06 – 10 Orr Street, Netherby, Ashburton

Geotechnical Design Report

Prepared for Kāinga Ora
Prepared by Beca Limited

25 March 2024

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Revision History

Revision N°	Prepared By	Description	Date
01	Kiri Moonen	Draft for information	28/02/2024
02	Kiri Moonen	For building and subdivision consent	12/03/2024
03	Kiri Moonen	Updated Groundwater and Infiltration Assessments	25/03/2024

Document Acceptance

Action	Name	Signed	Date
Prepared by	Kiri Moonen		12/03/2024
Reviewed by	Sam Glue		12/03/2024
Approved by	Paul Horrey		12/03/2024
on behalf of	Beca Limited		

Executive Summary

Ground Investigations Conducted	
Infiltration Test Pits	2
Hand Auger Tests (Completed by Kirk Roberts, February 2023)	16
Ground Conditions	
Gravel below the topsoil at the street side of the site and a layer of stiff silt present between the topsoil and the gravel at the rear of site.	
Site Classification	
NZS1170.5 Site Subsoil class	Soil class D
Liquefaction Hazard	
Vulnerability to liquefaction (ADC vulnerability map classification)	Low
The MBIE residential Foundation Technical Category at this site is:	TC1
Lateral Movement Hazard	
Distance to the nearest free face/ watercourse	180 m
Height of the free face	0.8 m
Risk of lateral movement-induced ground damage	Low
Design Depths	
Measured depth to groundwater at time of investigation	Not Encountered
Average depth to groundwater [Canterbury Maps Wells]	1.2 to 1.6 m bgl
Topsoil thickness	0.4 m / RL varies
Anticipated maximum depth to achieve 300 kPa geotechnical ultimate bearing capacity (accounting for topsoil removal)	0.4 m / RL varies
Foundation Solution	
Controlling factor for foundation solution	Finished Floor Levels
Recommended foundation type	TC1 Waffle Slab on a 200 - 400 mm thick AP65 engineered fill raft, designed for 300 kPa geotechnical ultimate bearing capacity. Gravel raft will extend 1 m beyond the building footprint.
Long term static settlement (50 years, excluding liquefaction settlement)	Negligible
Modulus of subgrade reaction for foundation	12 MPa
Driveway CBR	8 %

Preliminary Foundation Depths (m RL)

House Typology / Position	Existing ground Level (m RL)	ADC FFL (m RL*) (top of stiffened waffle slab)	TC2 waffle slab thickness (mm)	Base of TC2 waffle slab (m RL*)	Base of gravel pad / excavation level (m RL*)	Thickness of gravel pad (mm)
Houses 1 & 2	95.9	96.15	400	95.75	95.55	200
Houses 3 & 4	96.1	96.30		95.90	95.70	200
Houses 5 & 6	96.0	96.39		95.99	95.59	400
House 7	95.6	96.01		95.61	95.31	300
House 8 & 9	95.6	95.95		95.55	95.30	250

*Based on preliminary slab finished floor level (m RL, LVD1937) as stated. If this changes during detailed design, the finished ground level and base of raft will need to be updated on final construction drawings.

Infiltration Testing

Infiltration testing was conducted at site. Stormwater attenuation through soakage is possible on site. A design infiltration rate of; 203 mm/hour should be used in design for Houses 1 and 2, 109 mm/hour should be used for the remaining houses, 181 mm/hour should be used for the JOAL.

Retaining Wall Design

A standardised conservative timber pole retaining wall design will apply to retained heights less than 2 m, as per the following specification.

Timber Pile (SED) Retaining Wall Design

Max retained height	2000 mm	1500 mm	1000 mm	500 mm
Pole embedment	5000 mm	3500 mm	2500 mm	1500 mm
Normal pole length	7000 mm	5000 mm	3500 mm	2000 mm
Pole size (diameter, SED)	425 mm	275 mm	225 mm	150 mm
Socket size (diameter)	550 mm	400 mm	350 mm	300 mm
Pole spacing	1100 mm	1200 mm	1200 mm	1200 mm
Lagging dimensions	150 mm x 75 mm (THICK)		150 mm x 50 mm (THICK)	

Timber Retaining Wall Setbacks – Cohesionless Material

Max Retained Height	2 m	1.5 m	1.2 m	1.0 m	0.5 m
Setback requirements for cut boundary retaining walls	2.9 m	2.225 m	0.725 m	0.625 m	0.525 m

Notes:

Where the wall height is less than 1.2m, the batter may be cut vertical. In this situation some slope losses may occur if the cut is left open for extended periods of time.

1 Introduction

Kāinga Ora is redeveloping the site at 6 – 10 Orr Street in Netherby, Ashburton. Beca Limited (Beca) has been commissioned to undertake a geotechnical investigation and provide analysis and recommendations to support the development of the site. This report outlines the findings from the geotechnical investigations, desktop assessment, geotechnical design, and recommendations.

2 Site Description

The house development site is located in Netherby on 6 – 10 Orr Street to the north-east of the Ashburton CBD. The site is being uplifted from three houses to nine houses on a plot covering an area of 2,697 m^2 and has predominantly flat topography. The site is 180 m from the nearest waterway a drainage ditch to the north-east. The site location and basic details of the proposed development are presented in Figure 2-1.

Figure 2-1:Site Location Plan.

3 Geology

The Geology of the Aoraki Area 1:250 000 published geology map (Cox and Barrell, 2007) shows the site to be underlain by Late Pleistocene (Q1a) aged river alluvial gravel, sand and silt forming a modern floodplain or low-level terrace.

The Mt Hutt-Mt Peel fault zone (also called the Canterbury Range Front Faults and Geraldine-Mt Hutt Fault System) is the nearest mapped active fault system located approximately 33 km northwest of the site. Active faults within this fault zone include the Peel Forest Fault and the Montalto Fault (GNS, 2020). A study of this fault system by Pettinga et al. (2001) indicates that the average earthquake recurrence interval on this fault system is approximately every 5,000 to 10,000 years. This fault system has the potential to produce earthquakes up to magnitude 7.3 Mw (Pettinga et al., 2001). No other active faults are known to exist within a 30 km radius from the site.

The Canterbury plains typically have a shallow unconfined aquifer with a water table less than 20 m below the ground surface. Deeper confined aquifers are generally found at 30 m to 80 m and 130 m to 160 m depth (Forsyth et al., 2008).

The geological map of the site area is presented in Figure 3-1.

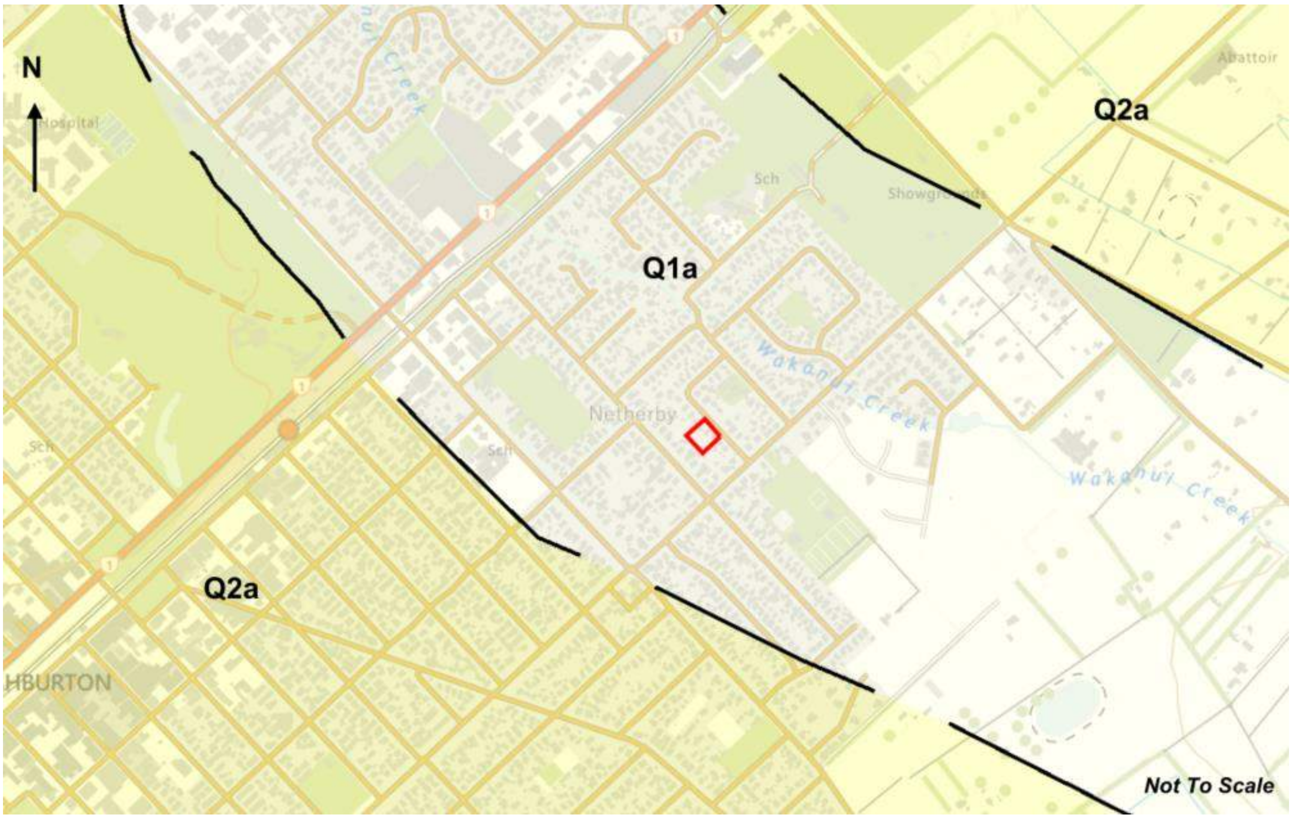

Figure 3-1: Geology at the site (GNS Science, 2024).

4 Desktop Information

A geotechnical desktop assessment was undertaken to understand the published information of the site. The following resources were reviewed:

- New Zealand Geotechnical Database (NZGD)
- Beca Reports Database
- Retrolens – historical aerial photographs
- Canterbury Maps
- Ashburton District Planning Maps
- Historical Black Maps (source Canterbury Maps)
- Geotechnical Report for a New Residential Dwelling – 6-10 Orr Street, Ashburton – Kirk Roberts, 2023
- Desktop Study Report 6-10 Orr Street, Ashburton – Beca, 2022

4.1 Desktop Review Summary

The findings of the assessment are as follows:

- There is existing geotechnical data on this site, completed by Kirk Roberts in 2023. This information showed the site is underlain by shallow gravels. No deep investigations were undertaken to confirm the thickness of the gravel.
- Nearby deep investigations, 400 m to the west of site, indicated that gravel extends from 1.5 m to borehole termination at 10.5 m depth. A nearby well log, 150 m to the southeast of site, indicated gravel extends from 0.3 m bgl to beyond 100 m bgl. This indicates that the gravel layer encountered on site is expected to be greater than 20 m thick.
- Previous Housing Development System (HDS) developments within proximity of the site include Dobson Street, 1.7 km to the west. The site revealed ground profiles similar to that described above and was classed as TC1 following site-specific investigations.
- The site is approximately 180 m from the nearest waterway, an unnamed drainage ditch to the northeast with a free face height of approximately 0.8 m.
- A groundwater assessment was conducted by analysing the wells shallower then 10m bgl in an 850m radius. This assessment showed the groundwater in the area typically ranges from 1.2 to 1.6m bgl.
- Early geomorphic maps of the city (‘Black Maps’) shows the site as grass and flax.
- A review of historic aerial imagery shows that the land was used as farmland prior to being developed in the mid 1950’s.
- The site is not in a tsunami evacuation zone.
- This site is not in a flood management zone.
- The site subsoil class is likely to be classified as class D according to AS/NZS1170.5:2002.
- The site is classified as having a small chance of liquefaction of small, isolated areas during strong earthquake shaking (Yetton & McCahon, 2002).
- The MBIE Residential Foundation Technical Category (TC) system is not applicable for the Ashburton area, however it can be used to compare the equivalent risk and it is expected the liquefaction risk at this site would result in an equivalent TC1 classification.

5 Geotechnical Investigation

The geotechnical investigation commenced on 28 February 2024 and was completed on the same day. The investigation locations have been surveyed post construction in terms of New Zealand Transverse Mercator Projection (NZTM2000). Elevations have been surveyed in Lyttleton Vertical Datum (LVD1937). Locations are presented on a site plan in Appendix B. The site investigations were observed and logged full time by a Beca Engineering Geologist, and the logs have been verified by a Beca Senior Engineering Geologist.

5.1 Standards and Calibration

The investigation was undertaken by Beca (2024 investigations) in general accordance with the New Zealand Ground Investigation Specification (2017), and a list of standards used during the site investigation is shown in Table 5-1.

Table 5-1: Summary of Standards used in this Investigation

Field Procedure	Standard Used
Soil and Rock Logging	In general accordance with New Zealand Geotechnical Society Guidelines (NZGS, 2005).
Scala (Dynamic Cone) Penetrometer Testing	NZS 4402.6.5.2 (1988)

Notes: (1) Standard widely adopted by contractors in NZ with the requirement of a maximum of half the allowable zero drift limit

5.2 Hand Augers

Geotechnical hand augers were undertaken by Kirk Roberts in 2023. The report produced by Kirk Roberts is attached in Appendix A. Hand augers were drilled and logged on site by Kirk Roberts and are summarised in Table 5-2. Locations were estimated from canterbury maps and elevations were estimated from the Topographical survey completed by Graham Surveying.

Table 5-2: Hand Auger Summary

Hand Auger ID	Location	Easting	Northing	Ground Level (m RL)	Total Depth (m bgl)
AR109526-GE-HA-001	6 Orr Street, Front Yard	1500818.8	5138426.1	95.6	0.1
AR109526-GE-HA-002	6 Orr Street, Front Yard	1500823.3	5138422.4	95.8	0.2
AR109526-GE-HA-003	6 Orr Street, Back Yard	1500757.7	5138438.2	95.7	0.3
AR109526-GE-HA-004	6 Orr Street, Back Yard	1500799.8	5138412.3	95.9	0.3
AR109526-GE-HA-005	6 Orr Street, Back Yard	1500806.6	5138402.2	95.6	0.5
AR109526-GE-HA-006	6 Orr Street, Back Yard	1500796.6	5138401.6	95.6	1.2
AR109526-GE-HA-007	6 Orr Street, Back Yard	1500784.0	5138396.2	95.6	1.3
AR109526-GE-HA-008	8 Orr Street, Front Yard	1500806.5	5138440.1	96.0	0.3
AR109526-GE-HA-009	8 Orr Street, Front Yard	1500798.2	5138433.8	96.3	0.3
AR109526-GE-HA-010	10 Orr Street, Front Yard	1500899.4	5138445.8	96.1	0.2
AR109526-GE-HA-011	10 Orr Street, Front Yard	1500794.8	5138451.8	95.9	0.3
AR109526-GE-HA-012	10 Orr Street, Back Yard	1500787.3	5138445.4	96.2	0.3
AR109526-GE-HA-013	10 Orr Street, Back Yard	1500783.2	5138428.4	96.1	0.8
AR109526-GE-HA-014	10 Orr Street, Back Yard	1500766.3	5138429.0	96.1	0.3
AR109526-GE-HA-015	10 Orr Street, Back Yard	1500775.9	5138419.7	96.0	0.6
AR109526-GE-HA-016	10 Orr Street, Back Yard	1500760.1	5138421.4	96.1	0.6

Hand Auger ID	Location	Easting	Northing	Ground Level (m RL)	Total Depth (m bgl)
Notes: RL (Relative Level) (CDD), Survey coordinates are given in NZTM2000, m bgl (metres below ground level)					

5.3 Infiltration Testing

Ashburton Contracting Limited (ACL Ltd) were contracted to excavate test pits using a 3T Airman hydraulic excavator. The pits were approximately 2.1 m by 1.5 m in plan area and ranged from 2.5 m to 2.0 m depth. Material excavated from the test pits were logged and sampled by a Beca Engineering Geologist.

Infiltration testing was carried out in accordance with the Ministry of Business, Innovation and Employment (MBIE) Acceptable Solutions and Verification Methods E1/VM1 (Surface Water), Section 9.0.2 (2017). Testing was conducted at a depth of 2.0m and 2.15m bgl within loosely and densely packed gravel using the falling head test method in ITP01 and ITP02 respectively. The excavated test pits were filled with potable water to within 0.60 m and 0.75 m above the base of the test pits due to pit wall stability in ITP01 and ITP02 respectively. Pre-soak lasted for 40 to 45 mins due to limited water. The drop in water level was then measured at intervals of between 20 seconds and 10 minutes.

Infiltration test locations are summarised in Table 5-3 and the logs are attached in Appendix C. The infiltration rates presented are based on the range of field measurements observed without a design factor applied.

Table 5-3: Infiltration Test Summary

Hand Auger ID	Location	Easting	Northing	Ground Level (m RL)	Total Depth (m bgl)
AR109526-GE-ITP-001	10 Orr, Front yard	1500796.0	5138452.0	95.9	2.5
AR109526-GE-ITP-002	8 Orr, Backyard	1500784.0	5138414.0	96.0	2.0
Notes: RL (Relative Level) (LVD1937) sourced from the site topographical map. Survey coordinates are given in NZTM2000, m bgl (metres below ground level) sourced from Canterbury Maps.					

5.4 Groundwater

No groundwater was encountered in the investigations.

6 Geotechnical Parameters

6.1 Generalised Soil Profile

The soil profile on the site has been developed based on the ground investigations conducted in March 2023. These investigations revealed a soil profile consisting of gravel below the topsoil at the street side of the site and a layer of silt present between the topsoil and the gravel at the rear of site. This profile was confirmed by the test pits conducted in February 2024 and is summarised within Table 6-1.

Table 6-1: Generalised Soil Profile

Soil Unit	**Description**	**Depth to Top of Layer (m bgl)**	**Layer Thickness (m)**	**Average Cone Resistance q_c (MPa)**
1	Dense, SILT, minor organics *[Topsoil]*	0	0.1 – 0.4	*No CPTs conducted*
2[1]	Stiff, SILT *[Holocene Alluvium]*	0.3 – 0.4	0.2 – 1.0	
3	Dense, GRAVEL *[Quaternary Alluvium]*	0.1 – 1.3	*undefined*	

Notes:

m bgl (metres below ground level)

[1] Only present in HA-05, HA-06, HA-07, HA-13, HA-15 and HA-16.

6.2 Design Soil Parameters

The soil strength parameters adopted for the geotechnical assessment and design are set out in Table 6-2. Listed soil units correspond with those described in Table 6-1.

Table 6-2: Soil Strength Parameters

Soil Unit	**Description**	**Unit Weight (kN/m^3)**	**Friction Angle, Φ (degree)**	**Effective Cohesion, c' (kPa)**	**Young's Modulus (MPa)**
2[1]	Stiff, SILT *[Holocene Alluvium]*	18	30	-	25
3	Dense, GRAVEL *[Quaternary Alluvium]*	21	35	-	100

7 Seismic Design Requirements

7.1 Design Life and Importance Level

The proposed structure is being designed as Importance Level of 2 (IL2) structure with a design life of 50 years, in accordance with AS/NZS 1170.0:2002 and as agreed upon with Kāinga Ora.

7.2 Site Subsoil Class

The site subsoil class in accordance with NZS 1170.5:2004 depends on the depth of the underlying soils or rock with each site being classified as either Site Class A, B, C, D or E. Class A refers to sites founded directly on very strong rock material, while Site Class B refers to slightly less competent rock. Class E refers to sites with more than 10 m of soils with SPT N values of less than 6 (i.e., soft soils). These classes are not applicable to the site as shown by the investigative data.

Class C refers to shallow soil sites, with a limit concerning the maximum depth of soils depending on the geology and density.

The geological map of the Christchurch area (Forsyth. Barrell & Jongens, 2008) indicates that alluvial deposits are likely to continue to depth beyond 100 m. A review of Beca data and publicly available information also shows the alluvial deposits of the extending beyond 100 m. As such, a Site Subsoil Class of D (deep soil site) has been adopted for this assessment.

7.3 Seismic Loads

Seismic (earthquake) loads were computed for the site according to the methodology outlined within the MBIE Earthquake Geotechnical Engineering Practice (Module 1, Section 5.1) for the site location (Ashburton). This module states recommended values for earthquake peak ground acceleration (PGA) and effective magnitude (Mw) for the Ashburton area.

Two limit state load cases were analysed: Serviceability Limit State (SLS) and Ultimate Limit State (ULS) design earthquakes:

- For a SLS design earthquake: The structure is "intended to be used without the need for repair".
- For a ULS design earthquake: The structure is required to maintain life safety of the building's occupants and ensure the structural integrity of the building is not lost following the event.

Peak Ground Acceleration (a_{max}) and Earthquake Magnitude (M) values recommended for Geotechnical Assessment were applied for the site as per Table A1 (MBIE, 2021) for Site Classes A, B, C, D and E, for level ground conditions. Recommended PGAs and Mw for liquefaction analyses are summarized in Table 7-1.

Table 7-1: Peak Ground Acceleration and Effective Magnitude for Liquefaction Analysis

Limit State Load	Annual Probability of Exceedance (yr)	Effective Magnitude (Mw)	Peak Ground Acceleration (PGA)
SLS	1/25	6.10	0.06
ULS	1/500	6.10	0.26

8 Liquefaction Assessment

Liquefaction may occur in loosely consolidated and saturated deposits as earthquake-induced cyclic shearing causes pore-water pressures to increase and exceed the static confining pressures, resulting in significant loss of stiffness and strength. Surface effects of liquefaction typically include surface cracking and permanent ground deformations such as vertical settlements and lateral displacements.

Fine grained cohesive soils that have ‘clay-like’ behaviour may be susceptible to cyclic softening under intense earthquake shaking. Cyclic softening induces a loss in shear strength to its residual/remoulded capacity as a result of monotonic and cyclic loading.

An assessment of the likelihood of liquefaction was not completed on this site due to the presence of shallow gravels and a deep water table. Therefore, based on the available site information, for the purposes of design this site is considered to be TC1 in line with the repairing and rebuilding houses affected by the Canterbury earthquakes guidance document (Ministry of Business, Innovation and Employment (MBIE), 2012). While the MBIE guidance is not applicable outside of the Canterbury area. The risk of liquefaction in the Ashburton area is similar to Christchurch, therefore, we recommend that it is used for classification and foundation recommendation requirements at this site.

9 Foundation Design Recommendations

9.1 Bearing Capacity

The ultimate bearing capacity was assessed from dynamic cone penetrometer (DCP) testing. Testing was conducted at the proposed house locations on site, and a depth to competent bearing was determined. Table 9-1 shows the depth to a competent bearing at each test location. Full bearing capacity results can be referred to in Appendix D.

Table 9-1: Depth to variable bearing capacity layers

Test ID	Location	Depth to 300kPa Ultimate Bearing Capacity (Static)		Depth to 200kPa Ultimate Bearing Capacity (Static)	
		m bgl	(m RL)	m bgl	(m RL)
AR109526-GE-HA-001	Front Yard, 6 Orr Street	0.0	95.6	0.0	95.6
AR109526-GE-HA-002	Front Yard, 6 Orr Street	0.0	95.8	0.0	95.8
AR109526-GE-HA-003	Back Yard, 6 Orr Street	0.0	95.7	0.0	95.7
AR109526-GE-HA-004	Back Yard, 6 Orr Street	0.0	95.9	0.0	95.9
AR109526-GE-HA-005	Back Yard, 6 Orr Street	0.0	95.6	0.0	95.6
AR109526-GE-HA-006	Back Yard, 6 Orr Street	0.1	95.5	0.0	95.6
AR109526-GE-HA-007	Back Yard, 6 Orr Street	0.0	95.6	0.0	95.6
AR109526-GE-HA-008	Front Yard, 8 Orr Street	0.0	96.0	0.0	96.0
AR109526-GE-HA-009	Front Yard, 8 Orr Street	0.0	96.3	0.0	96.3
AR109526-GE-HA-010	Front Yard, 10 Orr Street	0.0	96.1	0.0	96.1
AR109526-GE-HA-011	Front Yard, 10 Orr Street	0.0	95.9	0.0	95.9
AR109526-GE-HA-012	Front Yard, 10 Orr Street	0.0	96.2	0.0	96.2
AR109526-GE-HA-013	Back Yard, 10 Orr Street	0.0	96.1	0.0	96.1
AR109526-GE-HA-014	Back Yard, 10 Orr Street	0.0	96.1	0.0	96.1
AR109526-GE-HA-015	Back Yard, 10 Orr Street	0.0	96.0	0.0	96.0
AR109526-GE-HA-016	Back Yard, 10 Orr Street	0.0	96.1	0.0	96.1

We recommend a geotechnical ultimate bearing capacity of 300 kPa is adopted for the foundation design. NZS3604 “good ground” is present across the site from below the topsoil (0.1 to 0.4 m bgl).

9.2 Static Settlement

The site investigation and surrounding area suggests that the site is unlikely to be underlain by organic of soft cohesive soils. Additionally, the shallow gravels encountered between 0.1 m and 1.4 m below ground level will act to distribute the structural loads should any deeper soft layers exist under the site.

It is assumed that any settlement in the ground profile will be immediate in nature due to the ground profile and free-draining nature of the shallow non-cohesive, dense gravel soils. Considering the relatively low surcharge loads of 9 kPa and the density of the ground encountered, it is assumed any settlement resulting from construction is likely to be negligible over the 50-year design life.

9.3 Soil Modulus of Subgrade Reaction

Soil springs were determined on site based on an ultimate bearing capacity of 300 kPa for a maximum deformation of 25 mm based on recommendations in Foundation Analysis and Design, (Bowles, 1997). Due to the potential variation in soil stiffness under a slab we recommend a range of modulus of subgrade reaction is modelled from -50% to +200% of the estimated value. The estimated modulus of subgrade reaction is 12 MPa and the recommended range for design is 6 to 24 MPa.

9.4 Foundation Solution

MBIE Residential Foundation Technical Category (TC) maps exist for the Ashburton area. However, foundation recommendations are provided in accordance with the 'repairing and rebuilding houses affected by the Canterbury earthquakes' guidance (2012) to account for the site-specific risks which are similar to the conditions encountered in Christchurch.

Kāinga Ora's preferred foundation system for this site is a TC1 waffle slab over a gravel pad in accordance with the "Repairing and rebuilding houses affected by the Canterbury earthquakes" guidance, 2012.

A TC1 waffle slab suitable for 300 kPa geotechnical ultimate bearing capacity is recommended for this site. The waffle slab will sit on a minimum of 200 mm of compacted gravel hardfill (for a standard TC1 shallow foundation solution).

Topsoil removal considers the elevation of base of topsoil encountered at site, varying across site from 0.1 to 0.4 m bgl. Foundation levels are summarised within Table 9-3 and consider the advised preliminary finished floor level (FFL) requirement of 95.95 – 96.39 m RL (provided by the HDS civil engineering team), elevation of the base of topsoil and the thickness of the TC1 stiffened waffle (400 mm).

Note these levels are preliminary only and if amended during detailed design, final construction drawings will require updating.

Table 9-2: Summary of preliminary foundation levels

House Typology / Position	Existing ground Level (m RL)	ADC FFL (m RL*) (top of stiffened waffle slab)	TC2 waffle slab thickness (mm)	Base of TC2 waffle slab (m RL*)	Base of gravel pad / excavation level (m RL*)	Thickness of gravel pad (mm)
Houses 1 & 2	95.9	96.15	400	95.75	95.55	200
Houses 3 & 4	96.1	96.30		95.90	95.70	200
Houses 5 & 6	96.0	96.39		95.99	95.59	400
House 7	95.6	96.01		95.61	95.31	300
House 8 & 9	95.6	95.95		95.55	95.30	250

Notes:

*Based on preliminary slab finished floor level (m RL, LVD1937) as stated. If this changes during detailed design, the finished ground level and base of raft will need to be updated on final construction drawings.

The slab is to be constructed on a layer of non-woven geotextile at the bottom of the excavation and should be compacted in maximum 200 mm thick layers to 95% of maximum dry density. The waffle slab is to be constructed on top of the gravel raft. Please refer to the Beca Kāinga Ora Housing Development System (HDS) Specification for construction information, material details and testing requirements.

This design will limit the damage on the foundations from seismic induced ground movements and would be expected to be repairable if any damage does occur. However, the house may be out of level after a major seismic event.

The PS1 producer statement for the geotechnical foundation design is included in Appendix H.

An alternative solution for this site would be an NZS 3604 timber piled foundation supporting a raised floor level. Timber piles are to be founded at a minimum depth of 0.3 m bgl where NZS 3604 ‘good ground’ criteria is encountered as per section 9.1.

If this option is to be pursued, foundation (auger) diameter, concrete encasement, and timber pile dimensions are to be determined as per the requirements of NZS 3604 section 6.4.5, based on the span of bearers and joists as per Table 6.1. The piles shall be augered and concreted in place in accordance with the requirements of NZS 3604.

9.5 Construction Monitoring Requirements

In order to provide a Producer Statement Construction Review (PS4), Beca Ltd. is required to carry out specific construction monitoring, tests and inspections at certain points throughout the construction period. The specified construction monitoring service for the recommended design solution is CM2, appropriate for smaller projects where works are being undertaken by an experienced and competent contractor.

The engineering inspections required for this foundation solution are as follows. A minimum of one inspection is required for each item, however more inspections may be required depending on the contractor’s programme of works and staging of foundation excavations.

Table 9-3: Schedule of Inspections.

Inspection Item	Details
1	Subgrade inspection: *Cut base inspection* and *Dynamic Cone Penetration (DCP) tests (cohesionless soils) or shear vane tests (cohesive soils with a minimum undrained shear strength of 20 kPa) at the base of cut.*
2	Final raft inspection: *inspection of finished gravel raft, prior to sand being placed. Maximum Dry Density (MDD), Nuclear Density Meter (NDM), fill grading and optimum moisture content results to be provided to the engineer prior to this inspection.*
3	Hole inspection for retaining walls: *inspection of the excavated holes for the retaining structure to ensure they are clear of debris and groundwater.*
4	Retaining wall drainage: *inspection of the drainage installation before fill is placed behind the wall.*
5	Soak pit subgrade inspection: *Inspection of the base of the soakage pits to ensure the base is in the target material, clean and free of debris*
6	Soakage pit testing: *Each soak pit will require testing to ensure the design rates are acceptable. This will be completed by a hydrogeologist in conjunction with the contractor.*

10 Infiltration Risk and Recommendations

Infiltration testing was conducted, with results reviewed and assessed by Beca Hydrogeologists. Raw unfactored infiltration rates, design recommended rates and construction recommendations are communicated within an Infiltration Memorandum which can be referred to in Appendix E.

The following recommendations were made based on the testing:

- A factored soakage rate of:
  - 203 mm per hour is recommended for the design of houses 1 and 2,
  - 109 mm per hour is recommended for the design of houses 3 – 9,
  - 181 mm per hour is recommended for the design of the JOAL.
- Soak pits must terminate at least 0.5 m within the loose sandy gravel,
- A minimum soak pit depth of 2.5 m bgl,
- Soakage testing to confirm the soakage rates should be conducted in each soak pit during construction.

11 Retaining Wall Design

A standardised timber retaining wall design was completed by Beca to supply a conservative fit for purpose solution applicable to residential sites developed in the Housing Delivery System (HDS) for retaining structures up to 2.0 m high. The design basis was progressed iteratively to deliver an optimum solution considering conservative assumed geotechnical parameters and load cases. Target factor of safety (FoS) and allowable deformations are specified according to currently accepted New Zealand Codes and Standards.

A review of the site ground conditions, and the proposed development confirms the standardised design is applicable for this site. A memorandum detailing the design can be referred to in Appendix F. The proposed design for retained heights of 0.5, 1.0,1.5 and 2.0 m is summarised within Table 11-1.

Table 11-1: Standardised Timber SED Retaining Wall Design

TIMBER SED RETAINING WALL DESIGN				
Max retained height	2000 mm	1500 mm	1000 mm	500 mm
Pole embedment	5000 mm	3500 mm	2500 mm	1500 mm
Normal pole length (SED)	7000 mm	5000 mm	3500 mm	2000 mm
Pole size (diameter, SED)	425 mm	275 mm	225 mm	150 mm
Socket size (diameter)	550 mm	400 mm	350 mm	300 mm
Pole spacing	1100 mm	1200 mm	1200 mm	1200 mm
Lagging dimensions	150 mm x 75 mm (THICK)		150 mm x 50 mm (THICK)	

Table 11-2: Standardised Timber SED Retaining Wall Boundary Setbacks

Timber Retaining Wall Setbacks					
Max Retained Height	2 m	1.5 m	1.2 m	1.0 m	0.5 m
Setback requirements for cut boundary retaining walls	2.9 m	2.225 m	0.725 m	0.625 m	0.525 m

Notes:

Where the wall height is less than 1.2m, the batter may be cut vertical. In this situation some slope losses may occur if the cut is left open for extended periods of time.

12 Pavement Design Recommendations

The California Bearing Ratio (CBR) for pavement design was assessed from the DCP testing conducted across the site, with an average penetration depth (mm) per blow (e) was derived from the results of each test. Values within the upper 300 mm soil deposits were disregarded as they are considered unreliable due to insufficient lateral resistance on the rod tip and will likely be removed during initial site stripping. CBR percentages are calculated in accordance with Austroads – Guide to Pavement Technology Part 2 and are listed in Table 10-1. The insitu CBR is determined from a weighted average of the CBR below the cut level (70% weighting in top 300mm and 30% weighting for 900mm below this level). A recommended CBR of up to 8 % is proposed for pavement design. The CBR results are included in Appendix G.

Table 12-1: CBR Summary

Test ID	DCP Depth Considered (m bgl)	Insitu CBR (%)[1]
AR109526-GE-HA-001	0.3 – 1.5	50+
AR109526-GE-HA-002	0.3 – 1.5	50+
AR109526-GE-HA-003	0.3 – 1.5	8
AR109526-GE-HA-004	0.3 – 1.5	50
AR109526-GE-HA-005	0.3 – 1.5	29
AR109526-GE-HA-006	0.3 – 1.5	14
AR109526-GE-HA-007	0.3 – 1.5	16
AR109526-GE-HA-008	0.3 – 1.5	50+
AR109526-GE-HA-009	0.3 – 1.5	30
AR109526-GE-HA-010	0.3 – 1.5	50+
AR109526-GE-HA-011	0.3 – 1.5	50+
AR109526-GE-HA-012	0.3 – 1.5	50+
AR109526-GE-HA-013	0.3 – 1.5	19
AR109526-GE-HA-014	0.3 – 1.5	45
AR109526-GE-HA-015	0.3 – 1.5	46
AR109526-GE-HA-016	0.3 – 1.5	35

[1] Austroads – Guide to Pavement Technology Part 2: Pavement Structural Design, Section 5.2, Figure 5.3.

These CBR results are based on insitu testing representing the soil conditions and moisture content at the time of testing and may not reflect the worst case (e.g., saturated conditions).

13 Natural Hazard Assessment

The risk of natural hazards at the site has been assessed in accordance with section 106 of the Resource Management Act (RMA) and appropriate ADC and MBIE documents to support the subdivision consent application for the property. The statement of professional opinion for the development of the land is included in Appendix I. A summary of the section 106 hazards at the site is included in Table 13-1 below.

Table 13-1: RMA s106 Hazard Assessment Summary Table

Natural Hazard	Current Risk (as per s106(1)(a))	Effects from Development (as per s106(1)(a))
Flood Inundation	**Low Risk** the ADC District plan indicated it property not in a flood zone.	**Low Risk** Stormwater management is being designed in accordance with ADC standards.
Slips	**Low Risk** the property is not located near a slope or channel.	
Subsidence and Settlement (Static)	**Low Risk H**and augers have indicated there are no peat or soft cohesive soils within the ground profile.	**Low Risk** A review of the site data concluded there was negligible risk of consolidation settlement over the design life.
Subsidence and Settlement (Seismic)	**Low Risk** the land is currently unclassified. The site investigations have confirmed the site is considered TC1.	
Lateral Spreading	**Low Risk** the site is not situated near any free faces or watercourses that may cause lateral spreading in an earthquake.	
Erosion	**Low Risk** no surface water flow source of erosion has been identified near the site.	
Falling Debris	**Low Risk** no source of falling debris has been identified near the site.	

14 Geotechnical Risks

The investigation is based upon isolated investigation data over the site and there is a residual risk with geotechnical investigations and design that conditions may differ from those assumed or deteriorate on site during construction. A summary of the risks and proposed mitigation measures is included in Table 14-1 below.

Table 14-1: Development risks and recommended mitigation measures

Risk	Likelihood	Effects on development	Proposed mitigation measures
Isolated soft zones in subgrade cut (<300kPa ultimate bearing capacity)	Possible	Additional over-excavation required, minor delays	Test subgrade cut surface during construction and recommend additional excavation and replacement with AP65 in affected areas
Heavy rain during subgrade cut or backfilling works	Likely	Foundation softens and requires additional over excavation. Fill becomes contaminated with fines and cannot be compacted to target density, fill removed and replaced, significant delays	Aim to complete foundation excavation works only during fine weather. Install geotextile between cut subgrade and fill to reduce risk of fines migration into fill during rain events. Backfill the excavation promptly. Adjust the compaction methodology to match the subgrade and aggregate moisture content.
Long term static foundation settlement (organic/soft soils)	Rare	Tilting and settlement of house. Separation between house and adjacent access structures. Damage to services connections at edge of house.	Assess long term settlement risk for property. Where settlement is anticipated, add additional control measures such as flexible services connections or additional reinforcement in slab foundation. The foundations are able to be relevelled if required.
Encountering groundwater during foundation excavations	Unlikely	Dewatering may be required for foundation excavations.	Plan for dewatering if deep foundation excavations below groundwater level are required.

15 Applicability Statement

This report has been prepared by Beca Ltd (Beca) on the specific instructions of Kāinga Ora (Client). It is solely for our Client's use for the purpose for which it is intended in accordance with the agreed scope of work. Any use or reliance by any person contrary to the above, to which Beca has not given its prior written consent, is at that person's own risk.

Should you be in any doubt as to the applicability of this report and/or its recommendations for the proposed development as described herein, and/or encounter materials on site that differ from those described herein, it is essential that you discuss these issues with the authors before proceeding with any work based on this document.

In preparing this report Beca has relied on key information including the following:

- *Site survey information supplied by Kāinga Ora Housing Development System (HDS) Survey in February 2024.*
- *Site investigation data (boreholes, CPTs, etc) and CES data from the New Zealand Geotechnical Database, accessed in February 2024.*
- *Preliminary foundation floor levels supplied by Kāinga Ora Housing Development System (HDS) Civil Engineer in March 2024.*

Unless specifically stated otherwise in this report, Beca has relied on the accuracy, completeness, currency and sufficiency of all information provided to it by, or on behalf of, the Client, including the information listed above, and has not sought independently to verify the information provided.

This report should be read in full, having regard to all stated assumptions, limitations and disclaimers. No part of this report shall be taken out of context and, to the maximum extent permitted by law, no responsibility is accepted by Beca for the use of any part of this report in any context, or for any purpose, other than that stated herein.

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A

Appendix A – Kirk Roberts Geotechnical Report

KIRK ROBERTS
CONSULTING

5 March 2023

Project Ref: 2310011
Rev.: 01

Kainga Ora Homes and Communities
Emma Jenkins
Emma.jenkins@kaingaora.govt.nz

Geotechnical Report for a New Residential Dwelling – 6-10 Orr Street, Ashburton

Kirk Roberts Consulting has been engaged by the client to review available geotechnical investigation data, complete site investigations, and prepare a geotechnical report to advise foundation recommendations for a new residential dwelling at 6-10 Orr Street, Ashburton, with a total parcel area of 2,697 m^2 (LOT 3, LOT 4 and LOT 5 DP 18886).

This report is suitable to accompany a Building Consent application following a review of the finalised architectural/structural drawings by a Kirk Roberts geotechnical engineer.

1. Site Description

The subject property is physically located at 6-10 Orr Street, Ashburton, approximately 200 m south of Wakanui Creek and approximately 2.4 km northeast of Ashburton River. The site is flat and is bordered by Orr Street to the northeast and residential dwellings to all other boundaries. Refer to Attachment 2 for an aerial photo of the site.

2. Site Proposed Development

Kirk Roberts has been provided with a yield plan that indicates the proposed development involves the construction of nine residential units of the regular layout. Refer to Attachment 1.

We understand the proposed development is not final and could be changed. The proposed development yield plan does not indicate whether the units will comprise single to two-storey, lightweight or heavyweight construction. However, for the purpose of this report, the geotechnical recommendations made herein assume that the proposed development will fall within the scope of MBIE guidelines[1] and NZS 3604:2011.

Kirk Roberts Consulting shall review the finalised architectural and structural drawings for the development to ensure their compliance with the recommendations provided herein.

3. Sub-Surface Conditions

3.1. *Published Geology*

We have reviewed the data published by GNS Science on the New Zealand Geology Web Map[2] to determine the published geology. This is summarised in Table 1 below.

Table 1: Published geology

Geological group	Simple name	Geological age	Description
Holocene sediments	Holocene river deposits	0.01 – 0.012 million years	Modern river floodplain/low-level degradation tce. Unweathered, variably sorted gravel/sand/silt/clay. Surfaces <2 degree slope.

A review of the GNS Active Faults Database[3] indicates that there are no known active faults that are close to the site. Therefore, the site is located outside the minimum 20 m fault avoidance zone recommended by the Ministry for the Environment[4].

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[1] Ministry of Business Innovation and Employment (MBIE) Guidance: Repairing and rebuilding houses affected by the Canterbury earthquakes, Version 3, December 2012

[2] GNS Science – New Zealand Geology Web Map, date retrieved in February 2023 from http://data.gns.cri.nz/geology/

[3] Geological and Nuclear Sciences (2004). Active Faults Database, date retrieved in February 2023 from https://data.gns.cri.nz/af/

[4] Planning for Development of Land on or Close to Active Faults: A Guideline to Assist Resource Management Planners in New Zealand (Published July 2003).

3.2. Geotechnical Database Review

To supplement our geotechnical investigation, a review of the New Zealand Geotechnical Database[5] (NZGD) has been undertaken.

Review of the New Zealand Geotechnical Database[5] (NZGD) did not reveal any deep geotechnical investigations within 100 m of the site.

Review of the Canterbury Maps[6] uncovered several wells located in the wider area of the site within same geological formation. The wells are:

- L37/1139, located approximately 145 m east of the site, encountered sandy gravels and claybound gravels from 0.3 m below ground level (bgl) to at least 120 m bgl where the well was terminated. Groundwater was encountered on 14/10/2005 at 13.50 m bgl.
- L37/0741, located approximately 580 m northeast of the site, encountered sandy claybound gravels from 0.2 m bgl to at least 12.0 m bgl where the well was terminated. Groundwater was encountered on 23/10/1996 at 2.90 m bgl.

3.3. Geotechnical Investigation – Kirk Roberts October 2023

Kirk Roberts carried out a shallow ground investigation on the 15th of February 2023, completing sixteen hand-auger test holes (HA-1 to HA-16) with associated Scala penetrometer tests (SP-1 to SP-16).

Tests encountered topsoil to 0.3 – 0.4 m below ground level (bgl), over silts with a trace of gravels to 0.3 –1.3 m bgl where the holes met practical refusal on inferred gravel.

The Scala penetrometer tests (SP-1 to SP-16) generally returned minimum blow counts of 3 blows per 100 mm of penetration from 0.3 – 0.4 m bgl (below topsoil), indicating an index of geotechnical ultimate bearing capacity[7] of (GUBC) 300 kPa is available at this depth. All the Scala penetrometer tests met practical refusal on inferred gravel between 0.3 and 1.9 m bgl. Refer to the attached shallow soil test locations and test results (Attachment 2).

3.4. Groundwater

Groundwater was not encountered during the site investigation.

Canterbury map wells identified well (L37/1139) located approximately 145 m east of the site, the well record groundwater depth at 13.50 m bgl.

The Desktop Study Report[8] prepared by Beca stated "*investigations completed by WSP Opus (2018) compiled piezometric data across Ashburton and generated depth to groundwater maps for the Ashburton District Council (ADC) Urban Stormwater Management Area. These maps suggest that groundwater is likely to be relatively deeper and encountered between 5 m and 10 m bgl*."

4. <u>Flood Risk Assessment</u>

The Ashburton District Council Map Viewer[9] indicates that the site is not at risk of flooding. However, we recommend confirming this with the Ashburton District Council (ADC) before the Building Consent application. The Desktop Study Report[8] prepared by Beca stated indicates "*Flood maps published by ADC (2010) show the site is unlikely to flood up to a 1 in 200-year event (0.5% AEP) assuming the Ashburton River stop banks do not fail*".

Further assessment of site-specific flood issues is beyond the scope of this report.

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[5] New Zealand Geotechnical Database (NZGD), data retrieved in February 2023 from https://www.nzgd.org.nz/Default.aspx

[6] Environment Canterbury – Canterbury Maps Viewer, data retrieved in February 2023 from https://mapviewer.canterburymaps.govt.nz/

[7] The inferred bearing capacity is based on Scala penetrometer tests and have been estimated using the procedure presented by M.J. Stockwell in the paper '*Determination of allowable bearing pressure under small structures (June 1977)*'.

[8] Desktop Study Report, 6-10 Orr Street, Ashburton - Prepared by Beca Limited for Kāinga Ora – Homes and Communities, dated 17 June 2022

[9] Ashburton District Council – District Plan Publication, data retrieved in February 2023 from https://maps.adc.govt.nz/Viewer/?map=27f6a894aaf8471f986da5e21203f09d

5. Seismic Assessment

5.1. *Importance Level*

The proposed future residential units are considered as an Importance Level 2 (IL2) as defined by the NZS 1170.0:2002

5.1. *Site Seismic Class*

In accordance with NZS 1170.5:2004, site classification "*Class D*" applies to this particular site, defining it as a '*deep or soft soil site*'. Kirk Roberts Consulting have utilised a desktop investigation and site-specific shallow geotechnical investigations to determine the soil class for this site. No specific deep investigations were undertaken to confirm the subsoil class. If this is needed to be confirmed in a high certainty, site-specific deep investigations and assessment will need to be undertaken.

5.2. *Liquefaction Risk*

A review of the "*Ashburton Liquefaction Susceptibility*" map overlay presented on Canterbury Maps[6] indicates that the site is within a zone of "*very low liquefaction potential*" which is described as "*Areas of alluvium older than Holocene age. Very Small risk of liquefaction of local, isolated areas.*".

We note that the "*Ashburton Liquefaction Susceptibility*" overlay is based on a study undertaken by Geotech Consulting Ltd[10] to define areas of liquefaction susceptibility across the Ashburton District. The study mapped the Ashburton District's three zones describing different levels of liquefaction potential (Zone 1 being a zone of low potential and Zone 3 being a zone of nil to extremely low potential).

