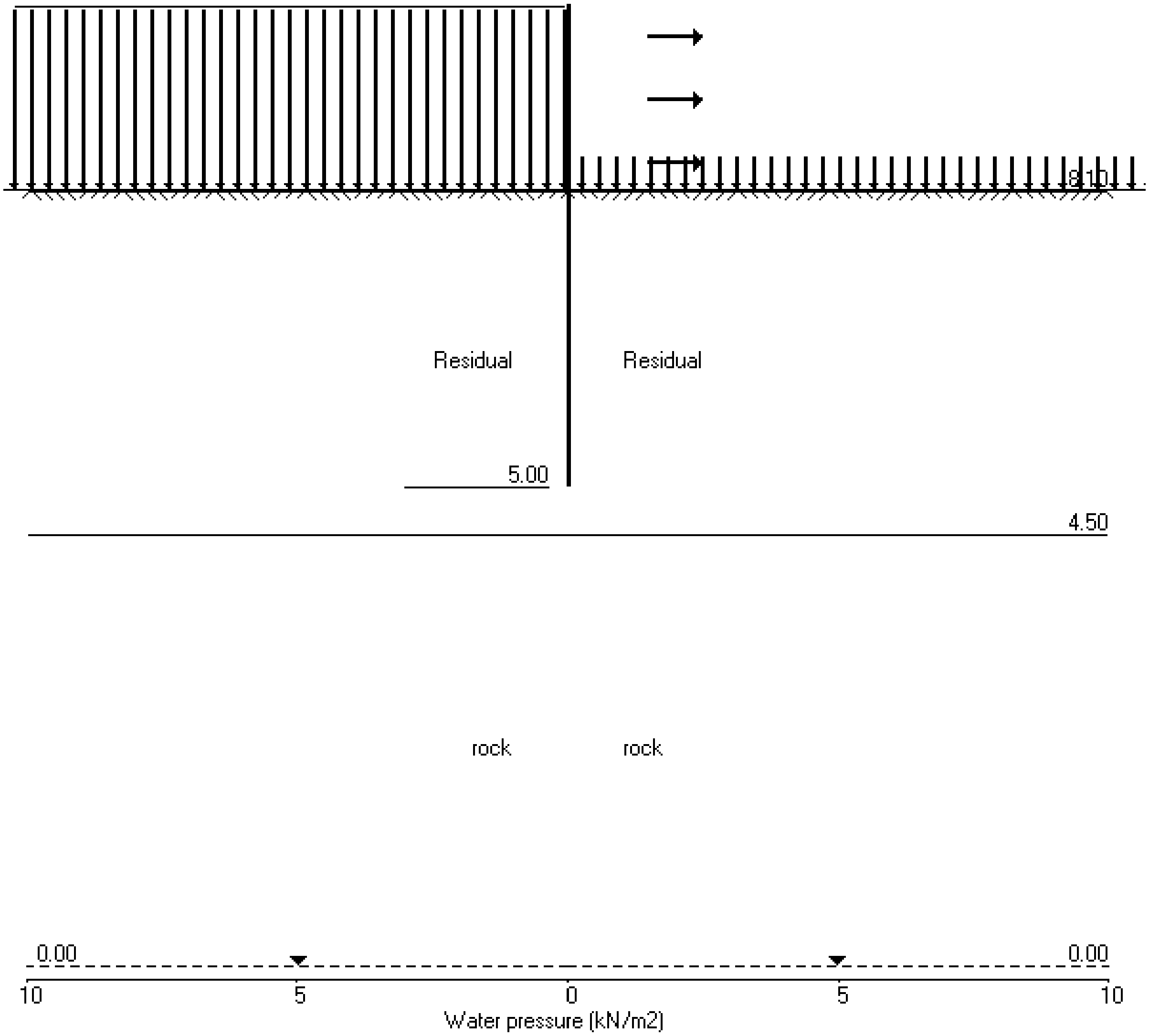
FACTORS OF SAFETY and ANALYSIS OPTIONS

```
 Stability analysis:
  Method of analysis  -  Strength Factor method
  Factor on soil strength for calculating wall depth = 1.50
  Active limit pressures calculated by Wedge Stability
  Passive limit pressures calculated by Wedge Stability

 Parameters for undrained strata:
  Minimum equivalent fluid density             =   5.00 kN/m3
  Maximum depth of water filled tension crack  =   0.00 m

 Bending moment and displacement calculation:
  Method  -  Subgrade reaction model using Influence Coefficients
  Open Tension Crack analysis? - No
  Non-linear Modulus Parameter (L) = 0 m

 Boundary conditions:
  Length of wall (normal to plane of analysis) = 20.00 m

  Width of excavation on Left  side of wall  = 20.00 m
  Width of excavation on Right side of wall  = 40.00 m

  Distance to rigid boundary on Left side  = 20.00 m
  Distance to rigid boundary on Right side = 20.00 m
```

OUTPUT OPTIONS

```
 Stage ------ Stage description ----------- ------- Output options -------
  no.                                       Displacement   Active,  Graph.
                                            Bending mom.   Passive  output
                                            Shear force   pressures
   1 Excav. to elev. 8.10 on RIGHT side          No           No      No
   2 Excav. to elev. 8.10 on LEFT side           No           No      No
   3 Apply surcharge no.3 at elev. 8.10          No           No      No
   4 Apply surcharge no.2 at elev. 8.10          No           No      No
   5 Apply load no.1 at elev. 9.70               No           No      No
   6 Apply load no.2 at elev. 9.05               No           No      No
   7 Apply load no.3 at elev. 8.40               No           No      No
   * Summary output                             Yes           -      Yes
```