Given the above, the likely depth to groundwater, and considering that the site is underlain by very dense shallow gravels which likely extend to >20 m bgl, it is our professional opinion that the "*very low liquefaction potential*" classification is appropriate and that the future performance of the site will likely be consistent with a Technical Category 1 (TC1)[11] classification in accordance with the MBIE Guidelines[1].

It is our professional opinion that the existing liquefaction hazard will not be worsened by the proposed works. On this basis, a deep geotechnical investigation (i.e., Cone Penetrometer or borehole Tests), or subsequent site-specific liquefaction analysis is not required.

5.3. *Lateral Spread*

Considering the "*very low liquefaction potential*", the distance to the nearest watercourse, and the absence of ground-cracking on-site, it is our professional opinion that the risk of liquefaction-induced lateral spread in terms of global lateral movement and lateral stretch is negligible to low.

6. Foundation Recommendations

6.1. *Discussion*

Given the results of the site-specific investigations and the assessed TC1-equivalent categorisation for the site, foundation options specified for TC1 sites as outlined in Part A of the MBIE guidelines[1] are generally considered appropriate, if the residential development is to comply with NZS 3604:2011.

The following foundation options are considered suitable for the site:

- Tied concrete foundation slab on grade to NZS 3604:2011;
- TC1 waffle slab foundation;
- Timber floor supported by braced timber piles, with or without a perimeter foundation, to NZS 3604:2011

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[10] Yetton & McCahon (2002): *Ashburton District Lifelines Project, Earthquake Hazard Assessment*, Environment Canterbury, Report No. U02/55

[11] Technical Category 1 (TC1, grey) means that future land damage from liquefaction is unlikely, and ground settlements are expected to be within normally accepted tolerances. Standard foundations (NZS 3604) are acceptable subject to shallow geotechnical investigation.

6.2. *In-Situ Soil Bearing Capacity*

The site-specific geotechnical investigations indicate that a geotechnical ultimate bearing capacity (GUBC) of 300 kPa is generally available from 0.3 – 0.4 m depth.

A capacity reduction factor as specified in Table 1, paragraph 3.5.1 of the NZBC B1/VM4 shall be used.

The topsoil and any unsuitable/uncontrolled fill are not suitable to support shallow foundations due to the risk of consolidation settlement.

At the time of construction, and in accordance with the Council requirements, all foundation excavations should be inspected by a suitably qualified geotechnical engineer to confirm that the soil profile is consistent with the findings of this geotechnical report. At this time, additional Scala penetrometer testing may be necessary to confirm the available geotechnical ultimate bearing capacity. Additional excavations may be required in localised soft spots or to remove any uncontrolled fill or organic material.

6.3. *Concrete Slab Foundations*

The concrete slab foundations are designed for '*Good Ground*' conditions as defined in NZS 3604:2011 and are therefore subject to a minimum bearing capacity requirement of 300 kPa (ultimate). As this bearing capacity is available immediately beneath the topsoil, a base course of approved granular fill such as CAP20 (75 mm thick) or CAP40 (100 mm thick) should be placed, compacted, and built up to the underside of the floor slab. Where the excavation exceeds the above thicknesses, additional approved granular fill such as AP65 shall be placed, compacted, and built up to the underside of the basecourse.

Fill material shall be compacted to 95% of the materials maximum dry density (MDD) and should comply with the requirements of ZNS4431:2022.

6.3.1. *Tied Concrete Foundation Slab on Grade to NZS 3604:2011*

The NZS 3604 foundation slab shall be designed as a 100 mm (minimum) thick, reinforced concrete slab with perimeter and internal (beneath load bearing walls) thickenings. The layout of the slab system will require all internal load-bearing wall foundations to be tied-in to the perimeter foundations and thickened where necessary, to minimise tension crack failure of the concrete slab during strong ground shaking.

6.3.2. *TC1 Waffle Slab Foundation*

The TC1 waffle slab is a foundation system that provides a practical alternative to the option described in Section 6.3.1 and delivers a similar level of performance. The TC1 waffle slab is designed for 'good ground' conditions as defined in NZS 3604:2011.

6.3.3. *Timber Floor Supported by Braced Timber Piles to NZS 3604:2011*

This option includes a timber floor supported by shallow timber piles, either with or without a perimeter concrete foundation, in accordance with Section 6 of NZS 3604:2011. The piles require embedment to a minimum depth of 0.5 m bgl and should be designed for a geotechnical ultimate bearing capacity of 300 kPa.

This foundation system would not mitigate the risk of settlement to the property; however, it would be relatively simple to re-level the timber floor through the packing of the timber piles. Lateral movement of the timber pole foundation and suspended floor system would be minimised by the provision of the diagonal bracing between the poles in accordance with Figures 6.6, and 6.7 from NZS 3604:2011.

7. Further Development Considerations

7.1. *Static Settlement*

The encountered soil profile within the shallow depth is likely to be susceptible to some amount of consolidation settlement. For the proposed development, and providing that the recommendations presented in this report are followed, we do not consider static settlement of the soils at the locations tested to be critical. However, in accordance with New Zealand Building Code, foundations should be designed to limit the differential settlement to 25 mm deflection over a 6 m span.

The topsoil and any localised soft soils or organics are not considered suitable to support shallow foundations due to the risk of consolidation settlement. These unsuitable materials should be excavated and removed from the site.

Filling above existing ground level could result in an increase to, or acceleration of, static settlement. As such, no earth filling above existing site levels should not be undertaken without further geotechnical input from a suitably qualified geotechnical engineer or engineering geologist.

7.2. Review of Developed Design

Kirk Roberts shall be provided with the developed design drawings as and when they are completed, in order to confirm that the recommendations presented herein are still applicable.

At the time of construction, an appropriate level of construction monitoring shall be required to confirm that the encountered soil profile, and founding layers, remain consistent with those outlined in this document. Significant variation of the exposed soil (organic material, soft spots etc.), identified during construction should be brought to the attention of the geotechnical engineer and the ground remediated as necessary.

8. Limitations

Whilst every care was taken during our desktop review, site investigation and interpretation of subsurface conditions, there may well be subsoil strata and features that were not detected.

It must be appreciated that the actual characteristics of the subsurface materials may vary significantly between adjacent test locations and sample intervals other than where observations, explorations and investigations have been made.

It should be noted that because of the inherent uncertainties in subsurface evaluations, changed or unanticipated subsurface conditions may occur that could affect total project cost and/or execution. Kirk Roberts Consulting does not accept responsibility for the consequences of significant variances in the conditions and the requirements for the execution of the works.

This report has been prepared to support the Building Consent application for the proposed dwelling. It is not intended to address the subdivision of the original site. This report assesses the land only, not the condition of any structures at the site.

Only our client is entitled to rely upon this report, and then only for the purpose stated above. Kirk Roberts Consulting Engineers Ltd accepts no liability to anyone other than these parties in any way in relation to this report and the content of it and any direct or indirect effect this report may have.

Should anyone wish to discuss the content of this report with Kirk Roberts Consulting Engineers Ltd, they are welcome to contact us on 03 379 8600 and www.kirkroberts.co.nz.

Yours faithfully,

Prepared by:	Reviewed by:	Reviewed and approved to release by:
Akbar Ali *BeTech Civil* *Geotechnical Technician*		**Firas Salman** *PhD, CMEngNZ, CPEng* *Senior Geotechnical Engineer*

Attachments:

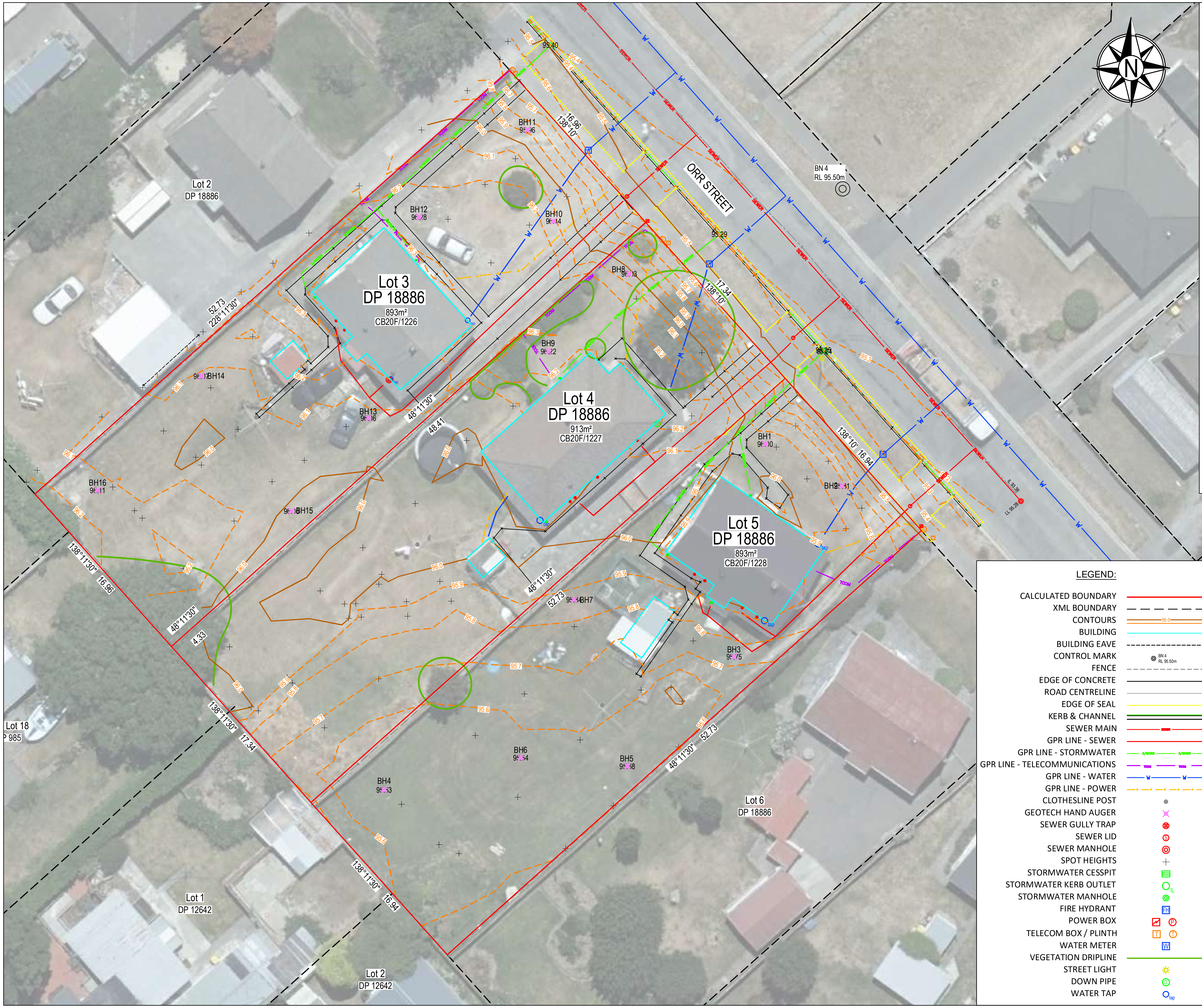
Yield plan	(1 page)
Site Survey	(1 page)
Site-specific investigation plans and bore-logs	(18 pages)

Attachment 1 : Yield Plan

Yield Plan

Beds	Typology	Units			Car Parks			Special Provisions
		Opt 1	Opt 2	Opt 3	Opt 1	Opt 2	Opt 3	
2	House	3			3			Full Universal Design subject to FFL
2	Duplex	4			4			
3	House	2			4			Full Universal Design subject to FFL
Total		**9**			**11**			

Attachment 2 : Site Survey



LEGEND:

CALCULATED BOUNDARY	
XML BOUNDARY	
CONTOURS	95.0
BUILDING	
BUILDING EAVE	
CONTROL MARK	◎ BN 4 RL 95.50m
FENCE	
EDGE OF CONCRETE	
ROAD CENTRELINE	
EDGE OF SEAL	
KERB & CHANNEL	
SEWER MAIN	
GPR LINE - SEWER	
GPR LINE - STORMWATER	
GPR LINE - TELECOMMUNICATIONS	
GPR LINE - WATER	W
GPR LINE - POWER	
CLOTHESLINE POST	●
GEOTECH HAND AUGER	✕
SEWER GULLY TRAP	⊕
SEWER LID	⊘
SEWER MANHOLE	◎
SPOT HEIGHTS	+
STORMWATER CESSPIT	▤
STORMWATER KERB OUTLET	○
STORMWATER MANHOLE	◎
FIRE HYDRANT	⊞
POWER BOX	☒ P
TELECOM BOX / PLINTH	T
WATER METER	W
VEGETATION DRIPLINE	
STREET LIGHT	☼
DOWN PIPE	D
WATER TAP	○

NOTES:
1. Levels are in terms of Lyttelton Vertical Datum 1937
Origin of Levels: U 77 (AC97) RL: 100.89m
2. Coordinates are in terms of NZGD2000, Gawler Circuit 2000
Origin of Coordinates: IT I DP 59139 (CM9T)
(783062.51 mN, 432632.41 mE)
3. Contours are at 0.1m intervals. Contours shown on this plan have been electronically computed from spot height determinations and may not represent the true ground levels. Any critical heights should be checked on site prior to design and construction commencing.
4. This plan has been carried out to topographical standards and all critical dimensions and levels should be verified.
5. *Boundary dimensions and locations have been adopted from DP 18886 . Boundaries shown on the plan face have an accuracy of +/- 40mm. A full legal survey is required to calculate boundary* dimensions, area and location within acceptable tolerances.
6. All easements, covenants and other legal instruments associated with this site may not be shown on this plan. An investigation of the most current legal records should be undertaken prior to design and construction commencing.
7. The Aerial photograph was obtained from LINZ dated 2020. Ground features surveyed may differ than those shown on aerial.
8. Legal Description:
6 Orr Street - Lot 5 DP 18886
Comprised in: CB20F/1228
Total Area: 893m²

8 Orr Street - Lot 4 DP 18886
Comprised in: CB20F/1227
Total Area: 913m²

10 Orr Street - Lot 3 DP 18886
Comprised in: CB20F/1226
Total Area: 893m²

9. All titles are Subject to -
- Part IV A Conservation Act 1987
- Section 11 Crown Minerals Act 1991

Rev.	Date	Description	Drwn	Chk	Appd
A	21/02/2023	DRAWING ISSUED	HB	CE	CE

GRAHAM
SURVEYING

Drawn HB Date 17/02/2023
Checked CE Date 21/02/2023
Surveyed HB Date 15/02/2023
Approved CE Date 21/02/2023

Project
6-10 ORR STREET
NETHERBY
ASHBURTON

Title
TOPOGRAPHICAL SURVEY

Status
AS SURVEYED

Scale 1:300 @ A3
Size A3
Horizontal Datum GAWLER 2000
Vertical Datum LYTTELTON V.D. 1937

Client
Kāinga Ora
Homes and Communities

Drawing Number GSL23040-SU-1000
Revision A

Attachment 3 : Site-Specific Investigation Plan and Bore-logs

Legend

⊕ Geotech Location Plan

☐ Site Boundary

0 5 m 10 m

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KIRK ROBERTS CONSULTING

Produced by **Datanest.earth**

Title: Geotech Location Plan

Client: Kāinga Ora - Homes and Communities

Project: 6-10 Orr St, Netherby, Ashburton

Drawn: AA

Figure No.: 01

Size: A4

Date: 16-02-2023

Checked: FS

Proj No.: 2310011

Scale: 1:500

Version: Final

Legend

- ⊕ Geotech Location Plan
- 🖼 Proposed Plan
- ☐ Site Boundary

LINZ CC BY 4.0 © Imagery Basemap contributors

KIRK ROBERTS CONSULTING

Produced by **Datanest.earth**

Title: Geotech Proposed Plan

Client: Kāinga Ora - Homes and Communities

Project: 6-10 Orr St, Netherby, Ashburton

Drawn: AA

Figure No.: 02

Size: A4

Date: 16-02-2023

Checked: FS

Proj No.: 2310011

Scale: 1:500

Version: Final

SCALA PENETROMETER & HAND AUGER RESULTS

KIRK ROBERTS CONSULTING

Client: Kainga Ora
Site Address: 6 -10 Orr Street
Town/City: Ashburton

Scala Penetrometer: SP-01
Hand Auger No. HA-01
Job No.: 2310011

Logged By: RSD
Logged Date: 16/02/23

Checked By: FS
Checked Date: 17/02/23

Date: 15/02/23
Weather: Overcast
Operator: RSD

Geological Formation	Depth (m)	Sample Description	Graphic	USCS	D_r	Water Table	Undrained Shear Strength Su (kPa)	Blows per 100 mm of Penetration
Topsoil	0.0–0.1	Silty TOPSOIL (OL); dark brown. Very stiff to hard, dry, non-plastic; [TOPSOIL].	TS	OL		Groundwater Not Encountered		3
	0.1–0.2	EOH: 0.10m 0.1m: [INFERRED GRAVEL]						8
	0.2–0.3							21 >>

Remarks:

Scala Penetrometer and Test Bore log tests give an indication of the ground condition at the location of the tests only. While they are representative of typical conditions across the site, they do not identify variations in the ground away from the test locations.

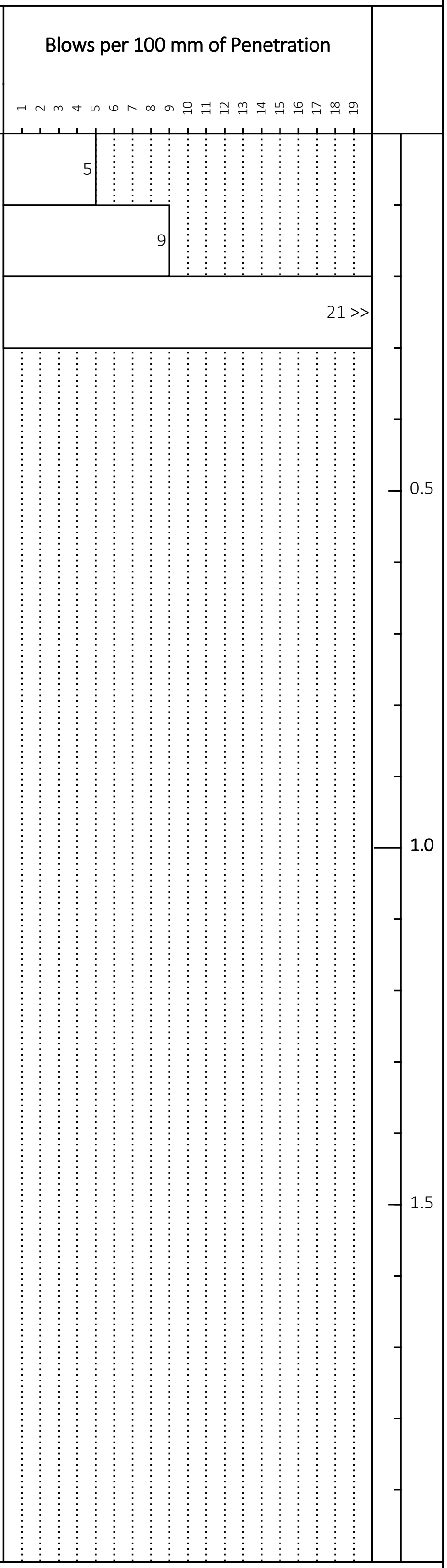
Generated with CORE-GS

SCALA PENETROMETER & HAND AUGER RESULTS

Client:	Kainga Ora	**Scala Penetrometer:**	SP-02
Site Address:	6 -10 Orr Street	**Hand Auger No.**	HA-02
Town/City:	Ashburton	**Job No.:**	2310011

Logged By:	RSD	**Checked By:**	FS	**Date:**	15/02/23
Logged Date:	16/02/23	**Checked Date:**	17/02/23	**Weather:**	Overcast
				Operator:	RSD

Geological Formation	Depth (m)	Sample Description	Graphic	USCS	D_r	Water Table	Undrained Shear Strength Su (kPa)
Topsoil	0.0–0.2	Silty TOPSOIL (OL); dark brown. Very stiff to hard, dry, non-plastic; [TOPSOIL]. EOH: 0.20m	TS	OL		Groundwater Not Encountered	
	0.2	0.2m: [INFERRED GRAVEL]					
	0.5						
	1.0						
	1.5						



Remarks:

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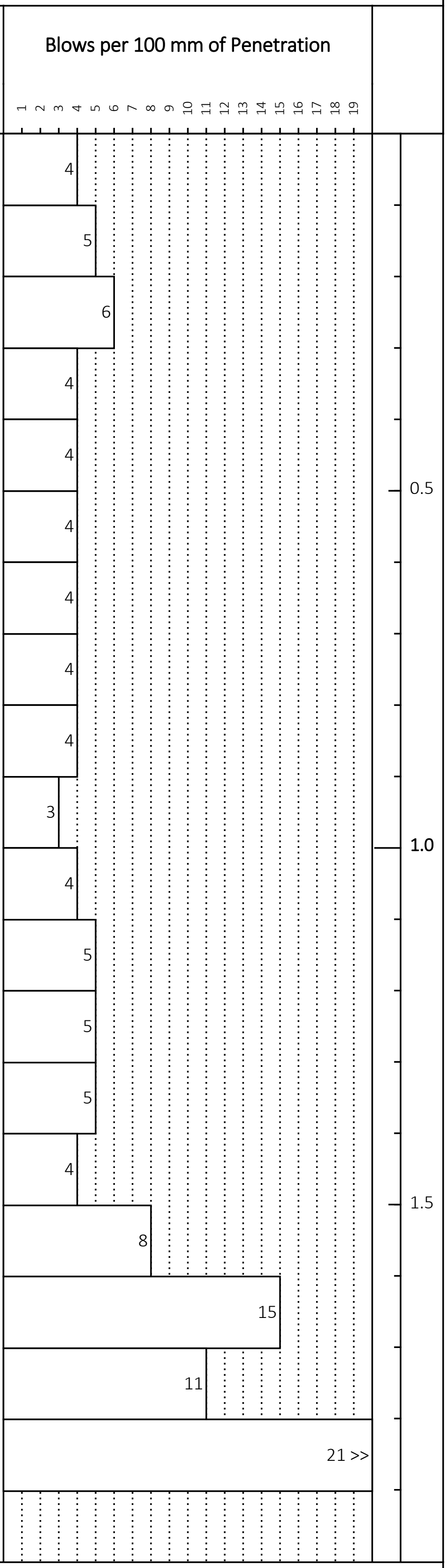
SCALA PENETROMETER & HAND AUGER RESULTS

KIRK ROBERTS
CONSULTING

Client:	Kainga Ora	**Scala Penetrometer:**	SP-03
Site Address:	6 -10 Orr Street	**Hand Auger No.**	HA-03
Town/City:	Ashburton	**Job No.:**	2310011

Logged By:	RSD	**Checked By:**	FS	**Date:**	15/02/23
Logged Date:	16/02/23	**Checked Date:**	17/02/23	**Weather:**	Overcast
				Operator:	RSD

Geological Formation	Depth (m)	Sample Description	Graphic	USCS	D_r	Water Table	Undrained Shear Strength Su (kPa)
Topsoil		Silty TOPSOIL (OL), with some gravel, with trace cobbles; dark brown. Firm, dry, non-plastic; gravel, fine to coarse, subangular to subround; cobbles, round to subround, up to 100mm; [TOPSOIL]. EOH: 0.30m	TS	OL		Groundwater Not Encountered	
		0.3m: [INFERRED GRAVEL]					
	0.5						
	1.0						
	1.5						



Remarks:

Scala Penetrometer and Test Bore log tests give an indication of the ground condition at the location of the tests only. While they are representative of typical conditions across the site, they do not identify variations in the ground away from the test locations.

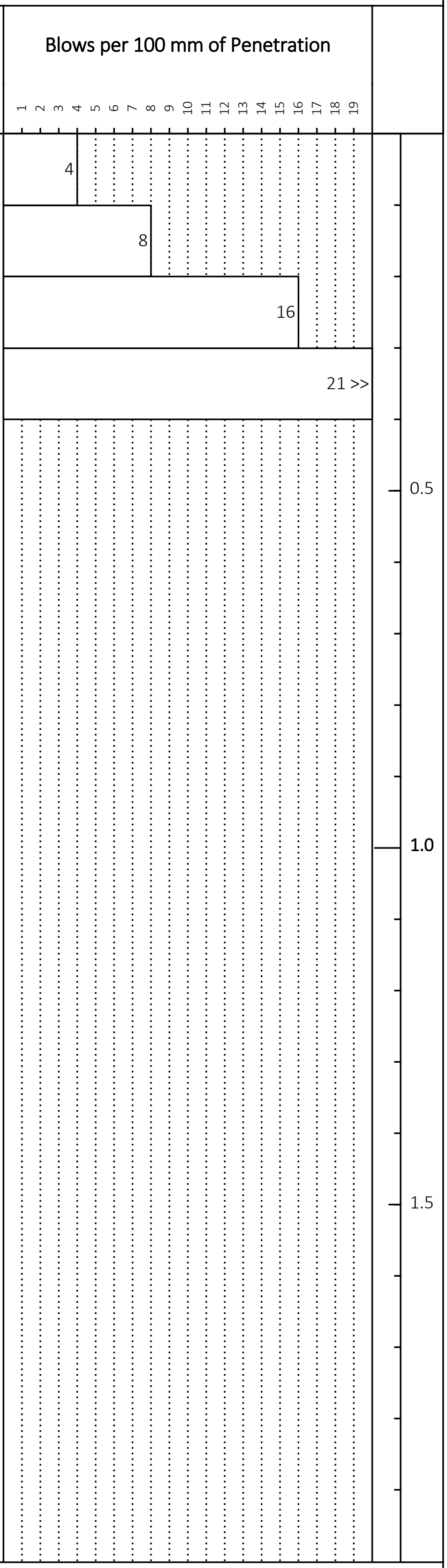
SCALA PENETROMETER & HAND AUGER RESULTS

Client:	Kainga Ora	**Scala Penetrometer:**	SP-04
Site Address:	6 -10 Orr Street	**Hand Auger No.**	HA-04
Town/City:	Ashburton	**Job No.:**	2310011

Logged By:	RSD	**Checked By:**	FS	**Date:**	15/02/23
Logged Date:	16/02/23	**Checked Date:**	17/02/23	**Weather:**	Overcast
				Operator:	RSD

Geological Formation	Depth (m)	Sample Description	Graphic	USCS	Dr	Water Table	Undrained Shear Strength Su (kPa)
Topsoil	0.0–0.30	Silty TOPSOIL (OL), with some gravel, with trace cobbles; dark brown. Stiff to very stiff, dry, non-plastic; gravel, fine to coarse, subangular to subround; cobbles, round to subround, up to 100mm; [TOPSOIL]. EOH: 0.30m	TS	OL		Groundwater Not Encountered	
	0.3	0.3m: [INFERRED GRAVEL]					

Depth scale: 0.5, 1.0, 1.5



Remarks:

Scala Penetrometer and Test Bore log tests give an indication of the ground condition at the location of the tests only. While they are representative of typical conditions across the site, they do not identify variations in the ground away from the test locations.

Generated with CORE-GS

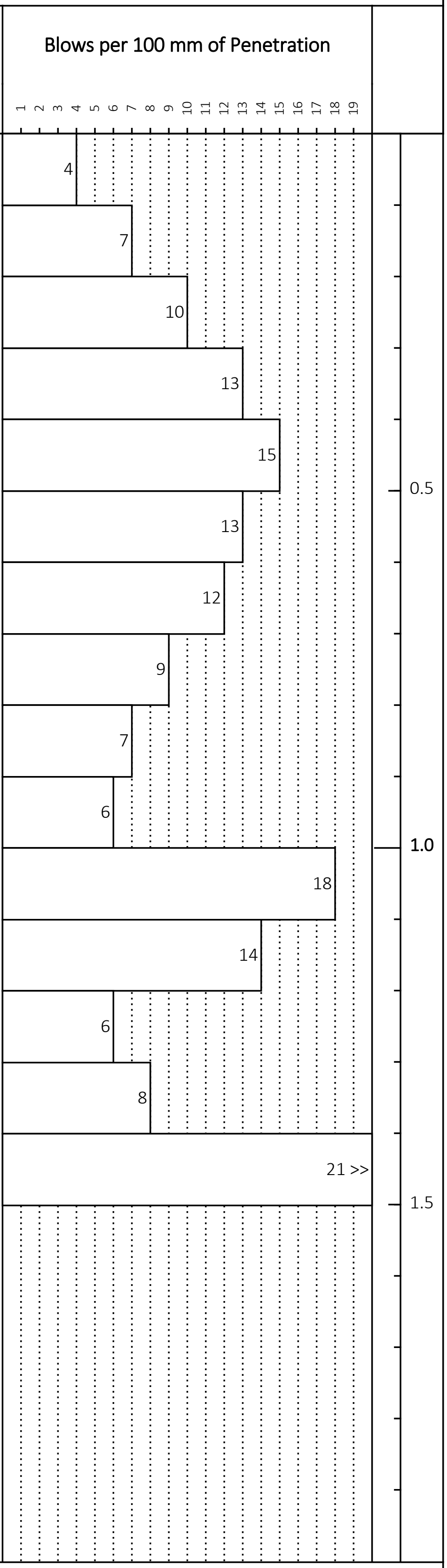
SCALA PENETROMETER & HAND AUGER RESULTS

KIRK ROBERTS CONSULTING

Client:	Kainga Ora	**Scala Penetrometer:**	SP-05
Site Address:	6 -10 Orr Street	**Hand Auger No.**	HA-05
Town/City:	Ashburton	**Job No.:**	2310011

Logged By:	RSD	**Checked By:**	FS	**Date:**	15/02/23
Logged Date:	16/02/23	**Checked Date:**	17/02/23	**Weather:**	Overcast
				Operator:	RSD

Geological Formation	Depth (m)	Sample Description	Graphic	USCS	D_r	Water Table	Undrained Shear Strength Su (kPa)
Topsoil	0.0–0.3	Silty TOPSOIL (OL), with some gravel, with trace cobbles; dark brown. Firm, dry, non-plastic; gravel, fine to medium, subangular to subround; cobbles, round to subround, up to 100mm; [TOPSOIL].	TS	OL		Groundwater Not Encountered	
Holocene Alluvial Deposits	0.3–0.5	SILT (M); yellowish brown. Firm to stiff, dry, non-plastic. EOH: 0.50m	×	M			
	0.5	0.5m: [INFERRED GRAVEL]					



Remarks:

Scala Penetrometer and Test Bore log tests give an indication of the ground condition at the location of the tests only. While they are representative of typical conditions across the site, they do not identify variations in the ground away from the test locations.

SCALA PENETROMETER & HAND AUGER RESULTS

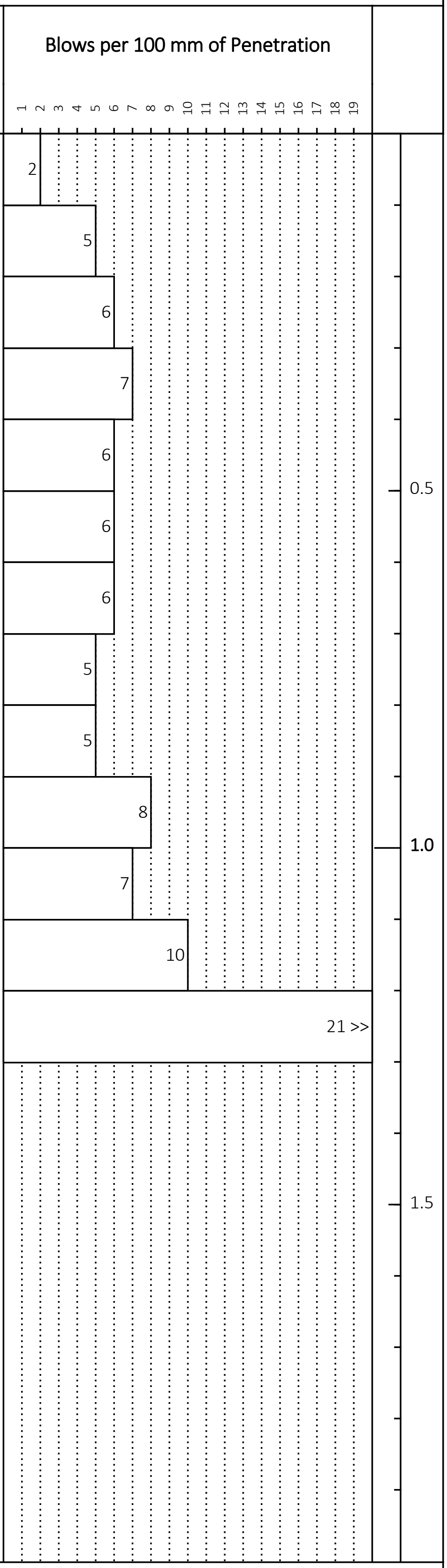
Client: Kainga Ora
Site Address: 6 -10 Orr Street
Town/City: Ashburton

Scala Penetrometer: SP-06
Hand Auger No. HA-06
Job No.: 2310011

Logged By: RSD
Logged Date: 16/02/23
Checked By: FS
Checked Date: 17/02/23

Date: 15/02/23
Weather: Overcast
Operator: RSD

Geological Formation	Depth (m)	Sample Description	USCS	Water Table
Topsoil	0.0–0.3	Silty TOPSOIL (OL), with some gravel; dark brown. Firm, dry, non-plastic; gravel, fine to medium, subround to subangular; [TOPSOIL].	OL	Groundwater Not Encountered
Holocene Alluvial Deposits	0.3–1.2	SILT (M); yellowish brown. Firm to stiff, dry, non-plastic. 0.9m: Moist. EOH: 1.20m 1.2m: [INFERRED GRAVEL]	M	



Remarks:

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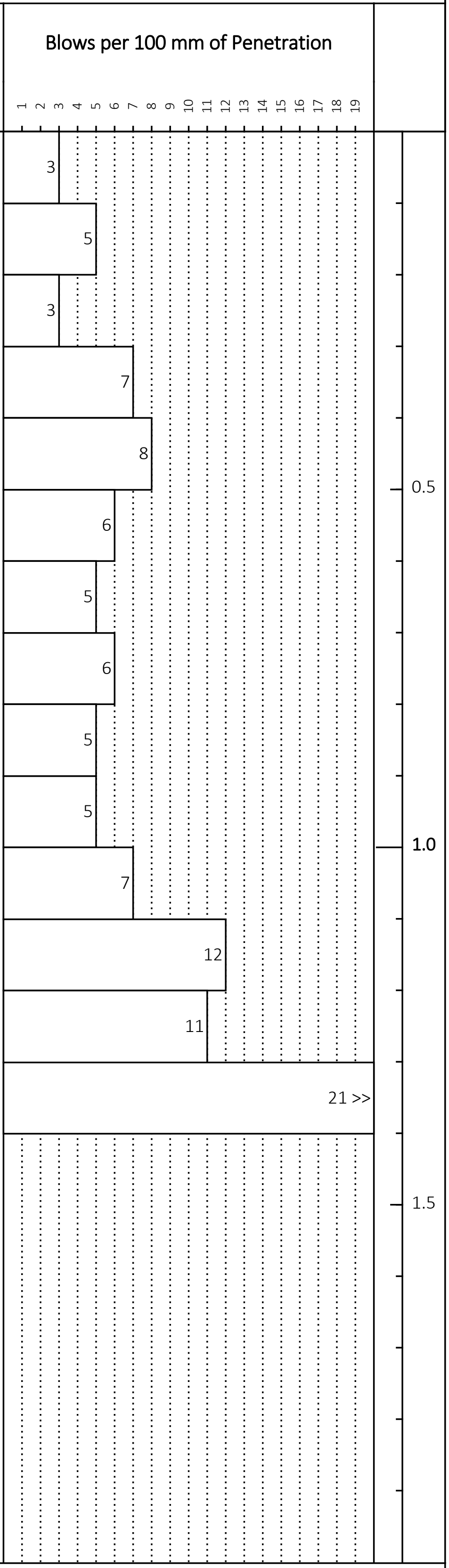
SCALA PENETROMETER & HAND AUGER RESULTS

KIRK ROBERTS CONSULTING

Client:	Kainga Ora	**Scala Penetrometer:**	SP-07
Site Address:	6 -10 Orr Street	**Hand Auger No.**	HA-07
Town/City:	Ashburton	**Job No.:**	2310011

Logged By:	RSD	**Checked By:**	FS	**Date:**	15/02/23
Logged Date:	16/02/23	**Checked Date:**	17/02/23	**Weather:**	Overcast
				Operator:	RSD

Geological Formation	Depth (m)	Sample Description	Graphic	USCS	Dr	Water Table	Undrained Shear Strength Su (kPa)
Topsoil		Silty TOPSOIL (OL), with minor gravel; dark brown. Firm, dry, non-plastic; gravel, fine to medium, subangular to subround; [TOPSOIL].	TS	OL		Groundwater Not Encountered	
Holocene Alluvial Deposits	0.5 1.0	SILT (M); yellowish brown. Firm to stiff, dry, non-plastic. 0.9m: Moist. EOH: 1.30m 1.3m: [INFERRED GRAVEL]		M			
	1.5						



Remarks:

Scala Penetrometer and Test Bore log tests give an indication of the ground condition at the location of the tests only. While they are representative of typical conditions across the site, they do not identify variations in the ground away from the test locations.

SCALA PENETROMETER & HAND AUGER RESULTS

KIRK ROBERTS CONSULTING

Client: Kainga Ora
Site Address: 6 -10 Orr Street
Town/City: Ashburton

Scala Penetrometer: SP-08
Hand Auger No. HA-08
Job No.: 2310011

Logged By: RSD
Logged Date: 16/02/23

Checked By: FS
Checked Date: 17/02/23

Date: 15/02/23
Weather: Overcast
Operator: AA

Geological Formation	Depth (m)	Sample Description	Graphic	USCS	D_r	Water Table	Undrained Shear Strength Su (kPa)	Blows per 100 mm of Penetration
Topsoil		Silty TOPSOIL (OL), with some gravel; dark brown. Stiff to very stiff, dry, non-plastic; gravel, fine to coarse, subround to subangular; [TOPSOIL]. EOH: 0.30m	TS	OL		Groundwater Not Encountered		4 11 21 >>
		0.3m: [INFERRED GRAVEL]						
	0.5							
	1.0							
	1.5							

Remarks:

Scala Penetrometer and Test Bore log tests give an indication of the ground condition at the location of the tests only. While they are representative of typical conditions across the site, they do not identify variations in the ground away from the test locations.

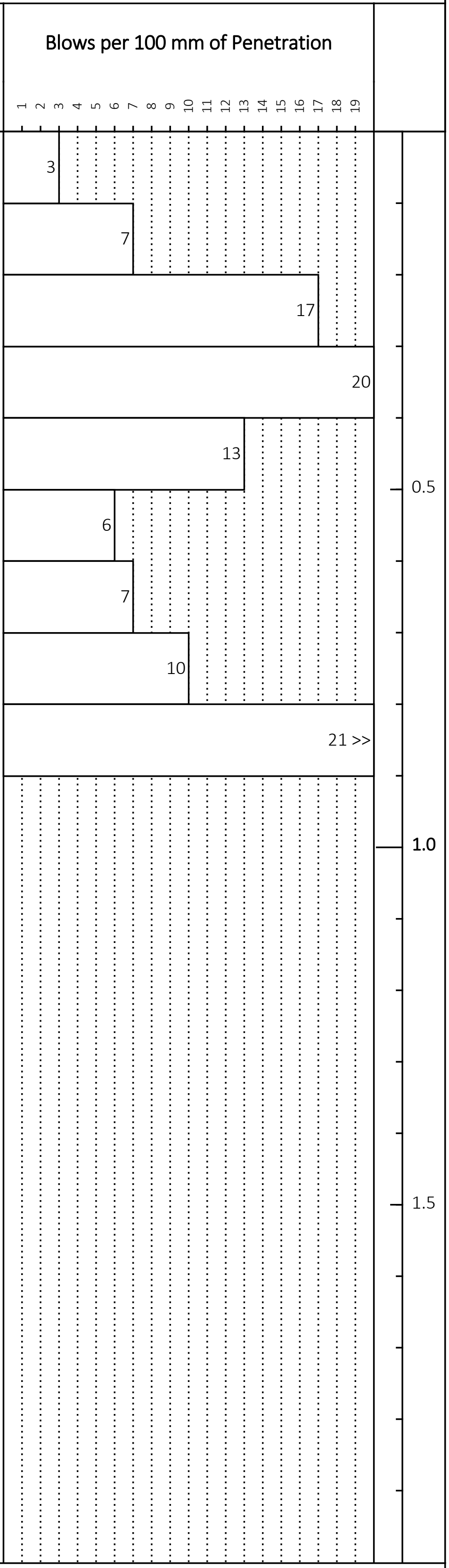
Generated with CORE-GS

SCALA PENETROMETER & HAND AUGER RESULTS

KIRK ROBERTS CONSULTING

Client:	Kainga Ora	**Scala Penetrometer:**	SP-09
Site Address:	6 -10 Orr Street	**Hand Auger No.**	HA-09
Town/City:	Ashburton	**Job No.:**	2310011
Logged By: RSD	**Checked By:** FS	**Date:**	15/02/23
Logged Date: 16/02/23	**Checked Date:** 17/02/23	**Weather:**	Overcast
		Operator:	AA

Geological Formation	Depth (m)	Sample Description	Graphic	USCS	D_r	Water Table	Undrained Shear Strength Su (kPa)
Topsoil	0.0–0.3	Silty TOPSOIL (OL), with some gravel; dark brown. Stiff to very stiff, dry, non-plastic; gravel, fine to coarse, subround to subangular; [TOPSOIL]. EOH: 0.30m	TS	OL		Groundwater Not Encountered	
	0.3	0.3m: [INFERRED GRAVEL]					



Depth scale: 0.5, 1.0, 1.5

Remarks:

Scala Penetrometer and Test Bore log tests give an indication of the ground condition at the location of the tests only. While they are representative of typical conditions across the site, they do not identify variations in the ground away from the test locations.

SCALA PENETROMETER & HAND AUGER RESULTS

KIRK ROBERTS
CONSULTING

Client:	Kainga Ora	**Scala Penetrometer:**	SP-10
Site Address:	6 -10 Orr Street	**Hand Auger No.**	HA-10
Town/City:	Ashburton	**Job No.:**	2310011

Logged By:	RSD	**Checked By:**	FS	**Date:**	15/02/23
Logged Date:	16/02/23	**Checked Date:**	17/02/23	**Weather:**	Overcast
				Operator:	AA

Geological Formation	Depth (m)	Sample Description	Graphic	USCS	D_r	Water Table	Undrained Shear Strength Su (kPa)	Blows per 100 mm of Penetration 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19
Topsoil		Silty TOPSOIL (OL), with some gravel; dark brown. Very stiff to hard, dry, non-plastic; gravel, fine to coarse, subround to subangular; [TOPSOIL]. EOH: 0.20m	TS	OL		Groundwater Not Encountered		12 15
		0.2m: [INFERRED GRAVEL]						21 >>
	0.5							
	1.0							
	1.5							

Remarks:

Scala Penetrometer and Test Bore log tests give an indication of the ground condition at the location of the tests only. While they are representative of typical conditions across the site, they do not identify variations in the ground away from the test locations.

SCALA PENETROMETER & HAND AUGER RESULTS

KIRK ROBERTS CONSULTING

Client:	Kainga Ora	**Scala Penetrometer:**	SP-11
Site Address:	6 -10 Orr Street	**Hand Auger No.**	HA-11
Town/City:	Ashburton	**Job No.:**	2310011

Logged By:	RSD	**Checked By:**	FS	**Date:**	15/02/23
Logged Date:	16/02/23	**Checked Date:**	17/02/23	**Weather:**	Overcast
				Operator:	AA

Geological Formation	Depth (m)	Sample Description	Graphic	USCS	D_r	Water Table	Undrained Shear Strength Su (kPa)	Blows per 100 mm of Penetration (1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19)
Topsoil		Silty TOPSOIL (OL), with some gravel; dark brown. Very stiff to hard, dry, non-plastic; gravel, fine to coarse, subangular to subround; [TOPSOIL]. EOH: 0.30m	TS	OL		Groundwater Not Encountered		8
								20
								21 >>
		0.3m: [INFERRED GRAVEL]						
	0.5							
	1.0							
	1.5							

Remarks:

Scala Penetrometer and Test Bore log tests give an indication of the ground condition at the location of the tests only. While they are representative of typical conditions across the site, they do not identify variations in the ground away from the test locations.

SCALA PENETROMETER & HAND AUGER RESULTS

KIRK ROBERTS CONSULTING

Client:	Kainga Ora	**Scala Penetrometer:**	SP-12
Site Address:	6 -10 Orr Street	**Hand Auger No.**	HA-12
Town/City:	Ashburton	**Job No.:**	2310011
Logged By: RSD	**Checked By:** FS	**Date:**	15/02/23
Logged Date: 16/02/23	**Checked Date:** 17/02/23	**Weather:**	Overcast
		Operator:	AA

Geological Formation	Depth (m)	Sample Description	Graphic	USCS	D_r	Water Table	Undrained Shear Strength Su (kPa)	Blows per 100 mm of Penetration
Topsoil		Silty TOPSOIL (OL), with some gravel, with trace cobbles; dark brown. Very stiff to hard, dry, non-plastic; gravel, fine to coarse, subangular to subround; cobbles, subround to round, up to 100mm; [TOPSOIL]. EOH: 0.30m	TS	OL		Groundwater Not Encountered		8 8 21 >>
	0.5	0.3m: [INFERRED GRAVEL]						
	1.0							
	1.5							

Remarks:

Scala Penetrometer and Test Bore log tests give an indication of the ground condition at the location of the tests only. While they are representative of typical conditions across the site, they do not identify variations in the ground away from the test locations.