Stage No.7   Apply load no.3 at elev. 8.40

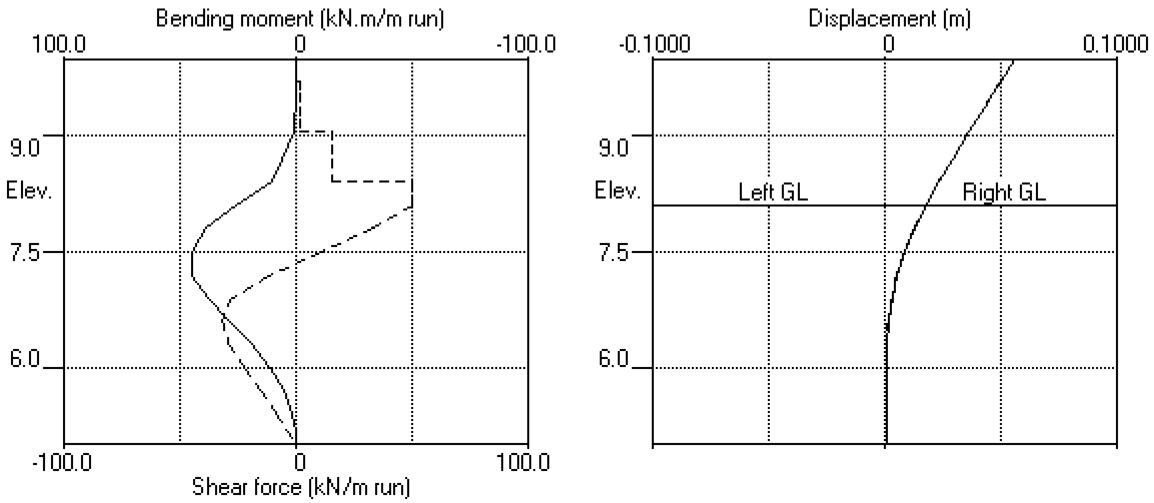


```
                                                       Units: kN,m
Stage No. 7   Apply load no.3 at elevation 8.40

STABILITY ANALYSIS of Soldier Pile Wall according to Strength Factor method
 Factor of safety on soil strength
 Active limit pressures calculated by Wedge Stability
 Passive limit pressures calculated by Wedge Stability

                                FoS for toe       Toe elev. for
                               elev. =     5.00    FoS = 1.500
 Stage  Ground level    Prop   Factor  Moment      Toe    Wall    Direction
  No.   Act.   Pass.    Elev.    of    equilib.   elev.  Penetr       of
                               Safety  at elev.          -ation     failure
   7    8.10    8.10    Cant.   1.486     5.50     ***     ***      L to R

   Legend: *** Result not found


BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall
  Analysis options
  Soldier Pile width = 0.60m;  spacing = 1.20m
  Passive mobilisation factor = 3.000
  Length of wall perpendicular to section = 20.00m
  Subgrade reaction model  -  Boussinesq Influence coefficients
  Soil deformations are elastic until the active or passive limit is reached
  Active limit pressures calculated by Wedge Stability
  Passive limit pressures calculated by Wedge Stability

  Rigid boundaries:     Left side 20.00 from wall
                       Right side 20.00 from wall

 Node   Y      Nett       Wall      Wall      Shear   Bending   Prop
 no.  coord  pressure     disp.   rotation    force   moment    forces
                kN/m2       m       rad.       kN/m    kN.m/m    kN/m
  1   10.00      0.00     0.056  2.05E-02       0.0       0.0
  2    9.70      0.00     0.050  2.05E-02       0.0      -0.0      1.4
                 0.00     0.050  2.05E-02       1.4       0.0
  3    9.54      0.00     0.046  2.05E-02       1.4       0.2
  4    9.38      0.00     0.043  2.05E-02       1.4       0.5
  5    9.21      0.00     0.040  2.04E-02       1.4       0.7
  6    9.05      0.00     0.036  2.04E-02       1.4       0.9     13.6
                 0.00     0.036  2.04E-02      15.0       0.9
  7    8.88      0.00     0.033  2.03E-02      15.0       3.5
  8    8.70      0.00     0.029  2.01E-02      15.0       6.2
  9    8.40      0.00     0.023  1.95E-02      15.0      10.8     35.0
                 0.00     0.023  1.95E-02      50.0      10.8
 10    8.10      0.00     0.018  1.81E-02      50.0      25.8
               -51.47     0.018  1.81E-02      50.0      25.8
 11    7.80    -66.85     0.013  1.56E-02      32.3      38.6
 12    7.50    -79.53     0.008  1.25E-02      10.3      44.8
 13    7.20    -69.43     0.005  9.14E-03     -12.1      45.0
 14    6.90    -33.48     0.003  5.99E-03     -27.5      38.2
 15    6.60      1.73     0.001  3.47E-03     -32.3      28.7
 16    6.30     18.15     0.001  1.67E-03     -29.3      19.1
 17    6.00     23.73     0.000  5.30E-04     -23.0      11.1
 18    5.70     24.56     0.000 -8.85E-05     -15.7       5.3
 19    5.40     23.05     0.000 -3.50E-04      -8.6       1.7
 20    5.20     21.54     0.000 -4.02E-04      -4.1       0.4
 21    5.00     19.88     0.001 -4.12E-04       0.0       0.0
```

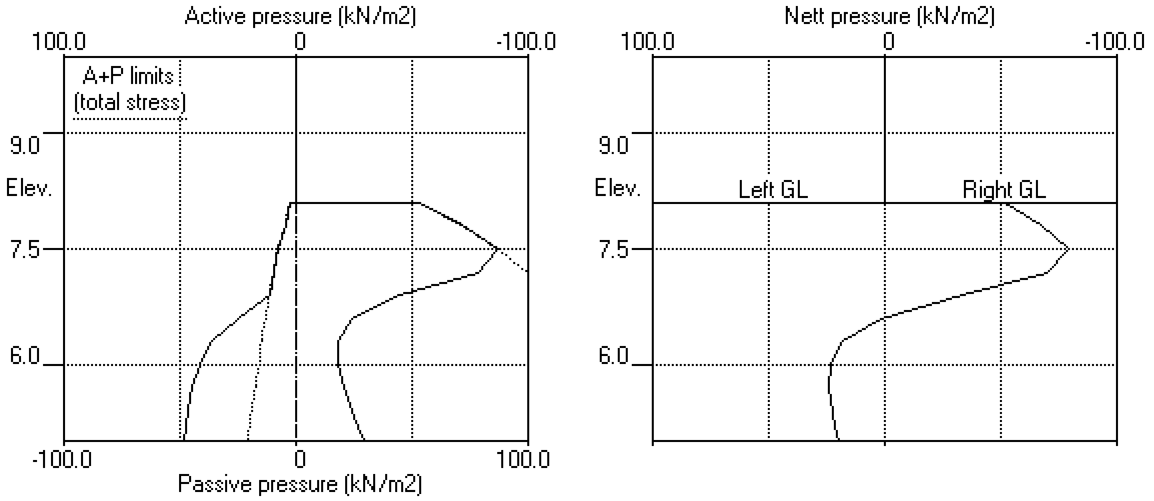
```
                                                                       (continued)
Stage No.7   Apply load no.3 at elevation 8.40

                                          LEFT side
                               Effective stresses                 Total    Coeff. of
Node    Y     Water    Vertic   Active    Passive    Earth        earth     subgrade
no.   coord   press.    -al     limit      limit    pressure    pressure   reaction
              kN/m2    kN/m2    kN/m2      kN/m2      kN/m2       kN/m2      kN/m3
  1   10.00    0.00     0.00     0.00       0.00       0.00        0.00        0.0
  2    9.70    0.00     0.00     0.00       0.00       0.00        0.00        0.0
  3    9.54    0.00     0.00     0.00       0.00       0.00        0.00        0.0
  4    9.38    0.00     0.00     0.00       0.00       0.00        0.00        0.0
  5    9.21    0.00     0.00     0.00       0.00       0.00        0.00        0.0
  6    9.05    0.00     0.00     0.00       0.00       0.00        0.00        0.0
  7    8.88    0.00     0.00     0.00       0.00       0.00        0.00        0.0
  8    8.70    0.00     0.00     0.00       0.00       0.00        0.00        0.0
  9    8.40    0.00     0.00     0.00       0.00       0.00        0.00        0.0
 10    8.10    0.00     0.00     0.00       0.00       0.00        0.00        0.0
               0.00    34.20     2.37     149.80       2.37        2.37a     18238
 11    7.80    0.00    39.60     4.60     189.68       4.60        4.60a     18238
 12    7.50    0.00    45.00     7.55     234.24       7.55        7.55a     18238
 13    7.20    0.00    50.39     9.27     258.74       9.27        9.27a     18238
 14    6.90    0.00    55.78    10.98     283.22      10.98       10.98a     18238
 15    6.60    0.00    61.15    12.70     307.65      25.50       25.50      18238
 16    6.30    0.00    66.52    14.41     332.04      36.38       36.38       9479
 17    6.00    0.00    71.88    16.11     356.38      41.84       41.84       9479
 18    5.70    0.00    77.22    17.81     380.65      44.93       44.93       9479
 19    5.40    0.00    82.55    19.51     404.87      46.85       46.85       9479
 20    5.20    0.00    86.10    20.36     380.88      47.86       47.86       9479
 21    5.00    0.00    89.64    21.18     353.79      48.81       48.81       9479


                                         RIGHT side
                               Effective stresses                 Total    Coeff. of
Node    Y     Water    Vertic   Active    Passive    Earth        earth     subgrade
no.   coord   press.    -al     limit      limit    pressure    pressure   reaction
              kN/m2    kN/m2    kN/m2      kN/m2      kN/m2       kN/m2      kN/m3
  1   10.00    0.00     0.00     0.00       0.00       0.00        0.00        0.0
  2    9.70    0.00     0.00     0.00       0.00       0.00        0.00        0.0
  3    9.54    0.00     0.00     0.00       0.00       0.00        0.00        0.0
  4    9.38    0.00     0.00     0.00       0.00       0.00        0.00        0.0
  5    9.21    0.00     0.00     0.00       0.00       0.00        0.00        0.0
  6    9.05    0.00     0.00     0.00       0.00       0.00        0.00        0.0
  7    8.88    0.00     0.00     0.00       0.00       0.00        0.00        0.0
  8    8.70    0.00     0.00     0.00       0.00       0.00        0.00        0.0
  9    8.40    0.00     0.00     0.00       0.00       0.00        0.00        0.0
 10    8.10    0.00     0.00     0.00       0.00       0.00        0.00        0.0
               0.00     7.20     0.00      53.84      53.84       53.84p     18238
 11    7.80    0.00    12.60     0.00      71.45      71.45       71.45p     18238
 12    7.50    0.00    18.00     0.00      87.09      87.09       87.09p     18238
 13    7.20    0.00    23.40     0.62     106.22      78.70       78.70      18238
 14    6.90    0.00    28.79     2.20     125.35      44.47       44.47      18238
 15    6.60    0.00    34.19     3.78     144.48      23.77       23.77      18238
 16    6.30    0.00    39.58     5.36     163.59      18.23       18.23       9479
 17    6.00    0.00    44.97     6.93     182.70      18.11       18.11       9479
 18    5.70    0.00    50.35     8.51     201.79      20.37       20.37       9479
 19    5.40    0.00    55.74    10.09     220.87      23.79       23.79       9479
 20    5.20    0.00    59.32    11.83     234.09      26.33       26.33       9479
 21    5.00    0.00    62.91    13.72     247.35      28.93       28.93       9479

Note:     10.98a  Soil pressure at active limit
          87.09p  Soil pressure at passive limit
```

Units: kN,m

Stage No.7 Apply load no.3 at elev. 8.40



Stage No.7 Apply load no.3 at elev. 8.40



```
                                                       Units: kN,m
Summary of results

STABILITY ANALYSIS of Soldier Pile Wall according to Strength Factor method
 Factor of safety on soil strength
 Active limit pressures calculated by Wedge Stability
 Passive limit pressures calculated by Wedge Stability

                                FoS for toe       Toe elev. for
                               elev. =    5.00     FoS = 1.500
 Stage  Ground level    Prop   Factor  Moment      Toe   Wall    Direction
  No.   Act.   Pass.    Elev.    of    equilib.   elev.  Penetr      of
                               Safety  at elev.          -ation    failure
   1   10.00    8.10    Cant.   2.314     5.33     6.91    1.19     L to R
   2    8.10    8.10    Cant.  Conditions not suitable for FoS calc.
   3    8.10    8.10    Cant.  Conditions not suitable for FoS calc.
   4    8.10    8.10           No analysis at this stage
   5    8.10    8.10    Cant.   2.952     5.31     7.41    0.69     L to R
   6    8.10    8.10    Cant.   2.241     5.41     6.28    1.82     L to R
   7    8.10    8.10    Cant.   1.486     5.50     ***     ***      L to R

   Legend: *** Result not found
```