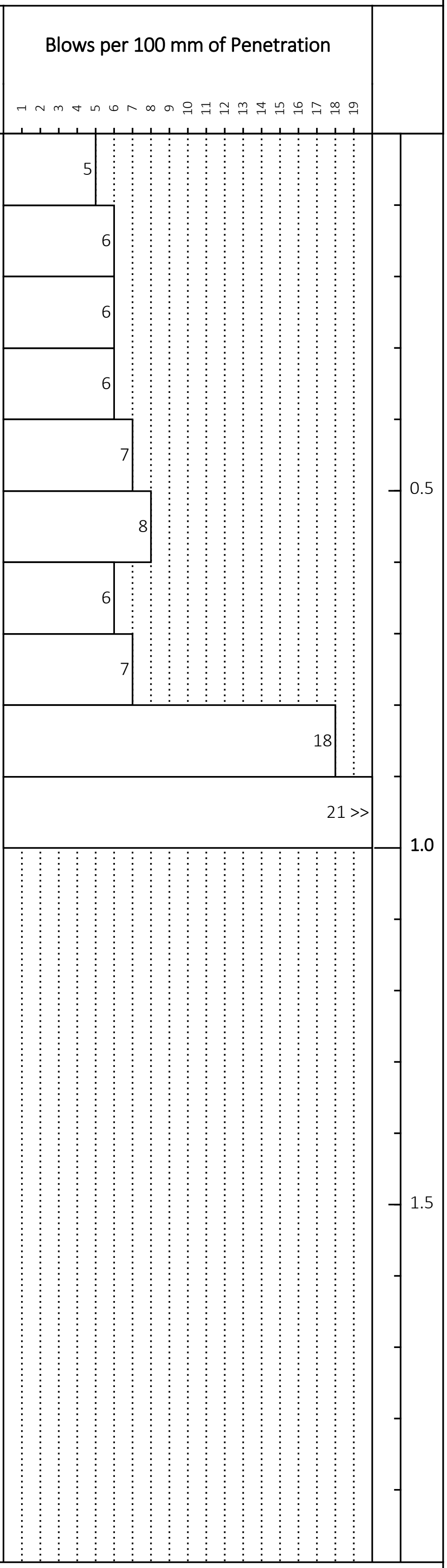
SCALA PENETROMETER & HAND AUGER RESULTS

KIRK ROBERTS CONSULTING

Client:	Kainga Ora	**Scala Penetrometer:**	SP-13
Site Address:	6 -10 Orr Street	**Hand Auger No.**	HA-13
Town/City:	Ashburton	**Job No.:**	2310011

Logged By:	RSD	**Checked By:**	FS	**Date:**	15/02/23
Logged Date:	16/02/23	**Checked Date:**	17/02/23	**Weather:**	Overcast
				Operator:	AA

Geological Formation	Depth (m)	Sample Description	Graphic	USCS	D_r	Water Table	Undrained Shear Strength Su (kPa)
Topsoil		Silty TOPSOIL (OL), with some gravel, with trace cobbles; dark brown. Very stiff to hard, dry, non-plastic; gravel, fine to coarse, subround to subangular; cobbles, subround to round, up to 100mm; [TOPSOIL].	TS	OL			
Holocene Alluvial Deposits	0.5	SILT (M); yellowish brown. Firm to stiff, dry, non-plastic. EOH: 0.80m		M			
	1.0 1.5	0.8m: [INFERRED GRAVEL]				Groundwater Not Encountered	



Remarks:

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SCALA PENETROMETER & HAND AUGER RESULTS

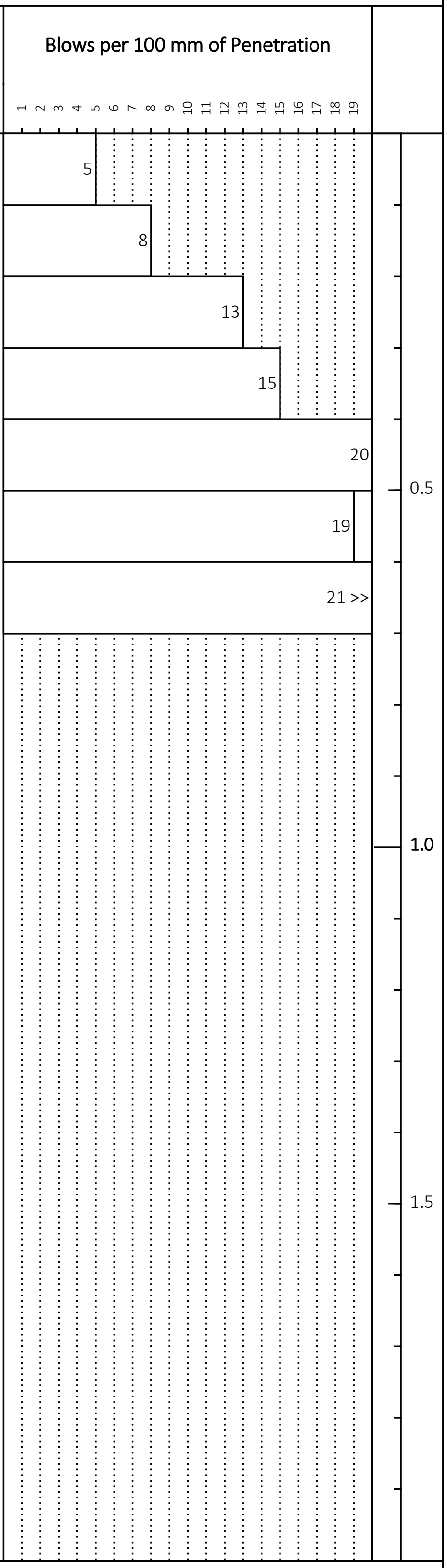
KIRK ROBERTS CONSULTING

Client:	Kainga Ora	**Scala Penetrometer:**	SP-14
Site Address:	6 -10 Orr Street	**Hand Auger No.**	HA-14
Town/City:	Ashburton	**Job No.:**	2310011

Logged By:	RSD	**Checked By:**	FS	**Date:**	15/02/23
Logged Date:	16/02/23	**Checked Date:**	17/02/23	**Weather:**	Overcast
				Operator:	AA

Geological Formation	Depth (m)	Sample Description	Graphic	USCS	D_r	Water Table	Undrained Shear Strength Su (kPa)
Topsoil	0.0 – 0.30	Silty TOPSOIL (OL), with some gravel; dark brown. Very stiff to hard, dry, non-plastic; gravel, fine to coarse, subangular to subround; [TOPSOIL]. EOH: 0.30m	TS	OL		Groundwater Not Encountered	
	0.3	0.3m: [INFERRED GRAVEL]					

Depth (m): 0.5, 1.0, 1.5



Remarks:

Scala Penetrometer and Test Bore log tests give an indication of the ground condition at the location of the tests only. While they are representative of typical conditions across the site, they do not identify variations in the ground away from the test locations.

SCALA PENETROMETER & HAND AUGER RESULTS

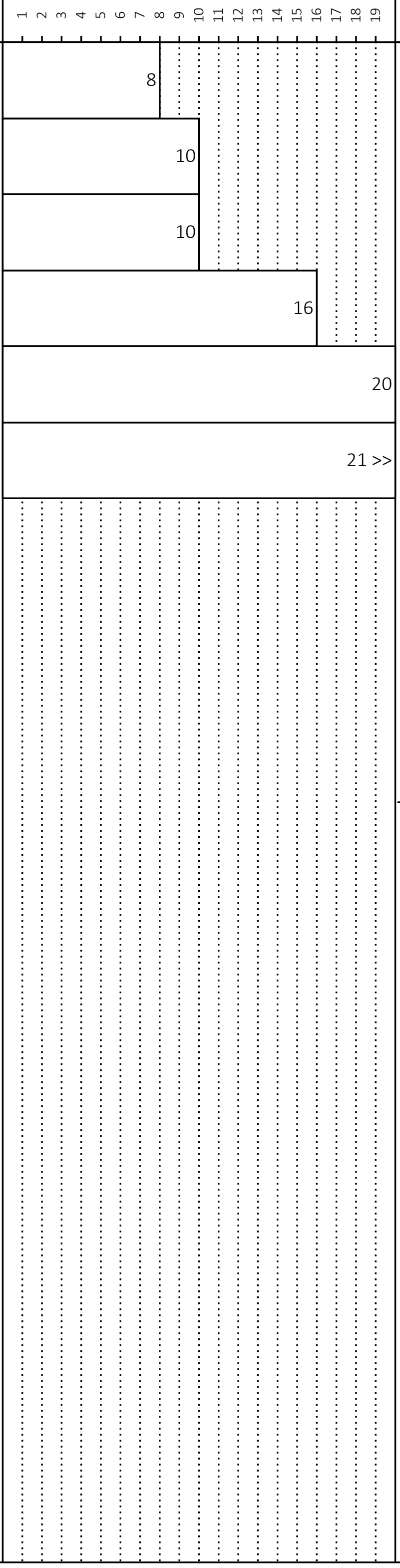
KIRK ROBERTS
CONSULTING

Client:	Kainga Ora	**Scala Penetrometer:**	SP-15
Site Address:	6 -10 Orr Street	**Hand Auger No.**	HA-15
Town/City:	Ashburton	**Job No.:**	2310011

Logged By:	RSD	**Checked By:**	FS	**Date:**	15/02/23
Logged Date:	16/02/23	**Checked Date:**	17/02/23	**Weather:**	Overcast
				Operator:	AA

Geological Formation	Depth (m)	Sample Description	Graphic	USCS	Dr	Water Table	Undrained Shear Strength Su (kPa)
Topsoil		Silty TOPSOIL (OL), with some gravel; dark brown. Very stiff to hard, dry, non-plastic; gravel, fine to coarse, subangular to subround; [TOPSOIL].	TS	OL		Groundwater Not Encountered	
Holocene Alluvial Deposits	0.5	SILT (M); yellowish brown. Firm to stiff, dry, non-plastic. EOH: 0.60m	×	M			
	1.0	0.6m: [INFERRED GRAVEL]					
	1.5						

Blows per 100 mm of Penetration



Remarks:

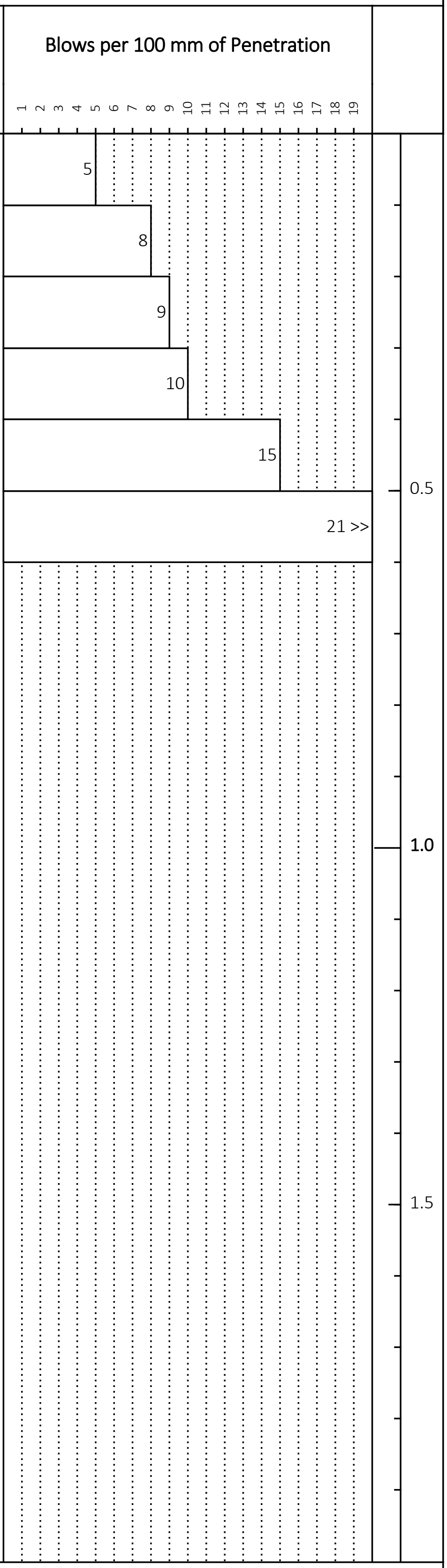
Scala Penetrometer and Test Bore log tests give an indication of the ground condition at the location of the tests only. While they are representative of typical conditions across the site, they do not identify variations in the ground away from the test locations.

SCALA PENETROMETER & HAND AUGER RESULTS

Client:	Kainga Ora	**Scala Penetrometer:**	SP-16
Site Address:	6 -10 Orr Street	**Hand Auger No.**	HA-16
Town/City:	Ashburton	**Job No.:**	2310011

Logged By:	RSD	**Checked By:**	FS	**Date:**	15/02/23
Logged Date:	16/02/23	**Checked Date:**	17/02/23	**Weather:**	Overcast
				Operator:	AA

Geological Formation	Depth (m)	Sample Description	Graphic	USCS	Dr	Water Table	Undrained Shear Strength Su (kPa)
Topsoil		Silty TOPSOIL (OL), with minor gravel; dark brown. Very stiff to hard, dry, non-plastic; gravel, fine to coarse, subangular to subround; [TOPSOIL].	TS	OL		Groundwater Not Encountered	
Holocene Alluvial Deposits	0.5	SILT (M); yellowish brown. Firm to stiff, dry, non-plastic. EOH: 0.60m		M			
		0.6m: [INFERRED GRAVEL]					
	1.0						
	1.5						



Remarks:

Scala Penetrometer and Test Bore log tests give an indication of the ground condition at the location of the tests only. While they are representative of typical conditions across the site, they do not identify variations in the ground away from the test locations.

B

Appendix B – Site Investigation Plan

No.	Revision	By	Chk	Appd	Date

Drawing Originator:

Original Scale (A1)

Reduced Scale (A3)

Design		
Drawn		
Dsg Verifier		
Dwg Check		

* Refer to Revision 1 for Original Signature

Client:

Kāinga Ora
Homes and Communities

Project:
Housing Delivery System
MBU4

Title:
GROUND INVESTIGATION PLAN
6 - 10 ORR STREET (ASHBURTON)

Discipline
GEOTECHNICAL

Drawing No.

Rev.

BLUEBEAM

DO NOT SCALE

IF IN DOUBT ASK.

C

Appendix C – Test Pit Logs and Photographs

Beca Geotechnical Log Key Sheet

SOIL AND ROCK DESCRIPTIONS

Soil and Rock Descriptions are in general accordance with the NZ Geotechnical Society (NZGS), 2005.
Hand-held Vane Shear Strength measurements are in general accordance with the NZGS, 2001.

METHODS

BH	Machine Borehole
CPT	Cone Penetration Test
DCP	Dynamic Cone Penetration
HA	Hand Auger
SPT	Standard Penetration Test
IVAN	In-situ Vane Test
MA	Machine Auger
OB	Open Barrel
SNC	Sonic Core Drilling
TP	Test Pit/Trench
TT	Triple Tube
PT	Thin-walled Open Drive Tube
VE	Vacuum Excavation
W	Wash Boring

WEATHERING

CW	Completely Weathered
HW	Highly Weathered
MW	Moderately Weathered
SW	Slightly Weathered
UW	Unweathered

SAMPLES

B	Bulk Disturbed Sample
C	Core Sample
D	Small Disturbed Sample
PT	Thin-wall Open Drive (Push) Tube Sample

WATER

▼ Groundwater Level (GWL)

IN-SITU TESTS

Shear Vane

Su	In-situ peak undrained shear strength and remoulded undrained shear strength
UTP	Unable to Penetrate
CB	Pilcon-type vane tested in Core Barrel
DH	Pilcon-type vane tested in-situ (downhole)
GV	Geonor vane, tested in-situ
IcV	Icone vane, tested in-situ

Standard Penetration Test (SPT)

N	SPTn Sampler (Split-spoon)
Nc	SPTn Solid Cone
HB	SPT Hammer Bouncing

TERMINOLOGY

RL	Relative Ground Level
RQD	Rock Quality Designation

GRAPHIC LOG (1 or a combination of the following)

- Clay
- Gravel
- Shells
- Cobbles / Boulders
- Silt
- Sand
- Organic Material
- Fill
- Sandstone (SST)
- Siltstone (ZST)
- Mudstone
- Interbedded SST & ZST
- Conglomerate
- Limestone
- Foliated Metamorphic
- Asphalt
- Fine Igneous
- Coarse Igneous
- Ignimbrite
- No Core

MONITORING INSTALLATION

Backfill Material

- Sand
- Gravel
- Grout
- Cement Mixes
- Bentonite

Standpipe

- Plain
- Slotted
- Vibrated Wire

ORGANIC SOILS

Von Post Degree of Humidification

H1	Completely unconverted and mud-free peat, when pressed gives clear water and plant structure is visible.
H2	Partially unconverted and mud-free peat, when pressed gives almost clear water and plant structure is visible.
H3	Very slightly decomposed or very slightly muddy peat, when pressed gives marked muddy water, no peat substance passes through the fingers and plant structure is less visible.
H4	Slightly decomposed or slightly muddy peat, when pressed gives muddy water and plant structure is less visible.
H5	Moderately decomposed or very muddy peat with growth structure evident but slightly obliterated.
H6	Moderately decomposed or very muddy peat with indistinct growth structure.
H7	Fairly well decomposed or very muddy peat but the growth structure can just be seen.
H8	Well decomposed or very muddy peat with very indistinct growth structure.
H9	Practically decomposed or mud-like peat in which almost no growth structure is evident.
H10	Completely decomposed or mud peat where no growth structure can be seen, entire substance passes through the fingers when pressed.

Test Pit Log

Test Pit ID: ITP-001
Sheet 1 of 1

Project:	HDS - 6-10 Orr Street	**Project number:**	3160491/AR109526
Site location:	6-10 Orr Street, Netherby, Ashburton	**Client:**	Kāinga Ora

Location:	10 Orr Street, Front yard	**Coordinate system:**	NZTM2000	**Vertical datum:**	NZVD 2016
		Northing:	5138452.0	**Ground level (mRL):**	96.00
		Easting:	1500796.0	**Location method:**	Canterbury Maps

Groundwater (m)	In Situ Tests: Su (kPa)	In Situ Tests: Scala blows/50mm	Samples	Depth (m)	RL (m)	Graphic Log	Soil/ Rock Description	Geological Unit
							SILT, some fine sand, some organics; light greyish brown; moist. Organics: roots and rootlets. [TOPSOIL]	Topsoil
							Tightly packed, fine to coarse sandy, fine to coarse GRAVEL, trace cobbles, trace silt; light greyish brown; dry, well graded. Gravel and cobbles: subrounded to subangular, SW to UW, greywacke.	Quaternary Alluvium
				0.5	95.5			
							Loosely packed, fine to coarse sandy, fine to coarse GRAVEL, some silt; light greyish brown; moist, well graded. Gravel: subrounded to subangular, SW to UW, greywacke.	
				1.0	95.0			
							1.10m: trace cobbles	
							Loosely packed, fine to coarse GRAVEL, minor fine to coarse sand, trace cobbles; light brownish grey; dry, poorly graded. Gravel and cobbles: subrounded to subangular, SW to UW, greywacke.	
				1.5	94.5		Loosely packed, fine to coarse sandy, fine to coarse GRAVEL, some cobbles, trace silt; light brownish grey; moist, well graded. Gravel and cobbles: subrounded to subangular, SW to UW, greywacke.	
							1.80 - 1.95m: absence of silt	
				2.0	94.0			
				2.5	93.5		2.50m - End of test pit, Target depth	
				3.0	93.0			
				3.5	92.5			
				4.0	92.0			
				4.5	91.5			

Date started: 28/02/2024
Vane ID: N/A
Vane type: N/A
Vane width: N/A
Face orientation: N/A

Logged by: JB
Contractor: ACL Ltd
Equipment: 3T Airman Hydraulic Excavator
Method: TP

Comments:
Groundwater not encountered. Loose material in pit walls led to ravelling during pre-soak and testing.

For Explanation of Symbols and Abbreviations See Key Sheet

Photo Log

Location ID: **ITP-001**

Sheet 1 of 1

Project:	HDS - 6-10 Orr Street	**Project number:**	3160491/AR109526
Site location:	6-10 Orr Street, Netherby, Ashburton	**Client Name:**	Kāinga Ora

Location:	10 Orr Street, Front yard	**Coordinate system:**	NZTM2000	**Vertical datum:**	NZVD 2016
		Northing:	5138452.0	**Ground level (mRL):**	96.00
		Easting:	1500796.0	**Location method:**	Canterbury Maps

ITP-001 - 0.00mbgl to 2.50mbgl

ITP-001 Arisings - 0.00mbgl to 2.50mbgl

Test Pit Log

Test Pit ID: **ITP-002**
Sheet 1 of 1

Project:	HDS - 6-10 Orr Street	**Project number:**	3160491/AR109526
Site location:	6-10 Orr Street, Netherby, Ashburton	**Client:**	Kāinga Ora

Location:	8 Orr Street, Back yard	**Coordinate system:**	NZTM2000	**Vertical datum:**	NZVD 2016
		Northing:	5138414.0	**Ground level (mRL):**	95.70
		Easting:	1500784.0	**Location method:**	Canterbury Maps

Groundwater (m)	In Situ Tests Su (kPa)	 Scala blows/50mm	Samples	Depth (m)	RL (m)	Graphic Log	Soil/ Rock Description	Geological Unit
					95.5		SILT, some fine sand, some organics; light greyish brown; moist. Organics: roots and rootlets. [TOPSOIL] *0.20m: trace fine to coarse gravel*	Topsoil
				0.5 1.0	95.0		Tightly packed, fine to coarse sandy, fine to coarse GRAVEL, trace cobbles, trace silt; light greyish brown; dry, well graded. Gravel and cobbles: subrounded to subangular, SW to UW, greywacke. *0.60m: loosely packed*	Quaternary Alluvium
					94.5		Loosely packed, fine to coarse GRAVEL, minor fine to coarse sand, trace cobbles; light brownish grey; moist, poorly graded. Gravel: subrounded to subangular, SW to UW, greywacke.	
				1.5	94.0		Tightly packed, fine to coarse sandy, fine to coarse GRAVEL, some cobbles, trace silt; light brownish grey; moist, well graded. Gravel and cobbles: subrounded to subangular, SW to UW, greywacke.	
				2.0 2.5 3.0 3.5 4.0 4.5	93.5 93.0 92.5 92.0 91.5 91.0		2.00m - End of test pit	

Date started:	28/02/2024	**Logged by:**	JB
Vane ID:	N/A	**Contractor:**	ACL Ltd
Vane type:	N/A	**Equipment:**	3T Airman Hydraulic Excavator
Vane width:	N/A	**Method:**	TP
Face orientation:	N/A		

Comments:
Groundwater not encountered. Minor ravelling in pit walls during excavation.

For Explanation of Symbols and Abbreviations See Key Sheet

Photo Log

Location ID: ITP-002

Sheet 1 of 1

Project:	HDS - 6-10 Orr Street	**Project number:**	3160491/AR109526
Site location:	6-10 Orr Street, Netherby, Ashburton	**Client Name:**	Kāinga Ora

Location:	8 Orr Street, Back yard	**Coordinate system:**	NZTM2000	**Vertical datum:**	NZVD 2016
		Northing:	5138414.0	**Ground level (mRL):**	95.70
		Easting:	1500784.0	**Location method:**	Canterbury Maps

ITP-002 - 0.00mbgl to 2.00mbgl

ITP-002 Arisings - 0.00mbgl to 2.00mbgl

D

Appendix D – Bearing Capacity Calculation Sheets

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-001 Page 1

M.J. STOCKWELL
DETERMINATION OF ALLOWABLE PRESSURE UNDER SMALL STRUCTURES
BEARING CAPACITY

Reference: Stockwell M.J. (1977). Determination of allowable bearing pressure under small structures. New Zealand Engineering, 132 - 135.

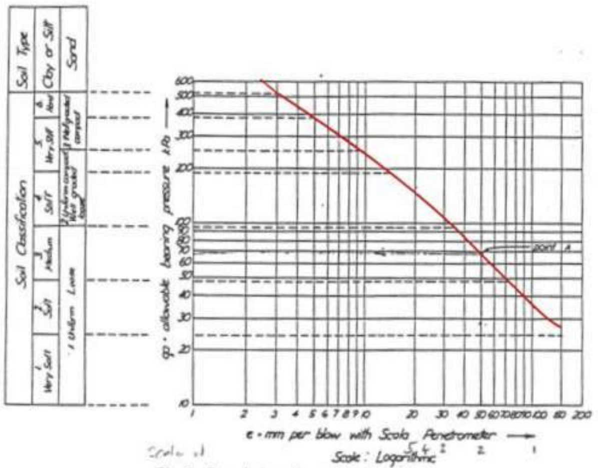


Fig. 2: Correlation of e and qa for design.

Ground Level (mRL)			95.6				
Depth (mm)			m RL	Measured No. Blows / 100mm	e mm/blow	Stockwell - qa kPa	Stockwell - qu kPa
0	-	100	95.50	3	33.3	100	300
100	-	200	95.40	8	12.5	195	585
200	-	300	95.30	21	4.8	260	780

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-002 Page 1

M.J. STOCKWELL
DETERMINATION OF ALLOWABLE PRESSURE UNDER SMALL STRUCTURES
BEARING CAPACITY

Reference: Stockwell M.J. (1977). Determination of allowable bearing pressure under small structures. New Zealand Engineering, 132 - 135.

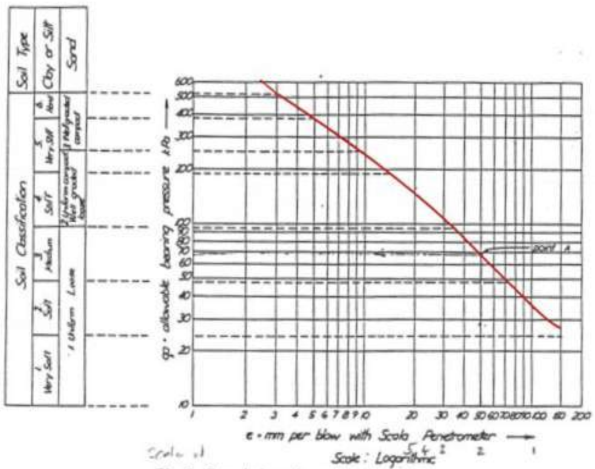


Fig. 2: Correlation of e and qa for design.

Ground Level (mRL)	95.8

Depth (mm)	m RL	Measured No. Blows / 100mm	e mm/blow	Stockwell - qa kPa	Stockwell - qu kPa
0 - 100	95.70	5	20.0	135	405
100 - 200	95.60	9	11.1	200	600
200 - 300	95.50	21	4.8	260	780

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-003 Page 1

M.J. STOCKWELL
DETERMINATION OF ALLOWABLE PRESSURE UNDER SMALL STRUCTURES
BEARING CAPACITY

Reference: Stockwell M.J. (1977). Determination of allowable bearing pressure under small structures. New Zealand Engineering, 132 - 135.

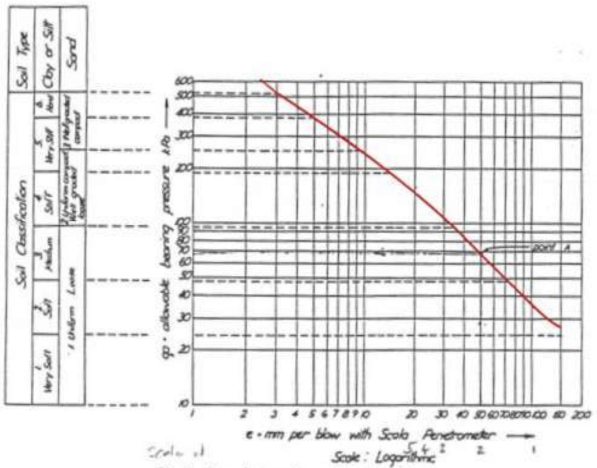


Fig. 2: Correlation of e and qa for design.

Ground Level (mRL)			95.7				
Depth (mm)			m RL	Measured No. Blows / 100mm	e mm/blow	Stockwell - qa kPa	Stockwell - qu kPa
0	-	100	95.60	4	25.0	116	348
100	-	200	95.50	5	20.0	135	405
200	-	300	95.40	6	16.7	150	450
300	-	400	95.30	4	25.0	116	348
400	-	500	95.20	4	25.0	116	348
500	-	600	95.10	4	25.0	116	348
600	-	700	95.00	4	25.0	116	348
700	-	800	94.90	4	25.0	116	348
800	-	900	94.80	4	25.0	116	348
900	-	1000	94.70	3	33.3	100	300
1000	-	1100	94.60	4	25.0	116	348
1100	-	1200	94.50	5	20.0	135	405
1200	-	1300	94.40	5	20.0	135	405
1300	-	1400	94.30	5	20.0	135	405
1400	-	1500	94.20	4	25.0	116	348
1500	-	1600	94.10	8	12.5	195	585
1600	-	1700	94.00	15	6.7	260	780
1700	-	1800	93.90	11	9.1	240	720
1800	-	1900	93.80	21	4.8	260	780

Job Name	Job Number	Date
HDS Christchurch MBU1	3160491	22/02/2024
Site Address		**Engineer**
6 - 10 Orr Street, Ashburton		Kiri Moonen

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-004 Page 1

M.J. STOCKWELL
DETERMINATION OF ALLOWABLE PRESSURE UNDER SMALL STRUCTURES
BEARING CAPACITY

Reference: Stockwell M.J. (1977). Determination of allowable bearing pressure under small structures. New Zealand Engineering, 132 - 135.

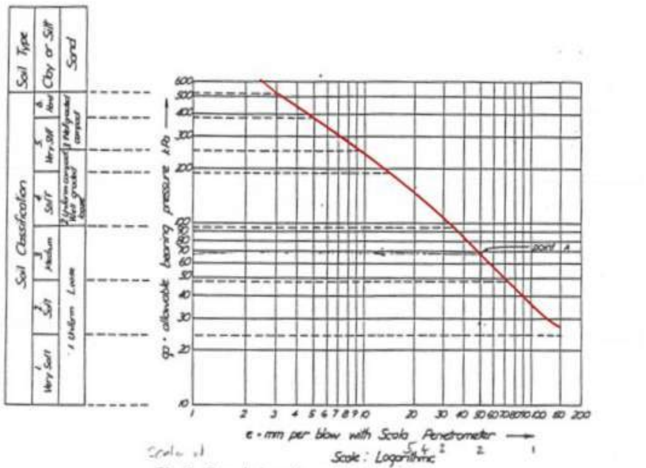


Fig. 2: Correlation of e and qa for design.

Ground Level (mRL)	95.9				
Depth (mm)	**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Stockwell - qa kPa**	**Stockwell - qu kPa**
0 - 100	95.80	4	25.0	116	348
100 - 200	95.70	8	12.5	195	585
200 - 300	95.60	16	6.3	260	780
300 - 400	95.50	21	4.8	260	780

Job Name	Job Number	Date
HDS Christchurch MBU1	**3160491**	**22/02/2024**
Site Address		**Engineer**
6 - 10 Orr Street, Ashburton		**Kiri Moonen**

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-005 Page 1

M.J. STOCKWELL
DETERMINATION OF ALLOWABLE PRESSURE UNDER SMALL STRUCTURES
BEARING CAPACITY

Reference: Stockwell M.J. (1977). Determination of allowable bearing pressure under small structures. New Zealand Engineering, 132 - 135.

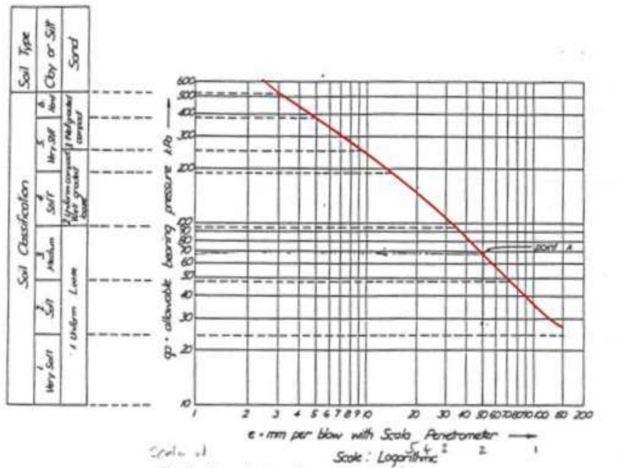


Fig. 2: Correlation of e and qa for design.

Ground Level (mRL)	95.6

Depth (mm)	**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Stockwell - qa kPa**	**Stockwell - qu kPa**
0 - 100	95.50	4	25.0	116	348
100 - 200	95.40	7	14.3	170	510
200 - 300	95.30	10	10.0	220	660
300 - 400	95.20	13	7.7	260	780
400 - 500	95.10	15	6.7	260	780
500 - 600	95.00	13	7.7	260	780
600 - 700	94.90	12	8.3	260	780
700 - 800	94.80	9	11.1	200	600
800 - 900	94.70	7	14.3	170	510
900 - 1000	94.60	6	16.7	150	450
1000 - 1100	94.50	18	5.6	260	780
1100 - 1200	94.40	14	7.1	260	780
1200 - 1300	94.30	6	16.7	150	450
1300 - 1400	94.20	8	12.5	195	585
1400 - 1500	94.10	21	4.8	260	780

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-006 Page 1

M.J. STOCKWELL
DETERMINATION OF ALLOWABLE PRESSURE UNDER SMALL STRUCTURES
BEARING CAPACITY

Reference: Stockwell M.J. (1977). Determination of allowable bearing pressure under small structures. New Zealand Engineering, 132 - 135.

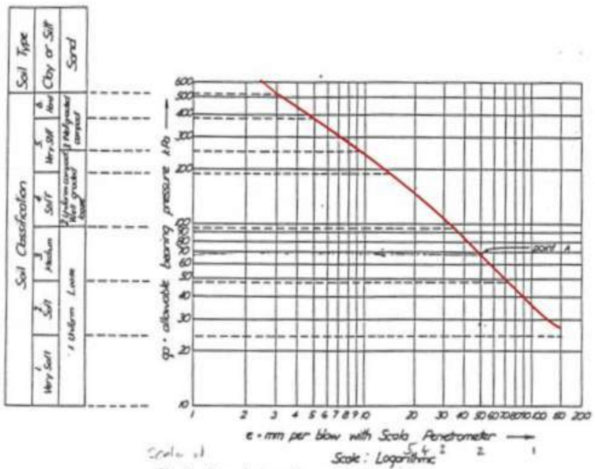


Fig. 2: Correlation of e and qa for design.

Ground Level (mRL)	95.6				
Depth (mm)	**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Stockwell - qa kPa**	**Stockwell - qu kPa**
0 - 100	95.50	2	50.0	67	200
100 - 200	95.40	5	20.0	135	405
200 - 300	95.30	6	16.7	150	450
300 - 400	95.20	7	14.3	170	510
400 - 500	95.10	6	16.7	150	450
500 - 600	95.00	6	16.7	150	450
600 - 700	94.90	6	16.7	150	450
700 - 800	94.80	5	20.0	135	405
800 - 900	94.70	5	20.0	135	405
900 - 1000	94.60	8	12.5	195	585
1000 - 1100	94.50	7	14.3	170	510
1100 - 1200	94.40	10	10.0	220	660
1200 - 1300	94.30	21	4.8	260	780

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-007 Page 1

M.J. STOCKWELL
DETERMINATION OF ALLOWABLE PRESSURE UNDER SMALL STRUCTURES
BEARING CAPACITY

Reference: Stockwell M.J. (1977). Determination of allowable bearing pressure under small structures. New Zealand Engineering, 132 - 135.

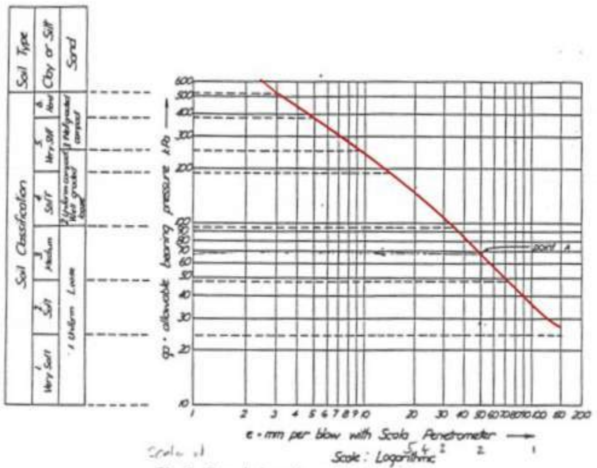


Fig. 2: Correlation of e and qa for design.

Ground Level (mRL)	95.6				
Depth (mm)	**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Stockwell - qa kPa**	**Stockwell - qu kPa**
0 - 100	95.50	3	33.3	100	300
100 - 200	95.40	5	20.0	135	405
200 - 300	95.30	3	33.3	100	300
300 - 400	95.20	7	14.3	170	510
400 - 500	95.10	8	12.5	195	585
500 - 600	95.00	6	16.7	150	450
600 - 700	94.90	5	20.0	135	405
700 - 800	94.80	6	16.7	150	450
800 - 900	94.70	5	20.0	135	405
900 - 1000	94.60	5	20.0	135	405
1000 - 1100	94.50	7	14.3	170	510
1100 - 1200	94.40	12	8.3	260	780
1200 - 1300	94.30	11	9.1	240	720
1300 - 1400	94.20	21	4.8	260	780

Job Name	Job Number	Date
HDS Christchurch MBU1	3160491	22/02/2024
Site Address		**Engineer**
6 - 10 Orr Street, Ashburton		Kiri Moonen

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-008 Page 1

M.J. STOCKWELL
DETERMINATION OF ALLOWABLE PRESSURE UNDER SMALL STRUCTURES
BEARING CAPACITY

Reference: Stockwell M.J. (1977). Determination of allowable bearing pressure under small structures. New Zealand Engineering, 132 - 135.

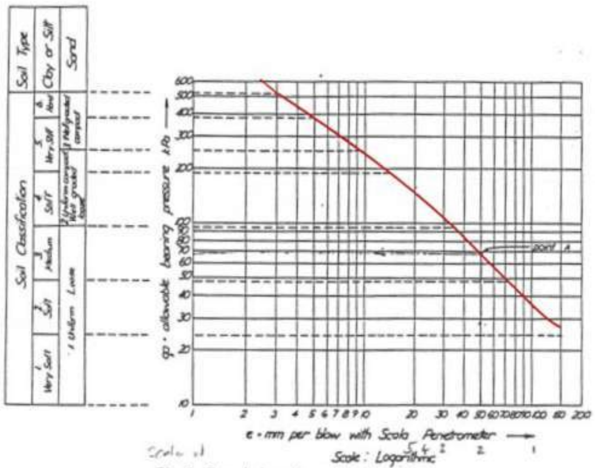


Fig. 2: Correlation of e and qa for design.

Ground Level (mRL)	96				
Depth (mm)	**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Stockwell - qa kPa**	**Stockwell - qu kPa**
0 - 100	95.90	4	25.0	116	348
100 - 200	95.80	11	9.1	240	720
200 - 300	95.70	21	4.8	260	780

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-009 Page 1

M.J. STOCKWELL
DETERMINATION OF ALLOWABLE PRESSURE UNDER SMALL STRUCTURES
BEARING CAPACITY

Reference: Stockwell M.J. (1977). Determination of allowable bearing pressure under small structures. New Zealand Engineering, 132 - 135.

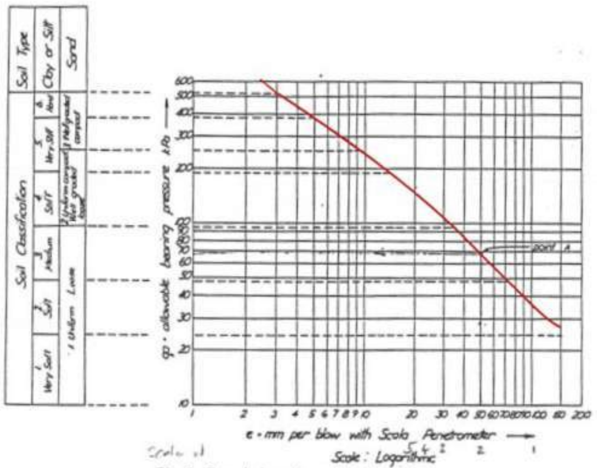


Fig. 2: Correlation of e and qa for design.

Ground Level (mRL)	96.3

Depth (mm)	m RL	Measured No. Blows / 100mm	e mm/blow	Stockwell - qa kPa	Stockwell - qu kPa
0 - 100	96.20	3	33.3	100	300
100 - 200	96.10	7	14.3	170	510
200 - 300	96.00	17	5.9	260	780
300 - 400	95.90	20	5.0	260	780
400 - 500	95.80	13	7.7	260	780
500 - 600	95.70	6	16.7	150	450
600 - 700	95.60	7	14.3	170	510
700 - 800	95.50	10	10.0	220	660
800 - 900	95.40	21	4.8	260	780

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-010 Page 1

M.J. STOCKWELL
DETERMINATION OF ALLOWABLE PRESSURE UNDER SMALL STRUCTURES
BEARING CAPACITY

Reference: Stockwell M.J. (1977). Determination of allowable bearing pressure under small structures. New Zealand Engineering, 132 - 135.

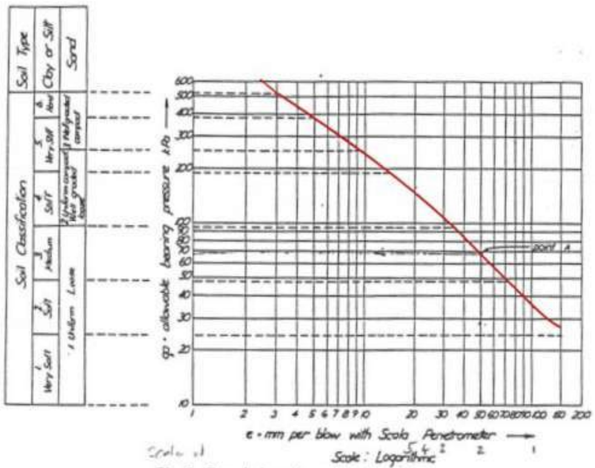


Fig. 2: Correlation of e and qa for design.

Ground Level (mRL) 96.1

Depth (mm)	m RL	Measured No. Blows / 100mm	e mm/blow	Stockwell - qa kPa	Stockwell - qu kPa
0 - 100	96.00	12	8.3	260	780
100 - 200	95.90	15	6.7	260	780
200 - 300	95.80	21	4.8	260	780

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-011 Page 1

M.J. STOCKWELL
DETERMINATION OF ALLOWABLE PRESSURE UNDER SMALL STRUCTURES
BEARING CAPACITY

Reference: Stockwell M.J. (1977). Determination of allowable bearing pressure under small structures. New Zealand Engineering, 132 - 135.

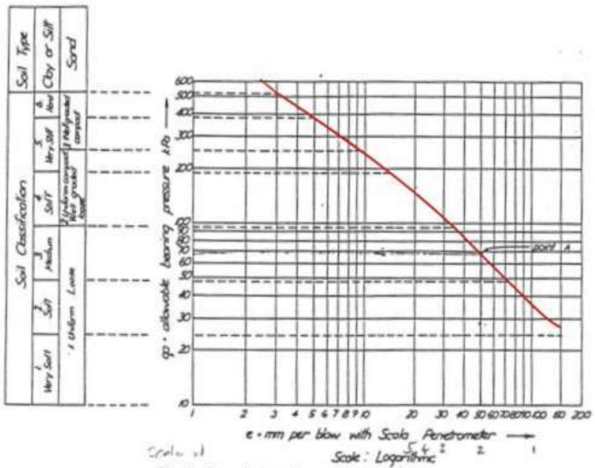


Fig. 2: Correlation of e and qa for design.

Ground Level (mRL)	95.9

Depth (mm)	**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Stockwell - qa kPa**	**Stockwell - qu kPa**
0 - 100	95.80	8	12.5	195	585
100 - 200	95.70	20	5.0	260	780
200 - 300	95.60	21	4.8	260	780

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-012 Page 1

M.J. STOCKWELL
DETERMINATION OF ALLOWABLE PRESSURE UNDER SMALL STRUCTURES
BEARING CAPACITY

Reference: Stockwell M.J. (1977). Determination of allowable bearing pressure under small structures. New Zealand Engineering, 132 - 135.

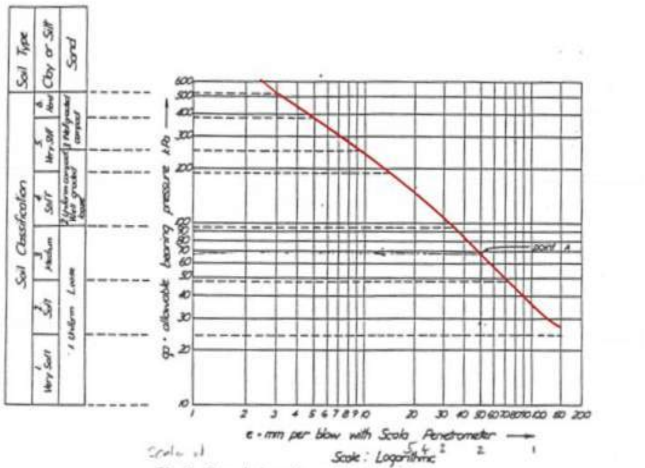


Fig. 2: Correlation of e and qa for design.

Ground Level (mRL)	96.2

Depth (mm)			**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Stockwell - qa kPa**	**Stockwell - qu kPa**
0	-	100	95.50	8	12.5	195	585
100	-	200	95.40	8	12.5	195	585
200	-	300	95.30	21	4.8	260	780

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-013 Page 1

M.J. STOCKWELL
DETERMINATION OF ALLOWABLE PRESSURE UNDER SMALL STRUCTURES
BEARING CAPACITY

Reference: Stockwell M.J. (1977). Determination of allowable bearing pressure under small structures. New Zealand Engineering, 132 - 135.

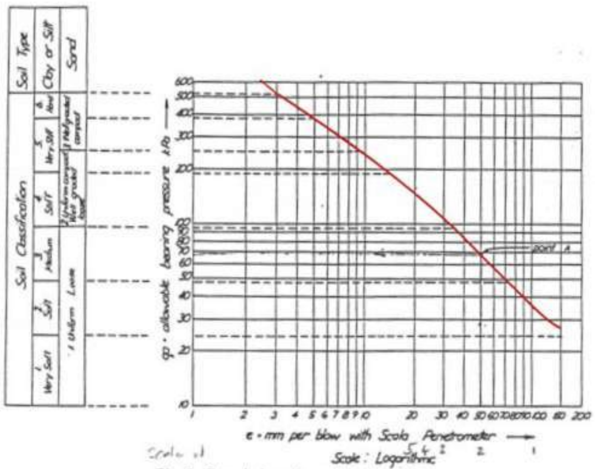


Fig. 2: Correlation of e and qa for design.

Ground Level (mRL)	96.1

Depth (mm)	**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Stockwell - qa kPa**	**Stockwell - qu kPa**
0 - 100	95.50	5	20.0	135	405
100 - 200	95.40	6	16.7	150	450
200 - 300	95.30	6	16.7	150	450
300 - 400	95.20	6	16.7	150	450
400 - 500	95.10	7	14.3	170	510
500 - 600	95.00	8	12.5	195	585
600 - 700	94.90	6	16.7	150	450
700 - 800	94.80	7	14.3	170	510
800 - 900	94.70	18	5.6	260	780
900 - 1000	94.60	21	4.8	260	780

Job Name	Job Number	Date
HDS Christchurch MBU1	3160491	22/02/2024
Site Address		**Engineer**
6 - 10 Orr Street, Ashburton		Kiri Moonen

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-014 Page 1

M.J. STOCKWELL
DETERMINATION OF ALLOWABLE PRESSURE UNDER SMALL STRUCTURES
BEARING CAPACITY

Reference: Stockwell M.J. (1977). Determination of allowable bearing pressure under small structures. New Zealand Engineering, 132 - 135.