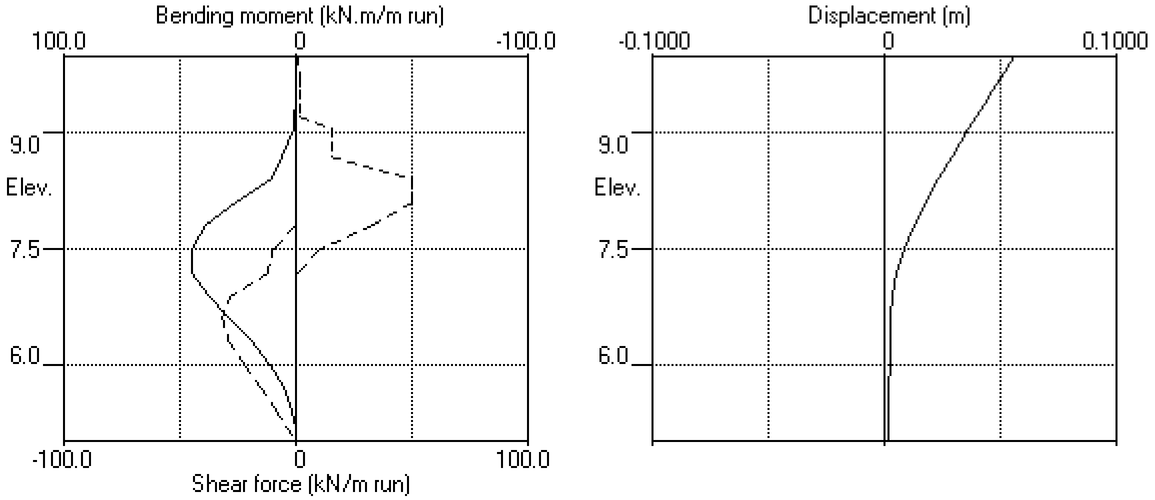
```
                                                       Units: kN,m
Summary of results

BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall
  Analysis options
  Soldier Pile width = 0.60m;  spacing = 1.20m
  Passive mobilisation factor = 3.000
  Length of wall perpendicular to section = 20.00m
  Subgrade reaction model  -  Boussinesq Influence coefficients
  Soil deformations are elastic until the active or passive limit is reached
  Active limit pressures calculated by Wedge Stability
  Passive limit pressures calculated by Wedge Stability

  Rigid boundaries:     Left side 20.00 from wall
                       Right side 20.00 from wall

Bending moment, shear force and displacement envelopes
 Node   Y       Displacement         Bending moment       Shear force
 no.  coord   maximum   minimum    maximum   minimum    maximum   minimum
                  m         m     kN.m/m    kN.m/m       kN/m      kN/m
  1   10.00     0.056     0.000        0.0      -0.0        0.0       0.0
  2    9.70     0.050     0.000        0.0      -0.0        1.4       0.0
  3    9.54     0.046     0.000        0.2      -0.0        1.4       0.0
  4    9.38     0.043     0.000        0.5      -0.0        1.4       0.0
  5    9.21     0.040     0.000        0.7      -0.0        1.4       0.0
  6    9.05     0.036     0.000        0.9      -0.0       15.0       0.0
  7    8.88     0.033     0.000        3.5      -0.0       15.0       0.0
  8    8.70     0.029     0.000        6.2      -0.0       15.0       0.0
  9    8.40     0.023     0.000       10.8       0.0       50.0       0.0
 10    8.10     0.018     0.000       25.8       0.0       50.0       0.0
 11    7.80     0.013     0.000       38.6       0.0       32.3       0.0
 12    7.50     0.008     0.000       44.8       0.0       10.3      -9.3
 13    7.20     0.005     0.000       45.0       0.0        0.8     -12.1
 14    6.90     0.003     0.000       38.2       0.0        0.2     -27.5
 15    6.60     0.002     0.000       28.7       0.0        0.0     -32.3
 16    6.30     0.002     0.000       19.1       0.0        0.0     -29.3
 17    6.00     0.002     0.000       11.1       0.0        0.0     -23.0
 18    5.70     0.002     0.000        5.3       0.0        0.0     -15.7
 19    5.40     0.002     0.000        1.7       0.0        0.0      -8.6
 20    5.20     0.002     0.000        0.4       0.0        0.0      -4.1
 21    5.00     0.002     0.000        0.0       0.0        0.0       0.0

Maximum and minimum bending moment and shear force at each stage
Stage  --------- Bending moment --------   ---------- Shear force ----------
 no.   maximum   elev.   minimum   elev.   maximum   elev.   minimum   elev.
        kN.m/m            kN.m/m             kN/m              kN/m
  1        1.3    7.50      -0.0    8.88       2.4    8.10      -0.8    6.60
  2        0.9    6.90      -0.0    8.70       1.0    7.50      -0.7    6.00
  3        0.9    6.90      -0.0    8.70       1.0    7.50      -0.7    6.00
  4    No calculation at this stage
  5        2.4    8.10      -0.0    9.70       1.4    9.70      -1.4    6.30
  6       17.9    7.80      -0.0   10.00      15.0    9.05     -12.0    7.20
  7       45.0    7.20      -0.0    9.70      50.0    8.40     -32.3    6.60
```

Summary of results   (continued)

Maximum and minimum displacement at each stage

```
Stage -------- Displacement ---------
 no.  maximum  elev.   minimum  elev.   Stage description                   _
           m                m
  1    0.004   10.00    0.000   10.00   Excav. to elev. 8.10 on RIGHT side
  2    0.002   10.00    0.000   10.00   Excav. to elev. 8.10 on LEFT side
  3    0.004   10.00    0.000   10.00   Apply surcharge no.3 at elev. 8.10
  4    No calculation at this stage     Apply surcharge no.2 at elev. 8.10
  5    0.006   10.00    0.000   10.00   Apply load no.1 at elev. 9.70
  6    0.018   10.00    0.000   10.00   Apply load no.2 at elev. 9.05
  7    0.056   10.00    0.000   10.00   Apply load no.3 at elev. 8.40
```

Bending moment, shear force, displacement envelopes



Project Name:

Dennis Matene

Subject:

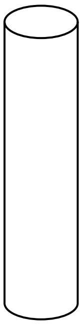
Input parameters for Wallap

By: Wayne Thorburn
Verified By:

Material Properties for Timber Pole

E = 8.70 GPa (Young Modulus) [MGS8, NZS3603 Amendement 4, Table 2.3]
8.70E+06 kPa
ρ = 450 kg/m^3 (Density)
S = 1.2 m c/c (Spacing between piles) 0.325 m ϕ

A = 0.083 m^2 (Sectional Area)
I = 5.47650E-04 m^4 (Area Moment of Inertia)
per pile



EA = **6.014E+05** kN/m = $[kN/m^2][m^2]/[m]$
EI = **3970.46** $kNm^2/m = [kN/m^2][m^4]/[m]$
w = **0.305** $kN/m/m = [kg/m^3][m/s^2][m^2]/[m]$

I 4.564E-04 m^4/m per unit length of wall
EI 3970.46 $kNm^2/m = [kN/m^2][m^4]/[m]$
per unit length of wall

	(m)	(kNm/m) BM	(kN/m SF	c/c (m)
Max Height	1.5	45.5	50	1.2

Load factor = 1

DESIGN

(kNm) BM	(kN) SF	fos	disp (mm)	pole size (mm)	Embedmer (m)	Total length (m)
54.6	60		56	325	3.5	5

pole design (maximum)

(kNm) BM	(kN) SF
58	97
OK	OK

Factored load on the plank at the base of the wall = 24.30 kPa

From Wallap	
1.5	Height (m)
16.2	kPa
1.5	Load factor
2	Rails Required
	Height (m)
	kPa
	Load factor
	Rails Required

Structural Design of Lagging to NZS 3603:1993

Timber Lagging: Structural actions

Lagging width b = 50 50

Lagging depth d = 150 150

For a maximum soil pressure of 24.3 kPa. The UDL on lagging "d" = 3.65 kN/m

Lagging Span "L" = 1.2 m

Maximum factored moment M* = 1/8 dL^2 0.656 kNm

Under Flexure, calculate the minimum lagging depth for moment capacity

Bending Stress, f_b = 11.7 MPa

Shear Stress, f_s = 2.4 MPa

No of parallel support elements, n = 1

Strength Reduction Factor, ϕ = 0.8

Duration Factor, k_1 = 0.6

Parallel Support Factor , k_4 = 1.00

Grid System Factor, k_5 = 1.00

Rails	Height	Approx. No.
Single	0 to 0.75	5
Double	0.9 to 1.5	5

Section modulus of lagging, Z = bd^2/6 = 62500 mm^3

$\phi M_n = \phi k_1 k_4 k_5 f_b Z$ = 0.351 kNm

Percentage of lagging moment capacity utilised 187%

Lagging moment capacity exceeded, try adjusting lagging size!

Check for Shear Capacity

For 150 x 50 lagging. Shear surface area = 5000.0 mm^2

$\phi V_n = \phi k_1 k_4 k_5 f_s A_s$ = 5.760 kN

Compare with V* = 2.734 kN

$$V^* = 0.625wL$$

Percentage of Shear capacity utilised 47%

Lagging OK for Shear Capacity!

Use 150 x 50 lagging, spanning continuously across a minimum of 2 pole spacings

Notes

1. This spreadsheet is applicable for low retaining walls. No seismic design is considered
2. Groundwater is modelled by adding hydrostatic pressures to the lateral soil loads calculated for the dry backfill.
Water pressure should be analysed with caution. Backfill should normally comprise granular material and with subsoil drainage.
3. Line Load and Point Load Surcharge are not considered
4. Compaction loads during construction are not considered
5. Soil Arching between piles is not considered. The lagging is designed to take full earth pressure, spanning a minimum of two pole spacings.
6. Waler design is not included for tie-back wall design

```
                                                       Units: kN,m
```

INPUT DATA

SOIL PROFILE

```
Stratum   Elevation of    ------------------ Soil types -------------------
  no.    top of stratum    Left side                 Right side
   1         10.00       1  Residual              1  Residual
   2          4.50       2  rock                  2  rock
```

SOIL PROPERTIES

```
                  Bulk    Young's   At rest  Consol  Active  Passive
-- Soil type --  density  Modulus    coeff.  state.  limit    limit   Cohesion
No. Description   kN/m3  Eh,kN/m2     Ko     NC/OC    Ka       Kp      kN/m2
  (Datum elev.)          (dEh/dy ) (dKo/dy) (  Nu ) ( Kac ) (  Kpc ) ( dc/dy )
 1  Residual      18.00     25000    0.500     OC    0.298    3.675     5.000d
                                            (0.350) (1.267) ( 4.825)
 2  rock          18.00     50000    0.500     OC    0.285    3.878     5.000d
                                            (0.350) (1.238) ( 4.985)
 3  fill          18.00     10000    0.580     OC    0.353    2.989
                                            (0.350) (0.000) ( 0.000)
```

Additional soil parameters associated with Ka and Kp

```
                          --- parameters for Ka ---  --- parameters for Kp ---
                            Soil      Wall    Back-    Soil      Wall    Back-
------- Soil type ------- friction  adhesion  fill   friction  adhesion  fill
No. Description             angle    coeff.   angle    angle    coeff.   angle
 1  Residual                29.00    0.633    0.00     29.00    0.304    0.00
 2  rock                    30.00    0.631    0.00     30.00    0.302    0.00
 3  fill                    25.00    0.642    0.00     25.00    0.311    0.00
```

GROUND WATER CONDITIONS

```
 Density of water = 9.810 kN/m3
                                   Left side     Right side
 Initial water table elevation        0.00            0.00

 Automatic water pressure balancing at toe of wall :  No
```

WALL PROPERTIES

```
                         Type of structure = Soldier Pile Wall
                        Soldier Pile width =  0.45 m
                      Soldier Pile spacing =  1.20 m
               Passive mobilisation factor =  3.00
                  Elevation of toe of wall =  7.50
             Maximum finite element length =  0.20 m
                  Youngs modulus of wall E = 8.7000E+06 kN/m2
               Moment of inertia of wall I = 6.5450E-05 m4/m run
                                           = 7.8540E-05 m4 per pile
                                       E.I = 569.42 kN.m2/m run
                      Yield Moment of wall = Not defined
```

HORIZONTAL and MOMENT LOADS/RESTRAINTS

```
 Load            Horizontal    Moment      Moment     Partial
  no.  Elevation     load        load      restraint   factor
                   kN/m run   kN.m/m run  kN.m/m/rad  (Category)
   1       9.70       1.400      0           0          N/A
   2       9.05       13.60      0           0          N/A
   3       8.40       35.00      0           0          N/A
```

SURCHARGE LOADS

```
Surch         Distance   Length    Width        Surcharge      Equiv. Partial
-arge           from    parallel  perpend. -----  kN/m2  -----  soil  factor/
 no.   Elev.    wall    to wall   to wall  Near edge  Far edge  type  Category
  1    11.50    0.00(L)   20.00      5.00      0.00     30.00     1     N/A
  2    11.50    5.00(L)   20.00     10.00     30.00     =         1     N/A
  3     8.10    0.00(L)   20.00     20.00     34.20     =         1     N/A

    Note: L = Left side,  R = Right side
          A trapezoidal surcharge is defined by two values:
          N = at edge near to wall,  F = at edge far from wall
```

CONSTRUCTION STAGES

```
Construction   Stage description
  stage no.    --------------------------------------------------------
      1        Fill to elevation 11.50 on LEFT side with soil type 3
      2        Apply surcharge no.1 at elevation 11.50
      3        Apply surcharge no.2 at elevation 11.50
      4        Apply seismic loading:
               0.100g horizontal
               Line of action of quasi-static seismic force = 0.333
               Seismic loading model: Quasi-static loading
```

FACTORS OF SAFETY and ANALYSIS OPTIONS

```
 Stability analysis:
  Method of analysis  -  Strength Factor method
  Factor on soil strength for calculating wall depth = 1.50
  Active limit pressures calculated by Wedge Stability
  Passive limit pressures calculated by Wedge Stability

 Parameters for undrained strata:
  Minimum equivalent fluid density            =   5.00 kN/m3
  Maximum depth of water filled tension crack  =   0.00 m

 Bending moment and displacement calculation:
  Method  -  Subgrade reaction model using Influence Coefficients
  Open Tension Crack analysis? - No
  Non-linear Modulus Parameter (L) = 0 m

 Boundary conditions:
  Length of wall (normal to plane of analysis) = 20.00 m

  Width of excavation on Left  side of wall  = 20.00 m
  Width of excavation on Right side of wall  = 40.00 m

  Distance to rigid boundary on Left side  = 20.00 m
  Distance to rigid boundary on Right side = 20.00 m
```