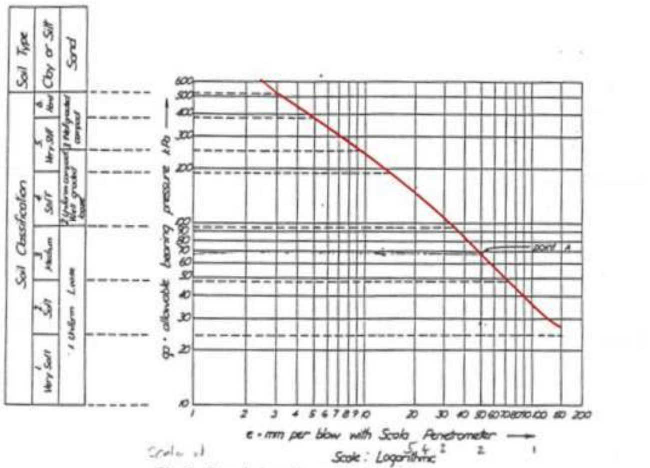


Fig. 2: Correlation of e and qa for design.

Ground Level (mRL)	96.1				
Depth (mm)	**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Stockwell - qa kPa**	**Stockwell - qu kPa**
0 - 100	95.50	5	20.0	135	405
100 - 200	95.40	8	12.5	195	585
200 - 300	95.30	13	7.7	260	780
300 - 400	95.20	15	6.7	260	780
400 - 500	95.10	20	5.0	260	780
500 - 600	95.00	19	5.3	260	780
600 - 700	94.90	21	4.8	260	780

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-015 Page 1

M.J. STOCKWELL
DETERMINATION OF ALLOWABLE PRESSURE UNDER SMALL STRUCTURES
BEARING CAPACITY

Reference: Stockwell M.J. (1977). Determination of allowable bearing pressure under small structures. New Zealand Engineering, 132 - 135.

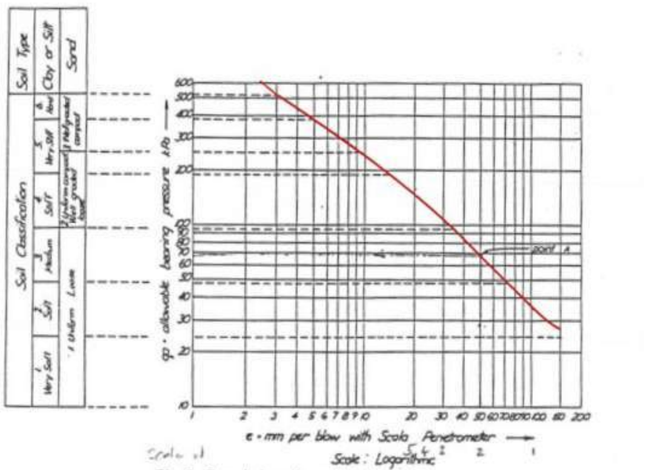


Fig. 2: Correlation of e and qa for design.

Ground Level (mRL)	96				
Depth (mm)	m RL	Measured No. Blows / 100mm	e mm/blow	Stockwell - qa kPa	Stockwell - qu kPa
0 - 100	95.90	8	12.5	195	585
100 - 200	95.80	10	10.0	220	660
200 - 300	95.70	10	10.0	220	660
300 - 400	95.60	16	6.3	260	780
400 - 500	95.50	20	5.0	260	780
500 - 600	95.40	21	4.8	260	780

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-016 Page 1

M.J. STOCKWELL
DETERMINATION OF ALLOWABLE PRESSURE UNDER SMALL STRUCTURES
BEARING CAPACITY

Reference: Stockwell M.J. (1977). Determination of allowable bearing pressure under small structures. New Zealand Engineering, 132 - 135.

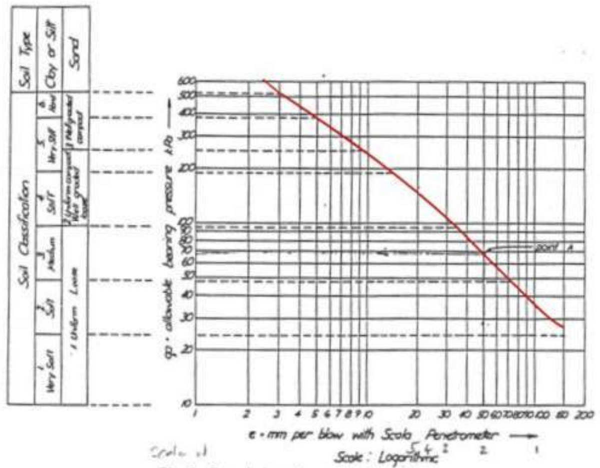


Fig. 2: Correlation of e and qa for design.

Ground Level (mRL)			96.1				
Depth (mm)			**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Stockwell - qa kPa**	**Stockwell - qu kPa**
0	-	100	96.20	5	20.0	135	405
100	-	200	96.10	8	12.5	195	585
200	-	300	96.00	9	11.1	200	600
300	-	400	95.90	10	10.0	220	660
400	-	500	95.80	15	6.7	260	780
500	-	600	95.70	21	4.8	260	780

E

Appendix E – Soakage Memorandum

Memorandum

To:	Dale Johnson	**Date:**	22 March 2024
From:	Jessica Boyd	**Our Ref:**	3160491-1666321878-67460

Copy: Sam Glue; Paul Horrey; Mike Thorley; Chris Hyslop, Kiri Moonen

Subject: ORR06-AR109526-GEO-MEM-Infiltration

This memorandum presents infiltration testing data from testing conducted at 8 and 10 Orr Street, (AR109526, Ashburton) on 28/02/2024.

It is a request by Ashburton District Council, that the proposed structures provide stormwater discharge via ground soakage. The testing was completed to support civil design of proposed soakage pits for the site as part of a Kāinga Ora Housing Delivery System residential development at the site.

Infiltration testing was conducted prior to the proposed soak pit design locations being known. The testing locations are shown on the plan included Attachment 1. The infiltration testing took place in machine excavated infiltration test pits within the target strata encountered from depths of 0.3 m bgl. The target strata comprise loosely and densely packed gravel. Target depths of the test pits were 2.0 m bgl.

Ashburton District Council have communicated infiltration to ground is preferred to stormwater attenuation (storage) options.

1.1 Infiltration Testing

Infiltration testing was carried out in accordance with the Ministry of Business, Innovation and Employment (MBIE) Acceptable Solutions and Verification Methods E1/VM1 (Surface Water), Section 9.0.2 (2017).

Testing was conducted at a depth of 2.0m and 2.15m bgl within loosely and densely packed gravel using the falling head test method in ITP01 and ITP02 respectively. The excavated test pits were filled with potable water to within 0.60 and 0.75 m above the base of the test pits due to pit wall stability in ITP01 and ITP02 respectively. Pre-soak lasted for 40 to 45 mins due to limited water.

Infiltration testing was conducted with the results summarised in Table 1-1, and detailed infiltration testing results are presented in Attachment 2. The infiltration rates presented are based on the range of field measurements observed without a design factor applied.

Table 1-1: Infiltration Testing Summary

Test Pit ID	Test Date	Measurement Intervals (sec)	Test Depth (m bgl)	Water Level at Start (m above base)	Minimum Measured Infiltration Rate (mm/hr)
AR109526-ITP-01	28/02/2024	20 - 120	2.15	0.58	1215
AR109526-ITP-02	28/02/2024	30 - 600	2.00	0.75	90[1]

1) We note that while the minimum measured infiltration rate is low for ITP-02.

1.2 Recommended Infiltration Rates

The observed infiltration rates are based on the field measurements without a design factor applied. Infiltration testing within the test pits included a component of horizontal (radial) infiltration in addition to vertical infiltration (i.e., the measured overall rate is higher than the vertical infiltration rate alone). The minimum stabilised infiltration results, which typically occur towards the end of the soakage test and are largely reflecting vertical flow with minimal hydraulic gradient effects, are recommended to be used for infiltration basin design purposes.

A mounding assessment was not considered to be required due to the measured depth to groundwater exceeding 2.5 m bgl during site testing (February 2024) considering publically available data (refer Kāinga Ora report ref: ORR06-AR109526-GEO-RP-Geotechnical Design Report). Site investigations (hand augers, February 2023) did not encounter groundwater to a depth of 1.3 m bgl (terminated on gravels).

The assessment of design rate is based on the minimum stabilised measured rate during the testing, or the minimum rate if a stabilised rate was not achieved.

The layouts of the soak pits are shown in plan AR109526-CV-111 REVB. Houses 1 & 2 are in the vicinity of ITP1 and that result should be used for soak pit design in that area. The remaining soak pits should consider the lower rate from ITP2, although noting the result is markedly lower than ITP1 and we do not consider ITP2 to representative of the infiltration capacity of the strata across site and in this area of Ashburton. The difference between the two infiltration tests is attributed to the lithology differences and depth of the test pits (ITP2 was shallower and located in a denser gravel which isn't consistently found elsewhere on site), and/or fines accumulating during the pre-soak period. It is recommended that the civil design use the average of the tested rates from ITP1 and ITP2 for the remaining areas of the site and that all the soak pit depths are 2.5m depth. Further testing at each soak pit will be required during construction and the soak pit sizing and/or depths are adjusted to ensure the design assumptions are met.

For soakage pit design purposes, we recommend a design factor of safety be applied to the minimum measured infiltration rate as recommended in the Stormwater Soakage and Groundwater Recharge in the Auckland Region (Auckland Council, 2021). We recommend the following factors as per the guidance (refer Attachment 3 for an extract of the guidance regarding consequence and testing):

- Consequence of Failure FoS (F_c): A consequence level of 2 is recommended for the soak pits in the JOAL due to the direct connection to the secondary overflow path. It is recommended that a consequence level 3 is used for the individual house soak pits.
- Testing Quality FoS (F_u): a quality level 4 is recommended, as the testing was conducted on site, but not at the location of each of the proposed soakage pit locations.

The above factors are based on discussion with Kāinga Ora civil engineers, confirming that the following measures are to be incorporated into the stormwater and soakage pit design;

- JOAL overflow of the soak pit system will backup and discharge at the sump as the lowest point, with secondary flow path via the accessway to the road (in accordance with the Stormwater Code of Practice).
- (Pre-treatment) Litter traps and leaf separators on the downpipes as anti-clogging measures
- (Pre-treatment) Type 2 sump in the accessway with submerged outlet and silt trap
- (Maintenance) A manhole will be incorporated into the soak pit for maintenance access, also accessing the silt trap
- Recommend maintenance to the client for the silt traps and measures on down pipes

Considering the tested rates and factors of safety, the recommended maximum infiltration rate for each location is summarised in Table 1-2. The final selection of design infiltration rates and factors of safety is subject to Civil Design considerations and may differ from those described herein and summarised below.

Table 1-2: Maximum Recommended Design Infiltration Rates

Test Pit ID	Minimum Observed Infiltration Rate (mm/hr)	Consequence of Failure FoS (F_c) [1]	Testing Quality FoS (F_u) [1]	Recommended FoS [1] (F_c x F_u)	Factored Infiltration Rate (mm/hr)
House1-2	1215	2.5	2.4	6	203
Houses rest of site	652[2]	2.5	2.4	6	109
JOAL	652[2]	1.5	2.4	3.6	181

Note: (1) Stormwater Soakage and Groundwater Recharge in the Auckland Region (Auckland Council, 2021)
(2) Average of the two ITP tests to account for infiltration testing variability

1.3 Construction Recommendations

The following general construction recommendations are based on the site investigation, testing, and analysis, as well as our experience with similar construction:

- Soak pits must terminate at least 0.5m within the loose sandy gravel. We recommend a minimum soak pit depth of 2.5 m bgl considering soil observations during site testing.
- The pre-treatment, maintenance access and maintenance plan, and secondary overflow options confirmed by Kāinga Ora civil engineers (refer section 1.2) must be incorporated into the stormwater design to meet the assumptions uses to determine the factored infiltration rates. Final selection of FoS and design infiltration rate to be made by Kāinga Ora civil engineers.
- A maintenance plan should be communicated to Kāinga Ora as per agreement with the Kāinga Ora civil engineer. Failure to maintain the system may result in compromised performance.
- Groundwater soakage pit bases should typically be above the groundwater table, as the depth to groundwater will affect infiltration performance.
- Seasonal groundwater variations and groundwater mounding may reduce infiltration performance depending on the depth to groundwater and duration of the discharge.
- We recommend that suitably uniform graded drainage material be used within the proposed infiltration system in accordance with building code recommendations.
- We do not recommend the use of filter cloth on the base of the infiltration system or around subsoil drains as they may clog over time and will be difficult to maintain. Filter cloth may be used on the sides and top of the infiltration system.
- Subsoil drainage materials should be reviewed by the project Geotechnical Engineer or Hydrogeologist
- Infiltration testing at each of the soakage pit(s) is/are recommended to determine a site-specific infiltration rate at each soak pit and the Civil Design assumptions of the soak pits are checked and

adjustments made as required. These risks and testing requirements should be communicated to the Kāinga Ora relevant project teams.

A minimum setback distance of 3 m is recommended for buildings and property boundaries according to the Stormwater Soakage and Groundwater Recharge in the Auckland Region (Auckland Council, 2021), unless advised by a Geotechnical Engineer.

Jessica Boyd

Engineering Geologist

Phone Number: +64 (3) 366 3521

Email: Jess.boyd@beca.com

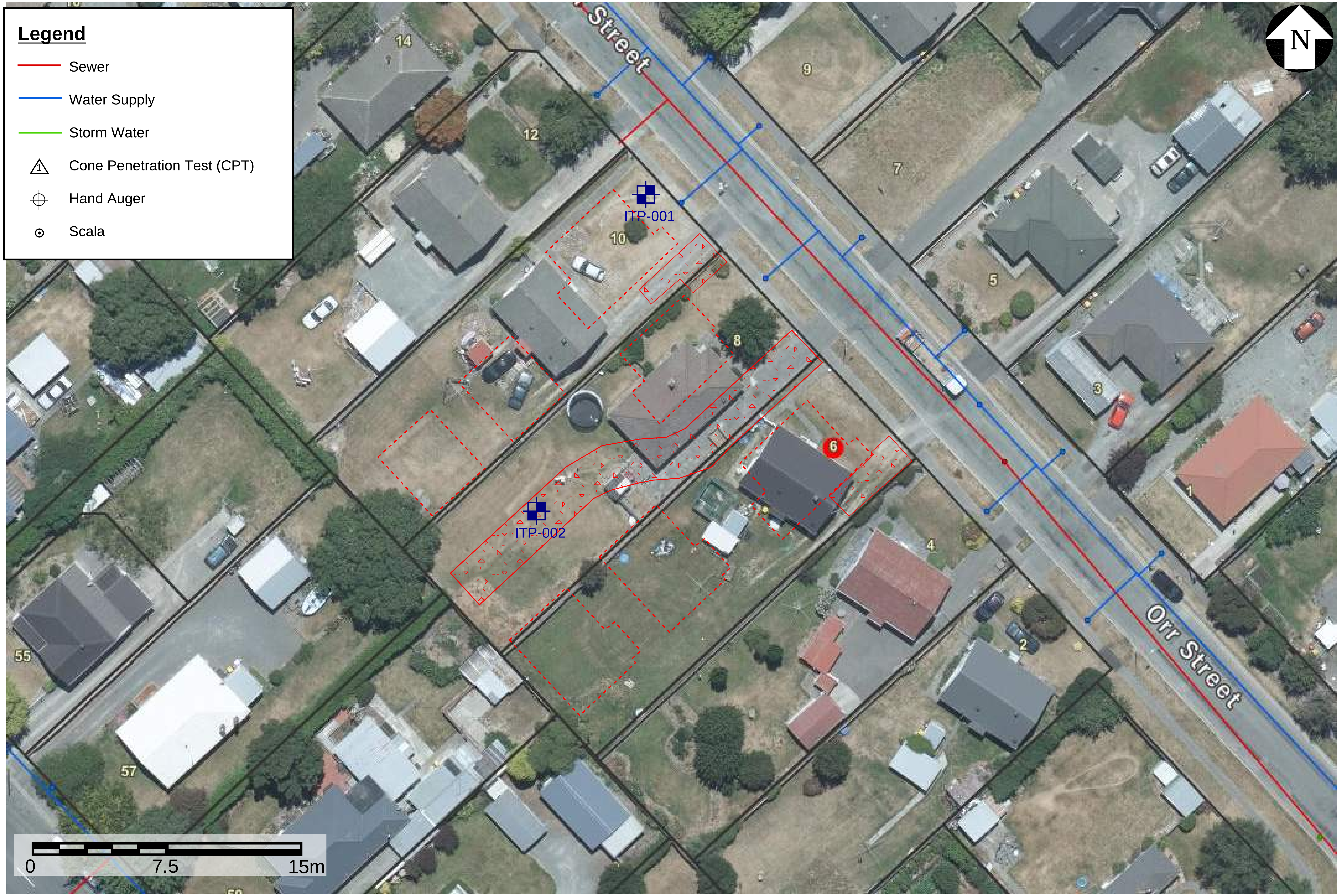
Mike Thorley

Technical Director - Hydrogeology

Phone Number: +64 (3) 366 3521

Email: Mike.Thorley@beca.com

Attachment 1: Testing Location Plan



No.	Revision	By	Chk	Appd	Date

Drawing Originator: Beca

Original Scale (A1)	Design		
	Drawn		
Reduced Scale (A3)	Dsg Verifier		
	Dwg Check		

* Refer to Revision 1 for Original Signature

Client: Kāinga Ora Homes and Communities

Project:
Housing Delivery System
MBU4

Title:
GROUND INVESTIGATION PLAN
6 - 10 ORR STREET (ASHBURTON)

Discipline
GEOTECHNICAL

Drawing No.

Rev.

BLUEBEAM

DO NOT SCALE

IF IN DOUBT ASK.

Geotechnical Log Key Sheet

SOIL AND ROCK DESCRIPTIONS

Soil and Rock Descriptions are in general accordance with the NZ Geotechnical Society (NZGS), 2005.
Hand-held Vane Shear Strength measurements are in general accordance with the NZGS, 2001.

METHODS

BH	Machine Borehole
CPT	Cone Penetration Test
DCP	Dynamic Cone Penetration
HA	Hand Auger
SPT	Standard Penetration Test
IVAN	In-situ Vane Test
MA	Machine Auger
OB	Open Barrel
SNC	Sonic Core Drilling
TP	Test Pit/Trench
TT	Triple Tube
PT	Thin-walled Open Drive Tube
VE	Vacuum Excavation
W	Wash Boring

WEATHERING

CW	Completely Weathered
HW	Highly Weathered
MW	Moderately Weathered
SW	Slightly Weathered
UW	Unweathered

SAMPLES

B	Bulk Disturbed Sample
C	Core Sample
D	Small Disturbed Sample
PT	Thin-wall Open Drive (Push) Tube Sample

WATER

▼	Groundwater Level (GWL)

IN-SITU TESTS

Shear Vane

Su	In-situ peak undrained shear strength and remoulded undrained shear strength
UTP	Unable to Penetrate
CB	Pilcon-type vane tested in Core Barrel
DH	Pilcon-type vane tested in-situ (downhole)
GV	Geonor vane, tested in-situ
IcV	Icone vane, tested in-situ

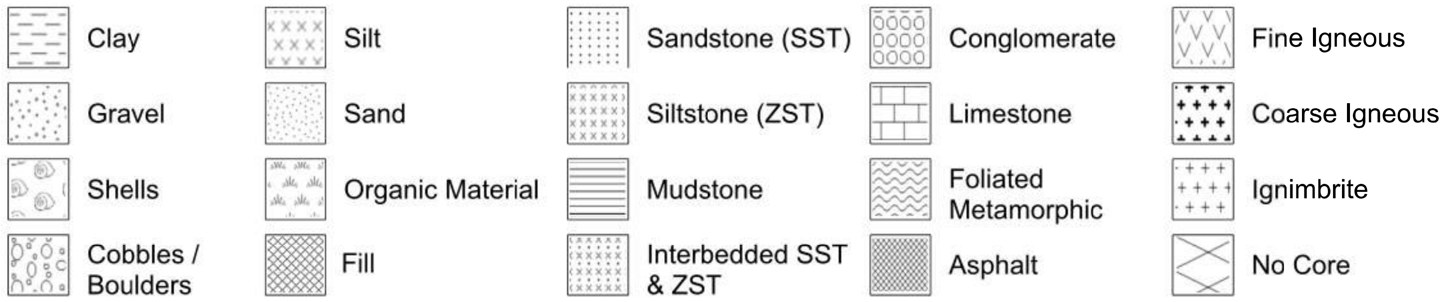
Standard Penetration Test (SPT)

N	SPTn Sampler (Split-spoon)
Nc	SPTn Solid Cone
HB	SPT Hammer Bouncing

TERMINOLOGY

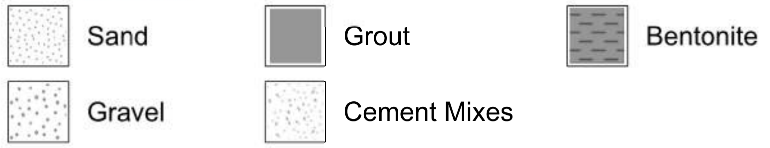
RL	Relative Ground Level
RQD	Rock Quality Designation

GRAPHIC LOG (1 or a combination of the following)

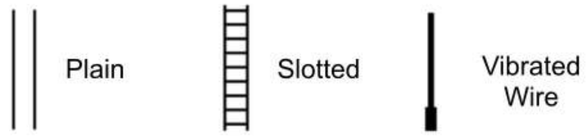


MONITORING INSTALLATION

Backfill Material



Standpipe



ORGANIC SOILS

Von Post Degree of Humidification

H1	Completely unconverted and mud-free peat, when pressed gives clear water and plant structure is visible.
H2	Partially unconverted and mud-free peat, when pressed gives almost clear water and plant structure is visible.
H3	Very slightly decomposed or very slightly muddy peat, when pressed gives marked muddy water, no peat substance passes through the fingers and plant structure is less visible.
H4	Slightly decomposed or slightly muddy peat, when pressed gives muddy water and plant structure is less visible.
H5	Moderately decomposed or very muddy peat with growth structure evident but slightly obliterated.
H6	Moderately decomposed or very muddy peat with indistinct growth structure.
H7	Fairly well decomposed or very muddy peat but the growth structure can just be seen.
H8	Well decomposed or very muddy peat with very indistinct growth structure.
H9	Practically decomposed or mud-like peat in which almost no growth structure is evident.
H10	Completely decomposed or mud peat where no growth structure can be seen, entire substance passes through the fingers when pressed.

Test Pit Log

Test Pit ID: **ITP-001**

Sheet 1 of 1

Project: HDS - 6-10 Orr Street

Project number: 3160491/AR109526

Site location: 6-10 Orr Street, Netherby, Ashburton

Client: Kāinga Ora

Location: 10 Orr Street, Front yard

Coordinate system: NZTM2000

Vertical datum: NZVD 2016

Northing: 5138452.0

Ground level (mRL): 96.00

Easting: 1500796.0

Location method: Canterbury Maps

Groundwater (m)	In Situ Tests		Samples	Depth (m)	RL (m)	Graphic Log	Soil/ Rock Description	Geological Unit
	Su (kPa)	Scala blows/50mm						
							SILT, some fine sand, some organics; light greyish brown; moist. Organics: roots and rootlets. [TOPSOIL]	Topsoil
				0.5	95.5		Tightly packed, fine to coarse sandy, fine to coarse GRAVEL, trace cobbles, trace silt; light greyish brown; dry, well graded. Gravel and cobbles: subrounded to subangular, SW to UW, greywacke.	Quaternary Alluvium
				1.0	95.0		Loosely packed, fine to coarse sandy, fine to coarse GRAVEL, some silt; light greyish brown; moist, well graded. Gravel: subrounded to subangular, SW to UW, greywacke. *1.10m: trace cobbles*	
							Loosely packed, fine to coarse GRAVEL, minor fine to coarse sand, trace cobbles; light brownish grey; dry, poorly graded. Gravel and cobbles: subrounded to subangular, SW to UW, greywacke.	
				1.5	94.5		Loosely packed, fine to coarse sandy, fine to coarse GRAVEL, some cobbles, trace silt; light brownish grey; moist, well graded. Gravel and cobbles: subrounded to subangular, SW to UW, greywacke. *1.80 - 1.95m: absence of silt*	
				2.0	94.0			
				2.5	93.5		2.50m - End of test pit, Target depth	
				3.0	93.0			
				3.5	92.5			
				4.0	92.0			
				4.5	91.5			

Date started: 28/02/2024

Vane ID: N/A

Vane type: N/A

Vane width: N/A

Face orientation: N/A

Logged by: JB

Contractor: ACL Ltd

Equipment: 3T Airman Hydraulic Excavator

Method: TP

Comments:

Groundwater not encountered. Loose material in pit walls led to ravelling during pre-soak and testing.

For Explanation of Symbols and Abbreviations See Key Sheet

Photo Log

Location ID: **ITP-001**

Sheet 1 of 1

Project:	HDS - 6-10 Orr Street	**Project number:**	3160491/AR109526
Site location:	6-10 Orr Street, Netherby, Ashburton	**Client Name:**	Kāinga Ora

Location:	10 Orr Street, Front yard	**Coordinate system:**	NZTM2000	**Vertical datum:**	NZVD 2016
		Northing:	5138452.0	**Ground level (mRL):**	96.00
		Easting:	1500796.0	**Location method:**	Canterbury Maps

ITP-001 - 0.00mbgl to 2.50mbgl

ITP-001 Arisings - 0.00mbgl to 2.50mbgl

Test Pit Log

Test Pit ID: ITP-002
Sheet 1 of 1

Project: HDS - 6-10 Orr Street
Site location: 6-10 Orr Street, Netherby, Ashburton
Project number: 3160491/AR109526
Client: Kāinga Ora
Location: 8 Orr Street, Back yard
Coordinate system: NZTM2000
Northing: 5138414.0
Easting: 1500784.0
Vertical datum: NZVD 2016
Ground level (mRL): 95.70
Location method: Canterbury Maps

Groundwater (m)	In Situ Tests		Samples	Depth (m)	RL (m)	Graphic Log	Soil/ Rock Description	Geological Unit
	Su (kPa)	Scala blows/50mm						
					95.5		SILT, some fine sand, some organics; light greyish brown; moist. Organics: roots and rootlets. [TOPSOIL] *0.20m: trace fine to coarse gravel*	Topsoil
				0.5			Tightly packed, fine to coarse sandy, fine to coarse GRAVEL, trace cobbles, trace silt; light greyish brown; dry, well graded. Gravel and cobbles: subrounded to subangular, SW to UW, greywacke. *0.60m: loosely packed*	Quaternary Alluvium
					95.0			
				1.0				
					94.5		Loosely packed, fine to coarse GRAVEL, minor fine to coarse sand, trace cobbles; light brownish grey; moist, poorly graded. Gravel: subrounded to subangular, SW to UW, greywacke.	
				1.5			Tightly packed, fine to coarse sandy, fine to coarse GRAVEL, some cobbles, trace silt; light brownish grey; moist, well graded. Gravel and cobbles: subrounded to subangular, SW to UW, greywacke.	
					94.0			
				2.0			2.00m - End of test pit	
					93.5			
				2.5				
					93.0			
				3.0				
					92.5			
				3.5				
					92.0			
				4.0				
					91.5			
				4.5				
					91.0			

Date started: 28/02/2024
Vane ID: N/A
Vane type: N/A
Vane width: N/A
Face orientation: N/A
Logged by: JB
Contractor: ACL Ltd
Equipment: 3T Airman Hydraulic Excavator
Method: TP

Comments:
Groundwater not encountered. Minor ravelling in pit walls during excavation.

For Explanation of Symbols and Abbreviations See Key Sheet

Photo Log

Location ID: ITP-002

Sheet 1 of 1

Project:	HDS - 6-10 Orr Street	**Project number:**	3160491/AR109526
Site location:	6-10 Orr Street, Netherby, Ashburton	**Client Name:**	Kāinga Ora

Location:	8 Orr Street, Back yard	**Coordinate system:**	NZTM2000	**Vertical datum:**	NZVD 2016
		Northing:	5138414.0	**Ground level (mRL):**	95.70
		Easting:	1500784.0	**Location method:**	Canterbury Maps

ITP-002 - 0.00mbgl to 2.00mbgl

ITP-002 Arisings - 0.00mbgl to 2.00mbgl

Attachment 2: Infiltration Testing Results

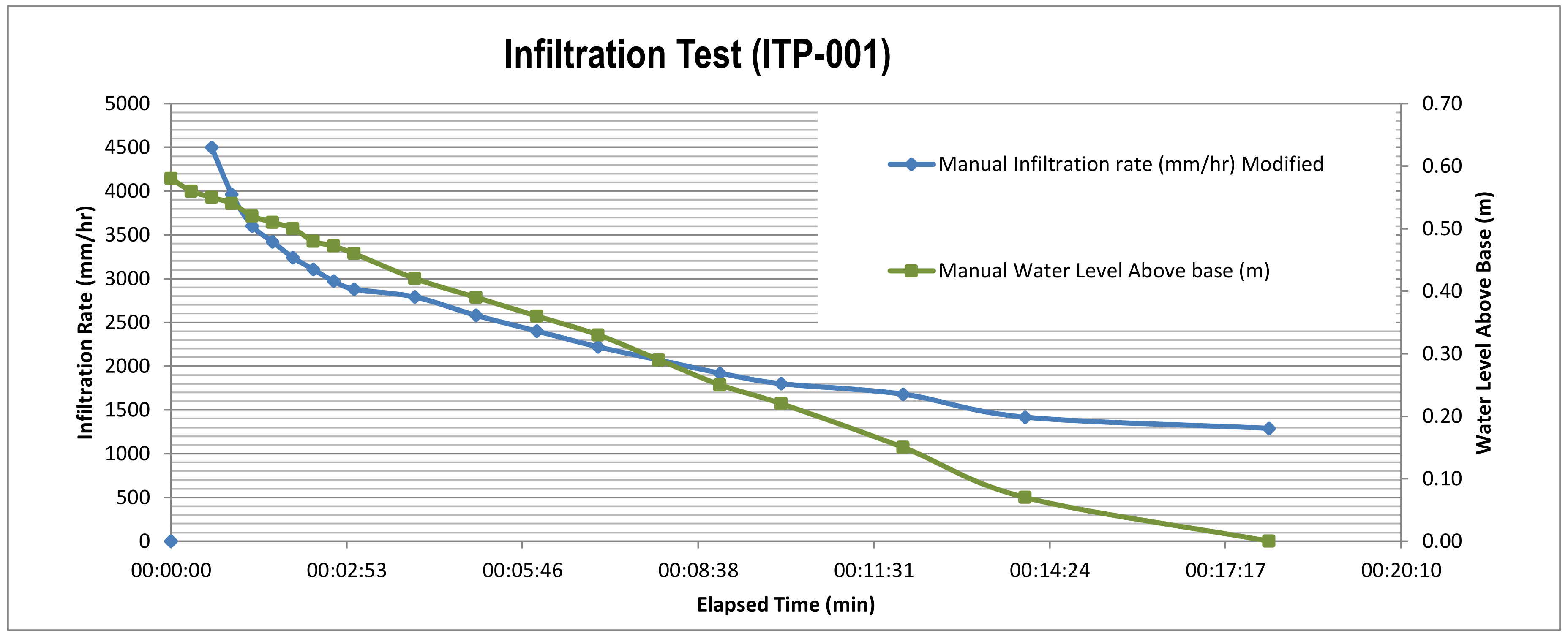
Location ID: ITP-001 (10 Orr Street, Ashburton)
Project Number: AR109526

3160491

Name:	ITP-001
Date Testing:	28/02/2024
Author:	JB
Checked:	

Depth of test pit (m)	2.50
Length of test pit (m)	2.10
Width of test pit (m)	1.50
Area Test pit (m)	7.88
Tape Point to Base Pit (m)	3.940
Timing Interval (seconds)	20

Infiltration (mm/hr)	
Minimum	1215



3.51

Manual Test Readings

Date	Manual Test Time Interval	Manual Test measurement (m)	Manual Water Level Above base (m)	WL bgl (m)	Lapsed Time (min)	Change level (mm)	Change level (mm) Modified	Change time (hr)	Manual Infiltration rate (mm/hr)	Manual Infiltration rate (mm/hr) Modified	Cumulative infiltration (mm)	Comment
28/02/2024	00:00:00	3.36	0.580	1.920	0.00			0.0000			0	
28/02/2024	00:00:20	3.38	0.560	1.940	0.33	20	25	0.0056	3600.0	4500.0	20	
28/02/2024	00:00:40	3.39	0.550	1.950	0.67	10	22	0.0056	1800.0	3960.0	30	
28/02/2024	00:01:00	3.4	0.540	1.960	1.00	10	20	0.0056	1800.0	3600.0	40	
28/02/2024	00:01:20	3.42	0.520	1.980	1.33	20	19	0.0056	3600.0	3420.0	60	
28/02/2024	00:01:40	3.43	0.510	1.990	1.67	10	18	0.0056	1800.0	3240.0	70	
28/02/2024	00:02:00	3.44	0.500	2.000	2.00	10	17.25	0.0056	1800.0	3105.0	80	
28/02/2024	00:02:20	3.46	0.480	2.020	2.33	20	16.5	0.0056	3600.0	2970.0	100	
28/02/2024	00:02:40	3.468	0.472	2.028	2.67	8	16	0.0056	1440.0	2880.0	108	
28/02/2024	00:03:00	3.48	0.460	2.040	3.00	12	15.5	0.0056	2160.0	2790.0	120	
28/02/2024	00:04:00	3.52	0.420	2.080	4.00	40	43	0.0167	2400.0	2580.0	160	
28/02/2024	00:05:00	3.55	0.390	2.110	5.00	30	40	0.0167	1800.0	2400.0	190	
28/02/2024	00:06:00	3.58	0.360	2.140	6.00	30	37	0.0167	1800.0	2220.0	220	
28/02/2024	00:07:00	3.61	0.330	2.170	7.00	30	34.5	0.0167	1800.0	2070.0	250	
28/02/2024	00:08:00	3.65	0.290	2.210	8.00	40	32	0.0167	2400.0	1920.0	290	
28/02/2024	00:09:00	3.69	0.250	2.250	9.00	40	30	0.0167	2400.0	1800.0	330	
28/02/2024	00:10:00	3.72	0.220	2.280	10.00	30	28	0.0167	1800.0	1680.0	360	
28/02/2024	00:12:00	3.79	0.150	2.350	12.00	70	47.25	0.0333	2100.0	1417.5	430	
28/02/2024	00:14:00	3.87	0.070	2.430	14.00	80	43	0.0333	2400.0	1290.0	510	
28/02/2024	00:18:00	3.94	0.000	2.500	18.00	70	76	0.0667	1050.0	1140.0	580	Drained hole terminated

Testing Notes:

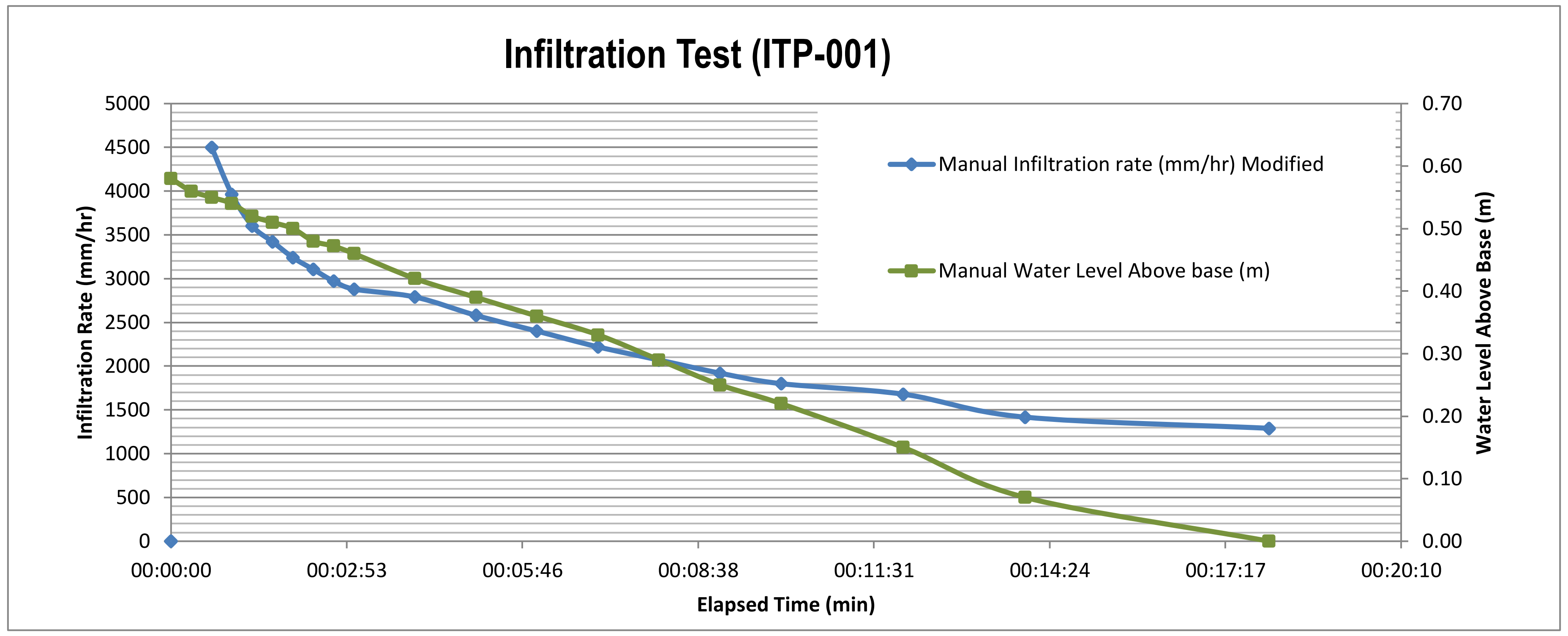
- **9,000 L poured and maintained at 3.47 mbgl for 40mins until ran out of water and let drain until water fully drained from hole 10mins**
- **Due to loose material in the pit walls ravelling was evident. Flow rate was reduced to minimise this but lead to a decrease in the water level.**
- **Test pit depth started at 2.5 m bgl after pre-soak the ravelling caused infill leading to a measured depth of 2.15 m bgl for the start of the test.**
- **10,000 L poured and maintained at 3.34 m bgl for 45 min until ran out of water timed and measured draining of water in hole.**

Location ID: ITP-001 (10 Orr Street, Ashburton)
Project Number: AR109526
3160491

Name:	ITP-001
Date Testing:	28/02/2024
Author:	JB
Checked:	

Depth of test pit (m)	2.50
Length of test pit (m)	2.10
Width of test pit (m)	1.50
Area Test pit (m)	7.88
Tape Point to Base Pit (m)	3.940
Timing Interval (seconds)	20

Infiltration (mm/hr)	
Minimum	1215



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28/02/2024	00:01:00	3.4	0.540	1.960	1.00	10	20	0.0056	1800.0	3600.0	40	
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28/02/2024	00:02:20	3.46	0.480	2.020	2.33	20	16.5	0.0056	3600.0	2970.0	100	
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28/02/2024	00:06:00	3.58	0.360	2.140	6.00	30	37	0.0167	1800.0	2220.0	220	
28/02/2024	00:07:00	3.61	0.330	2.170	7.00	30	34.5	0.0167	1800.0	2070.0	250	
28/02/2024	00:08:00	3.65	0.290	2.210	8.00	40	32	0.0167	2400.0	1920.0	290	
28/02/2024	00:09:00	3.69	0.250	2.250	9.00	40	30	0.0167	2400.0	1800.0	330	
28/02/2024	00:10:00	3.72	0.220	2.280	10.00	30	28	0.0167	1800.0	1680.0	360	
28/02/2024	00:12:00	3.79	0.150	2.350	12.00	70	47.25	0.0333	2100.0	1417.5	430	
28/02/2024	00:14:00	3.87	0.070	2.430	14.00	80	43	0.0333	2400.0	1290.0	510	
28/02/2024	00:18:00	3.94	0.000	2.500	18.00	70	76	0.0667	1050.0	1140.0	580	Drained hole terminated

Testing Notes:

- **9,000 L poured and maintained at 3.47 mbgl for 40mins until ran out of water and let drain until water fully drained from hole 10mins**
- **Due to loose material in the pit walls ravelling was evident. Flow rate was reduced to minimise this but lead to a decrease in the water level.**
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- **10,000 L poured and maintained at 3.34 m bgl for 45 min until ran out of water timed and measured draining of water in hole.**

Attachment 3: Extract of Stormwater Soakage and Groundwater Recharge in the Auckland Region (Auckland Council, 2021);

Consequence of Failure FoS (Fc) 2

Testing Quality FoS (Fu) 4

B.4.0 Factors of safety for soakage device sizing

There are many uncertainties in the design process, not least the assumed soakage rate. Soakage rates may change significantly over time and can vary by orders of magnitude. In addition, failure consequences vary depending upon the device’s design and location. To account for these issues a factor of safety that reduces observed soakage rates needs to be introduced into the design process. When choosing an appropriate factor of safety, engineering judgement, depending upon the consequences of failure and subsequent design uncertainties, is needed. Key risks that are addressed with the factor of safety are:

- Insufficient confidence in input data, e.g., soakage testing
- Insufficient pre-treatment of stormwater inflow into the device
- Difficult access to the proposed device for maintenance
- Frequency of maintenance of proposed device is likely to be low.

The observed soakage rate used in the design process should be divided by the safety factor. The safety factor is generated by multiplying together two partial factors. These are:

- A factor for the consequences of failure, and
- A factor to account for uncertainty in input data.

Equation 1 should be used to calculate the required Factor of Safety ($F_{(total)}$):

$$F_{(total)} = F_{(c)} \times F_{(u)} \qquad \text{Equation 1}$$

Where:

$F_{(total)}$ - Total combined Factor of Safety to be applied

$F_{(c)}$ - Factor of Safety representing the consequences of failure from Table 5

$F_{(u)}$ - Factor of Safety representing testing uncertainty from Table 6

Table 5, which has been adapted and modified from the CIRIA SuDS Manual C753 (Woods Ballard, et al., 2015), shows suggested safety factors for the consequences of failure. Note that the figures are not based on actual observation of performance loss. Table 6 shows suggested safety factors for the uncertainty in input data.

Table 5: Suggested partial factor of safety ($F_{(c)}$) for consequences of failure

Device	Consequences of failure (see table notes for definitions of Consequence Levels)			
	Consequence Level 1	Consequence Level 2	Consequence Level 3	Consequence Level 4
Soakpit	1	1.5	2.5	5
Groundwater recharge pit	1	1	Not acceptable	Not acceptable
Rockbore	1	1.5	2.5	5

- **Consequence Level 1**: The secondary flow path complies with the Stormwater Code of Practice and all of the following apply:
    - Pre-treatment will be present
    - Access for maintenance will be easy, frequency of maintenance will be high, and a maintenance plan will be implemented.

- **Consequence Level 2**: The secondary flow path complies with the Stormwater Code of Practice and one **or** more of the following applies:
    - Pre-treatment will be present
    - Access for maintenance will be easy, frequency of maintenance will be high, and a maintenance plan will be implemented.

- **Consequence Level 3**: The secondary flow path does not meet the Stormwater Code of Practice but will only cause minor damage to external areas, or non-habitable floor flooding (e.g., surface water on car parking), and one or more of the below points applies:
    - Pre-treatment will be present
    - Access for maintenance will be easy, frequency of maintenance will be high, and a maintenance plan will be implemented.

- **Consequence Level 4:** Any other scenario, including all situations where the secondary flow path is likely to cause damage to buildings or structures, or major flooding of roads.

Table 6: Suggested partial factor of safety ($F_{(u)}$) for uncertainty in input data

Testing situation	Testing quality (see table notes for definitions of Quality Levels)			
	Quality Level 1	Quality Level 2	Quality Level 3	Quality Level 4
Falling head test in soil	1.2	1.4	1.8	2.4
Constant head test in soil	1.0	1.2	1.5	2.0
Rockbore test	1.0	1.2	1.5	2.0

- **Quality Level 1**: All of the following apply:
    - Test undertaken at the location and depth of the proposed device
    - Test undertaken at a time when groundwater is at an annual high. For rock bores, this must be after heavy rain at a time when the rainfall-induced groundwater level peak is likely to be present
    - Groundwater monitoring with a duration of over 12 months and measurements taken in winter and summer is available within 100 m of the proposed device. For rockbore tests, this must include monitoring at short intervals (1 hour or less) to identify short-term response to heavy rainfall.

- **Quality Level 2:** All of the following apply:
    - Test undertaken at the location and depth of the proposed device
    - Test undertaken at a time when groundwater is likely to be at an annual high. For rock bores, this must be after heavy rain at a time when the rainfall-induced groundwater level peak is likely to be present.
- **Quality Level 3:** One of the following apply:
    - Test undertaken at the location and depth of the proposed device, but at a time of year when the groundwater may be lower than the seasonal high
    - Test undertaken at a time when groundwater is likely to be at an annual high, but not at the exact device location. For this to apply, the test must be in a location where the geological and hydrogeological conditions are expected to be the same as the actual proposed device location, and no more than 10 m (horizontally) and 1 m (vertically) from the actual proposed device location.
- **Quality Level 4:** Any other scenario. The designer will still have to demonstrate that the testing is representative of the proposed device location.

F

Appendix F – Retaining Wall Memorandum

Memorandum

To:	Kāinga Ora	**Date:**	7 September 2023
From:	David Dobson, Kiri Moonen	**Our Ref:**	3160491-1666321878-35110
Copy:	Sam Glue; Paul Horrey; Oliver Rees		
Subject:	GEO-MEM-STD Timber Retain Design		

This memorandum presents a standardised timber pole retaining wall design to meet the requirements of Kāinga Ora HDS residential sites. This design is applicable for the following towns:

- Rotorua
- Christchurch
- Dunedin
- Timaru
- Invercargill

The purpose of the standardised timber retaining wall design is to supply a conservative fit for purpose solution which can be applied to residential sites developed in the Housing Delivery System (HDS) for retaining structures up to 1.5m high. The standardised design is specified to meet the stability requirements as per current New Zealand Codes and Standards.