OUTPUT OPTIONS

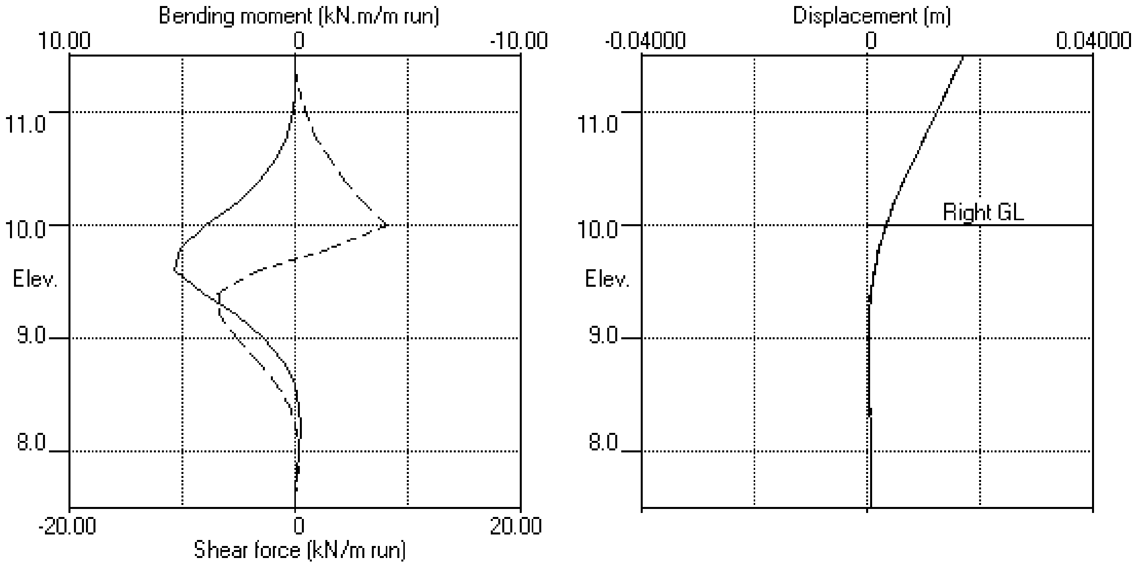
```
 Stage ------ Stage description ----------- ------- Output options -------
  no.                                       Displacement   Active,  Graph.
                                            Bending mom.   Passive  output
                                            Shear force   pressures
   1 Fill to elev. 11.50 on LEFT side           Yes          Yes     Yes
   2 Apply surcharge no.1 at elev. 11.50         No           No      No
   3 Apply surcharge no.2 at elev. 11.50         No           No      No
   4 Quasi-static Seismic load: 0.100g(H)        No           No      No
   * Summary output                             Yes           -      Yes
```

```
                                                            (continued)
Stage No.3   Apply surcharge no.2 at elevation 11.50

                                  LEFT side
                         Effective stresses
 Node    Y     Water   Vertic  Active   Passive    Earth       Total    Coeff. of
 no.   coord   press.   -al    limit    limit     pressure     earth    subgrade
                                                             pressure   reaction
               kN/m2   kN/m2   kN/m2    kN/m2     kN/m2        kN/m2      kN/m3
  1    11.50    0.00    0.00    0.00     0.00      0.00         0.00      36966
  2    11.35    0.00    3.27    0.61    14.65      0.61         0.61a     36966
  3    11.20    0.00    6.54    2.16    39.01      2.17         2.17a      4209
  4    11.00    0.00   10.90    3.60    64.99      3.61         3.61a      4209
  5    10.80    0.00   15.25    5.04    90.93      5.04         5.04a      4209
  6    10.60    0.00   19.59    6.48   116.81      6.48         6.48       4209
  7    10.40    0.00   23.92    7.91   142.62      7.92         7.92       4209
  8    10.20    0.00   28.24    9.33   168.33      9.36         9.36       4209
  9    10.00    0.00   32.53   10.75   193.95     10.80        10.80       4209
                0.00   32.53    3.79   204.20      3.84         3.84      10522
 10     9.80    0.00   36.81    5.22   226.54      5.34         5.34      10522
 11     9.60    0.00   41.07    6.66   248.77      7.67         7.67      10522
 12     9.40    0.00   45.30    8.08   270.89     16.77        16.77      10522
 13     9.20    0.00   49.51    9.50   292.89     22.24        22.24      10522
 14     9.00    0.00   53.70   10.90   314.75     25.35        25.35      10522
 15     8.80    0.00   57.86   12.30   336.49     27.18        27.18      10522
 16     8.60    0.00   62.00   13.69   358.09     28.49        28.49      10522
 17     8.40    0.00   66.11   15.08   379.55     29.71        29.71      10522
 18     8.20    0.00   70.19   16.45   400.88     31.05        31.05      10522
 19     8.00    0.00   74.25   17.81   422.07     32.54        32.54      10522
 20     7.80    0.00   78.28   19.17   443.12     34.15        34.15      10522
 21     7.65    0.00   81.29   20.33   427.83     35.39        35.39      10522
 22     7.50    0.00   84.28   21.49   410.36     36.63        36.63      10522


                                 RIGHT side
                         Effective stresses
 Node    Y     Water   Vertic  Active   Passive    Earth       Total    Coeff. of
 no.   coord   press.   -al    limit    limit     pressure     earth    subgrade
                                                             pressure   reaction
               kN/m2   kN/m2   kN/m2    kN/m2     kN/m2        kN/m2      kN/m3
  1    11.50    0.00    0.00    0.00     0.00      0.00         0.00        0.0
  2    11.35    0.00    0.00    0.00     0.00      0.00         0.00        0.0
  3    11.20    0.00    0.00    0.00     0.00      0.00         0.00        0.0
  4    11.00    0.00    0.00    0.00     0.00      0.00         0.00        0.0
  5    10.80    0.00    0.00    0.00     0.00      0.00         0.00        0.0
  6    10.60    0.00    0.00    0.00     0.00      0.00         0.00        0.0
  7    10.40    0.00    0.00    0.00     0.00      0.00         0.00        0.0
  8    10.20    0.00    0.00    0.00     0.00      0.00         0.00        0.0
  9    10.00    0.00    0.00    0.00     0.00      0.00         0.00        0.0
                0.00    0.00    0.00    23.83     23.83        23.83p     10522
 10     9.80    0.00    3.60    0.00    36.90     36.90        36.90p     10522
 11     9.60    0.00    7.20    0.00    49.96     37.75        37.75      10522
 12     9.40    0.00   10.80    0.00    63.03     22.55        22.55      10522
 13     9.20    0.00   14.40    0.00    76.09     16.18        16.18      10522
 14     9.00    0.00   18.00    0.00    89.16     15.54        15.54      10522
 15     8.80    0.00   21.60    0.00   102.23     17.96        17.96      10522
                0.00   21.60    0.08   102.23     17.96        17.96      10522
 16     8.60    0.00   25.20    0.95   115.29     21.59        21.59      10522
 17     8.40    0.00   28.80    1.82   128.36     25.38        25.38      10522
 18     8.20    0.00   32.40    2.68   141.42     28.84        28.84      10522
 19     8.00    0.00   36.00    3.55   154.49     31.89        31.89      10522
 20     7.80    0.00   39.60    4.42   167.56     34.61        34.61      10522
 21     7.65    0.00   42.30    5.07   177.36     36.55        36.55      10522
 22     7.50    0.00   45.00    5.73   187.16     38.45        38.45      10522
```