1.1 Design Basis

The design basis was progressed iteratively to deliver an optimum solution considering conservative assumed geotechnical parameters and load cases that will cover a large variety of sites. Target factor of safety (FoS) and allowable deformations are specified according to current New Zealand Codes and Standards. The following assumptions were used in the generic design of the retaining wall:

- Wall heights designed for <0.5m, <1.0m, <1.5m and <2.0m retained height.
- House load of 15kPa founded 1.0m behind top of wall. (Factored permanent load)
- Driveway traffic load case of 12kPa acting 0.3m behind top of wall. (Live traffic load for emergency vehicles)
- 1.8m high fence on top of wall with a wind load of 1.0kPa to create an equivalent bending moment of 3.8kNm/m at top of wall. (Wind live load)
- Loss of toe support from a 0.5m deep trench excavation directly in front of wall. (Temporary excavation)
- Groundwater from 1.0mbgl with a short-term elevated groundwater case at ground level. Drainage will be installed behind the wall at front ground water level to minimise risk of groundwater build up behind the wall.
- Design ULS seismic load of 0.36g using Mononobe-Okabe dynamic loads on wall, triangular distributed load is approximated to 2 point loads in Wallap (see Mononobe-Okabe equation spreadsheets in Attachment C).
- Angle of wall friction of 2/3 of the angle of soil friction.
- The ground is flat in front and on top of the wall.
- Moderately conservative founding non-cohesive soils and increased density retaining non-cohesive soils assumed for the design.
- Wall to have a 1H:20V raked pile profile, however, a vertical wall has been considered for design.
- Designed in accordance with MBIE Module 6: Earthquake resistant retaining wall design guidance document using WALLAP design software with the subgrade reaction model method.

- Displacements are reset after the wall and permanent loads are added for all load cases to confirm the additional displacement from the new loads only. For total wall displacements, the load case displacement should be added to the load case 1 displacement.

The standardised timber retaining wall design considers conservative backfill and soil founding units, which have been specified according to observed soil types on sloping Kāinga Ora sites typically associated with retained solutions. The design assumes a flat backslope which may be surcharged depending on the residential scenario considered. The assumed geotechnical parameters are presented within Table 1-1.

Target FoS for local stability and load factors applied for bending moment and shear capacities are summarised in Table 1-2.

Static and dynamic load cases are summarised in Table 1-3 and consider typical HDS residential plans with regard to worst case surcharge location of houses and driveways.

Table 1-1: Geotechnical Parameters Summary

Unit I.D.	Unit Weight (kN/m3)	Friction Angle (deg, °)	Effective Cohesion (c)	Youngs Modulus of Elasticity (E) (MPa)	Note
Backfill Unit	22	30	0	50	Flat backslope
Founding Unit	18	30	0	10	-

Table 1-2: Target Factor of Safety and Load Factors Summary

Load Case	Target Local Stability Factor of Safety[1]	Bending Moment and Shear Capacity Load Factor[2]	Deflection Limit (mm)[3]	
			Static	ULS
Design Case 1 – Permanent Loads	1.5	1.5	25	-
Design Case 2 – Fence Wind Load	1.2	1.2	25	-
Design Case 3 – Temporary Excavation	1.2	1.2	25	-
Design Case 4 – Raised Groundwater	1.2	1.2	25	-
Design Case 5 – Driveway Load	1.5	1.2	25	-
Design Case 6 – ULS Seismic Load	1.2	1.0	25	100

Notes;

1 Recommended target factor of safety as per MBIE Earthquake Geotechnical Engineering Practice Module 6: Earthquake resistant retaining wall design considering pseudo-static assessment of ground stability (although this predominantly refers to global stability, it has been adopted here for local stability cases)

2 Bending moment and shear load factors applied to assessed loads consider possible site specific variation from the standardised design assumptions.

3 Acceptable deformation according to MBIE Earthquake Geotechnical Engineering Practice Module 6: Earthquake resistant retaining wall design considering a type 3 retaining wall supporting a building foundation. Recommended SLS values have been applied for static cases (also equivalent to allowable settlement of structures as per MBIE guidance [25 mm]).

Table 1-3: Static and Dynamic Load Cases Summary

Load Case	Surcharge (kPa)	Load (kN.m/m)	Excavation	Groundwater (m bgl)
Design Case 1 – Permanent Loads	House dead load	-	-	1.0
Design Case 2 – Fence Wind Load	House dead load	Wind load	-	1.0
Design Case 3 – Temporary Excavation	House dead load	-	Temporary services cut	1.0
Design Case 4 – Raised Groundwater	House dead load	-	-	0.0
Design Case 5 – Driveway Load	Driveway traffic	-	-	1.0
Design Case 6 – ULS Seismic Load	House dead load	ULS Seismic load	-	1.0

1.2 Standardised Design

Standardised designs were developed for 0.5, 1.0, 1.5 and 2.0 m maximum retained heights based on experience of typical retaining wall demands within the Kāinga Ora HDS scheme to date. Following the iterative design process, standardised retained designs which present the most economic and efficient options to meet the target factor of safety and deflection limitations are summarised within Table 1-4. The tables with summarising the local stability (conducted in the software Wallap) results for each individual load stage and are included in Attachment A, the Mathcad sheets for checking bending and shear capacities can be referred to in Attachment B and ULS earthquake dynamic loading assessments can be referred to in Attachment C (Mononobe Okabe Method).

Table 1-4: Standardised Design Specification

Retained Height	Pole Diameter (mm, SED)	Socket Diameter (mm)	Pole Spacing (m)	Pole Length (m)	Pole Embedment (m bgl)
0.5[1]	150	300	1.2	2.00	1.50
1.0[1]	225	350	1.2	3.50	2.50
1.5[2]	275	400	1.2	5.00	3.50
2.0[2]	425	550	1.1	7.00	5.0

Lagging Specification;

1 Lagging requirement = 150 mm depth, 50 mm minimum thickness

2 Lagging requirement = 150 mm depth, 75 mm minimum thickness

1.3 Construction Specification Notes

The following general construction recommendations are based on the standardised design materials, as well as our experience with similar construction:

- The contractor shall locate and protect all services prior to commencing work and shall inform the engineer should any conflicts the contractor shall be responsible for any damage to services caused by their activities.

- All timber shall be treated to NZS 3604 hazard class H5 as specified. timber poles and posts shall be Radiata pine or Corsican pine. The poles and rails shall be straight and free of decay, knots, splits, checks or any other defect that may affect the strength of the pole.
- All cut timber shall be treated via site application of a suitable product to the supplier's specification to achieve a level of treatment equal to or greater than the member's original level of treatment.
- All poles shall be placed large end into the base of the hole.
- Bored holes shall not remain open overnight. Holes must be thoroughly cleaned out before placing concrete. Poles shall be installed and concreted in a hit and miss pattern within the same day as boring.
- Poles shall be braced (where necessary) during and after concreting such that the required alignment is maintained.
- All steel components shall be hot dipped galvanised in accordance with AS/NZS 4680, to HDG 900 in accordance with AS/NZS 2312.
- Lagging joints shall occur at posts only. Lagging joints shall be staggered between poles. Lagging to be secured to posts with ø4.0mm, 200mm long nails.
- Material for backfilling behind the wall shall be drainage AP40 in accordance with the Beca Geotechnical and Civil Specification (appropriate the specific Kāinga Ora HDS region). Backfill is to be placed and compacted in horizontal layers of max. 200mm layer depth.
- Geotextile shall be BIDIM A19 or equivalent.
- Backfill base drains shall be ø100mm Novocoil (or equivalent) slotted drains with outflow through the base of the wall at strategic points to pavement/driveway or garden areas (excluding walls which support above driveways, which will have curbs for stormwater capture).

We recommend the following table layout is applied for structural construction drawings;

TABLE 1: TIMBER SED RETAINING WALL DESIGN

MAX RETAINED HEIGHT	<2000 mm	<1500 mm	<1000 mm	<500 mm
POLE EMBEDMENT	5000 mm	3500 mm	2500 mm	1500 mm
NORMAL POLE LENGTH (SED)	7000 mm	5000 mm	3500 mm	2000 mm
POLE SIZE (DIAMETER)	425 mm	275 mm	225 mm	150 mm
SOCKET SIZE (DIAMETER)	550 mm	400 mm	350 mm	300 mm
POLE SPACING	1100 mm	1200 mm	1200 mm	1200 mm
LAGGING DIMENSIONS	150 mm x 75 mm (THICK)		150 mm x 50 mm (THICK)	

1.4 Limitations

The presented standardised design is solely for our Client's use for the purpose for which it is intended in accordance with the agreed scope of work (Kāinga Ora HDS residential developments only). Any use or reliance by any person contrary to the above, to which Beca has not given its prior written consent, is at that person's own risk.

The following limitations apply to the proposed design. Should conditions lay out with these conditions, a specific engineered design (SED) shall apply.

- The engineer should consider the site specific conditions in reference to the conservative soil parameters assumed for the standardised design. For example if particularly soft cohesive soils are encountered, an SED may apply.

- Groundwater levels are conservative and assume a standard case level of 1.0 m bgl, with a high groundwater case (4) of 0.5 m bgl. Conditions which exceed these conservative values may require SED.
- Backslope and front slope angle is assumed to be zero (flat) for all cases. Walls supporting slopes above or below the wall will require specific engineering design.
- The design is based on allowable deflection and target FoS as per MBIE Earthquake Geotechnical Engineering Practice Module 6: Earthquake resistant retaining wall design considering a type 3 retaining wall supporting a building foundation. Any other scenario may require specific engineering design.
- Dead loads assume a house surcharge of 15 kPa based on the maximum surcharge to date experienced within the HDS scheme. This typically concerns dual storey light weight cladding options. The house is offset by 1.0 m from the top of the wall.
- Traffic load is assumed to be 12 kPa at an offset of 0.3 m from the top of the wall.
- Wind loads are based on a maximum fence height of 1.80 m from the top of the proposed retaining wall and wind load of 1.0 kPa as specified by the Kāinga Ora HDS structural Engineer.
- For static case 3 (temporary excavation case), we recommend a maximum 3.0 m length of trench open at any given time. Trench sections to be excavated and backfilled on the same day. A maximum trench excavation depth of 0.5 m has been assumed. No trenching or excavation shall be allowed within 0.5 m of the base of the wall (design case based on input from civil engineer)
- Dynamic (seismic) loads assume a ULS load of 0.36g, being the largest PGA to date experienced on the Kāinga Ora HDS project (at Rotorua, Christchurch, Timaru, Dunedin and Invercargill). The standardised designs assume the wall supports an importance level 2 (IL2) structure with a design life of 50 years.
- All cases assume a residential setting. Walls which support public access, public roads or other public infrastructure may require SED in alignment with appropriate codes and standards.

Kiri Moonen

Geotechnical Engineer

Email: Kiri.Moonen@beca.com

Sam Glue

Senior Associate - Geotechnical

Email: Sam.Glue@beca.com

Attachment 1: Local Stability Assessments

```
                                                       Units: kN,m
INPUT DATA

SOIL PROFILE
Stratum   Elevation of    ------------------ Soil types -------------------
  no.    top of stratum    Left side                Right side
   1          0.00       1  Founding unit (std s   1  Founding unit (std s
   2         -5.00       1  Founding unit (std s   1  Founding unit (std s

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  Founding     18.00     10000    0.500     OC     0.294    4.369
    unit (std s                             (0.300) (0.000) ( 0.000)
 2  Back Fill..  22.00     50000    0.500     OC     0.294    4.288
    (    0.00 )          ( 0.3000)          (0.300) (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  Founding unit (std s    33.03    0.000    0.00    30.00     0.500    0.00
 2  Back Fill (std spec)    33.03    0.000    0.00    30.00     0.464    0.00

GROUND WATER CONDITIONS
 Density of water = 10.00 kN/m3
                                     Left side     Right side
 Initial water table elevation         -1.00          -1.00

 Automatic water pressure balancing at toe of wall : Yes


WALL PROPERTIES
                         Type of structure = Soldier Pile Wall
                        Soldier Pile width =  0.50 m
                      Soldier Pile spacing =  1.10 m
               Passive mobilisation factor =  3.00
                  Elevation of toe of wall = -5.00
             Maximum finite element length =  0.40 m
                  Youngs modulus of wall E = 7.8520E+06 kN/m2
               Moment of inertia of wall I = 1.4559E-03 m4/m run
                                           = 1.6015E-03 m4 per pile
                                       E.I = 11432 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      Not defined
            2      Not defined
   3      2.00        0          3.800        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     2.00    1.00(L)    9.00      9.00     15.00     =         0     N/A

    Note: L = Left side,  R = Right side
```

CONSTRUCTION STAGES

```
Construction   Stage description
  stage no.    --------------------------------------------------------
      1        Fill to elevation 2.00 on LEFT side with soil type 2
      2        Apply surcharge no.1 at elevation 2.00
      3        Change EI of wall to 11432 kN.m2/m run
               Reset wall displacements to zero at this stage
      4        Apply load no.3 at elevation 2.00
```

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) = 100.00 m

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

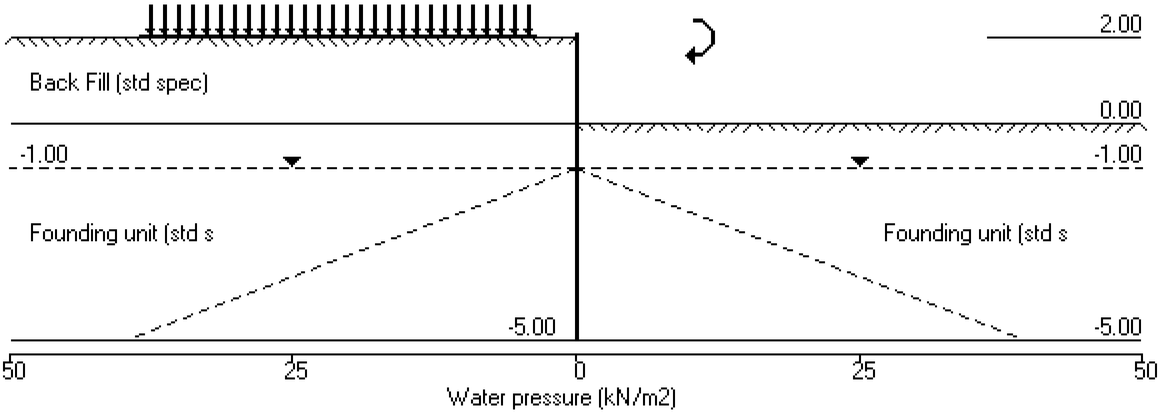
  Distance to rigid boundary on Left side  = 20.00 m
  Distance to rigid boundary on Right side = 20.00 m
```

OUTPUT OPTIONS

```
 Stage ------ Stage description ----------- ------- Output options -------
  no.                                       Displacement   Active,  Graph.
                                            Bending mom.   Passive  output
                                            Shear force   pressures
   1 Fill to elev. 2.00 on LEFT side            Yes          Yes     Yes
   2 Apply surcharge no.1 at elev. 2.00         Yes          Yes      No
   3 Change EI of wall to 11432kN.m2/m run       No           No      No
   4 Apply load no.3 at elev. 2.00              Yes          Yes      No
   * Summary output                             Yes           -      Yes
```

Units: kN,m

Stage No.4   Apply load no.3 at elev. 2.00



```
                                                       Units: kN,m
Stage No. 1   Fill to elevation 2.00 on LEFT side with soil type 2

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    2.00    0.00    Cant.   1.869    -4.53    -3.38   3.38     L to R


BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall
  Analysis options
  Soldier Pile width = 0.50m;  spacing = 1.10m
  Passive mobilisation factor = 3.000
  Length of wall perpendicular to section = 100.00m
  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    2.00      0.00     0.021   5.10E-03      0.0       0.0
  2    1.60      2.58     0.019   5.10E-03      0.5       0.1
  3    1.20      5.17     0.017   5.09E-03      2.1       0.6
  4    0.80      7.75     0.015   5.05E-03      4.6       1.9
  5    0.40     10.33     0.013   4.94E-03      8.3       4.4
  6    0.00     12.91     0.011   4.71E-03     12.9       8.6
  7   -0.40    -16.60     0.009   4.31E-03     12.2      14.0
  8   -0.70    -15.80     0.008   3.90E-03      7.3      17.3
  9   -1.00    -13.26     0.007   3.43E-03      3.0      18.8
 10   -1.30    -10.60     0.006   2.93E-03     -0.6      19.1
 11   -1.60     -8.43     0.005   2.44E-03     -3.5      18.4
 12   -2.00     -5.46     0.004   1.82E-03     -6.3      16.6
 13   -2.40     -1.82     0.004   1.29E-03     -7.7      13.7
 14   -2.80      0.71     0.003   8.75E-04     -7.9      10.5
 15   -3.20      2.37     0.003   5.64E-04     -7.3       7.3
 16   -3.60      3.43     0.003   3.54E-04     -6.2       4.6
 17   -4.00      4.10     0.003   2.31E-04     -4.6       2.4
 18   -4.40      4.57     0.002   1.73E-04     -2.9       0.9
 19   -4.70      4.86     0.002   1.58E-04     -1.5       0.2
 20   -5.00      5.13     0.002   1.55E-04      0.0       0.0
```

```
-----------------------------------------------------------------------------
                                                              (continued)
Stage No.1    Fill to elevation 2.00 on LEFT side with soil type 2

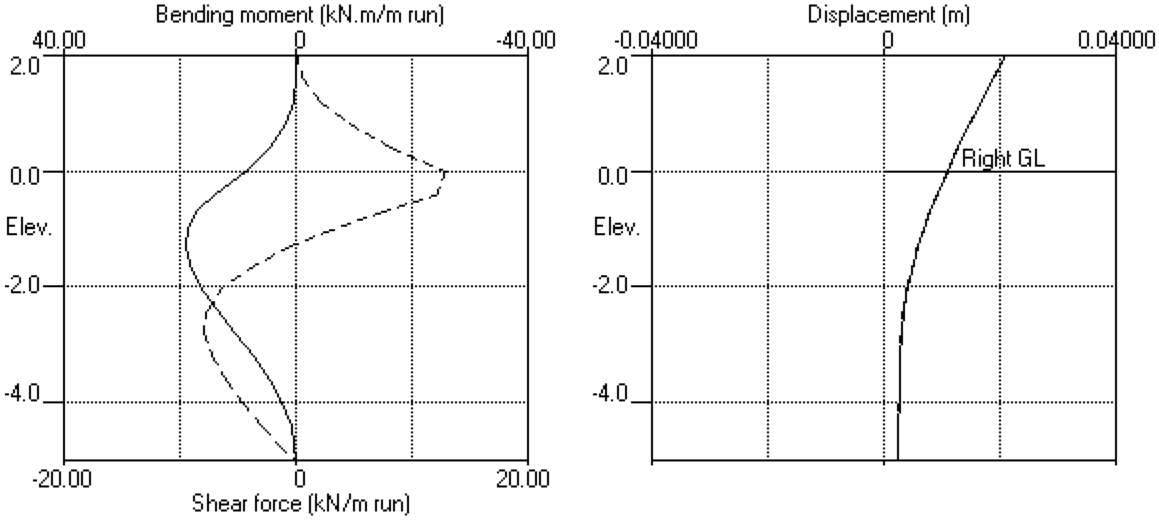
                                         LEFT side                         _
                             Effective stresses
 Node   Y    Water   Vertic  Active   Passive    Earth     Total   Coeff. of
 no.  coord  press.    -al   limit     limit   pressure   pressure  subgrade
                                                                    reaction
              kN/m2   kN/m2   kN/m2     kN/m2    kN/m2      kN/m2     kN/m3
  1    2.00     0.00    0.00    0.00      0.00     0.00      0.00      12669
  2    1.60     0.00    8.80    2.58     39.06     2.58      2.58a     12669
  3    1.20     0.00   17.60    5.17     78.13     5.17      5.17a     12669
  4    0.80     0.00   26.40    7.75    117.19     7.75      7.75a     12669
  5    0.40     0.00   35.20   10.33    156.26    10.33     10.33a     12669
  6    0.00     0.00   44.00   12.91    195.32    12.91     12.91a     12669
                0.00   44.00   12.91    186.91    12.91     12.91a      2534
  7   -0.40     0.00   51.20   15.03    217.49    15.03     15.03a      2534
  8   -0.70     0.00   56.60   16.61    240.43    16.61     16.61a      2534
  9   -1.00     0.00   62.00   18.20    263.37    18.20     18.20a      2534
 10   -1.30     3.00   64.40   18.89    273.48    18.89     21.89a      2534
 11   -1.60     6.00   66.80   19.59    283.59    19.59     25.59a      2534
 12   -2.00    10.00   70.00   20.52    297.07    21.33     31.33       2534
 13   -2.40    14.00   73.20   21.44    310.56    24.51     38.51       2534
 14   -2.80    18.00   76.40   22.37    324.04    27.20     45.20       2534
 15   -3.20    22.00   79.60   23.30    337.52    29.52     51.52       2534
 16   -3.60    26.00   82.80   24.23    351.00    31.58     57.58       2534
 17   -4.00    30.00   86.00   25.15    364.48    33.47     63.47       2534
 18   -4.40    34.00   89.20   26.08    377.97    35.27     69.27       2534
 19   -4.70    37.00   91.60   26.77    388.08    36.60     73.60       2534
 20   -5.00    40.00   94.00   27.47    398.19    37.91     77.91       2534


                                        RIGHT side                         _
                             Effective stresses
 Node   Y    Water   Vertic  Active   Passive    Earth     Total   Coeff. of
 no.  coord  press.    -al   limit     limit   pressure   pressure  subgrade
                                                                    reaction
              kN/m2   kN/m2   kN/m2     kN/m2    kN/m2      kN/m2     kN/m3
  1    2.00     0.00    0.00    0.00      0.00     0.00      0.00        0.0
  2    1.60     0.00    0.00    0.00      0.00     0.00      0.00        0.0
  3    1.20     0.00    0.00    0.00      0.00     0.00      0.00        0.0
  4    0.80     0.00    0.00    0.00      0.00     0.00      0.00        0.0
  5    0.40     0.00    0.00    0.00      0.00     0.00      0.00        0.0
  6    0.00     0.00    0.00    0.00      0.00     0.00      0.00        0.0
                0.00    0.00    0.00      0.00     0.00      0.00       3319
  7   -0.40     0.00    7.20    2.12     31.63    31.63     31.63p      3319
  8   -0.70     0.00   12.60    3.70     55.35    32.42     32.42       3319
  9   -1.00     0.00   18.00    5.29     79.07    31.46     31.46       3319
 10   -1.30     3.00   20.40    5.99     89.63    29.49     32.49       3319
 11   -1.60     6.00   22.80    6.69    100.19    28.02     34.02       3319
 12   -2.00    10.00   26.00    7.62    114.27    26.79     36.79       3319
 13   -2.40    14.00   29.20    8.55    128.35    26.32     40.32       3319
 14   -2.80    18.00   32.40    9.48    142.43    26.49     44.49       3319
 15   -3.20    22.00   35.60   10.41    156.51    27.15     49.15       3319
 16   -3.60    26.00   38.80   11.34    170.59    28.15     54.15       3319
 17   -4.00    30.00   42.00   12.27    184.67    29.37     59.37       3319
 18   -4.40    34.00   45.20   13.21    198.75    30.70     64.70       3319
 19   -4.70    37.00   47.60   13.90    209.31    31.74     68.74       3319
 20   -5.00    40.00   50.00   14.60    219.87    32.78     72.78       3319

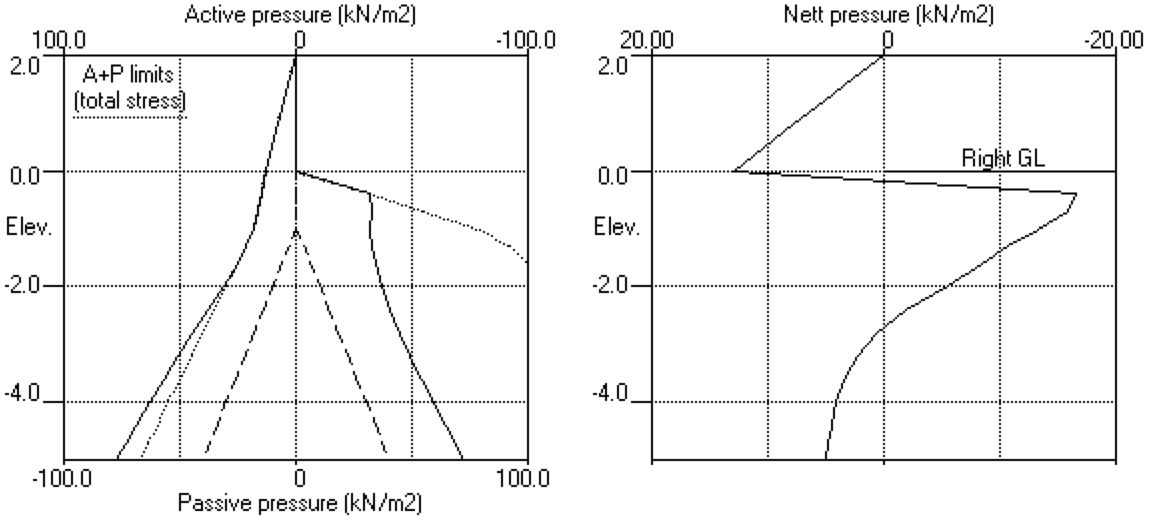
Note:     25.59 a  Soil pressure at active limit
          31.63 p  Soil pressure at passive limit
```

```
                                                       Units: kN,m
```

Stage No.1 Fill to elev. 2.00 on LEFT side



Stage No.1 Fill to elev. 2.00 on LEFT side



```
                                                       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    2.00    0.00    Cant.   1.869    -4.53    -3.38    3.38     L to R
   2    2.00    0.00    Cant.   1.685    -4.43    -3.88    3.88     L to R
   3    2.00    0.00           No analysis at this stage
   4    2.00    0.00    Cant.   1.677    -4.43    -3.95    3.95     L to R
```

```
                                                       Units: kN,m
Summary of results

BENDING MOMENT and DISPLACEMENT ANALYSIS of Soldier Pile Wall
  Analysis options
  Soldier Pile width = 0.50m;  spacing = 1.10m
  Passive mobilisation factor = 3.000
  Length of wall perpendicular to section = 100.00m
  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    2.00     0.003     0.000        3.8       0.0        0.0       0.0
  2    1.60     0.002     0.000        3.9       0.0        0.5       0.0
  3    1.20     0.002     0.000        4.4       0.0        2.2       0.0
  4    0.80     0.002     0.000        5.8       0.0        5.0       0.0
  5    0.40     0.001     0.000        8.5       0.0        9.0       0.0
  6    0.00     0.001     0.000       13.1       0.0       14.2       0.0
  7   -0.40     0.001     0.000       19.2       0.0       14.3       0.0
  8   -0.70     0.000     0.000       23.3       0.0        9.3       0.0
  9   -1.00     0.000     0.000       25.0       0.0        4.1       0.0
 10   -1.30     0.000     0.000       25.2       0.0        0.0      -1.4
 11   -1.60     0.000     0.000       24.1       0.0        0.0      -4.8
 12   -2.00     0.000    -0.000       21.7       0.0        0.0      -8.1
 13   -2.40     0.000    -0.000       18.0       0.0        0.0      -9.9
 14   -2.80     0.000    -0.000       13.8       0.0        0.0     -10.3
 15   -3.20     0.000    -0.000        9.7       0.0        0.0      -9.6
 16   -3.60     0.000    -0.000        6.1       0.0        0.0      -8.1
 17   -4.00     0.000    -0.000        3.2       0.0        0.0      -6.1
 18   -4.40     0.000    -0.000        1.2       0.0        0.0      -3.9
 19   -4.70     0.000    -0.000        0.3       0.0        0.0      -2.0
 20   -5.00     0.000    -0.000        0.0       0.0        0.0       0.0

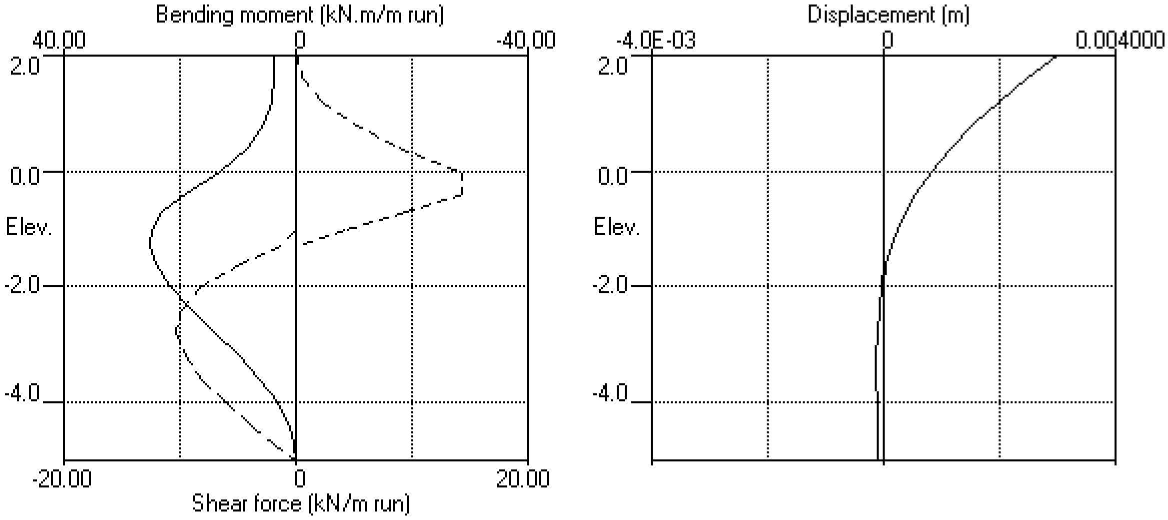
Maximum and minimum bending moment and shear force at each stage
Stage  --------- Bending moment --------   ---------- Shear force ----------
 no.   maximum   elev.   minimum   elev.   maximum   elev.   minimum   elev.
        kN.m/m            kN.m/m             kN/m              kN/m
 1        19.1   -1.30       0.0    2.00      12.9    0.00      -7.9   -2.80
 2        21.9   -1.30       0.0    2.00      14.3   -0.40      -9.2   -2.80
 3     No calculation at this stage
 4        25.2   -1.30       0.0    2.00      14.3   -0.40     -10.3   -2.80

Maximum and minimum displacement at each stage
Stage -------- Displacement ---------
 no.  maximum  elev.   minimum  elev.   Stage description                   _
          m                m
 1     0.021    2.00    0.000    2.00   Fill to elev. 2.00 on LEFT side
 2     0.025    2.00    0.000    2.00   Apply surcharge no.1 at elev. 2.00
 3     Wall displacements reset to zero Change EI of wall to 11432kN.m2/m run
 4     0.003    2.00   -0.000   -3.60   Apply load no.3 at elev. 2.00
```

Summary of results (continued)

```
                                                       Units: kN,m
```

Bending moment, shear force, displacement envelopes



```
                                                       Units: kN,m
INPUT DATA

SOIL PROFILE
Stratum   Elevation of    ------------------ Soil types -------------------
  no.    top of stratum    Left side                Right side
   1          0.00        1  Founding unit (std s   1  Founding unit (std s
   2         -5.00        1  Founding unit (std s   1  Founding unit (std s

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  Founding      18.00     10000    0.500     OC    0.294    4.358
    unit (std s                             (0.300) (0.000) ( 0.000)
 2  Back Fill..   22.00     50000    0.500     OC    0.294    4.288
    (    0.00 )          ( 0.3000)          (0.300) (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  Founding unit (std s    30.00    0.464    0.00     30.00    0.495    0.00
 2  Back Fill (std spec)    30.00    0.464    0.00     30.00    0.464    0.00

GROUND WATER CONDITIONS
 Density of water = 10.00 kN/m3
                                   Left side     Right side
 Initial water table elevation       -1.00           -1.00

 Automatic water pressure balancing at toe of wall : Yes


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 = -4.50
             Maximum finite element length =  0.30 m
                  Youngs modulus of wall E = 7.8520E+06 kN/m2
               Moment of inertia of wall I = 3.3134E-04 m4/m run
                                           = 3.9761E-04 m4 per pile
                                       E.I = 2601.7 kN.m2/m run
                      Yield Moment of wall = Not defined

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     2.00    1.00(L)    9.00      9.00     15.00      =         0     N/A

    Note: L = Left side,  R = Right side
```

CONSTRUCTION STAGES

```
Construction   Stage description
  stage no.    -------------------------------------------------------
      1        Fill to elevation 2.00 on LEFT side with soil type 2
      2        Apply surcharge no.1 at elevation 2.00
      3        Change EI of wall to 2602 kN.m2/m run
               Reset wall displacements to zero at this stage
      4        Excavate to elevation 0.00 on RIGHT side
               Toe of berm at elevation -0.50
               Width of top of berm = 0.50
               Width of toe of berm = 0.60
```

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) = 100.00 m

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

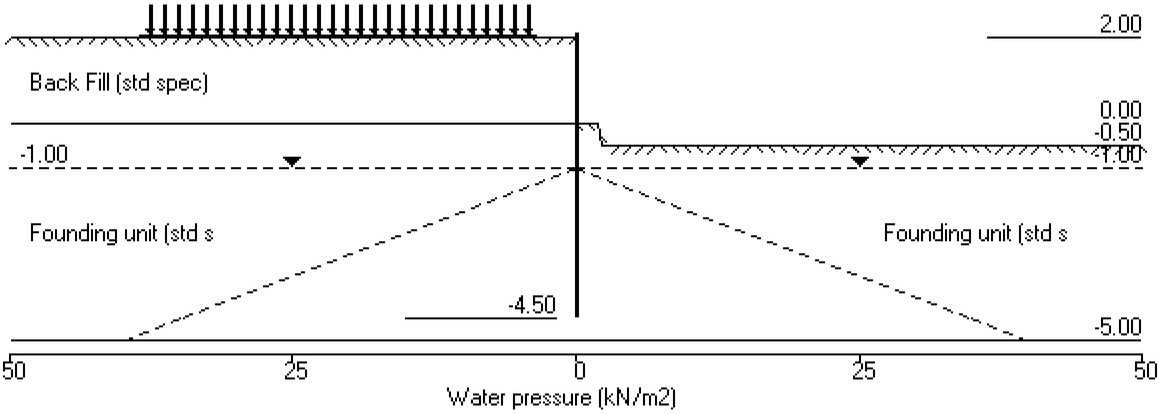
  Distance to rigid boundary on Left side  = 20.00 m
  Distance to rigid boundary on Right side = 20.00 m
```

OUTPUT OPTIONS

```
 Stage ------ Stage description ----------- ------- Output options -------
  no.                                       Displacement   Active,  Graph.
                                            Bending mom.   Passive  output
                                            Shear force   pressures
    1 Fill to elev. 2.00 on LEFT side           Yes          Yes     Yes
    2 Apply surcharge no.1 at elev. 2.00        Yes          Yes      No
    3 Change EI of wall to 2602kN.m2/m run       No           No      No
    4 Excav. to elev. 0.00 on RIGHT side        Yes          Yes     Yes
    * Summary output                            Yes           -      Yes
```

```
BECA LIMITED (NZ)                                            | Sheet No.
Program: WALLAP  Version 6.07  Revision A55.B74.R58          | Job No. 3160491
                             Licensed from GEOSOLVE          | Made by :    KM
Data filename/Run ID: STD-2_300mm_Static_Case_3              |
Std Retaining Wall - 2m - Drained Case 3                     | Date:12-10-2023
Standardised Design                                          | Checked :
-----------------------------------------------------------------------------
                                                      Units: kN,m
```

Stage No.4   Excav. to elev. 0.00 on RIGHT side



```
                                                       Units: kN,m
Stage No. 1   Fill to elevation 2.00 on LEFT side with soil type 2

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. =   -4.50      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    2.00    0.00    Cant.   1.775    -4.07    -3.36    3.36       L to R


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 = 100.00m
  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     2.00     0.00      0.041   1.35E-02      0.0      -0.0
  2     1.75     1.61      0.038   1.35E-02      0.2       0.0
  3     1.50     3.21      0.034   1.35E-02      0.8       0.1
  4     1.20     5.14      0.030   1.35E-02      2.1       0.5
  5     0.90     7.06      0.026   1.34E-02      3.9       1.4
  6     0.60     8.99      0.022   1.31E-02      6.3       2.9
  7     0.30    10.92      0.018   1.27E-02      9.3       5.3
  8     0.00    12.84      0.015   1.19E-02     12.8       8.6
  9    -0.25    -5.58      0.012   1.09E-02     13.7      12.0
 10    -0.50   -24.00      0.009   9.62E-03     10.1      15.1
 11    -0.75   -29.55      0.007   8.07E-03      3.4      17.2
 12    -1.00   -20.42      0.005   6.43E-03     -2.9      17.1
 13    -1.25   -13.04      0.004   4.85E-03     -7.1      15.7
 14    -1.50    -6.19      0.003   3.44E-03     -9.5      13.6
 15    -1.80     0.92      0.002   2.05E-03    -10.3      10.5
 16    -2.10     4.85      0.002   1.01E-03     -9.4       7.5
 17    -2.40     6.51      0.001   3.01E-04     -7.7       4.9
 18    -2.70     6.66      0.001  -1.45E-04     -5.7       2.9
 19    -3.00     5.92      0.001  -3.94E-04     -3.8       1.5
 20    -3.30     4.71      0.002  -5.11E-04     -2.2       0.6
 21    -3.60     3.31      0.002  -5.50E-04     -1.0       0.1
 22    -3.90     1.87      0.002  -5.53E-04     -0.3      -0.1
 23    -4.20     0.43      0.002  -5.48E-04      0.1      -0.0
 24    -4.50    -1.00      0.002  -5.45E-04      0.0      -0.0
```

```
-----------------------------------------------------------------------------
                                                              (continued)
Stage No.1   Fill to elevation 2.00 on LEFT side with soil type 2

                                         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    2.00    0.00    0.00    0.00      0.00     0.00       0.00     15804
  2    1.75    0.00    5.50    1.61     24.42     1.61       1.61a    15804
  3    1.50    0.00   11.00    3.21     48.83     3.21       3.21a    15804
  4    1.20    0.00   17.60    5.14     78.13     5.14       5.14a    15804
  5    0.90    0.00   24.20    7.06    107.43     7.06       7.06a    15804
  6    0.60    0.00   30.80    8.99    136.73     8.99       8.99a    15804
  7    0.30    0.00   37.40   10.92    166.02    10.92      10.92a    15804
  8    0.00    0.00   44.00   12.84    195.32    12.84      12.84a    15804
               0.00   44.00   12.84    188.74    12.84      12.84a     3161
  9   -0.25    0.00   48.50   14.15    208.04    14.15      14.15a     3161
 10   -0.50    0.00   53.00   15.47    227.35    15.47      15.47a     3161
 11   -0.75    0.00   57.50   16.78    246.65    16.78      16.78a     3161
 12   -1.00    0.00   62.00   18.09    265.95    18.09      18.09a     3161
 13   -1.25    2.50   64.00   18.66    274.49    18.66      21.16a     3161
 14   -1.50    5.00   66.00   19.22    283.03    20.77      25.77      3161
 15   -1.80    8.00   68.40   19.89    293.28    24.55      32.55      3161
 16   -2.10   11.00   70.80   20.57    303.53    27.17      38.17      3161
 17   -2.40   14.00   73.20   21.24    313.77    28.98      42.98      3161
 18   -2.70   17.00   75.60   21.92    324.02    30.23      47.23      3161
 19   -3.00   20.00   78.00   22.59    334.27    31.16      51.16      3161
 20   -3.30   23.00   80.40   23.26    344.52    31.92      54.92      3161
 21   -3.60   26.00   82.80   23.94    354.76    32.62      58.62      3161
 22   -3.90   29.00   85.20   24.61    365.01    33.29      62.29      3161
 23   -4.20   32.00   87.60   25.29    375.26    33.97      65.97      3161
 24   -4.50   35.00   90.00   25.96    385.51    34.65      69.65      3161


                                        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    2.00    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  2    1.75    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  3    1.50    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  4    1.20    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  5    0.90    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  6    0.60    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  7    0.30    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  8    0.00    0.00    0.00    0.00      0.00     0.00       0.00       0.0
               0.00    0.00    0.00      0.00     0.00       0.00      5550
  9   -0.25    0.00    4.50    1.32     19.73    19.73      19.73p     5550
 10   -0.50    0.00    9.00    2.64     39.47    39.47      39.47p     5550
 11   -0.75    0.00   13.50    3.96     59.20    46.33      46.33      5550
 12   -1.00    0.00   18.00    5.28     78.94    38.51      38.51      5550
 13   -1.25    2.50   20.00    5.85     87.72    31.70      34.20      5550
 14   -1.50    5.00   22.00    6.42     96.51    26.96      31.96      5550
 15   -1.80    8.00   24.40    7.11    107.05    23.63      31.63      5550
 16   -2.10   11.00   26.80    7.80    117.60    22.32      33.32      5550
 17   -2.40   14.00   29.20    8.49    128.14    22.47      36.47      5550
 18   -2.70   17.00   31.60    9.18    138.68    23.57      40.57      5550
 19   -3.00   20.00   34.00    9.87    149.23    25.24      45.24      5550
 20   -3.30   23.00   36.40   10.56    159.77    27.21      50.21      5550
 21   -3.60   26.00   38.80   11.25    170.31    29.30      55.30      5550
 22   -3.90   29.00   41.20   11.94    180.86    31.43      60.43      5550
```

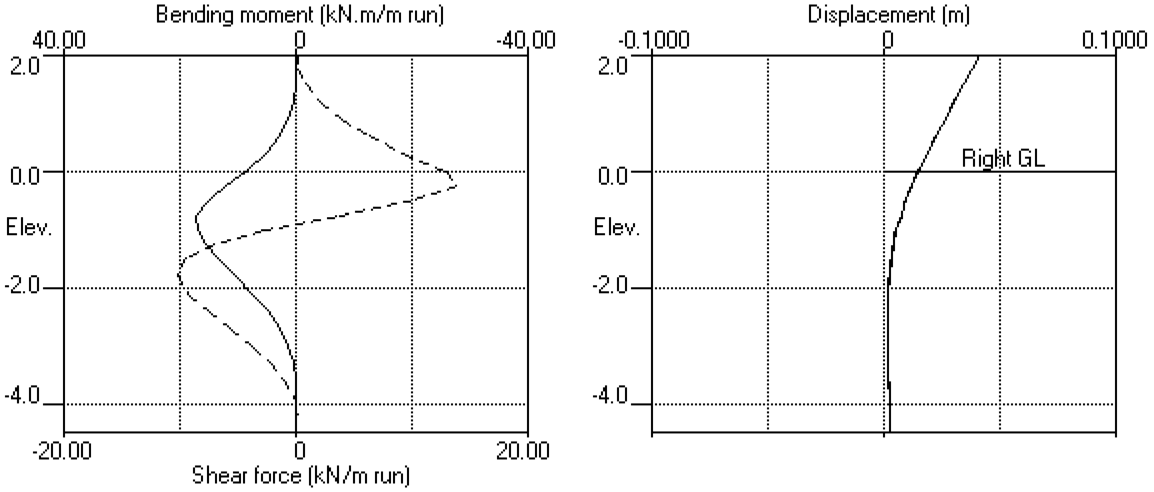
```
                                                              (continued)
Stage No.1   Fill to elevation 2.00 on LEFT side with soil type 2

                                        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
 23   -4.20   32.00   43.60   12.63    191.40    33.54      65.54      5550
 24   -4.50   35.00   46.00   13.32    201.94    35.65      70.65      5550

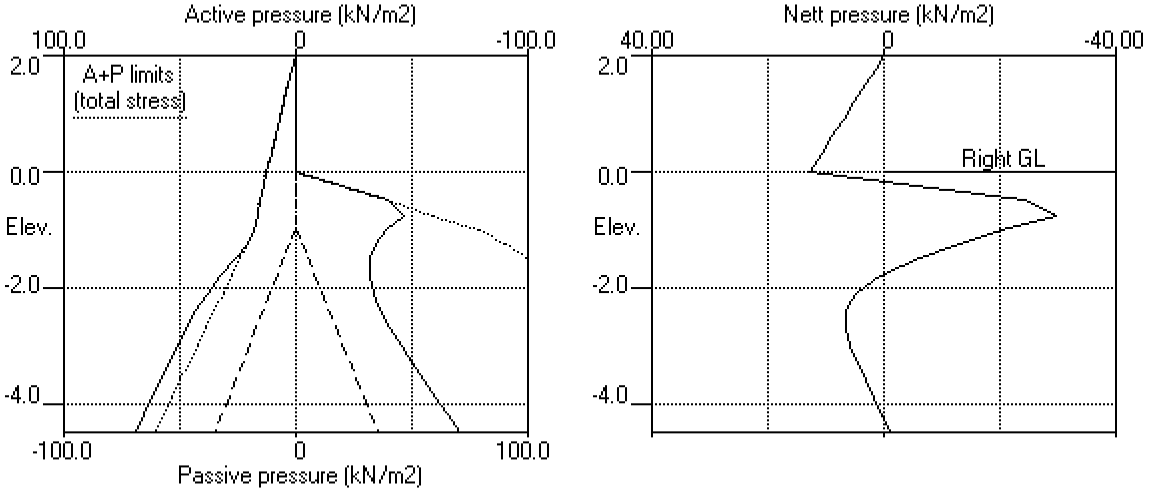
Note:     21.16 a  Soil pressure at active limit
          39.47 p  Soil pressure at passive limit
```

Units: kN,m

Stage No.1 Fill to elev. 2.00 on LEFT side



Stage No.1 Fill to elev. 2.00 on LEFT side



```
                                                       Units: kN,m
Stage No. 4   Excavate to elevation 0.00 on RIGHT side
              Toe of berm at elevation -0.50
              Width of top of berm = 0.50
              Width of toe of berm = 0.60

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. =   -4.50     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
   4    2.00    0.00    Cant.   1.359  -4.23       ***     ***      L to R

   Legend: *** Result not found


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 = 100.00m
  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

    *** Wall displacements reset to zero at stage 3

 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    2.00      0.00     0.026   6.72E-03      0.0      -0.0
  2    1.75      1.65     0.024   6.72E-03      0.2       0.0
  3    1.50      3.42     0.022   6.72E-03      0.8       0.1
  4    1.20      5.71     0.020   6.72E-03      2.2       0.6
  5    0.90      8.06     0.018   6.72E-03      4.3       1.5
  6    0.60     10.39     0.016   6.72E-03      7.0       3.2
  7    0.30     12.66     0.014   6.72E-03     10.5       5.8
  8    0.00     14.86     0.012   6.72E-03     14.6       9.6
                14.89     0.012   6.72E-03     14.6       9.6
  9   -0.25     11.22     0.010   6.72E-03     17.9      13.7
 10   -0.50     -1.98     0.009   6.66E-03     19.0      18.4
 11   -0.75    -24.71     0.007   6.45E-03     15.7      23.3
 12   -1.00    -37.97     0.006   6.02E-03      7.9      26.2
 13   -1.25    -35.22     0.004   5.35E-03     -1.3      27.1
 14   -1.50    -22.64     0.003   4.53E-03     -8.5      25.8
 15   -1.80    -11.71     0.002   3.48E-03    -13.7      22.2
 16   -2.10     -0.80     0.001   2.50E-03    -15.5      17.6
 17   -2.40      6.16     0.000   1.65E-03    -14.7      12.9
 18   -2.70      9.20    -0.000   9.94E-04    -12.4       8.8
 19   -3.00      9.80    -0.000   5.10E-04     -9.6       5.5
 20   -3.30      9.05    -0.001   1.86E-04     -6.7       3.1
 21   -3.60      7.57    -0.001  -8.34E-06     -4.3       1.4
```

```
                                                              (continued)
Stage No.4   Excavate to elevation 0.00 on RIGHT side
              Toe of berm at elevation -0.50
              Width of top of berm = 0.50
              Width of toe of berm = 0.60

 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
 22   -3.90      5.73    -0.001  -1.07E-04     -2.3       0.5
 23   -4.20      3.78    -0.001  -1.43E-04     -0.8       0.1
 24   -4.50      1.79    -0.000  -1.50E-04      0.0      -0.0

                                         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    2.00    0.00    0.00    0.00      0.00     0.00       0.00     13569
  2    1.75    0.00    5.59    1.65     29.45     1.65       1.65a    13569
  3    1.50    0.00   11.60    3.42     61.10     3.42       3.42a    13569
  4    1.20    0.00   19.36    5.71    101.96     5.71       5.71a    13569
  5    0.90    0.00   27.33    8.06    143.96     8.06       8.06a    13569
  6    0.60    0.00   35.22   10.39    185.51    10.39      10.39a    13569
  7    0.30    0.00   42.91   12.66    226.01    12.66      12.66a    13569
  8    0.00    0.00   50.38   14.86    265.36    14.86      14.86a    13569
               0.00   50.38   14.89    208.87    14.89      14.89a     2714
  9   -0.25    0.00   55.46   16.39    229.91    16.39      16.39a     2714
 10   -0.50    0.00   60.41   17.85    250.45    17.85      17.85a     2714
 11   -0.75    0.00   65.27   19.29    270.58    19.29      19.29a     2714
 12   -1.00    0.00   70.04   20.70    290.34    20.70      20.70a     2714
 13   -1.25    2.50   72.23   21.35    299.32    21.35      23.85a     2714
 14   -1.50    5.00   74.36   21.99    308.04    21.99      26.99a     2714
 15   -1.80    8.00   76.85   22.74    318.22    22.74      30.74a     2714
 16   -2.10   11.00   79.29   23.47    328.15    27.37      38.37      2714
 17   -2.40   14.00   81.67   24.19    337.88    31.13      45.13      2714
 18   -2.70   17.00   84.01   24.89    347.45    33.61      50.61      2916
 19   -3.00   20.00   86.32   25.58    356.88    35.18      55.18      2916
 20   -3.30   23.00   88.61   26.27    366.21    36.16      59.16      2916
 21   -3.60   26.00   90.87   26.95    375.46    36.80      62.80      2916
 22   -3.90   29.00   93.13   27.63    384.65    37.28      66.28      2916
 23   -4.20   32.00   95.37   28.51    321.52    37.69      69.69      2916
 24   -4.50   35.00   97.60   29.42    254.24    38.09      73.09      2916


                                        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    2.00    0.00    0.00    0.00      0.00     0.00       0.00        0.0
  2    1.75    0.00    0.00    0.00      0.00     0.00       0.00        0.0
  3    1.50    0.00    0.00    0.00      0.00     0.00       0.00        0.0
  4    1.20    0.00    0.00    0.00      0.00     0.00       0.00        0.0
  5    0.90    0.00    0.00    0.00      0.00     0.00       0.00        0.0
  6    0.60    0.00    0.00    0.00      0.00     0.00       0.00        0.0
  7    0.30    0.00    0.00    0.00      0.00     0.00       0.00        0.0
  8    0.00    0.00    0.00    0.00      0.00     0.00       0.00        0.0
               0.00    0.00    0.00      0.00     0.00       0.00       4611
  9   -0.25    0.00    4.50    1.27      5.17     5.17       5.17p     4611
 10   -0.50    0.00    9.00    2.46     19.83    19.83      19.83p     4611
 11   -0.75    0.00   13.50    3.59     44.00    44.00      44.00p     4611
 12   -1.00    0.00   18.00    4.78     58.67    58.67      58.67p     4611
```

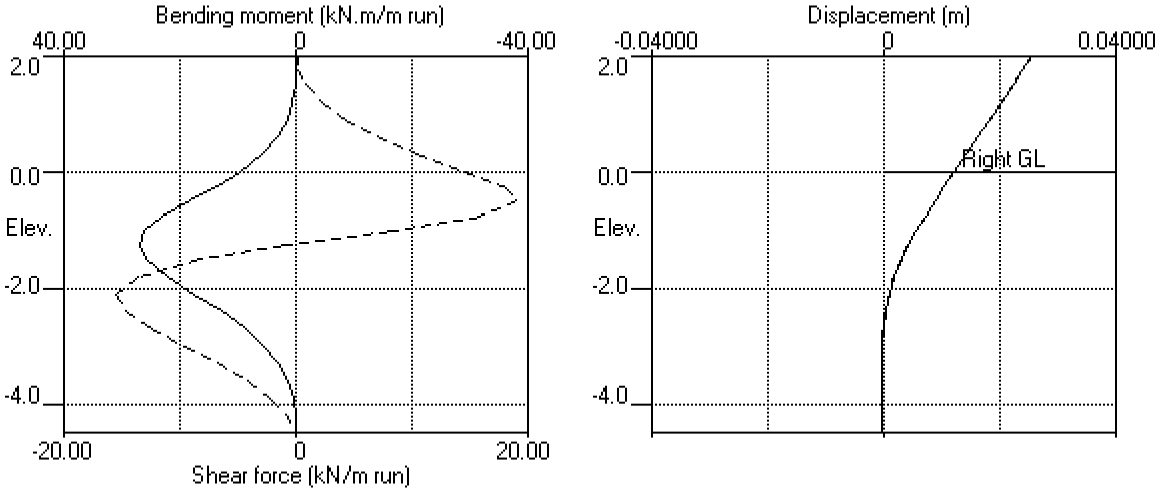
```
                                                                      (continued)
Stage No.4   Excavate to elevation 0.00 on RIGHT side
              Toe of berm at elevation -0.50
              Width of top of berm = 0.50
              Width of toe of berm = 0.60

                                        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
 13   -1.25    2.50  20.00    5.07     64.56     56.58       59.08         4611
 14   -1.50    5.00  22.00    5.36     70.45     44.64       49.64         4611
 15   -1.80    8.00  24.40    5.70     77.51     34.46       42.46         4611
 16   -2.10   11.00  26.80    6.05     84.58     28.17       39.17         4611
 17   -2.40   14.00  29.20    6.40     91.64     24.97       38.97         4611
 18   -2.70   17.00  31.60    6.74     98.71     24.41       41.41         2916
 19   -3.00   20.00  34.00    7.09    105.77     25.38       45.38         2916
 20   -3.30   23.00  36.40    7.43    112.84     27.10       50.10         2916
 21   -3.60   26.00  38.80    7.78    119.91     29.23       55.23         2916
 22   -3.90   29.00  41.20    8.12    126.97     31.54       60.54         2916
 23   -4.20   32.00  43.60    8.47    134.04     33.91       65.91         2916
 24   -4.50   35.00  46.00    8.82    141.10     36.30       71.30         2916

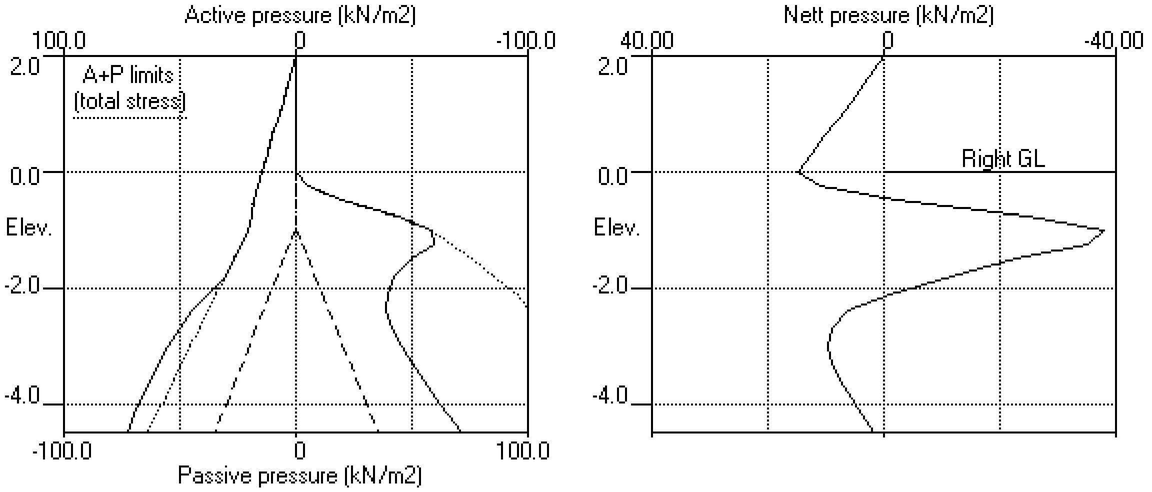
Note:     30.74 a  Soil pressure at active limit
          58.67 p  Soil pressure at passive limit
```

```
                                                        Units: kN,m
```

Stage No.4 Excav. to elev. 0.00 on RIGHT side



Stage No.4 Excav. to elev. 0.00 on RIGHT side



```
BECA LIMITED (NZ)                                           | Sheet No.
Program: WALLAP  Version 6.07  Revision A55.B74.R58         | Job No. 3160491
                             Licensed from GEOSOLVE         | Made by :    KM
Data filename/Run ID: STD-2_300mm_Static_Case_3             |
Std Retaining Wall - 2m - Drained Case 3                    | Date:12-10-2023
Standardised Design                                         | Checked :
-----------------------------------------------------------------------------
                                                       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. =   -4.50     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    2.00    0.00    Cant.   1.775    -4.07    -3.36    3.36     L to R
   2    2.00    0.00    Cant.   1.623    -4.10    -3.88    3.88     L to R
   3    2.00    0.00           No analysis at this stage
   4    2.00    0.00    Cant.   1.359    -4.23     ***     ***      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 = 100.00m
  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

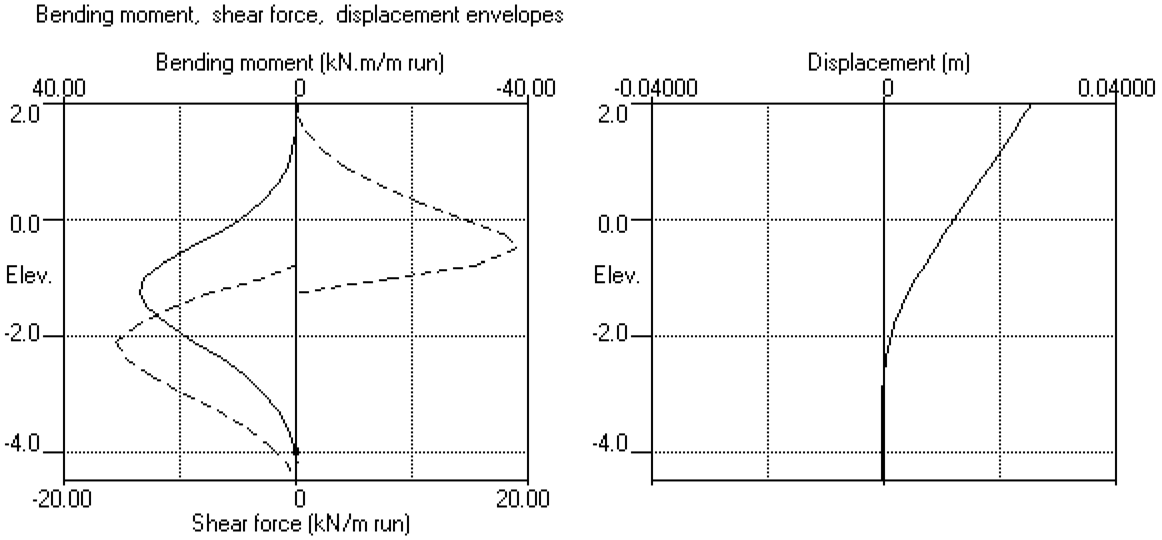
Bending moment, shear force and displacement envelopes
 Node    Y       Displacement         Bending moment       Shear force
 no.  coord   maximum   minimum    maximum   minimum    maximum   minimum
                  m         m       kN.m/m    kN.m/m      kN/m      kN/m
  1    2.00     0.026     0.000        0.0      -0.0        0.0       0.0
  2    1.75     0.024     0.000        0.0       0.0        0.2       0.0
  3    1.50     0.022     0.000        0.1       0.0        0.8       0.0
  4    1.20     0.020     0.000        0.6       0.0        2.2       0.0
  5    0.90     0.018     0.000        1.5       0.0        4.3       0.0
  6    0.60     0.016     0.000        3.2       0.0        7.0       0.0
  7    0.30     0.014     0.000        5.8       0.0       10.5       0.0
  8    0.00     0.012     0.000        9.6       0.0       14.6       0.0
  9   -0.25     0.010     0.000       13.7       0.0       17.9       0.0
 10   -0.50     0.009     0.000       18.4       0.0       19.0       0.0
 11   -0.75     0.007     0.000       23.3       0.0       15.7       0.0
 12   -1.00     0.006     0.000       26.2       0.0        7.9      -2.9
 13   -1.25     0.004     0.000       27.1       0.0        0.0      -7.4
 14   -1.50     0.003     0.000       25.8       0.0        0.0     -10.6
 15   -1.80     0.002     0.000       22.2       0.0        0.0     -13.7
 16   -2.10     0.001     0.000       17.6       0.0        0.0     -15.5
 17   -2.40     0.000     0.000       12.9       0.0        0.0     -14.7
 18   -2.70     0.000    -0.000        8.8       0.0        0.0     -12.4
 19   -3.00     0.000    -0.000        5.5       0.0        0.0      -9.6
 20   -3.30     0.000    -0.001        3.1       0.0        0.0      -6.7
 21   -3.60     0.000    -0.001        1.4       0.0        0.0      -4.3
 22   -3.90     0.000    -0.001        0.5      -0.1        0.0      -2.3
 23   -4.20     0.000    -0.001        0.1      -0.0        0.1      -0.8
 24   -4.50     0.000    -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       17.2   -0.75      -0.1   -3.90      13.7   -0.25     -10.3   -1.80
  2       20.4   -1.00      -0.0   -4.20      16.1   -0.25     -12.1   -1.80
  3    No calculation at this stage
  4       27.1   -1.25      -0.0    2.00      19.0   -0.50     -15.5   -2.10
```

Summary of results (continued)

Maximum and minimum displacement at each stage

```
Stage -------- Displacement ---------
 no.  maximum  elev.   minimum  elev.   Stage description                     _
         m                 m
  1    0.041    2.00    0.000    2.00   Fill to elev. 2.00 on LEFT side
  2    0.050    2.00    0.000    2.00   Apply surcharge no.1 at elev. 2.00
  3    Wall displacements reset to zero Change EI of wall to 2602kN.m2/m run
  4    0.026    2.00   -0.001   -3.60   Excav. to elev. 0.00 on RIGHT side
```



Bending moment, shear force, displacement envelopes

```
                                                       Units: kN,m
INPUT DATA

SOIL PROFILE
Stratum   Elevation of    ------------------ Soil types -------------------
  no.    top of stratum    Left side                Right side
   1          0.00       1  Founding unit (std s   1  Founding unit (std s
   2         -8.00       1  Founding unit (std s   1  Founding unit (std s

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  Founding      18.00     10000    0.500     OC    0.294    4.369
    unit (std s                             (0.300) (0.000) ( 0.000)
 2  Back Fill..   22.00     50000    0.500     OC    0.294    4.288
    (    0.00 )          ( 0.3000)          (0.300) (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  Founding unit (std s    30.00    0.464    0.00     30.00    0.500    0.00
 2  Back Fill (std spec)    30.00    0.464    0.00     30.00    0.464    0.00

GROUND WATER CONDITIONS
 Density of water = 10.00 kN/m3
                                    Left side     Right side
 Initial water table elevation        0.00           0.00

 Automatic water pressure balancing at toe of wall : Yes


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 = -4.50
             Maximum finite element length =  0.30 m
                  Youngs modulus of wall E = 7.8520E+06 kN/m2
               Moment of inertia of wall I = 3.3134E-04 m4/m run
                                           = 3.9761E-04 m4 per pile
                                       E.I = 2601.7 kN.m2/m run
                      Yield Moment of wall = Not defined

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     2.00    1.00(L)    9.00      9.00     15.00      =         0     N/A

    Note: L = Left side,  R = Right side

CONSTRUCTION STAGES
Construction   Stage description
  stage no.    -------------------------------------------------------
      1        Fill to elevation 2.00 on LEFT side with soil type 2
      2        Change EI of wall to 2602 kN.m2/m run
               Reset wall displacements to zero at this stage
      3        Apply surcharge no.1 at elevation 2.00
```

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) = 100.00 m

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

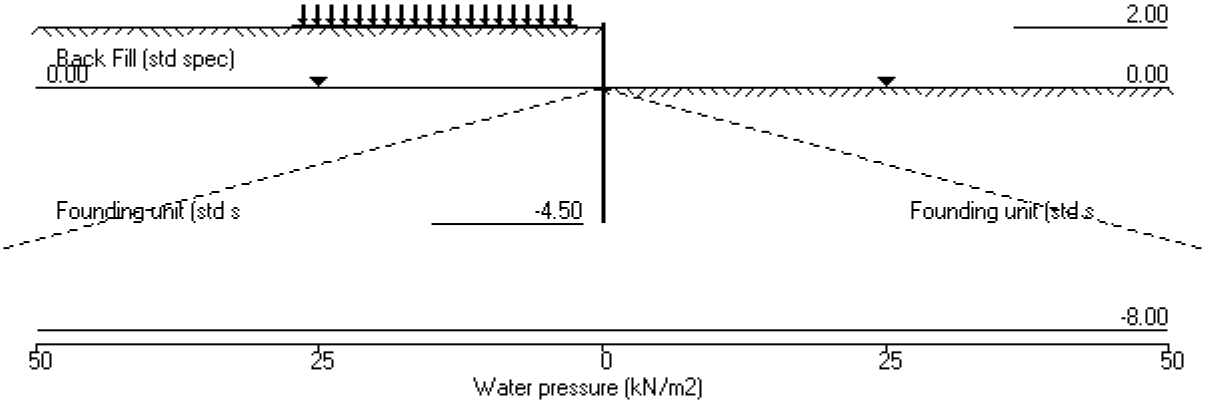
  Distance to rigid boundary on Left side  = 20.00 m
  Distance to rigid boundary on Right side = 20.00 m
```

OUTPUT OPTIONS

```
 Stage ------ Stage description ----------- ------- Output options -------
  no.                                       Displacement   Active,  Graph.
                                            Bending mom.   Passive  output
                                            Shear force   pressures
   1 Fill to elev. 2.00 on LEFT side            Yes          Yes     Yes
   2 Change EI of wall to 2602kN.m2/m run        No           No      No
   3 Apply surcharge no.1 at elev. 2.00         Yes          Yes      No
   * Summary output                             Yes           -      Yes
```

Units: kN,m

Stage No.3 Apply surcharge no.1 at elev. 2.00



```
                                                       Units: kN,m
Stage No. 1   Fill to elevation 2.00 on LEFT side with soil type 2

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. =   -4.50     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    2.00     0.00    Cant.   1.389    -4.12      ***     ***        L to R

   Legend: *** Result not found


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 = 100.00m
  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    2.00      0.00     0.074   2.19E-02       0.0      -0.0
   2    1.75      1.61     0.069   2.19E-02       0.2       0.0
   3    1.50      3.22     0.063   2.19E-02       0.8       0.1
   4    1.20      5.15     0.057   2.18E-02       2.1       0.5
   5    0.90      7.08     0.050   2.17E-02       3.9       1.4
   6    0.60      9.01     0.044   2.15E-02       6.3       2.9
   7    0.30     10.94     0.037   2.10E-02       9.3       5.3
   8    0.00     12.87     0.031   2.02E-02      12.9       8.6
   9   -0.30      2.85     0.025   1.90E-02      15.2      12.9
  10   -0.60     -7.18     0.020   1.72E-02      14.6      17.4
  11   -0.90    -17.21     0.015   1.50E-02      10.9      21.3
  12   -1.20    -27.23     0.011   1.24E-02       4.3      23.7
  13   -1.50    -26.26     0.008   9.67E-03      -3.8      24.3
  14   -1.80    -14.71     0.005   7.01E-03      -9.9      22.0
  15   -2.10     -6.80     0.003   4.68E-03     -13.1      18.3
  16   -2.40      1.72     0.002   2.81E-03     -13.9      14.2
  17   -2.70      6.54     0.002   1.40E-03     -12.7      10.1
  18   -3.00      8.61     0.001   4.40E-04     -10.4       6.6
  19   -3.30      8.87     0.001  -1.65E-04      -7.8       3.9
  20   -3.60      8.05     0.001  -5.03E-04      -5.2       2.0
  21   -3.90      6.67     0.002  -6.60E-04      -3.0       0.8
  22   -4.20      5.05     0.002  -7.13E-04      -1.3       0.2
  23   -4.50      3.38     0.002  -7.22E-04       0.0      -0.0
```

```
-----------------------------------------------------------------------------
                                                              (continued)
Stage No.1   Fill to elevation 2.00 on LEFT side with soil type 2

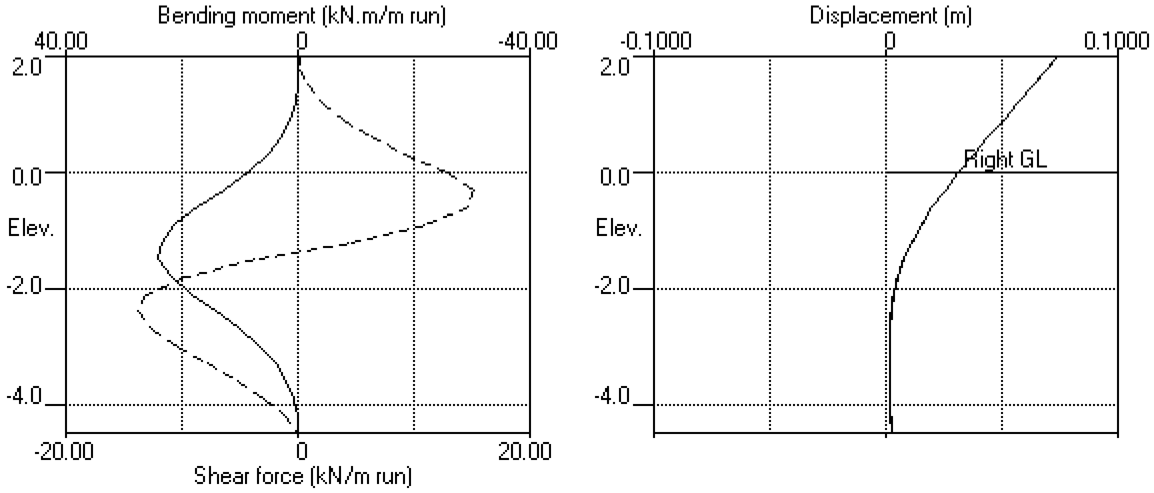
                                         LEFT side                         _
                             Effective stresses            Total   Coeff. of
 Node   Y    Water   Vertic  Active   Passive    Earth     earth    subgrade
 no.  coord  press.    -al   limit     limit   pressure   pressure  reaction
              kN/m2   kN/m2   kN/m2     kN/m2    kN/m2      kN/m2     kN/m3
  1    2.00    0.00    0.00    0.00      0.00     0.00       0.00     14610
  2    1.75    0.00    5.50    1.61     24.42     1.61       1.61a    14610
  3    1.50    0.00   11.00    3.22     48.83     3.22       3.22a    14610
  4    1.20    0.00   17.60    5.15     78.13     5.15       5.15a    14610
  5    0.90    0.00   24.20    7.08    107.43     7.08       7.08a    14610
  6    0.60    0.00   30.80    9.01    136.73     9.01       9.01a    14610
  7    0.30    0.00   37.40   10.94    166.02    10.94      10.94a    14610
  8    0.00    0.00   44.00   12.87    195.32    12.87      12.87a    14610
               0.00   44.00   12.87    186.74    12.87      12.87a     2922
  9   -0.30    3.00   46.40   13.56    196.84    13.56      16.56a     2922
 10   -0.60    6.00   48.80   14.24    206.94    14.24      20.24a     2922
 11   -0.90    9.00   51.20   14.92    217.04    14.92      23.92a     2922
 12   -1.20   12.00   53.60   15.61    227.14    15.61      27.61a     2922
 13   -1.50   15.00   56.00   16.29    237.24    16.29      31.29a     2922
 14   -1.80   18.00   58.40   16.98    247.34    16.98      34.98a     2922
 15   -2.10   21.00   60.80   17.66    257.44    17.66      38.66a     2922
 16   -2.40   24.00   63.20   18.34    267.54    22.00      46.00      2922
 17   -2.70   27.00   65.60   19.03    277.64    25.02      52.02      2922
 18   -3.00   30.00   68.00   19.71    287.74    27.00      57.00      2922
 19   -3.30   33.00   70.40   20.39    297.84    28.29      61.29      2922
 20   -3.60   36.00   72.80   21.08    307.94    29.18      65.18      2922
 21   -3.90   39.00   75.20   21.76    318.04    29.86      68.86      2922
 22   -4.20   42.00   77.60   22.44    328.14    30.46      72.46      2922
 23   -4.50   45.00   80.00   23.13    338.24    31.03      76.03      2922


                                        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    2.00    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  2    1.75    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  3    1.50    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  4    1.20    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  5    0.90    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  6    0.60    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  7    0.30    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  8    0.00    0.00    0.00    0.00      0.00     0.00       0.00       0.0
               0.00    0.00    0.00      0.00     0.00       0.00      4833
  9   -0.30    3.00    2.40    0.70     10.71    10.71      13.71p     4833
 10   -0.60    6.00    4.80    1.39     21.42    21.42      27.42p     4833
 11   -0.90    9.00    7.20    2.09     32.13    32.13      41.13p     4833
 12   -1.20   12.00    9.60    2.78     42.84    42.84      54.84p     4833
 13   -1.50   15.00   12.00    3.48     53.55    42.55      57.55      4833
 14   -1.80   18.00   14.40    4.17     64.26    31.69      49.69      4833
 15   -2.10   21.00   16.80    4.87     74.97    24.46      45.46      4833
 16   -2.40   24.00   19.20    5.56     85.68    20.28      44.28      4833
 17   -2.70   27.00   21.60    6.26     96.39    18.48      45.48      4833
 18   -3.00   30.00   24.00    6.95    107.10    18.39      48.39      4833
 19   -3.30   33.00   26.40    7.65    117.81    19.42      52.42      4833
 20   -3.60   36.00   28.80    8.34    128.53    21.14      57.14      4833
 21   -3.90   39.00   31.20    9.04    139.24    23.20      62.20      4833
 22   -4.20   42.00   33.60    9.74    149.95    25.40      67.40      4833
 23   -4.50   45.00   36.00   10.43    160.66    27.65      72.65      4833
```

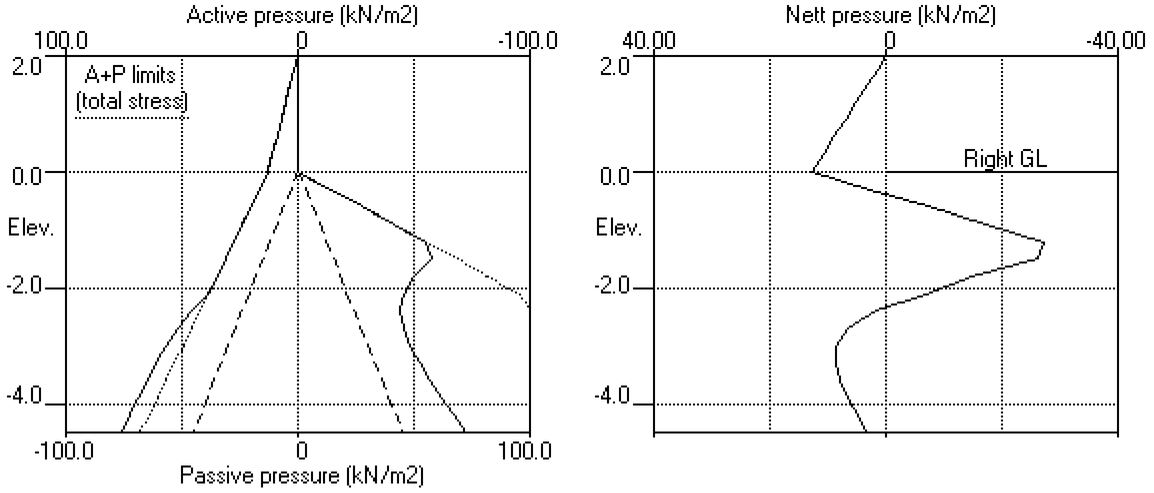
```
                                                         (continued)
Stage No.1   Fill to elevation 2.00 on LEFT side with soil type 2
Note:     38.66 a  Soil pressure at active limit
          54.84 p  Soil pressure at passive limit
```

Units: kN,m

Stage No.1 Fill to elev. 2.00 on LEFT side



Stage No.1 Fill to elev. 2.00 on LEFT side



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                                                       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. =    -4.50     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    2.00     0.00    Cant.   1.389    -4.12     ***     ***       L to R
   2    2.00     0.00           No analysis at this stage
   3    2.00     0.00    Cant.   1.274    -4.04     ***     ***       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 = 100.00m
  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

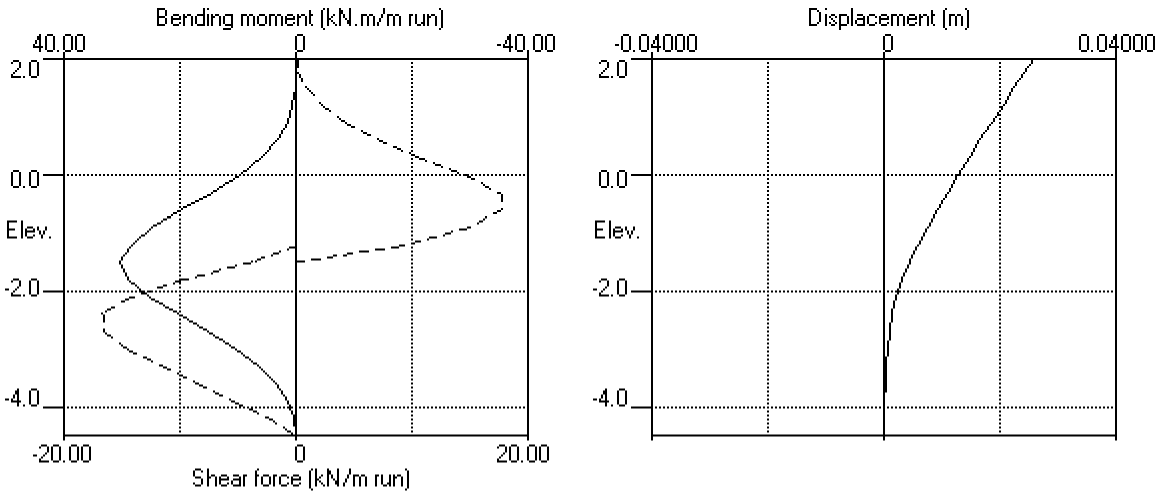
Bending moment, shear force and displacement envelopes
 Node    Y        Displacement         Bending moment        Shear force
 no.   coord    maximum   minimum    maximum   minimum    maximum   minimum
                   m         m        kN.m/m    kN.m/m      kN/m      kN/m
  1     2.00    0.026     0.000        0.0      -0.0        0.0       0.0
  2     1.75    0.024     0.000        0.0       0.0        0.2       0.0
  3     1.50    0.022     0.000        0.1       0.0        0.8       0.0
  4     1.20    0.021     0.000        0.6       0.0        2.2       0.0
  5     0.90    0.019     0.000        1.5       0.0        4.3       0.0
  6     0.60    0.017     0.000        3.2       0.0        7.0       0.0
  7     0.30    0.015     0.000        5.8       0.0       10.5       0.0
  8     0.00    0.013     0.000        9.6       0.0       14.6       0.0
  9    -0.30    0.011     0.000       14.5       0.0       17.6       0.0
 10    -0.60    0.009     0.000       19.9       0.0       17.7       0.0
 11    -0.90    0.007     0.000       24.8       0.0       14.8       0.0
 12    -1.20    0.006     0.000       28.4       0.0        8.9       0.0
 13    -1.50    0.004     0.000       30.4       0.0        0.1      -3.8
 14    -1.80    0.003     0.000       29.0       0.0        0.0      -9.9
 15    -2.10    0.002     0.000       25.3       0.0        0.0     -14.4
 16    -2.40    0.001     0.000       20.6       0.0        0.0     -16.8
 17    -2.70    0.001     0.000       15.3       0.0        0.0     -16.7
 18    -3.00    0.000     0.000       10.5       0.0        0.0     -14.6
 19    -3.30    0.000     0.000        6.6       0.0        0.0     -11.6
 20    -3.60    0.000     0.000        3.6       0.0        0.0      -8.4
 21    -3.90    0.000     0.000        1.5       0.0        0.0      -5.3
 22    -4.20    0.000    -0.000        0.4       0.0        0.0      -2.5
 23    -4.50    0.000    -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        24.3   -1.50      -0.0    2.00      15.2   -0.30     -13.9   -2.40
  2     No calculation at this stage
  3        30.4   -1.50      -0.0    2.00      17.7   -0.60     -16.8   -2.40

Maximum and minimum displacement at each stage
Stage -------- Displacement ---------
 no.   maximum  elev.   minimum  elev.   Stage description                    _
          m                m
  1     0.074    2.00    0.000    2.00   Fill to elev. 2.00 on LEFT side
  2     Wall displacements reset to zero Change EI of wall to 2602kN.m2/m run
  3     0.026    2.00   -0.000   -4.50   Apply surcharge no.1 at elev. 2.00
```

Summary of results (continued)

Bending moment, shear force, displacement envelopes



```
                                                       Units: kN,m
INPUT DATA

SOIL PROFILE
Stratum   Elevation of    ------------------ Soil types -------------------
  no.    top of stratum    Left side                Right side
   1          0.00       1  Founding unit (std s   1  Founding unit (std s
   2         -5.00       1  Founding unit (std s   1  Founding unit (std s

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  Founding      18.00     10000    0.500     OC    0.294    4.369
    unit (std s                             (0.300) (0.000) ( 0.000)
 2  Back Fill..   22.00     50000    0.500     OC    0.294    4.288
    (    0.00 )          ( 0.3000)          (0.300) (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  Founding unit (std s    30.00    0.464    0.00     30.00    0.500    0.00
 2  Back Fill (std spec)    30.00    0.464    0.00     30.00    0.464    0.00

GROUND WATER CONDITIONS
 Density of water = 10.00 kN/m3
                                   Left side     Right side
 Initial water table elevation       -1.00           -1.00

 Automatic water pressure balancing at toe of wall : Yes


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 = -4.50
               Maximum finite element length =  0.30 m
                    Youngs modulus of wall E = 7.8520E+06 kN/m2
                 Moment of inertia of wall I = 3.3134E-04 m4/m run
                                             = 3.9761E-04 m4 per pile
                                         E.I = 2601.7 kN.m2/m run
                        Yield Moment of wall = Not defined

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     2.00    0.30(L)    5.00      6.00     12.00      =         0     N/A

    Note: L = Left side,  R = Right side

CONSTRUCTION STAGES
Construction   Stage description
  stage no.    -------------------------------------------------------
      1        Fill to elevation 2.00 on LEFT side with soil type 2
      2        Change EI of wall to 2602 kN.m2/m run
               Reset wall displacements to zero at this stage
      3        Apply surcharge no.1 at elevation 2.00
```

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) = 100.00 m

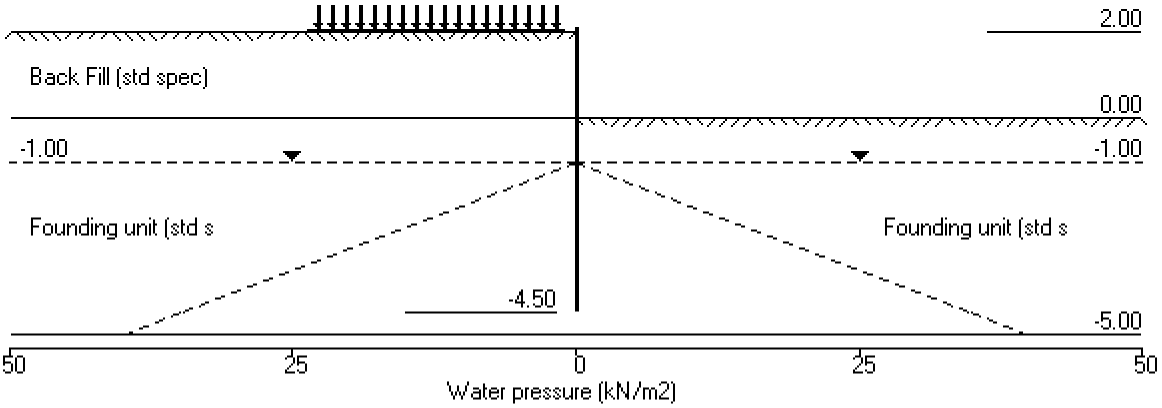
  Width of excavation on Left  side of wall  = 20.00 m
  Width of excavation on Right side of wall  = 20.00 m

  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. 2.00 on LEFT side            Yes          Yes     Yes
   2 Change EI of wall to 2602kN.m2/m run        No           No      No
   3 Apply surcharge no.1 at elev. 2.00         Yes          Yes      No
   * Summary output                             Yes           -      Yes
```

Stage No.3   Apply surcharge no.1 at elev. 2.00



```
                                                      Units: kN,m
Stage No. 1   Fill to elevation 2.00 on LEFT side with soil type 2

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. =   -4.50      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    2.00    0.00      Cant.   1.777    -4.07     -3.36    3.36      L to R


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 = 100.00m
  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     2.00     0.00      0.040   1.33E-02      0.0       0.0
  2     1.75     1.61      0.037   1.33E-02      0.2       0.0
  3     1.50     3.21      0.034   1.33E-02      0.8       0.1
  4     1.20     5.14      0.030   1.32E-02      2.1       0.5
  5     0.90     7.06      0.026   1.31E-02      3.9       1.4
  6     0.60     8.99      0.022   1.29E-02      6.3       2.9
  7     0.30    10.92      0.018   1.24E-02      9.3       5.3
  8     0.00    12.84      0.014   1.16E-02     12.8       8.6
  9    -0.30    -9.35      0.011   1.04E-02     13.4      12.7
 10    -0.60   -31.53      0.008   8.78E-03      7.2      15.9
 11    -0.80   -26.47      0.007   7.52E-03      1.4      17.0
 12    -1.00   -19.58      0.005   6.22E-03     -3.2      16.8
 13    -1.25   -12.46      0.004   4.68E-03     -7.2      15.4
 14    -1.50    -5.62      0.003   3.30E-03     -9.4      13.3
 15    -1.80     1.19      0.002   1.95E-03    -10.1      10.2
 16    -2.10     4.92      0.002   9.52E-04     -9.2       7.2
 17    -2.40     6.45      0.001   2.66E-04     -7.5       4.7
 18    -2.70     6.54      0.001  -1.60E-04     -5.5       2.7
 19    -3.00     5.77      0.001  -3.96E-04     -3.7       1.4
 20    -3.30     4.57      0.002  -5.05E-04     -2.1       0.5
 21    -3.60     3.19      0.002  -5.40E-04     -1.0       0.1
 22    -3.90     1.77      0.002  -5.41E-04     -0.2      -0.1
 23    -4.20     0.36      0.002  -5.35E-04      0.1      -0.0
 24    -4.50    -1.03      0.002  -5.32E-04      0.0       0.0
```

```
-----------------------------------------------------------------------------
                                                              (continued)
Stage No.1   Fill to elevation 2.00 on LEFT side with soil type 2

                                         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    2.00    0.00    0.00    0.00      0.00     0.00       0.00     15838
  2    1.75    0.00    5.50    1.61     24.42     1.61       1.61a    15838
  3    1.50    0.00   11.00    3.21     48.83     3.21       3.21a    15838
  4    1.20    0.00   17.60    5.14     78.13     5.14       5.14a    15838
  5    0.90    0.00   24.20    7.06    107.43     7.06       7.06a    15838
  6    0.60    0.00   30.80    8.99    136.73     8.99       8.99a    15838
  7    0.30    0.00   37.40   10.92    166.02    10.92      10.92a    15838
  8    0.00    0.00   44.00   12.84    195.32    12.84      12.84a    15838
               0.00   44.00   12.84    189.18    12.84      12.84a     3168
  9   -0.30    0.00   49.40   14.42    212.40    14.42      14.42a     3168
 10   -0.60    0.00   54.80   15.99    235.62    15.99      15.99a     3168
 11   -0.80    0.00   58.40   17.04    251.10    17.04      17.04a     3168
 12   -1.00    0.00   62.00   18.09    266.58    18.09      18.09a     3168
 13   -1.25    2.50   64.00   18.66    275.14    18.66      21.16a     3168
 14   -1.50    5.00   66.00   19.22    283.70    20.97      25.97      3168
 15   -1.80    8.00   68.40   19.89    293.97    24.64      32.64      3168
 16   -2.10   11.00   70.80   20.57    304.24    27.20      38.20      3168
 17   -2.40   14.00   73.20   21.24    314.51    28.95      42.95      3168
 18   -2.70   17.00   75.60   21.92    324.78    30.18      47.18      3168
 19   -3.00   20.00   78.00   22.59    335.05    31.11      51.11      3168
 20   -3.30   23.00   80.40   23.26    345.33    31.87      54.87      3168
 21   -3.60   26.00   82.80   23.94    355.60    32.57      58.57      3168
 22   -3.90   29.00   85.20   24.61    365.87    33.25      62.25      3168
 23   -4.20   32.00   87.60   25.29    376.14    33.94      65.94      3168
 24   -4.50   35.00   90.00   25.96    386.41    34.63      69.63      3168


                                        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    2.00    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  2    1.75    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  3    1.50    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  4    1.20    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  5    0.90    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  6    0.60    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  7    0.30    0.00    0.00    0.00      0.00     0.00       0.00       0.0
  8    0.00    0.00    0.00    0.00      0.00     0.00       0.00       0.0
               0.00    0.00    0.00      0.00     0.00       0.00      5555
  9   -0.30    0.00    5.40    1.58     23.76    23.76      23.76p     5555
 10   -0.60    0.00   10.80    3.17     47.53    47.53      47.53p     5555
 11   -0.80    0.00   14.40    4.22     63.37    43.51      43.51      5555
 12   -1.00    0.00   18.00    5.28     79.21    37.67      37.67      5555
 13   -1.25    2.50   20.00    5.85     88.03    31.11      33.61      5555
 14   -1.50    5.00   22.00    6.42     96.85    26.59      31.59      5555
 15   -1.80    8.00   24.40    7.11    107.43    23.45      31.45      5555
 16   -2.10   11.00   26.80    7.80    118.01    22.28      33.28      5555
 17   -2.40   14.00   29.20    8.49    128.59    22.50      36.50      5555
 18   -2.70   17.00   31.60    9.18    139.18    23.65      40.65      5555
 19   -3.00   20.00   34.00    9.87    149.76    25.33      45.33      5555
 20   -3.30   23.00   36.40   10.56    160.34    27.30      50.30      5555
 21   -3.60   26.00   38.80   11.25    170.92    29.38      55.38      5555
 22   -3.90   29.00   41.20   11.94    181.51    31.48      60.48      5555
```

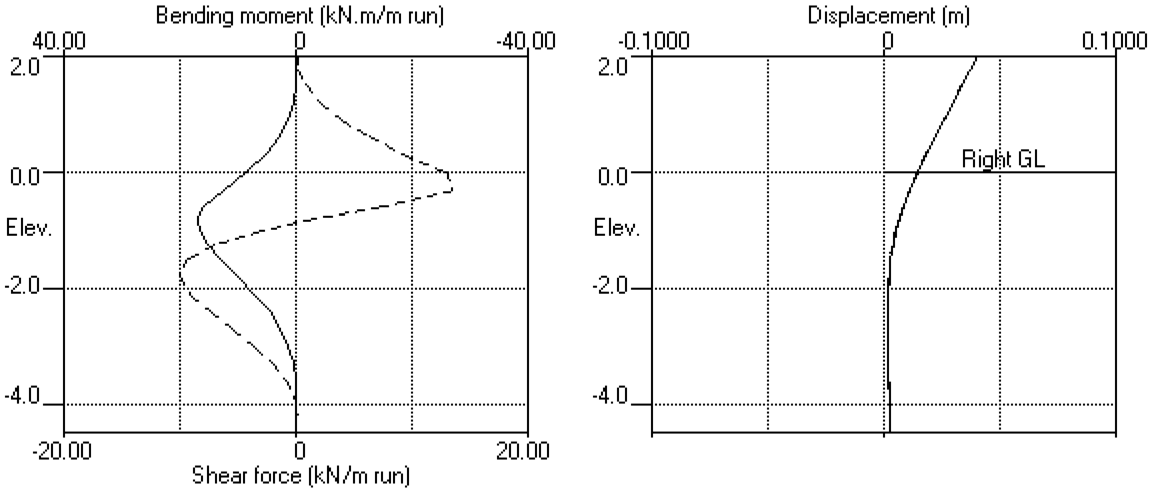
```
                                                              (continued)
Stage No.1   Fill to elevation 2.00 on LEFT side with soil type 2

                                        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
 23   -4.20   32.00   43.60   12.63    192.09    33.58      65.58      5555
 24   -4.50   35.00   46.00   13.32    202.67    35.67      70.67      5555