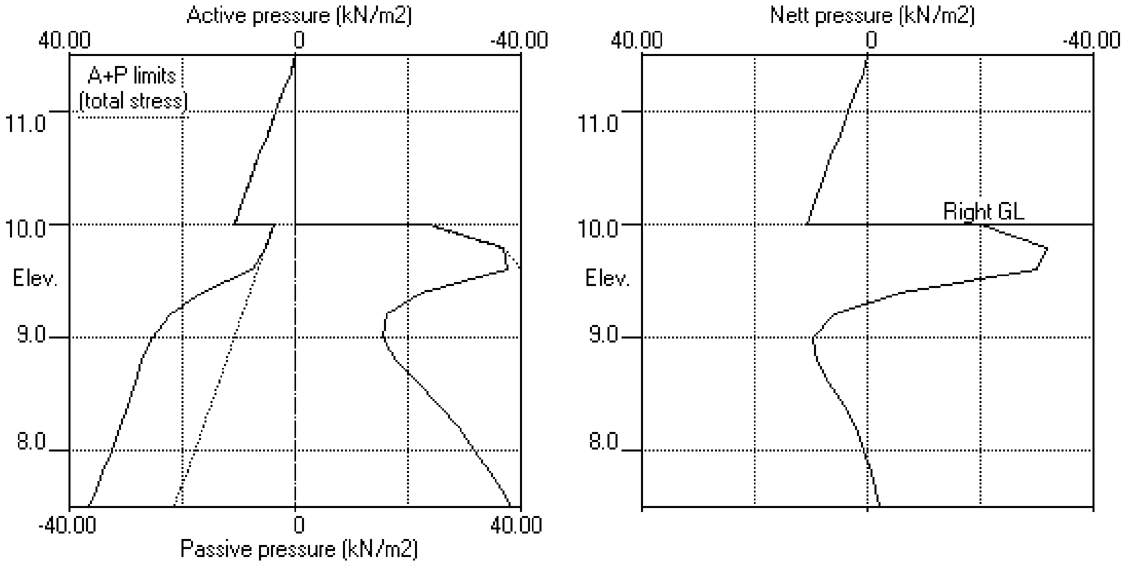
lagging pressure

```
                                                   Units: kN,m
```

Stage No.3 Apply surcharge no.2 at elev. 11.50



Stage No.3 Apply surcharge no.2 at elev. 11.50



```
                                                       Units: kN,m
Summary of results

STABILITY ANALYSIS of Soldier Pile Wall according to Strength Factor method
 Factor of safety on soil strength
 Active limit pressures calculated by Wedge Stability
 Passive limit pressures calculated by Wedge Stability

                                FoS for toe       Toe elev. for
                               elev. =     7.50    FoS = 1.500
 Stage  Ground level    Prop   Factor  Moment      Toe    Wall    Direction
  No.   Act.   Pass.    Elev.    of    equilib.   elev.  Penetr      of
                               Safety  at elev.          -ation    failure
   1   11.50   10.00    Cant.   2.127     7.81     8.50    1.50     L to R
   2   11.50   10.00    Cant.   1.695     7.77     8.08    1.92     L to R
   3   11.50   10.00    Cant.   1.724     7.69     8.16    1.84     L to R
   4   11.50   10.00    Cant.   1.429     7.68     ***     ***      L to R

   Legend: *** Result not found
```

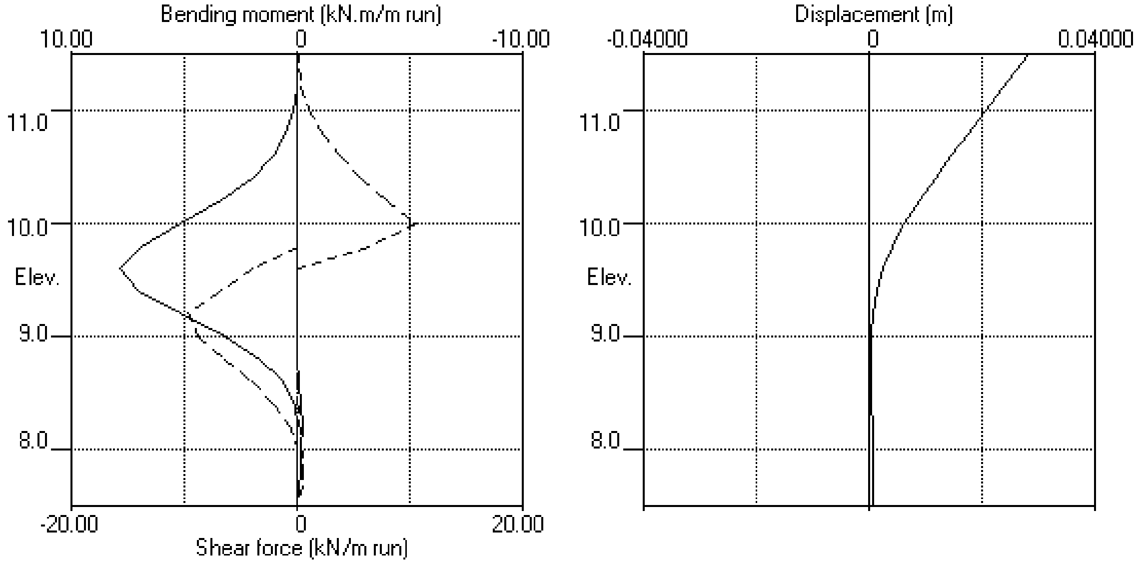
```
                                                        Units: kN,m
Summary of results

BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall
  Analysis options
  Soldier Pile width = 0.45m;  spacing = 1.20m
  Passive mobilisation factor = 3.000
  Length of wall perpendicular to section = 20.00m
  Subgrade reaction model  -  Boussinesq Influence coefficients
  Soil deformations are elastic until the active or passive limit is reached
  Active limit pressures calculated by Wedge Stability
  Passive limit pressures calculated by Wedge Stability

  Rigid boundaries:      Left side 20.00 from wall
                        Right side 20.00 from wall

Bending moment, shear force and displacement envelopes
 Node    Y       Displacement         Bending moment       Shear force
 no.   coord    maximum   minimum    maximum   minimum    maximum   minimum
                   m         m        kN.m/m    kN.m/m      kN/m      kN/m
  1    11.50    0.028     0.000        0.0      -0.0        0.0       0.0
  2    11.35    0.026     0.000        0.0       0.0        0.1       0.0
  3    11.20    0.024     0.000        0.0       0.0        0.3       0.0
  4    11.00    0.020     0.000        0.2       0.0        1.1       0.0
  5    10.80    0.017     0.000        0.5       0.0        2.2       0.0
  6    10.60    0.014     0.000        1.1       0.0        3.7       0.0
  7    10.40    0.012     0.000        2.0       0.0        5.6       0.0
  8    10.20    0.009     0.000        3.4       0.0        7.9       0.0
  9    10.00    0.006     0.000        5.2       0.0       10.6       0.0
 10     9.80    0.004     0.000        6.9       0.0        6.1       0.0
 11     9.60    0.002     0.000        7.8       0.0        0.0      -3.9
 12     9.40    0.001     0.000        7.0       0.0        0.0      -6.9
 13     9.20    0.001     0.000        5.2       0.0        0.0      -9.7
 14     9.00    0.000     0.000        3.2       0.0        0.0      -8.8
 15     8.80    0.000     0.000        1.7       0.0        0.0      -6.4
 16     8.60    0.000     0.000        0.6      -0.0        0.0      -4.0
 17     8.40    0.000     0.000        0.1      -0.2        0.0      -2.0
 18     8.20    0.000     0.000        0.0      -0.2        0.1      -0.7
 19     8.00    0.000     0.000        0.0      -0.2        0.3       0.0
 20     7.80    0.001     0.000        0.0      -0.1        0.4       0.0
 21     7.65    0.001     0.000        0.0      -0.1        0.4       0.0
 22     7.50    0.001     0.000        0.0      -0.0        0.0       0.0

Maximum and minimum bending moment and shear force at each stage
Stage  --------- Bending moment --------   ---------- Shear force ----------
 no.   maximum  elev.   minimum  elev.   maximum  elev.   minimum  elev.
        kN.m/m           kN.m/m            kN/m             kN/m
  1       4.3    9.80      -0.2   8.20       7.0  10.00      -6.0   9.40
  2       5.4    9.60      -0.2   8.20       8.0  10.00      -6.9   9.40
  3       5.4    9.60      -0.2   8.20       8.0  10.00      -6.9   9.40
  4       7.8    9.60      -0.2   8.00      10.6  10.00      -9.7   9.20
```