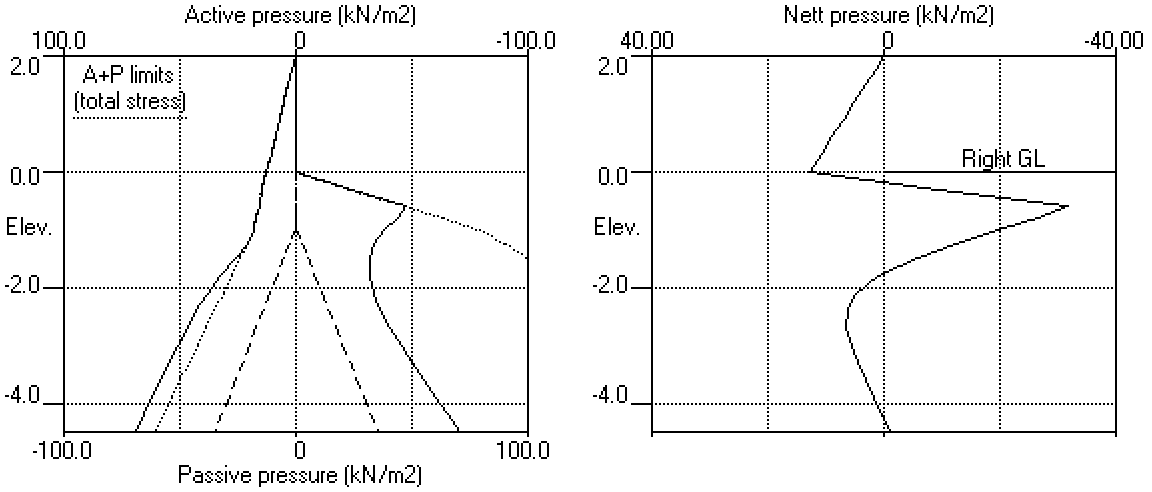
Note:     21.16 a  Soil pressure at active limit
          47.53 p  Soil pressure at passive limit
```

```
                                                Units: kN,m
```

Stage No.1 Fill to elev. 2.00 on LEFT side



Stage No.1 Fill to elev. 2.00 on LEFT side



```
                                                       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. =   -4.50     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    2.00    0.00    Cant.   1.777    -4.07    -3.36    3.36     L to R
   2    2.00    0.00           No analysis at this stage
   3    2.00    0.00    Cant.   1.621    -4.06    -3.90    3.90     L to R
```

```
                                                       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 = 100.00m
  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     2.00     0.013     0.000        0.0      -0.0        0.0       0.0
  2     1.75     0.012     0.000        0.0       0.0        0.2       0.0
  3     1.50     0.011     0.000        0.1       0.0        0.8       0.0
  4     1.20     0.010     0.000        0.5       0.0        2.1       0.0
  5     0.90     0.009     0.000        1.4       0.0        4.3       0.0
  6     0.60     0.007     0.000        2.9       0.0        7.4       0.0
  7     0.30     0.006     0.000        5.7       0.0       11.2       0.0
  8     0.00     0.005     0.000        9.7       0.0       15.5       0.0
  9    -0.30     0.004     0.000       14.7       0.0       16.9       0.0
 10    -0.60     0.003     0.000       19.1       0.0       11.5       0.0
 11    -0.80     0.003     0.000       21.1       0.0        4.9       0.0
 12    -1.00     0.002     0.000       21.4       0.0        0.0      -3.2
 13    -1.25     0.001     0.000       20.1       0.0        0.0      -7.3
 14    -1.50     0.001     0.000       17.8       0.0        0.0     -10.8
 15    -1.80     0.001     0.000       14.1       0.0        0.0     -12.5
 16    -2.10     0.000     0.000       10.3       0.0        0.0     -11.9
 17    -2.40     0.000     0.000        7.0       0.0        0.0     -10.1
 18    -2.70     0.000     0.000        4.3       0.0        0.0      -7.7
 19    -3.00     0.000     0.000        2.3       0.0        0.0      -5.4
 20    -3.30     0.000     0.000        1.0       0.0        0.0      -3.3
 21    -3.60     0.000     0.000        0.3       0.0        0.0      -1.7
 22    -3.90     0.000     0.000        0.0      -0.1        0.0      -0.6
 23    -4.20     0.000     0.000        0.0      -0.0        0.1      -0.0
 24    -4.50     0.000     0.000        0.0       0.0        0.0       0.0

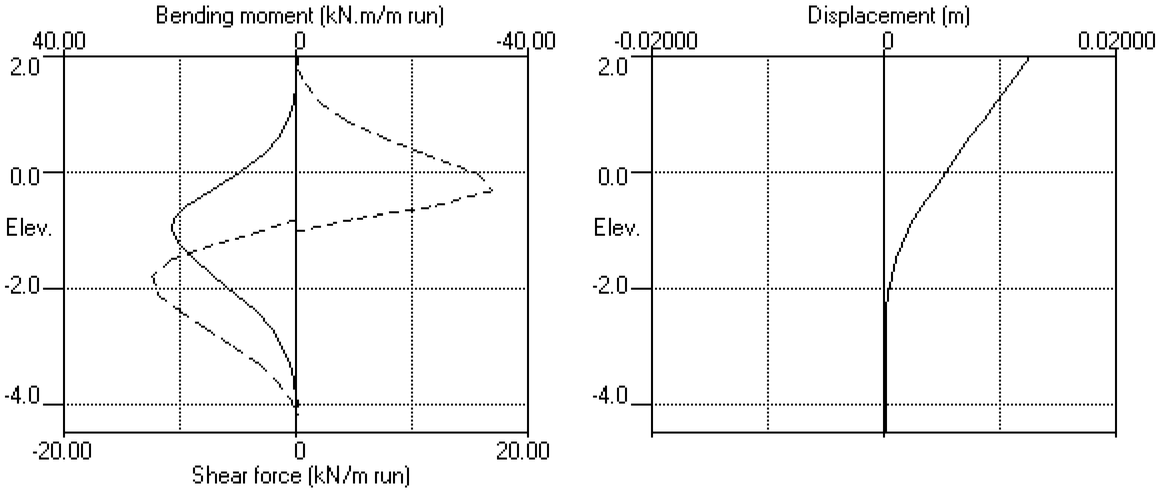
Maximum and minimum bending moment and shear force at each stage
Stage  --------- Bending moment --------   ---------- Shear force ----------
 no.   maximum  elev.   minimum  elev.     maximum  elev.   minimum  elev.
        kN.m/m           kN.m/m             kN/m             kN/m
  1      17.0   -0.80     -0.1   -3.90       13.4   -0.30    -10.1   -1.80
  2    No calculation at this stage
  3      21.4   -1.00     -0.0   -4.20       16.9   -0.30    -12.5   -1.80
```

Summary of results   (continued)

Maximum and minimum displacement at each stage

```
Stage -------- Displacement ---------
 no.  maximum  elev.   minimum  elev.   Stage description                   _
          m                 m
  1    0.040    2.00    0.000    2.00   Fill to elev. 2.00 on LEFT side
  2    Wall displacements reset to zero Change EI of wall to 2602kN.m2/m run
  3    0.013    2.00    0.000    2.00   Apply surcharge no.1 at elev. 2.00
```

Bending moment, shear force, displacement envelopes



```
                                                       Units: kN,m
INPUT DATA

SOIL PROFILE
Stratum   Elevation of    ------------------ Soil types -------------------
  no.    top of stratum    Left side                Right side
   1           0.00       1  Founding unit (std s   1  Founding unit (std s
   2          -5.00       1  Founding unit (std s   1  Founding unit (std s

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  Founding      18.00     10000    0.500     OC    0.294    4.369
    unit (std s                             (0.300) (0.000) ( 0.000)
 2  Back Fill..   22.00     50000    0.500     OC    0.320    3.812
    (    0.00 )          ( 0.3000)          (0.300) (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  Founding unit (std s    30.00    0.464   0.00     30.00    0.500   0.00
 2  Back Fill (std spec)    28.00    0.469   0.00     28.00    0.469   0.00

GROUND WATER CONDITIONS
 Density of water = 10.00 kN/m3
                                   Left side     Right side
 Initial water table elevation       -1.00          -1.00

 Automatic water pressure balancing at toe of wall : Yes


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 = -4.50
             Maximum finite element length =  0.30 m
                  Youngs modulus of wall E = 7.8520E+06 kN/m2
               Moment of inertia of wall I = 3.3134E-04 m4/m run
                                           = 3.9761E-04 m4 per pile
                                       E.I = 2601.7 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       1.33       4.040       0            0          N/A
   2       0.44       12.11       0            0          N/A
   3       0.00       0           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     2.00    1.00(L)    9.00      9.00     15.00      =         0     N/A

    Note: L = Left side,  R = Right side
```

CONSTRUCTION STAGES

```
Construction   Stage description
  stage no.    --------------------------------------------------------
      1        Fill to elevation 2.00 on LEFT side with soil type 2
      2        Apply surcharge no.1 at elevation 2.00
      3        Change EI of wall to 1839 kN.m2/m run
               Reset wall displacements to zero at this stage
      4        Apply load no.1 at elevation 1.33
      5        Apply load no.2 at elevation 0.44
```

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) = 100.00 m

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

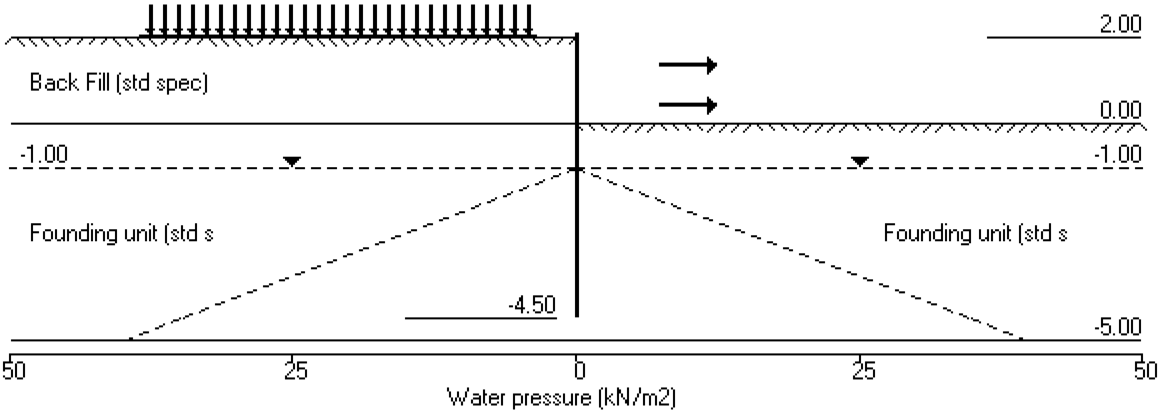
  Distance to rigid boundary on Left side  = 20.00 m
  Distance to rigid boundary on Right side = 20.00 m
```

OUTPUT OPTIONS

```
 Stage ------ Stage description ----------- ------- Output options -------
  no.                                       Displacement   Active,  Graph.
                                            Bending mom.   Passive  output
                                            Shear force   pressures
   1 Fill to elev. 2.00 on LEFT side            Yes          Yes     Yes
   2 Apply surcharge no.1 at elev. 2.00         Yes          Yes      No
   3 Change EI of wall to 1839kN.m2/m run        No           No      No
   4 Apply load no.1 at elev. 1.33              Yes          Yes     Yes
   5 Apply load no.2 at elev. 0.44              Yes          Yes     Yes
   * Summary output                             Yes           -      Yes
```

Units: kN,m

Stage No.5 Apply load no.2 at elev. 0.44



```
                                                       Units: kN,m
Stage No. 1   Fill to elevation 2.00 on LEFT side with soil type 2

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. =    -4.50    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    2.00    0.00    Cant.   1.758    -4.07    -3.43    3.43     L to R


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 = 100.00m
  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      EI of
 no.  coord  pressure     disp.   rotation    force   moment    forces     wall
                kN/m2       m      rad.       kN/m    kN.m/m    kN/m   kN.m2/m
  1    2.00      0.00     0.044   1.45E-02      0.0       0.0              2602
  2    1.83      1.16     0.042   1.45E-02      0.1       0.0              2602
  3    1.67      2.32     0.039   1.45E-02      0.4       0.0              2602
  4    1.50      3.48     0.037   1.45E-02      0.9       0.1              2602
  5    1.33      4.64     0.034   1.45E-02      1.6       0.3              2602
  6    1.11      6.12     0.031   1.45E-02      2.7       0.8              2602
  7    0.90      7.61     0.028   1.44E-02      4.2       1.5              2602
  8    0.67      9.20     0.025   1.42E-02      6.1       2.7              2602
  9    0.44     10.80     0.022   1.39E-02      8.4       4.4              2602
 10    0.22     12.32     0.019   1.34E-02     11.0       6.5              2602
 11    0.00     13.84     0.016   1.27E-02     13.8       9.2              2602
                12.87     0.016   1.27E-02     13.8       9.2
 12   -0.30     -9.31     0.012   1.14E-02     14.4      13.6              2602
 13   -0.60    -31.50     0.009   9.70E-03      8.3      17.2              2602
 14   -0.80    -29.72     0.007   8.33E-03      2.1      18.5              2602
 15   -1.00    -21.94     0.006   6.91E-03     -3.0      18.3              2602
 16   -1.25    -13.90     0.004   5.22E-03     -7.5      16.9              2602
 17   -1.50     -6.82     0.003   3.71E-03    -10.1      14.7              2602
 18   -1.80      0.89     0.002   2.21E-03    -11.0      11.3              2602
 19   -2.10      5.16     0.002   1.09E-03    -10.1       8.1              2602
 20   -2.40      6.97     0.001   3.30E-04     -8.3       5.3              2602
 21   -2.70      7.15     0.001  -1.50E-04     -6.2       3.1              2602
 22   -3.00      6.36     0.001  -4.19E-04     -4.1       1.6              2602
 23   -3.30      5.07     0.002  -5.45E-04     -2.4       0.6              2602
 24   -3.60      3.57     0.002  -5.87E-04     -1.1       0.1              2602
 25   -3.90      2.01     0.002  -5.91E-04     -0.3      -0.1              2602
 26   -4.20      0.47     0.002  -5.85E-04      0.1      -0.0              2602
 27   -4.50     -1.07     0.002  -5.83E-04      0.0      -0.0               ---
```

```
                                                              (continued)
Stage No.1   Fill to elevation 2.00 on LEFT side with soil type 2

                                         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     2.00    0.00    0.00    0.00      0.00     0.00       0.00    15846
  2     1.83    0.00    3.68    1.16     14.45     1.16       1.16a    15846
  3     1.67    0.00    7.37    2.32     28.90     2.32       2.32a    15846
  4     1.50    0.00   11.05    3.48     43.35     3.48       3.48a    15846
  5     1.33    0.00   14.74    4.64     57.81     4.64       4.64a    15846
  6     1.11    0.00   19.47    6.12     76.36     6.12       6.12a    15846
  7     0.90    0.00   24.20    7.61     94.91     7.61       7.61a    15846
  8     0.67    0.00   29.26    9.20    114.75     9.20       9.20a    15846
  9     0.44    0.00   34.32   10.80    134.59    10.80      10.80a    15846
 10     0.22    0.00   39.16   12.32    153.58    12.32      12.32a    15846
 11     0.00    0.00   44.00   13.84    172.56    13.84      13.84a    15847
                0.00   44.00   12.87    189.04    12.87      12.87a     3169
 12    -0.30    0.00   49.40   14.45    212.24    14.45      14.45a     3169
 13    -0.60    0.00   54.80   16.03    235.44    16.03      16.03a     3169
 14    -0.80    0.00   58.40   17.08    250.90    17.08      17.08a     3169
 15    -1.00    0.00   62.00   18.14    266.37    18.14      18.14a     3169
 16    -1.25    2.50   64.00   18.71    274.92    18.71      21.21a     3169
 17    -1.50    5.00   66.00   19.27    283.47    20.57      25.57      3169
 18    -1.80    8.00   68.40   19.96    293.73    24.56      32.56      3169
 19    -2.10   11.00   70.80   20.64    304.00    27.30      38.30      3169
 20    -2.40   14.00   73.20   21.32    314.26    29.16      43.16      3169
 21    -2.70   17.00   75.60   22.01    324.52    30.42      47.42      3169
 22    -3.00   20.00   78.00   22.69    334.78    31.34      51.34      3169
 23    -3.30   23.00   80.40   23.37    345.04    32.07      55.07      3169
 24    -3.60   26.00   82.80   24.05    355.30    32.73      58.73      3169
 25    -3.90   29.00   85.20   24.74    365.56    33.36      62.36      3169
 26    -4.20   32.00   87.60   25.42    375.82    34.00      66.00      3169
 27    -4.50   35.00   90.00   26.10    386.08    34.65      69.65      3169


                                        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     2.00    0.00    0.00    0.00      0.00     0.00       0.00      0.0
  2     1.83    0.00    0.00    0.00      0.00     0.00       0.00      0.0
  3     1.67    0.00    0.00    0.00      0.00     0.00       0.00      0.0
  4     1.50    0.00    0.00    0.00      0.00     0.00       0.00      0.0
  5     1.33    0.00    0.00    0.00      0.00     0.00       0.00      0.0
  6     1.11    0.00    0.00    0.00      0.00     0.00       0.00      0.0
  7     0.90    0.00    0.00    0.00      0.00     0.00       0.00      0.0
  8     0.67    0.00    0.00    0.00      0.00     0.00       0.00      0.0
  9     0.44    0.00    0.00    0.00      0.00     0.00       0.00      0.0
 10     0.22    0.00    0.00    0.00      0.00     0.00       0.00      0.0
 11     0.00    0.00    0.00    0.00      0.00     0.00       0.00      0.0
                0.00    0.00    0.00      0.00     0.00       0.00     5591
 12    -0.30    0.00    5.40    1.58     23.76    23.76      23.76p     5591
 13    -0.60    0.00   10.80    3.17     47.53    47.53      47.53p     5591
 14    -0.80    0.00   14.40    4.22     63.37    46.80      46.80      5591
 15    -1.00    0.00   18.00    5.28     79.21    40.08      40.08      5591
 16    -1.25    2.50   20.00    5.85     88.03    32.61      35.11      5591
 17    -1.50    5.00   22.00    6.42     96.85    27.39      32.39      5591
 18    -1.80    8.00   24.40    7.11    107.43    23.67      31.67      5591
 19    -2.10   11.00   26.80    7.80    118.01    22.14      33.14      5591
```

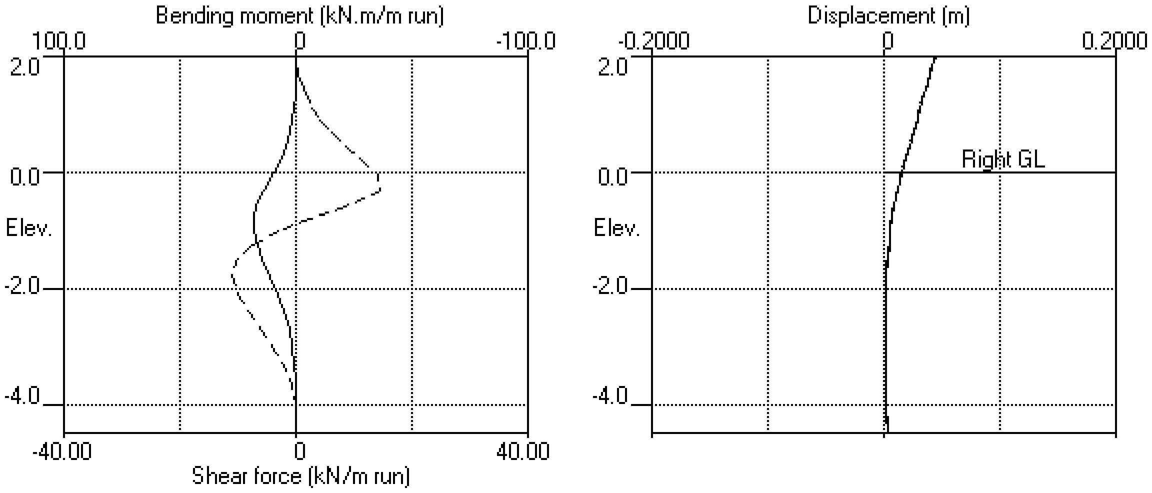
```
                                                              (continued)
Stage No.1   Fill to elevation 2.00 on LEFT side with soil type 2

                                         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
 20   -2.40    14.00   29.20    8.49    128.59     22.19      36.19       5591
 21   -2.70    17.00   31.60    9.18    139.18     23.27      40.27       5591
 22   -3.00    20.00   34.00    9.87    149.76     24.98      44.98       5591
 23   -3.30    23.00   36.40   10.56    160.34     27.00      50.00       5591
 24   -3.60    26.00   38.80   11.25    170.92     29.16      55.16       5591
 25   -3.90    29.00   41.20   11.94    181.51     31.35      60.35       5591
 26   -4.20    32.00   43.60   12.63    192.09     33.54      65.54       5591
 27   -4.50    35.00   46.00   13.32    202.67     35.72      70.72       5591

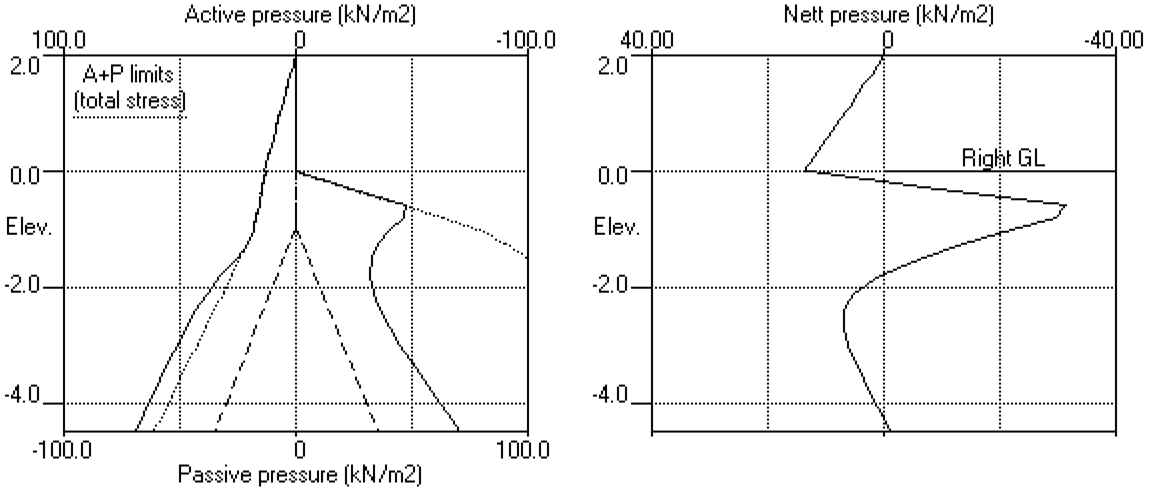
Note:     21.21 a  Soil pressure at active limit
          47.53 p  Soil pressure at passive limit
```

```
                                                       Units: kN,m
```

Stage No.1 Fill to elev. 2.00 on LEFT side



Stage No.1 Fill to elev. 2.00 on LEFT side



```
                                                             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. =    -4.50    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    2.00    0.00    Cant.   1.758   -4.07     -3.43    3.43      L to R
   2    2.00    0.00    Cant.   1.601   -4.08     -3.98    3.98      L to R
   3    2.00    0.00           No analysis at this stage
   4    2.00    0.00    Cant.   1.545   -4.06     -4.27    4.27      L to R
   5    2.00    0.00    Cant.   1.421   -4.01      ***      ***      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 = 100.00m
  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    2.00     0.109     0.000        0.0       0.0        0.0       0.0
  2    1.83     0.103     0.000        0.0       0.0        0.1       0.0
  3    1.67     0.097     0.000        0.0       0.0        0.4       0.0
  4    1.50     0.092     0.000        0.2       0.0        0.9       0.0
  5    1.33     0.086     0.000        0.4       0.0        5.7       0.0
  6    1.11     0.079     0.000        1.7       0.0        7.0       0.0
  7    0.90     0.071     0.000        3.4       0.0        8.7       0.0
  8    0.67     0.064     0.000        5.7       0.0       11.0       0.0
  9    0.44     0.056     0.000        8.5       0.0       25.8       0.0
 10    0.22     0.049     0.000       14.5       0.0       28.8       0.0
 11    0.00     0.042     0.000       21.2       0.0       32.3       0.0
 12   -0.30     0.033     0.000       31.3       0.0       33.4       0.0
 13   -0.60     0.024     0.000       40.7       0.0       28.0       0.0
 14   -0.80     0.019     0.000       45.9       0.0       20.8       0.0
 15   -1.00     0.015     0.000       49.2       0.0       10.5      -3.0
 16   -1.25     0.010     0.000       49.7       0.0        0.0     -10.2
 17   -1.50     0.006     0.000       46.8       0.0        0.0     -18.7
 18   -1.80     0.002     0.000       39.3       0.0        0.0     -27.7
 19   -2.10     0.000    -0.000       30.2       0.0        0.0     -29.8
 20   -2.40     0.000    -0.001       21.4       0.0        0.0     -27.2
 21   -2.70     0.000    -0.002       13.9       0.0        0.0     -22.1
 22   -3.00     0.000    -0.002        8.2       0.0        0.0     -16.3
 23   -3.30     0.000    -0.002        4.2       0.0        0.0     -10.8
 24   -3.60     0.000    -0.001        1.7       0.0        0.0      -6.2
 25   -3.90     0.000    -0.001        0.4      -0.1        0.0      -2.8
 26   -4.20     0.000    -0.001        0.0      -0.1        0.1      -0.7
 27   -4.50     0.000    -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       18.5   -0.80      -0.1   -3.90      14.4   -0.30     -11.0   -1.80
  2       22.5   -1.00      -0.0   -4.20      17.3   -0.30     -13.2   -1.80
  3    No calculation at this stage
  4       31.8   -1.00      -0.1   -4.20      21.3   -0.30     -18.7   -1.80
  5       49.7   -1.25      -0.0   -4.20      33.4   -0.30     -29.8   -2.10
```

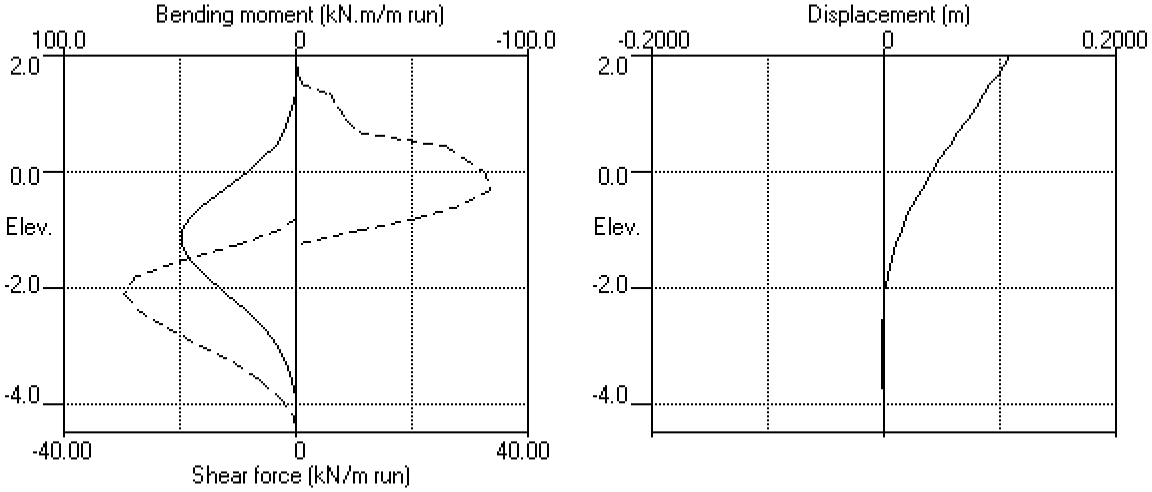
Summary of results (continued)

Maximum and minimum displacement at each stage

```
Stage -------- Displacement ---------
 no.  maximum  elev.   minimum  elev.   Stage description                    _
          m                m
  1    0.044    2.00    0.000    2.00   Fill to elev. 2.00 on LEFT side
  2    0.056    2.00    0.000    2.00   Apply surcharge no.1 at elev. 2.00
  3    Wall displacements reset to zero Change EI of wall to 1839kN.m2/m run
  4    0.033    2.00   -0.001   -2.70   Apply load no.1 at elev. 1.33
  5    0.109    2.00   -0.002   -3.00   Apply load no.2 at elev. 0.44
```

```
BECA LIMITED (NZ)                                           | Sheet No.
Program: WALLAP  Version 6.07  Revision A55.B74.R58         | Job No. 3160491
                             Licensed from GEOSOLVE         | Made by :    KM
Data filename/Run ID: STD-2_300mm_Dynamic_Case              |
Std Retaining Wall - 2m - Drained Case 6                    | Date:12-10-2023
Standardised Design                                         | Checked :
-----------------------------------------------------------------------------
                                                       Units: kN,m
```

Bending moment, shear force, displacement envelopes



Attachment 2: Bending Moment and Shear Capacity Checks

2.00m Timber Retaining Wall
(Kainga Ora HDS)
(Standardised Design Cases) [Rev2]

Design Calculations for Timber Pole Retaining Wall (max 2.0m retained height)

Contents:

Page 01: Cover
Page 02: Contents
Page 03 Notes
Page 04: Soil Profile/Soil Properties
Page 05: Methodology of Analysis
Page 06: Summary of Analysis/Design Actions and Factoring of Actions
Page 07: Pole Properties
Page 08: Design of Pole: Bending/Shear-Static
Page 09: Design of Pole: Bending/Shear-Seismic
Page 10: Design of Pole: Deflection/Actions on Rails
Page 11: Design of Rails: Bending/Shear
Page 12: Design of Rails: Deflection and
Page 12: Rail Connections/Drainage/Socket Diameter

Appendix A: Drawings
Appendix B: WALLAP Outputs
Appendix C: Hand Auger Log
Appendix D: Specification

Notes:

Design notes:

Timber pole retaining wall design designed to NZS:3603:1993
Timber pole wall design with sloping ground above the wall.

Notes to be on Drawings:

Notes:

- During all phases of work the engineer shall be informed on a daily basis as to the work anticipated to be carried out, to enable monitoring to be undertaken.
- The contractor shall locate and protect all services prior to commencing work & shall inform the engineer should any conflicts arise. The contractor shall be responsible for any damage to services caused by their activities.
- All timber shall be treated to NZS 3604 hazard class H5. Timber poles, waler & rails shall be radiata pine or corsican pine. The poles & rails shall be straight & free of decay, knots, splits, checks or any other defect that may affect the strength of the pole.
- All cut timber to be treated via site application of a suitable product to the suppliers specification to achieve a level of treatment equal to or greater than the members original level of treatment.
- All poles shall be placed large end into the base of the hole.
- Pile holes are not to remain open over night. Holes must be thoroughly cleaned out before placing concrete. Poles shall be installed as hit & miss pattern within the same day as boring.
- The Contractor is responsible for all temporary stability.
- Poles are to be braced during & after concreting to make sure the required alignment is maintained.
- All steel fixing components shall be hot dipped galvanised in accordance with AS/NZS 4680.
- Rail joints to occur at posts only. rail joints are to be staggered. rails to be secured to posts with 4.0mm, 120mm long self drilling screws
- Soil profiles, parameters and groundwater levels are standard conservative values based on observations across multiple Kainga Ora HDS hill sites, as specified by the Beca Geotechnical Verifier. The aim being to provide a standardised conservative retaining wall design which can be applied to HDS sites with retained heights of 0.5 - 1.5 m.
- Excavation shall be filled with approved AP40 hardfill. Compaction shall be in accordance with the appropriate Beca Geotechnical and Civil Specification for the Kainga Ora Project in question (varies per region).

Soil Profile

depth from top of wall

0.00m-1.50m = L1 - Engineered FILL (standard design specified parameters)
1.50m-10.0m = L2 - Standard design specified parameters for founding unit

Soil Properties

Layer 1 $\gamma_1 := 22 \ \frac{kN}{m^3}$ $\phi'_1 := 30 \ deg$ $c'_1 := 0 \ kPa$ $E_1 := 50000 \ \frac{kN}{m^2}$

Layer 2 $\gamma_2 := 18 \ \frac{kN}{m^3}$ $\phi'_2 := 30 \ deg$ $c'_2 := 0 \ kPa$ $E_2 := 10000 \ \frac{kN}{m^2}$

Wall Friction

$\frac{\delta}{\phi}$ ratio Active=0.67
Passive=0.5

Methodology of Analysis

Analysis of the wall section has been completed using the Wallap software to determine the pole embedment depth, wall loads (shear and bending) and estimate displacements.

Six load cases were analysed in WALLAP:
- Static case: dead load with house surcharge
- Extreme / seismic cases: static dead load with house surcharge and wind load applied to 1.8m fence on wall, static dead load with house surcharge and 0.5 m bgl excavation in front of toe, dynamic (ULS, 0.36g) load by monobe okabe, high GWL (-0.5 m bgl). For all cases the ground is considered to be flat at the top of the wall, and GWL is at -1 m bgl, driveway scenario with 12 kPa traffic surcharge set 0.3m from top of wall.

The soil/structure interaction has been modeled with unfactored loads with moderately conservative soil parameters. Bending moments and shear forces are then factored in the structural analysis of the poles and rails (see below). Embedment depths were calculated for a target Factor of Safety (FOS) of 1.5 for static loading (representative GWL) and 1.2 for seismic loading (representative GWL) using the Strength Factor Method

Wallap analysis in stages, this generally follows the following stages:

Staging:

- Stage 1 - Construct and fill active side to design levels
- Stage 2 - Reset wall stiffness (EI) and wall displacements
- Stage 3 - Apply house surcharge and/or wind loads, and/or over-excavation (depending on case)
- Extreme / Seismic Analysis
- (considers house dead load only) Stage 4 + 5 - Apply ULS Seismic Loading (at intervals set by Mononobe for a flexible wall)

Summary of Analysis

Design Action on Pole (Wallap Analysis)

Maximum Moment

$M := 15.73\ \frac{kNm}{m}$ (static) .23.6 with a factor of saftey of 1.3 $M = 15.73\ \frac{kNm}{m}$

$M_s := 41.42\ \frac{kNm}{m}$ (seismic)or(Extreme loads) $M_s = 41.42\ \frac{kNm}{m}$

Maximum Shear

$V := 9.53\ \frac{kN}{m}$ (static) .14.3 with a factor of saftey of 1.3 $V = 9.53\ \frac{kN}{m}$

$V_s := 27.8\ \frac{kN}{m}$ (seismic)or(Extreme loads) $V_s = 27.8\ \frac{kN}{m}$

Design of Pole

Pole Properties

Pole Diameter (SED) $SED := 300\ mm$

Pole Spacing $s_s := 1.2\ m$

Anchor Spacing $s_a := 0\ m$

Angle of Anchor $\omega := 0\ deg$

Factored Actions on Pole

Static Load Factor $LF := 1.5$

$M_{star} := M \cdot LF \cdot s_s$ $M_{star} = 28.314\ kNm$

$V_{star} := V \cdot LF \cdot s_s$ $V_{star} = 17.154\ kN$

Seismic Load Factor $LF_s := 1.2$

$M_{s.star} := M_s \cdot LF_s \cdot s_s$ $M_{s.star} = 59.64\ kNm$

$V_{s.star} := V_s \cdot LF_s \cdot s_s$ $V_{s.star} = 40.032\ kN$

Pole Properties

Second Moment of Area of Pole $I_p := \frac{\pi}{64} \cdot SED^4$ $I_p = 3.976 \times 10^8 \ \mathrm{mm}^4$

$I_{ps} := \frac{I_p}{s_s \cdot 1000^4}$ $I_{ps} = 3.313 \times 10^{-4} \ \frac{\mathrm{m}^4}{\mathrm{m}}$

Young's Modulus of Elasticity

$E_p := 8.7 \quad \mathrm{Gpa}$ (Table 7.1) NZS3606

$k20 := 0.95$ (Table 7.2) shaved and E

$k21 := 0.95$ (Table 7.3) steamed and E

Working Modulus of Elasticity $E_{pw} := k20 \cdot k21 \cdot E_p \cdot 1000$ $E_{pw} = 7.852 \times 10^3 \quad \mathrm{MPa}$

Stiffness of wall (input for Wallap)

$E_{pw} = 7.852 \times 10^3 \quad \mathrm{MPa}$

$I_p = 3.976 \times 10^8 \quad \mathrm{mm}^4$

$EI_p := \frac{\left(E_{pw} \cdot I_p\right)}{s_s \cdot 1000^3}$ $EI_p = 2.602 \times 10^3 \ \frac{\mathrm{kNm}^2}{\mathrm{m}}$

Static Design
Strength of Pole
Check Bending (maximum bending)

$M_{star} = 28.314\ \ kN$

$M_{s.star} < \phi Mn$

$\phi := 0.8$ (Clause 2.5) Pole

$Mn := k1 \cdot k8 \cdot k20 \cdot k21 \cdot k22 \cdot fb \cdot z$

$k1 := 0.6$ (Table 2.4) soil pressures

$k8 := 1.0$ (Table 2.8) green wet pole

$k20 := 0.85$ (Table 7.2) shaved and in bending

$k21 := 0.85$ (Table 7.3) steamed and in bending

$k22 := 1.0$ (Table 7.4) full pole wet

$fb := 38\ \ MPa$ (Table 7.1) green pole category 350

Length to maximum bending $x := 1.7\ \ m$

Taper rate (diameter taper) $tp := 6\ \dfrac{mm}{m}$

$df := SED + 0.8 \cdot x \cdot tp$ (80% length to bending) $df = 308.1($ mm pole diameter at bending

$z := \dfrac{\pi}{32} \cdot df^3 \qquad z = 2.873 \times 10^6\ \ mm^3$

$Mn := k1 \cdot k8 \cdot k20 \cdot k21 \cdot k22 \cdot fb \cdot z \cdot 10^{-6}$ $Mn = 47.326\ \ kNm$

$\phi Mn := \phi \cdot Mn \qquad \phi Mn = 37.861$ $M_{star} < \phi Mn$ $\phi Mn = 37.861\ \ kNm$

$M_{star} = 28.314$ < $\phi Mn = 37.861$ kNm $BM_{static} = "OK"$

Check Shear (maximum shear)

$V_{star} = 17.154\ \ kN$

$Vstar < \phi V$

$\phi := 0.8$ (Clause 2.4) Timber Pole

$V := k1 \cdot k20 \cdot k21 \cdot k22 \cdot fs \cdot As$

$k1 := 0.6$ (Table 2.4) permanent loading

$k20 := 1.0$ (Table 7.2) shaved and in shear

$k21 := 0.90$ (Table 7.3) steamed and in shear

$k22 := 1.0$ (Table 7.4) full pole wet

$fs := 3.1\ \ MPa$ (Table 7.1) green pole category 350

$As := \dfrac{\pi}{4} \cdot \left(\dfrac{SED}{1000}\right)^2 \qquad As = 0.071\ \ m^2$

$V := k1 \cdot k20 \cdot k21 \cdot k22 \cdot fs \cdot 10^3 \cdot As$ $V = 118.328\ \ kN$

$\phi V := \phi \cdot V \qquad \phi V = 94.662\ \ kN$ $\phi V = 94.662\ \ kN$

$Vstar < \phi V$

$V_{star} = 17.154$ < $\phi V = 94.662$ kN $Shear_{static} = "OK"$

Seismic Design

Strength of Pole

Check Bending (maximum bending)

$M_{s.star} = 59.645\ kN$

$M_{s.star} < \phi Mn$

$\phi := 0.8$ (Clause 2.5) Pole

$Mn := k1 \cdot k8 \cdot k20 \cdot k21 \cdot k22 \cdot fb \cdot z$

$k1_s := 1.0$ (Table 2.4) seismic actions

$k8 = 1$ (Table 2.8) green wet pole

$k20 := 0.85$ (Table 7.2) shaved and in bending

$k21 := 0.85$ (Table 7.3) steamed and in bending

$k22 = 1$ (Table 7.4) full pole wet

$fb := 38\ MPa$ (Table 7.1) green pole category 350

Length to maximum bending $x_s := 0.7\ m$

Taper rate (diameter taper) $tp = 6\ \frac{mm}{m}$

$df_s := SED + 0.8 \cdot x_s \cdot tp$ (80% length to bending) $df_s = 303.36\ mm$ pole diameter at bending

$z_s := \frac{\pi}{32} \cdot df_s^3 \quad z_s = 2.741 \times 10^6\ mm^3$

$Mn_s := k1_s \cdot k8 \cdot k20 \cdot k21 \cdot k22 \cdot fb \cdot z_s \cdot 10^{-6}$ $Mn_s = 75.248\ kNm$

$\phi Mn_s := \phi \cdot Mn_s \quad \phi Mn_s = 60.199$ $\phi Mn_s = 60.199\ kNm$

Mstar < ϕMn

$M_{s.star} = 59.645 < \phi Mn_s = 60.199\ kNm \qquad BM_{seismic} = \text{"OK"}$

Check Shear (maximum shear)

$V_{s.star} = 40.032\ kN$

$Vstar < \phi V$

$\phi := 0.8$ (Clause 2.4) Timber Pole

$V := k1 \cdot k20 \cdot k21 \cdot k22 \cdot fs \cdot As$

$k1_s := 1.0$ (Table 2.4) seismic action

$k20 := 1.0$ (Table 7.2) shaved and in shear

$k21 := 0.9$ (Table 7.3) steamed and in shear

$k22 = 1$ (Table 7.4) full pole wet

$fs := 3.1\ MPa$ (Table 7.1) green pole category 350

$As := \frac{\pi}{4} \cdot \left(\frac{SED}{1000}\right)^2 \qquad As = 0.071\ m^2$

$V_s := k1_s \cdot k20 \cdot k21 \cdot k22 \cdot fs \cdot 10^3 \cdot As$ $V_s = 197.213\ kN$

$\phi V_s := \phi \cdot V_s \qquad \phi V_s = 157.771\ kN$ $\phi V_s = 157.771\ kN$

Vstar < ϕV

$V_{s.star} = 40.032 < \phi V_s = 157.771\ kN \qquad Shear_{seismic} = \text{"OK"}$

Deflection

Maximum deflection at the top of pole, is equal to 38mm during the extreme seismic case (ULS).
Allowable deflection in seismic cases is considered to be 100 mm (MBIE EGEP Mod6, 2021)
Adopt wall as a 275mm SED pole placed SED up at 1.2m centres min density of 350 kg/m3, 1:10 wall angle (6%).

Actions on Rails (Ka conditions at base of rail)

Active pressures

From Wallap Analysis

Maximum Pressure on Rails (All cases) $P_r := 12.05$ kPa $P_r = 12.05$ kPa

Try min 50mm by 150mm - consider single simply supported span

Width of rail $b := 0.15$ m (vertical depth of rail)

Thickness of rail $d := 0.075$ m

Spacing of posts $s_s = 1.2$ m (span of rail)

Bending moment $M_b := \frac{\left(P_r \cdot b \cdot s_s^{\,2}\right)}{8}$ $M_b = 0.325$ kNm

Shear $V_b := \frac{\left(P_r \cdot b \cdot s_s\right)}{2}$ $V_b = 1.085$ kN

Design loads on rail

Load Factor $LF = 1.5$

$M_{star} := LF \cdot M_b$ $M_{star} = 0.488$ kNm

$V_{star} := LF \cdot V_b$ $V_{star} = 1.627$ kN

Check Bending

$M_{star} = 0.488 \quad kNm$

$M_{star} < \phi Mn$

$\phi := 0.8$ (Clause 2.5) Pole

$Mn := k1 \cdot k8 \cdot fb \cdot z$

$k1 := 0.6$ (Table 2.4) soil pressures

$k8 := 1.0$ (Table 2.8) green wet pole

$fb := 11.7\ MPa$ (Table 2.2) characteristic stresses for visually graded timber

$b := 0.15\ m \quad d = 0.075\ m$ rail dimensions

$z_r := \frac{b \cdot d^2 \cdot 1000^3}{6} \qquad z_r = 1.406 \times 10^5 \quad mm^3$

$Mn_r := k1 \cdot k8 \cdot fb \cdot z_r \cdot 10^{-6} \qquad Mn_r = 0.987 \quad kNm$

$\phi Mn_r := \phi \cdot Mn_r \quad \phi Mn_r = 0.79 \qquad \phi Mn_r = 0.79 \quad kNm$

$M_{star} < \phi Mn$

$M_{star} = 0.488 \quad < \quad \phi Mn_r = 0.79 \quad kNm \qquad BM_{rail} = \text{"OK"}$

Check Shear

$V_{star} = 1.627 \quad kN$

$Vstar < \phi V$

$\phi := 0.8$ (Clause 2.4) Timber Pole

$V := k1 \cdot fs \cdot As$

$k1 := 0.6$ (Table 2.4) permanent loading

$fs := 2.4 \quad MPa$ (Table 2.2) characteristic stresses for green Radiata pine

$As := \frac{\left(2 \cdot b \cdot d \cdot 1000^2\right)}{3} \quad As = 7.5 \times 10^3 \quad mm^2$

$V_r := k1 \cdot fs \cdot 10^{-3} \cdot As \qquad V_r = 10.8 \quad kN$

$\phi V_r := \phi \cdot V_r \qquad \phi V_r = 8.64 \quad kN \qquad \phi V_r = 8.64 \quad kN$

$Vstar < \phi V$

$V_{star} = 1.627 < \quad \phi V_r = 8.64 \quad kN \qquad Shear_{rail} = \text{"OK"}$

Adopt 150mmx75mm VSG8 rails spanning 1.2m between poles.

Check Deflection

Pressure on rail $P_r = 12.05$ kPa

Diameter of rail $b = 0.15$ m

Long term deflection $k2 := 3$

UDL on Rail $w_r := P_r \cdot \frac{b}{1}$ $w_r = 1.808 \ \frac{kN}{m}$

Rail Span $s_s = 1.2$

Second Moment of Area of rail $I_r := \frac{b \cdot d^3 \cdot 1000^4}{12}$ $I_r = 5.273 \times 10^6 \ mm^4$

Young's Modulus of Elasticity

$E_p := 8.'\ Gpa$ (Table 7.1) NZS3606

$k20 := 0.95$ Table 7.2 shaved and E

$k21 := 0.95$ (Table 7.3) steamed and E

Working Modulus of Elasticity $E_{pw} := k20 \cdot k21 \cdot E_p \cdot 1000$ $E_{pw} = 7.852 \times 10^3$ MPa

Stiffness of rail

$E_{pw} = 7.852 \times 10^3$ MPa

$I_r = 5.273 \times 10^6 \ mm^4$

$EI_r := E_{pw} \cdot 1000 \cdot \frac{I_r}{1000^4}$ $EI_r = 41.406 \ kNm^2$

$\Delta_r := \frac{\left(5 \cdot k2 \cdot w_r \cdot s_s^{\,4}\right)}{\left(384 \cdot EI_r\right)}$ $\Delta_r = 3.536 \times 10^{-3}$ mm

Adopt 150mmx75mm VSG8 rails spanning 1.2m between poles.

Rail Connection

Rails to be screwed to poles using 4.0mm self drilling screw, coach bolt top rail to pole.