Summary of results (continued)

Maximum and minimum displacement at each stage

```
Stage -------- Displacement ---------
 no.  maximum  elev.   minimum  elev.   Stage description                     _
          m                m
  1    0.013   11.50    0.000   11.50   Fill to elev. 11.50 on LEFT side
  2    0.017   11.50    0.000   11.50   Apply surcharge no.1 at elev. 11.50
  3    0.017   11.50    0.000   11.50   Apply surcharge no.2 at elev. 11.50
  4    0.028   11.50    0.000   11.50   Quasi-static Seismic load: 0.100g(H)
```

Bending moment, shear force, displacement envelopes



Project Name:

Dennis Matene

Subject:

Input parameters for Wallap

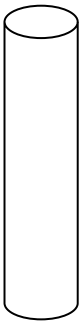
By: Wayne Thorburn
Verified By:

Material Properties for Timber Pole

E = 8.70 GPa (Young Modulus) [MGS8, NZS3603 Amendement 4, Table 2.3]
8.70E+06 kPa
ρ = 450 kg/m^3 (Density)
S = 1.2 m c/c (Spacing between piles) 0.200 m ϕ

A = 0.031 m^2 (Sectional Area)
I = 7.85398E-05 m^4 (Area Moment of Inertia)
per pile

EA = **2.278E+05** kN/m = [kN/m^2][m^2]/[m]
EI = **569.41** kNm^2/m = [kN/m^2][m^4]/[m]
w = **0.116** kN/m/m = [kg/m^3][m/s^2][m^2]/[m]



I 6.545E-05 m^4/m per unit length of wall
EI 569.41 kNm^2/m = [kN/m^2][m^4]/[m]
per unit length of wall

	(m)	(kNm/m) BM	(kN/m SF	c/c (m)
Max Height	1.5	5.4	8	1.2

Load factor = 1.5

DESIGN

(kNm) BM	(kN) SF	fos	disp (mm)	pole size (mm)	Embedmer (m)	Total length (m)
9.72	14.4		17	200	2.5	4

pole design (maximum)	
(kNm) BM	(kN) SF
16	40
OK	OK

Factored load on the plank at the base of the wall = 16.20 kPa

From Wallap

1.5	Height (m)
10.8	kPa
1.5	Load factor
2	Rails Required
	Height (m)
	kPa
	Load factor
	Rails Required

Structural Design of Lagging to NZS 3603:1993

Timber Lagging: Structural actions

Lagging width b = 50 50

Lagging depth d = 150 150

For a maximum soil pressure of 16.2 kPa. The UDL on lagging "d" = 2.43 kN/m

Lagging Span "L" = 1.2 m

Maximum factored moment M* = 1/8 dL^2 0.437 kNm

Under Flexure, calculate the minimum lagging depth for moment capacity

Bending Stress, f_b = 11.7 MPa

Shear Stress, f_s = 2.4 MPa

No of parallel support elements, n = 2

Strength Reduction Factor, ϕ = 0.8

Duration Factor, k_1 = 0.6

Parallel Support Factor , k_4 = 1.00

Grid System Factor, k_5 = 1.00

Rails	Height	Approx. No.
Single	0 to 1.05	7
Double	1.05 to 1.5m	3

Section modulus of lagging, Z = bd^2/6 = 125000 mm^3

$\phi M_n = \phi k_1 k_4 k_5 f_b Z$ = 0.702 kNm

Percentage of lagging moment capacity utilised 62%

Lagging OK for Moment Capacity!

Check for Shear Capacity

For 150 x 50 lagging. Shear surface area = 5000.0 mm^2

$\phi V_n = \phi k_1 k_4 k_5 f_s A_s$ = 5.760 kN

Compare with V* = 1.823 kN

$$V^* = 0.625wL$$

Percentage of Shear capacity utilised 32%

Lagging OK for Shear Capacity!

Use 150 x 100 lagging, spanning continuously across a minimum of 2 pole spacings

Notes

1. This spreadsheet is applicable for low retaining walls. No seismic design is considered
2. Groundwater is modelled by adding hydrostatic pressures to the lateral soil loads calculated for the dry backfill.
Water pressure should be analysed with caution. Backfill should normally comprise granular material and with subsoil drainage.
3. Line Load and Point Load Surcharge are not considered
4. Compaction loads during construction are not considered
5. Soil Arching between piles is not considered. The lagging is designed to take full earth pressure, spanning a minimum of two pole spacings.
6. Waler design is not included for tie-back wall design

Appendix D – PS4 Advisory Note

IMPORTANT ADVISORY NOTE

PRODUCER STATEMENT – CONSTRUCTION REVIEW (PS4)

The Building Consent Authority (BCA) frequently requires Producer Statements–Construction Review (PS4) to be submitted to the BCA in order for a Code of Compliance Certificate (CCC) to be issued. A PS4 is usually required for each specialist area. The requirement for a consultant to issue a PS4 related to their area of work will appear as a condition in the Building Consent documents.

It is the consent holder's responsibility to notify Haigh Workman Limited for geotechnical construction monitoring and testing required for subsequent issue of a PS4. An initial inspection of stripped or excavated ground must take place before any fill or blinding concrete is placed. Retrospective site monitoring of completed or partially completed geotechnical work is not possible and a PS4 will not be issued without all the required observations.

In order to secure our construction monitoring services and avoid delays on site, Haigh Workman Limited require at least 24 hours' notice prior to the time the site visit is required. Construction monitoring is limited to items that have been recommended, designed and detailed by Haigh Workman Limited. We are unable to inspect non-consented or unauthorised work. Haigh Workman Limited do not carry out construction monitoring or issue PS4's for work that has been recommended, designed or detailed by other consultants without prior approval from Haigh Workman Limited. Haigh Workman Limited will not issue a PS4 where construction monitoring and/or testing have been carried out by any other consultant. The PS4 must be sought from the consultant who carried out those inspections.

The full Building Consent, with stamped plans with consent numbers (or a legible copy of the same) including all amendments, shall be made available to us during inspections. We will not commence construction monitoring until the documentation is available or provided to us prior to our site visit.

Unless stated otherwise in our terms of engagement, the fees associated with construction monitoring and the issue of PS4's are separate from any work carried out prior to commencement of construction. We are able to provide a fee estimate for this work if required. We cannot provide a fixed quote because the quantum of work required frequently depends on the construction program and the performance of others. These things are not known to us in advance of construction. Our normal terms of trade require payment of fees monthly during the inspection period and full settlement prior to release of any PS4.