Socket

$SED = 300$ mm

$tp := 6 \ \frac{mm}{m}$

$PL := 5.0$ m Pole length

Average thickness of cover $t_c := 50$ mm

Diameter of Socket $D_s := SED + PL \cdot tp + 2 \cdot t_c$ $D_s = 430$ mm

Adopt Minimum Socket Diameter of 0.40m with 20MPa concrete

Attachment 3: ULS Dynamic Earthquake Loading (Mononobe-Okabe Method)

GE - Retaining Systems - F - Calculation of seismic pressures and loads for rigid and flexible retaining walls

Calculation: Seismic Earth Pressures and Loads for Yielding or Flexible Walls

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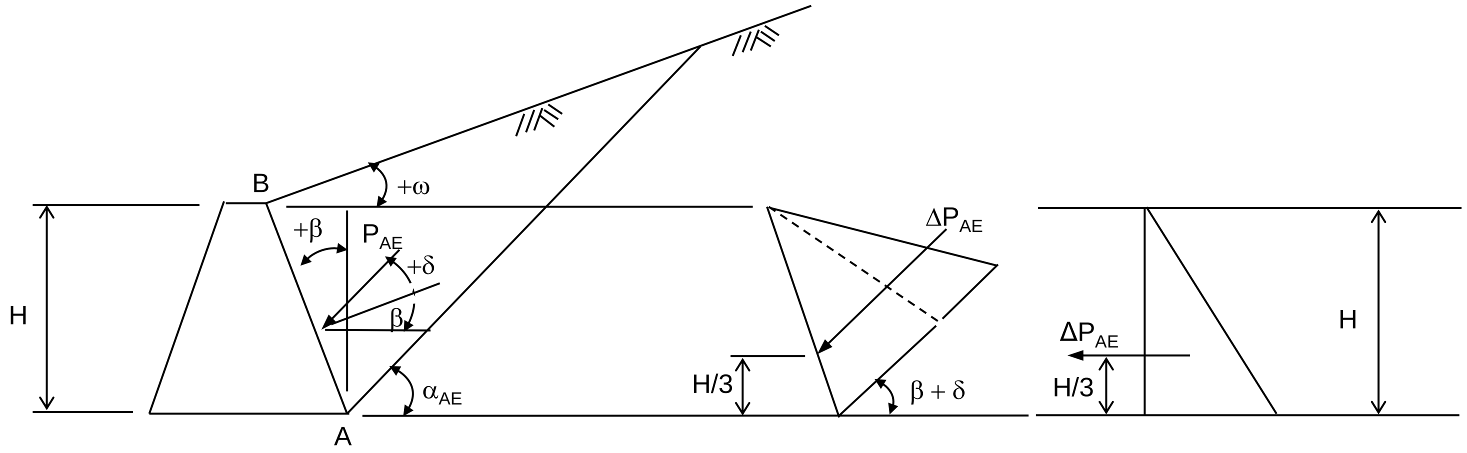
Mononobe-Okabe Equation for Increment in Lateral Earth Pressure Under Seismic Loading (Cohesionless Soil) for Yielding or Flexible Walls (Wall displacements > 0.5% x Height of Wall)

Reference: Wood, J. H.; Elms, D. G. (1990): "Seismic Design of Bridge Abutments and Retaining Walls" 84(2), Wellington, N.Z. : Road Research Unit, Transit New Zealand, RRU bulletin, 0549-0030



Failure Plane For Earthquake Loading

Active Earthquake Pressure on A-B

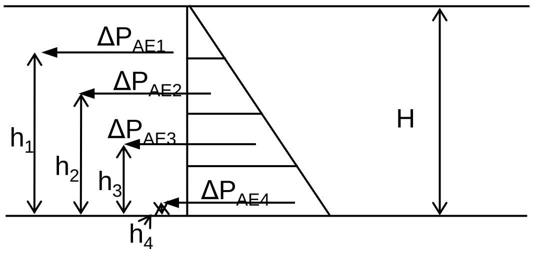
Seismic coefficient for earth retaining structures	$C_0 = k_h =$	0.36	g	
	$\theta = \tan^{-1}(k_h/(1-k_v)) =$	19.8	º	(Assume $k_v = 0$)
Slope of the back of wall (from vertical, +ve anticlockwise)	$\beta =$	0	º	
Angle of ground slope	$\omega =$	0	º	
Angle of shearing resistance / friction angle (total stress)	$\phi =$	30	º	
Angle of wall friction, in general 1/2 or 2/3 of angle of soil friction	$\delta =$	0.6667	ϕ	
	$\delta =$	20.0	º	
Angle of failure plane	$\alpha_{AE} =$	31.3	º	
	Top Level of Wall	2	m RL	
	Bottom Level of Wall	0	m RL	
Vertical height of wall on which earth pressure is calculated	H =	2	m	
Number of distributed horizontal seismic loads along height of exposed retaining wall (maximum input number 20)	i=	2		
Unit weight of soil	$\gamma =$	22	kN/m^3	
Mononobe Okabe active pressure coefficient (total gravity plus earthquake component)	$K_{AE} =$	0.664		
Coefficient of active earth pressure	$K_A =$	0.297		
Coefficient of earth pressure at rest	$K_O =$	0.500		
Coefficient of earthquake earth pressure	$\Delta K_{AE} = K_{AE} - K_A =$	0.367		

Earhquake increment of wall force	$\Delta P_{AE} =$	**16.1**	**kN/m**
Location of ΔP_{AE} - 1/3 *H* from base of wall		**0.67**	**m from base**

Seismic Loads for input in Wallap and other programs

i	Distance from top of wall (m)	Location of ΔP_{AE} (mRL)	ΔP_{AE} (kN/m)
1	**0.67**	**1.33**	**4.04**
2	**1.56**	**0.44**	**12.11**

Example of load distribution for for loads



$$K_{AE} = \frac{\cos^2(\phi-\beta-\theta)}{\cos\theta\cos^2\beta\cos(\delta+\beta+\theta)\left(1+\left(\frac{\sin(\phi+\delta)\sin(\phi-\omega-\theta)}{\cos(\delta+\beta+\theta)\cos(\beta-\omega)}\right)^{0.5}\right)^2}$$

$$K_A = \frac{\cos^2(\phi-\beta)}{\cos^2\beta\cos(\delta+\beta)\left(1+\left(\frac{\sin(\phi+\delta)\sin(\phi-\omega)}{\cos(\delta+\beta)\cos(\omega-\beta)}\right)^{0.5}\right)^2}$$

$$\cot(\alpha_{AE}-\omega) = -\tan(\phi+\delta+\beta-\omega)+\sec(\phi+\delta+\beta-\omega)\left(\frac{\cos(\beta+\delta+\theta)\sin(\phi+\delta)}{\cos(\beta-\omega)\sin(\phi-\theta-\omega)}\right)^{0.5}$$

$$\Delta P_{AE} = \frac{1}{2}(K_{AE}-K_A)\gamma H^2$$

G

Appendix G – CBR Calculation Sheets

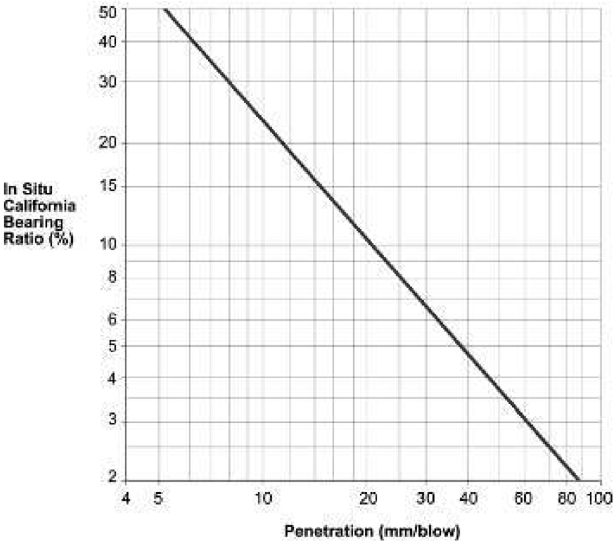
Job Name	Job Number	Date
HDS Christchurch MBU1	3160491	22/02/2024
Site Address		**Engineer**
6 - 10 Orr Street, Ashburton		Kiri Moonen

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-001

AUSTROADS
PART 2: Pavement Structural Design
CALIFORNIA BEARING RATIO

Reference: Austroads Ltd. (2017). Guide to Pavement Technology Part 2: Pavement Structural Design. Sydney: Austroads Ltd. (Section 5.5.2, Figure 5.3)



Ground Level (mRL)	95.6			
Depth (mm)	**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Austroads Correlated CBR (%)**
0 - 100	95.50	*3*	33.3	6
100 - 200	95.40	*8*	12.5	17
200 - 300	95.30	*21*	4.8	50

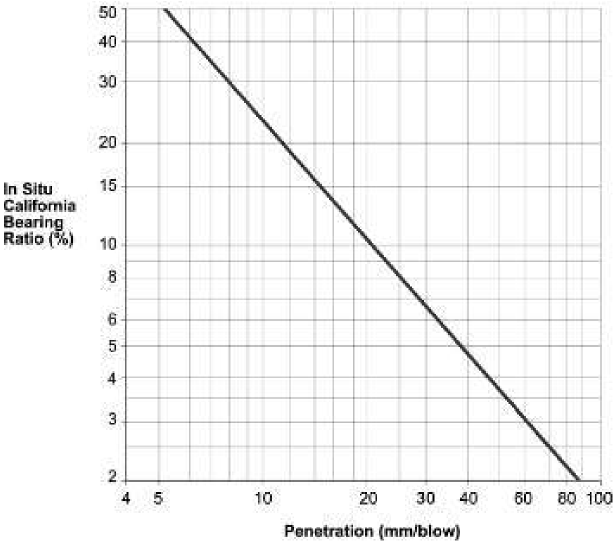
Weighted average: 50+

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-002

AUSTROADS
PART 2: Pavement Structural Design
CALIFORNIA BEARING RATIO

Reference: Austroads Ltd. (2017). Guide to Pavement Technology Part 2: Pavement Structural Design. Sydney: Austroads Ltd. (Section 5.5.2, Figure 5.3)



Ground Level (mRL)	95.8

Depth (mm)	m RL	Measured No. Blows / 100mm	e mm/blow	Austroads Correlated CBR (%)
0 - 100	95.70	5	20.0	10
100 - 200	95.60	9	11.1	20
200 - 300	95.50	21	4.8	50

Weighted average: 50+

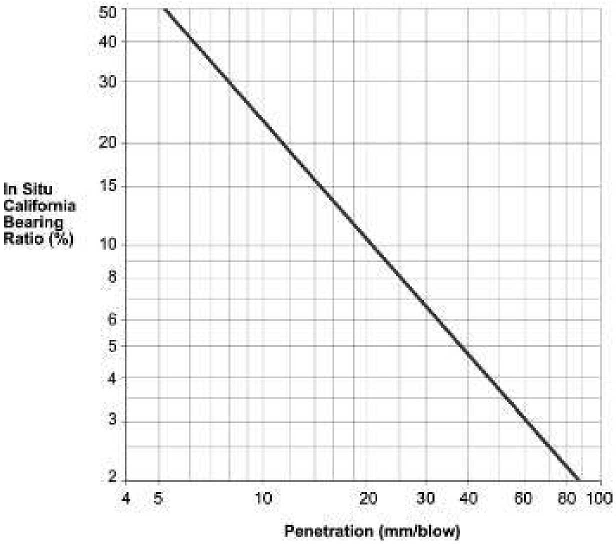
Job Name	Job Number	Date
HDS Christchurch MBU1	3160491	22/02/2024
Site Address		**Engineer**
6 - 10 Orr Street, Ashburton		Kiri Moonen

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-003

AUSTROADS
PART 2: Pavement Structural Design
CALIFORNIA BEARING RATIO

Reference: Austroads Ltd. (2017). Guide to Pavement Technology Part 2: Pavement Structural Design. Sydney: Austroads Ltd. (Section 5.5.2, Figure 5.3)



Ground Level (mRL)	95.7			
Depth (mm)	**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Austroads Correlated CBR (%)**
0 - 100	95.60	4	25.0	8
100 - 200	95.50	5	20.0	10
200 - 300	95.40	6	16.7	12
300 - 400	95.30	4	25.0	8
400 - 500	95.20	4	25.0	8
500 - 600	95.10	4	25.0	8
600 - 700	95.00	4	25.0	8
700 - 800	94.90	4	25.0	8
800 - 900	94.80	4	25.0	8
900 - 1000	94.70	3	33.3	6
1000 - 1100	94.60	4	25.0	8
1100 - 1200	94.50	5	20.0	10
1200 - 1300	94.40	5	20.0	10
1300 - 1400	94.30	5	20.0	10
1400 - 1500	94.20	4	25.0	8

Weighted average: 8.1

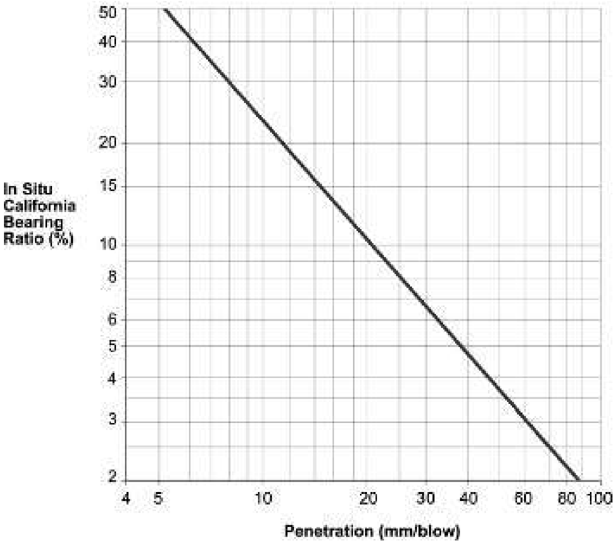
Job Name	Job Number	Date
HDS Christchurch MBU1	3160491	22/02/2024
Site Address		**Engineer**
6 - 10 Orr Street, Ashburton		Kiri Moonen

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-004

AUSTROADS
PART 2: Pavement Structural Design
CALIFORNIA BEARING RATIO

Reference: Austroads Ltd. (2017). Guide to Pavement Technology Part 2: Pavement Structural Design. Sydney: Austroads Ltd. (Section 5.5.2, Figure 5.3)



Ground Level (mRL)	95.9			
Depth (mm)	**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Austroads Correlated CBR (%)**
0 - 100	95.80	*4*	25.0	8
100 - 200	95.70	*8*	12.5	17
200 - 300	95.60	*16*	6.3	40
300 - 400	95.50	21	4.8	50

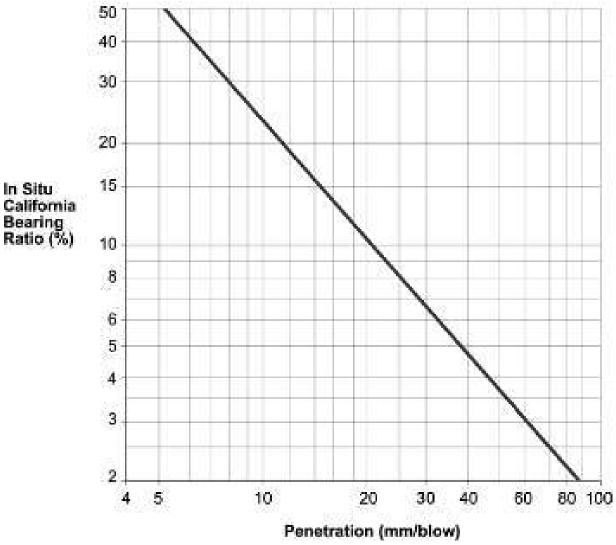
Weighted average: 50.0

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-005

AUSTROADS
PART 2: Pavement Structural Design
CALIFORNIA BEARING RATIO

Reference: Austroads Ltd. (2017). Guide to Pavement Technology Part 2: Pavement Structural Design. Sydney: Austroads Ltd. (Section 5.5.2, Figure 5.3)



Ground Level (mRL)			95.6			
Depth (mm)			**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Austroads Correlated CBR (%)**
0	-	100	95.50	*4*	25.0	8
100	-	200	95.40	*7*	14.3	15
200	-	300	95.30	*10*	10.0	20
300	-	400	95.20	13	7.7	30
400	-	500	95.10	15	6.7	35
500	-	600	95.00	13	7.7	30
600	-	700	94.90	12	8.3	25
700	-	800	94.80	9	11.1	20
800	-	900	94.70	7	14.3	15
900	-	1000	94.60	6	16.7	12
1000	-	1100	94.50	18	5.6	45
1100	-	1200	94.40	14	7.1	30
1200	-	1300	94.30	6	16.7	12
1300	-	1400	94.20	8	12.5	17
1400	-	1500	94.10	21	4.8	50

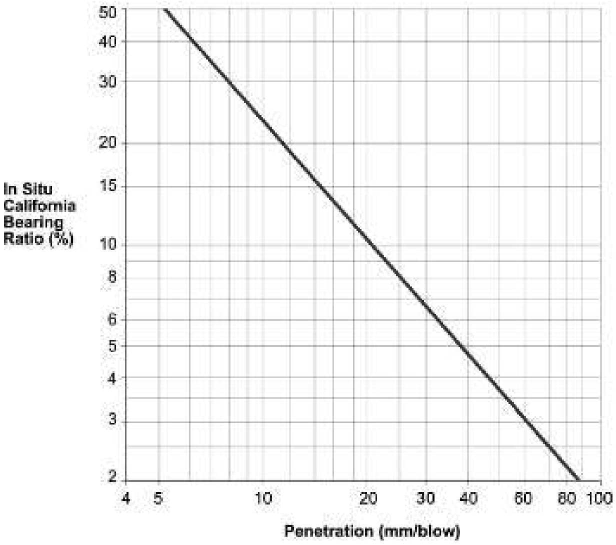
Weighted average: 29.7

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-006

AUSTROADS
PART 2: Pavement Structural Design
CALIFORNIA BEARING RATIO

Reference: Austroads Ltd. (2017). Guide to Pavement Technology Part 2: Pavement Structural Design. Sydney: Austroads Ltd. (Section 5.5.2, Figure 5.3)



Ground Level (mRL)	95.6			
Depth (mm)	**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Austroads Correlated CBR (%)**
0 - 100	95.50	2	50.0	4
100 - 200	95.40	5	20.0	10
200 - 300	95.30	6	16.7	12
300 - 400	95.20	7	14.3	15
400 - 500	95.10	6	16.7	12
500 - 600	95.00	6	16.7	12
600 - 700	94.90	6	16.7	12
700 - 800	94.80	5	20.0	10
800 - 900	94.70	5	20.0	10
900 - 1000	94.60	8	12.5	17
1000 - 1100	94.50	7	14.3	15
1100 - 1200	94.40	10	10.0	20
1200 - 1300	94.30	21	4.8	50

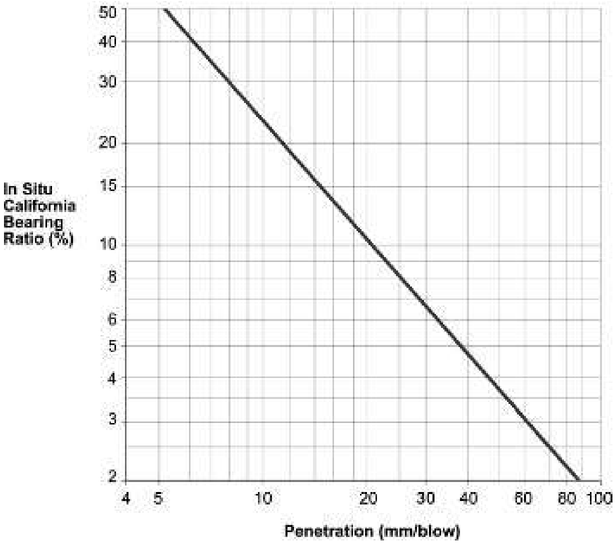
Weighted average: 14.8

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-007

AUSTROADS
PART 2: Pavement Structural Design
CALIFORNIA BEARING RATIO

Reference: Austroads Ltd. (2017). Guide to Pavement Technology Part 2: Pavement Structural Design. Sydney: Austroads Ltd. (Section 5.5.2, Figure 5.3)



Ground Level (mRL) 95.6

Depth (mm)			m RL	Measured No. Blows / 100mm	e mm/blow	Austroads Correlated CBR (%)
0	-	100	95.50	3	33.3	6
100	-	200	95.40	5	20.0	10
200	-	300	95.30	3	33.3	6
300	-	400	95.20	7	14.3	15
400	-	500	95.10	8	12.5	17
500	-	600	95.00	6	16.7	12
600	-	700	94.90	5	20.0	10
700	-	800	94.80	6	16.7	12
800	-	900	94.70	5	20.0	10
900	-	1000	94.60	5	20.0	10
1000	-	1100	94.50	7	14.3	15
1100	-	1200	94.40	12	8.3	25
1200	-	1300	94.30	11	9.1	25
1300	-	1400	94.20	21	4.8	50

Weighted average: 16.2

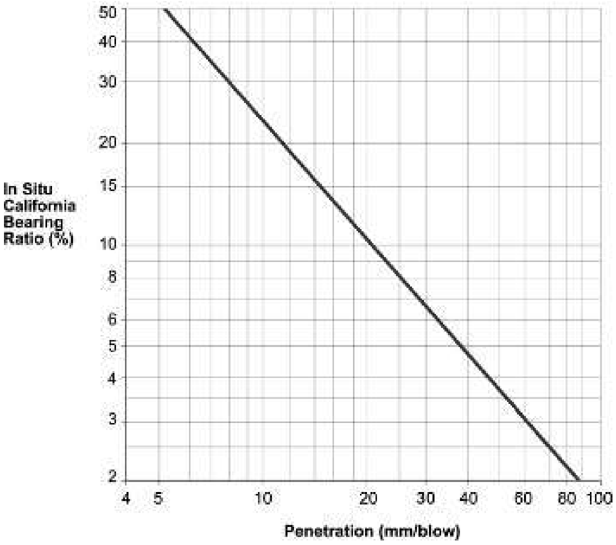
Job Name	Job Number	Date
HDS Christchurch MBU1	3160491	22/02/2024
Site Address		**Engineer**
6 - 10 Orr Street, Ashburton		Kiri Moonen

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-008

AUSTROADS
PART 2: Pavement Structural Design
CALIFORNIA BEARING RATIO

Reference: Austroads Ltd. (2017). Guide to Pavement Technology Part 2: Pavement Structural Design. Sydney: Austroads Ltd. (Section 5.5.2, Figure 5.3)



Ground Level (mRL)	96			
Depth (mm)	**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Austroads Correlated CBR (%)**
0 - 100	95.90	*4*	25.0	8
100 - 200	95.80	*11*	9.1	25
200 - 300	95.70	*21*	4.8	50

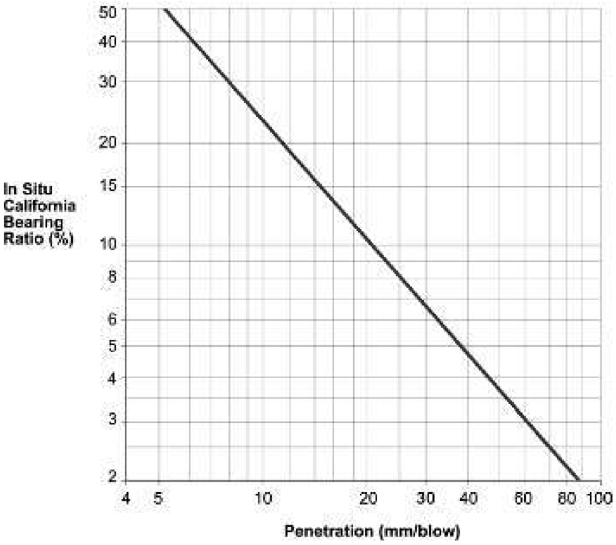
Weighted average: 50+

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-009

AUSTROADS
PART 2: Pavement Structural Design
CALIFORNIA BEARING RATIO

Reference: Austroads Ltd. (2017). Guide to Pavement Technology Part 2: Pavement Structural Design. Sydney: Austroads Ltd. (Section 5.5.2, Figure 5.3)



Ground Level (mRL) 96.3

Depth (mm)	m RL	Measured No. Blows / 100mm	e mm/blow	Austroads Correlated CBR (%)
0 - 100	96.20	3	33.3	6
100 - 200	96.10	7	14.3	15
200 - 300	96.00	17	5.9	40
300 - 400	95.90	20	5.0	50
400 - 500	95.80	13	7.7	30
500 - 600	95.70	6	16.7	12
600 - 700	95.60	7	14.3	15
700 - 800	95.50	10	10.0	20
800 - 900	95.40	21	4.8	50

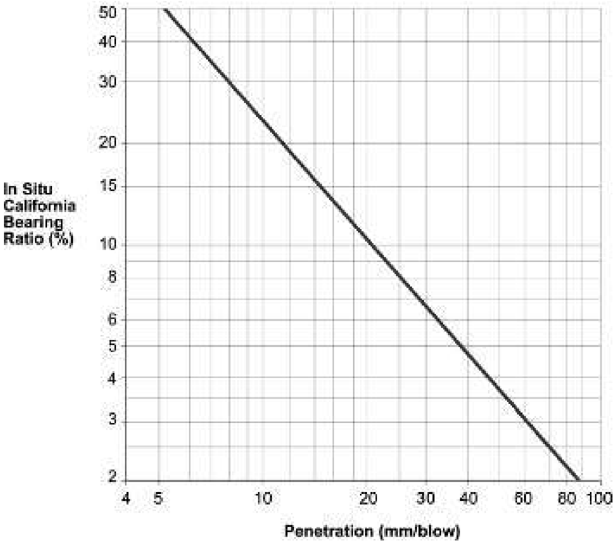
Weighted average: 30.0

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-010

AUSTROADS
PART 2: Pavement Structural Design
CALIFORNIA BEARING RATIO

Reference: Austroads Ltd. (2017). Guide to Pavement Technology Part 2: Pavement Structural Design. Sydney: Austroads Ltd. (Section 5.5.2, Figure 5.3)



Ground Level (mRL)	96.1			
Depth (mm)	**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Austroads Correlated CBR (%)**
0 - 100	96.00	*12*	8.3	25
100 - 200	95.90	*15*	6.7	35
200 - 300	95.80	*21*	4.8	50

Weighted average: 50+

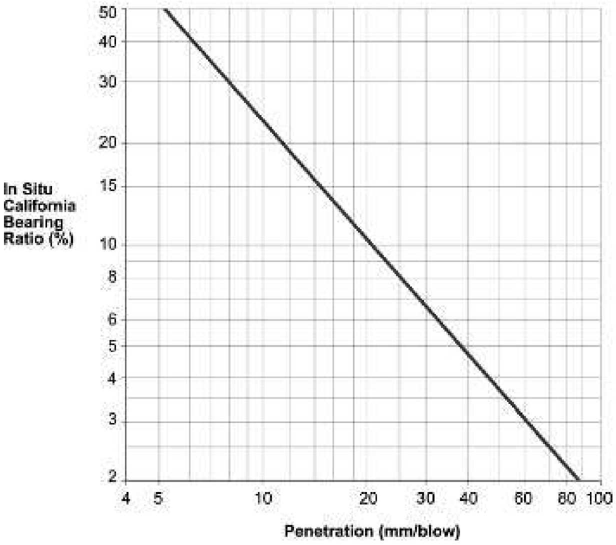
Job Name	Job Number	Date
HDS Christchurch MBU1	3160491	22/02/2024
Site Address		**Engineer**
6 - 10 Orr Street, Ashburton		Kiri Moonen

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-011

AUSTROADS
PART 2: Pavement Structural Design
CALIFORNIA BEARING RATIO

Reference: Austroads Ltd. (2017). Guide to Pavement Technology Part 2: Pavement Structural Design. Sydney: Austroads Ltd. (Section 5.5.2, Figure 5.3)



Ground Level (mRL)	95.9			
Depth (mm)	**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Austroads Correlated CBR (%)**
0 - 100	95.50	*8*	12.5	17
100 - 200	95.40	*20*	5.0	50
200 - 300	95.30	*21*	4.8	50

Weighted average: 50+

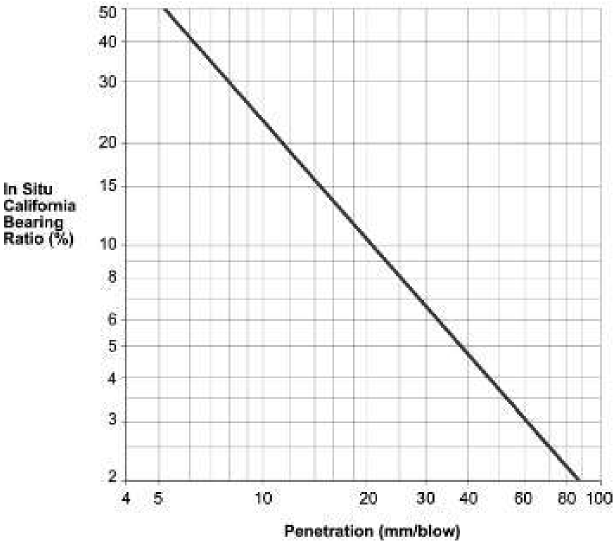
Job Name	Job Number	Date
HDS Christchurch MBU1	3160491	22/02/2024
Site Address		**Engineer**
6 - 10 Orr Street, Ashburton		Kiri Moonen

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-012

AUSTROADS
PART 2: Pavement Structural Design
CALIFORNIA BEARING RATIO

Reference: Austroads Ltd. (2017). Guide to Pavement Technology Part 2: Pavement Structural Design. Sydney: Austroads Ltd. (Section 5.5.2, Figure 5.3)



Ground Level (mRL)	96.2

Depth (mm)			m RL	Measured No. Blows / 100mm	e mm/blow	Austroads Correlated CBR (%)
0	-	100	95.50	*8*	12.5	17
100	-	200	95.40	*8*	12.5	17
200	-	300	95.30	*21*	4.8	50

Weighted average: 50+

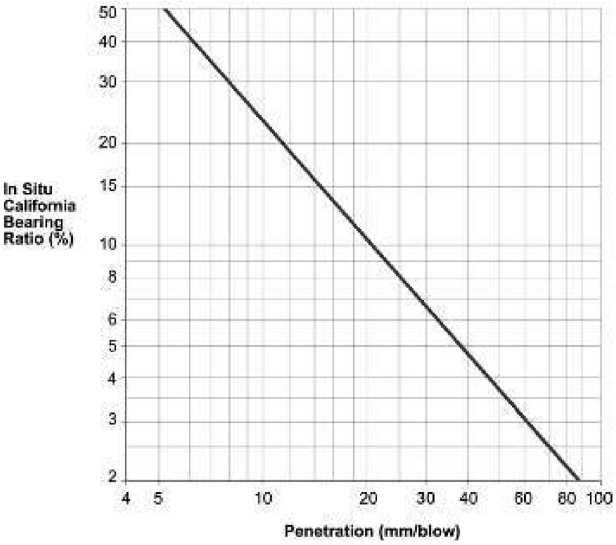
Job Name	Job Number	Date
HDS Christchurch MBU1	3160491	22/02/2024
Site Address		**Engineer**
6 - 10 Orr Street, Ashburton		Kiri Moonen

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-013

AUSTROADS
PART 2: Pavement Structural Design
CALIFORNIA BEARING RATIO

Reference: Austroads Ltd. (2017). Guide to Pavement Technology Part 2: Pavement Structural Design. Sydney: Austroads Ltd. (Section 5.5.2, Figure 5.3)



Ground Level (mRL)			96.1			
Depth (mm)			**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Austroads Correlated CBR (%)**
0	-	100	95.50	5	20.0	10
100	-	200	95.40	6	16.7	12
200	-	300	95.30	6	16.7	12
300	-	400	95.20	6	16.7	12
400	-	500	95.10	7	14.3	15
500	-	600	95.00	8	12.5	17
600	-	700	94.90	6	16.7	12
700	-	800	94.80	7	14.3	15
800	-	900	94.70	18	5.6	45
900	-	1000	94.60	21	4.8	50

Weighted average: 19.4

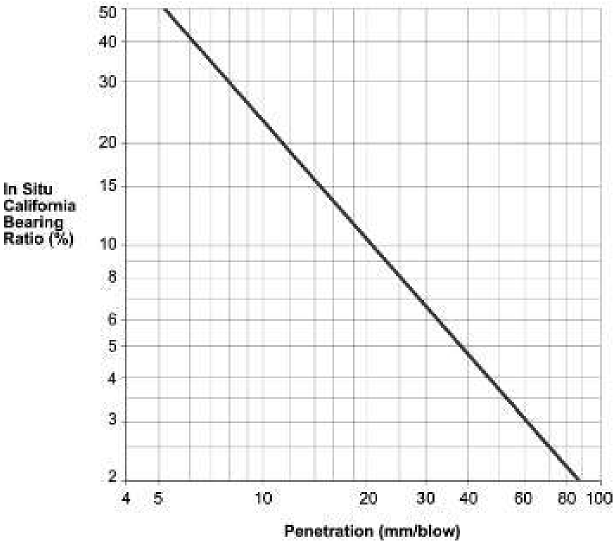
Job Name	Job Number	Date
HDS Christchurch MBU1	3160491	22/02/2024
Site Address		**Engineer**
6 - 10 Orr Street, Ashburton		Kiri Moonen

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-014

AUSTROADS
PART 2: Pavement Structural Design
CALIFORNIA BEARING RATIO

Reference: Austroads Ltd. (2017). Guide to Pavement Technology Part 2: Pavement Structural Design. Sydney: Austroads Ltd. (Section 5.5.2, Figure 5.3)



Ground Level (mRL)	96.1			
Depth (mm)	**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Austroads Correlated CBR (%)**
0 - 100	95.90	5	20.0	10
100 - 200	95.80	8	12.5	17
200 - 300	95.70	13	7.7	30
300 - 400	95.60	15	6.7	35
400 - 500	95.50	20	5.0	50
500 - 600	95.40	19	5.3	45
600 - 700	95.30	21	4.8	50

Weighted average: 45.3

Job Name	Job Number	Date
HDS Christchurch MBU1	3160491	22/02/2024

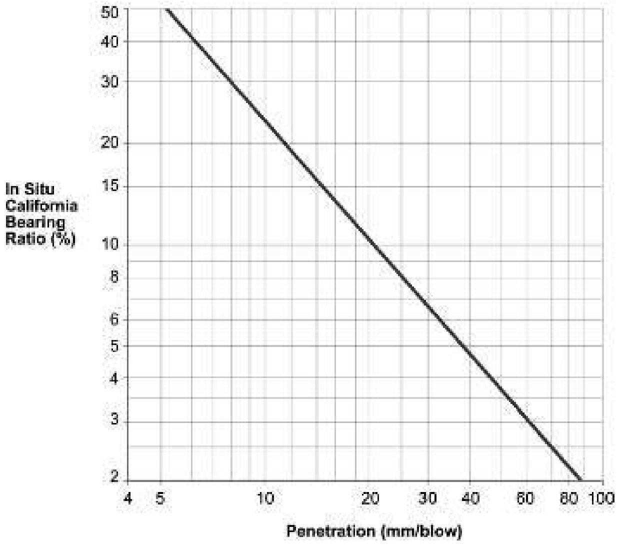
Site Address	Engineer
6 - 10 Orr Street, Ashburton	Kiri Moonen

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-015

AUSTROADS
PART 2: Pavement Structural Design
CALIFORNIA BEARING RATIO

Reference: Austroads Ltd. (2017). Guide to Pavement Technology Part 2: Pavement Structural Design. Sydney: Austroads Ltd. (Section 5.5.2, Figure 5.3)



Ground Level (mRL)	96

Depth (mm)			m RL	Measured No. Blows / 100mm	e mm/blow	Austroads Correlated CBR (%)
0	-	100	96.20	*8*	12.5	17
100	-	200	96.10	*10*	10.0	20
200	-	300	96.00	*10*	10.0	20
300	-	400	95.90	16	6.3	40
400	-	500	95.80	20	5.0	50
500	-	600	95.70	21	4.8	50

Weighted average: 46.7

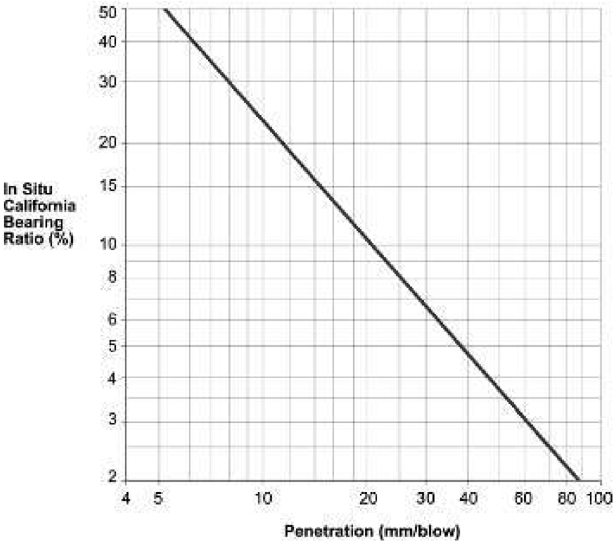
Job Name	Job Number	Date
HDS Christchurch MBU1	3160491	22/02/2024
Site Address		**Engineer**
6 - 10 Orr Street, Ashburton		Kiri Moonen

GE - Determination of Bearing Pressures and California Bearing Ratios

AR109526-GE-HA-016

AUSTROADS
PART 2: Pavement Structural Design
CALIFORNIA BEARING RATIO

Reference: Austroads Ltd. (2017). Guide to Pavement Technology Part 2: Pavement Structural Design. Sydney: Austroads Ltd. (Section 5.5.2, Figure 5.3)



Ground Level (mRL)	96.1			
Depth (mm)	**m RL**	**Measured No. Blows / 100mm**	**e mm/blow**	**Austroads Correlated CBR (%)**
0 - 100	96.00	5	20.0	10
100 - 200	95.90	8	12.5	17
200 - 300	95.80	9	11.1	20
300 - 400	95.70	10	10.0	20
400 - 500	95.60	15	6.7	35
500 - 600	95.50	21	4.8	50

Weighted average: 35.0

H

Appendix H – Geotechnical PS1 Producer Statement

PRODUCER STATEMENT – PS1
DESIGN

BUILDING CODE CLAUSE(S): B1 **JOB NUMBER:** 3160491

ISSUED BY: Beca Limited
(Engineering Design Firm)

TO: Kainga Ora
(Owner/Developer)

TO BE SUPPLIED TO: Consentium
(Building Consent Authority)

IN RESPECT OF: Geotechnical design of foundations and timber pole retaining walls for new state houses
(Description of Building Work)

AT: 6, 8 and 10 Orr Street, Netherby, Ashburton
(Address, Town/City)

LEGAL DESCRIPTION: Lots 3, 4 and 5, DP 18886 **N/A** ☐

We have been engaged by the owner/developer referred to above to provide *(Extent of Engagement)*:
Geotechnical design services for new state house development and subdivision of land
in respect of the requirements of the Clause(s) of the Building Code specified above for Part only , as specified in the Schedule, of the proposed building work.

The design carried out by us has been prepared in accordance with:

- ☐Compliance documents issued by the Ministry of Business, Innovation & Employment *(Verification method/acceptable solution)* and/or;
- ☑Alternative solution as per the attached Schedule.

The proposed building work covered by this producer statement is described on the drawings specified in the Schedule, together with the specification, and other documents set out in the Schedule.

On behalf of the Engineering Design Firm, and subject to:

- Site verification of the following design assumptions: Depth to 300kPa geotechnical ultimate bearing capacity. .
- All proprietary products meeting their performance specification requirements;

I believe on reasonable grounds that:

- the building, if constructed in accordance with the drawings, specifications, and other documents provided or listed in the Schedule, will comply with the relevant provisions of the Building Code and that;
- the persons who have undertaken the design have the necessary competency to do so.

I recommend the CM2 level of **construction monitoring**.

I, *(Name of Engineering Design Professional)* Samuel Glue , am:

- ☑CPEng number 248637

and hold the following qualifications BE Hons (Civil)

The Engineering Design Firm holds a current policy of Professional Indemnity Insurance no less than $200,000
The Engineering Design Firm is a member of ACE New Zealand.

SIGNED BY *(Name of Engineering Design Professional)*: Samuel Glue
(Signature below):

SGlue

ON BEHALF OF *(Engineering Design Firm)*: Beca Limited Date: 12/03/2024

Note: This statement has been prepared solely for the Building Consent Authority named above and shall not be relied upon by any other person or entity. Any liability in relation to this statement accrues to the Engineering Design Firm only. As a condition of reliance on this statement, the Building Consent Authority accepts that the total maximum amount of liability of any kind arising from this statement and all other statements provided to the Building Consent Authority in relation to this building work, whether in tort or otherwise, is limited to the sum of $200,000.

This form is to accompany **Form 2 of the Building (Forms) Regulations 2004** for the application of a Building Consent.

SCHEDULE to PS1

Please include an itemised list of all referenced documents, drawings, or other supporting materials in relation to this producer statement below:

6-10 Orr Street, Netherby, Ashburton. Geotechnical Design Report

GUIDANCE ON USE OF PRODUCER STATEMENTS

Information on the use of Producer Statements and Construction Monitoring Guidelines can be found on the Engineering New Zealand website
https://www.engineeringnz.org/engineer-tools/engineering-documents/producer-statements/

Producer statements were first introduced with the Building Act 1991. The producer statements were developed by a combined task committee consisting of members of the New Zealand Institute of Architects (NZIA), Institution of Professional Engineers New Zealand (now Engineering New Zealand), Association of Consulting and Engineering New Zealand (ACE NZ) in consultation with the Building Officials Institute of New Zealand (BOINZ). The original suite of producer statements has been revised at the date of this form to ensure standard use within the industry.

The producer statement system is intended to provide Building Consent Authorities (BCAs) with part of the reasonable grounds necessary for the issue of a Building Consent or a Code Compliance Certificate, without necessarily having to duplicate review of design or construction monitoring undertaken by others.

PS1 DESIGN Intended for use by a suitably qualified independent engineering design professional in circumstances where the BCA accepts a producer statement for establishing reasonable grounds to issue a Building Consent;

PS2 DESIGN REVIEW Intended for use by a suitably qualified independent engineering design review professional where the BCA accepts an independent design professional’s review as the basis for establishing reasonable grounds to issue a Building Consent;

PS3 CONSTRUCTION Forms commonly used as a certificate of completion of building work are Schedule 6 of NZS 3910:2013 or Schedules E1/E2 of NZIA’s SCC 2011[2]

PS4 CONSTRUCTION REVIEW Intended for use by a suitably qualified independent engineering construction monitoring professional who either undertakes or supervises construction monitoring of the building works where the BCA requests a producer statement prior to issuing a Code Compliance Certificate.

This must be accompanied by a statement of completion of building work (Schedule 6).

The following guidelines are provided by ACE New Zealand and Engineering New Zealand to interpret the Producer Statement.

Competence of Engineering Professional

This statement is made by an engineering firm that has undertaken a contract of services for the services named, and is signed by a person authorised by that firm to verify the processes within the firm and competence of its personnel.

The person signing the Producer Statement on behalf of the engineering firm will have a professional qualification and proven current competence through registration on a national competence-based register such as a Chartered Professional Engineer (CPEng).

Membership of a professional body, such as Engineering New Zealand provides additional assurance of the designer’s standing within the profession. If the engineering firm is a member of ACE New Zealand, this provides additional assurance about the standing of the firm.

Persons or firms meeting these criteria satisfy the term “suitably qualified independent engineering professional”.

Professional Indemnity Insurance

As part of membership requirements, ACE New Zealand requires all member firms to hold Professional Indemnity Insurance to a minimum level.

The PI Insurance minimum stated on the front of this form reflects standard practice for the relationship between the BCA and the engineering firm.

Professional Services during Construction Phase

There are several levels of service that an engineering firm may provide during the construction phase of a project (CM1-CM5 for engineers[3]). The building Consent Authority is encouraged to require that the service to be provided by the engineering firm is appropriate for the project concerned.

Requirement to provide Producer Statement PS4

Building Consent Authorities should ensure that the applicant is aware of any requirement for producer statements for the construction phase of building work at the time the building consent is issued as no design professional should be expected to provide a producer statement unless such a requirement forms part of the Design Firm’s engagement.

Refer Also:

[1] Conditions of Contract for Building & Civil Engineering Construction NZS 3910: 2013
[2] NZIA Standard Conditions of Contract SCC 2011
[3] Guideline on the Briefing & Engagement for Consulting Engineering Services (ACE New Zealand/Engineering New Zealand 2004)
[4] PN01 Guidelines on Producer Statements

www.acenz.org.nz
www.engineeringnz.org

I

Appendix I – Statement of Professional Opinion

Statement of Professional Opinion on the Suitability of Land for Subdivision

(Appendix I to the Infrastructure Design Standard)

Issued by: Beca Limited
(Geotechnical engineering firm or suitably qualified engineer)

To: Kainga Ora
(Owner/Developer)

To be supplied to: Ashburton District Council
(Territorial authority)

In respect of: State housing development and land subdivision
(Description of proposed infrastructure/land development)

At: 6, 8 and 10 Orr Street, Netherby, Ashburton
(Address)

I Samuel Birdling Glue *(Geotechnical engineer)* on behalf of Beca Limited *(Geotechnical engineering firm)*

hereby confirm:

1. I am a suitably qualified and experienced geotechnical engineer and was retained by the owner/developer as the geotechnical engineer on the above proposed development.
2. My/the geotechnical assessment report, dated March 2024 has been carried out in accordance with the Department of Building and Housing *Guidelines for geotechnical investigation and assessment of subdivisions* and includes:
    - (i) Details of and the results of my/the site investigations.
    - (ii) A liquefaction assessment.
    - (iii) An assessment of rockfall and slippage, including hazards resulting from seismic activity.
    - (iv) An assessment of the slope stability and ground bearing capacity confirming the location and appropriateness of building sites.
    - (v) Recommendations proposing measures to avoid, remedy or mitigate any potential hazards on the land subject to the application, in accordance with the provisions of Section 106 of the Resource Management Act 1991.
3. In my professional opinion, I consider that Council is justified in granting consent incorporating the following conditions:

   Foundations to be TC1 waffle slab foundation on a 200 to 400mm thick gravel raft and designed for 300kPa geotechnical ultimate bearing capacity.

4. This professional opinion is furnished to the territorial authority and the owner/developer for their purposes alone, on the express condition that it will not be relied upon by any other person and does not remove the necessity for the normal inspection of foundation conditions at the time of erection of any building.

5. This certificate shall be read in conjunction with my/the geotechnical report referred to in Clause 2 above, and shall not be copied or reproduced except in conjunction with the full geotechnical completion report.

6. The geotechnical engineering firm issuing this statement holds a current policy of professional indemnity

   insurance of no less than $ 200,000
   (Minimum amount of insurance shall be commensurate with the current amounts recommended by IPENZ, ACENZ, TNZ, INGENIUM.)

(Signature of Engineer)

Date: 12/03/2024

Qualifications and experience:

BE Civil (Hons), CPEng, CMEngNZ, 16 years experience in Geotechnical Engineering design.