

28 August 2026

Southern Links 1 – Industrial Development

Hamilton

GEOTECHNICAL INVESTIGATION REPORT

Colliers Project Leaders

Job No. HAM2024-0120 | Version 5



EXECUTIVE SUMMARY

This report presents the results of geotechnical investigations and a geohazards assessment to support the substantive application for the Southern Links 1 project ("SL1" or the "Project") under the Fast-track Approvals Act 2024 ("FTAA"). This report relates to the industrial portion of the SL1 Project, referred to as the "Industrial Area" and the "Industrial Development" in this report.

The 61Ha Industrial Development site, as shown conceptually in **Figure 1**, is located on the southern boundary of the current Hamilton City Council limits in, Hamilton, New Zealand. The site contains areas of gravel yards, material stockpiles, and pastureland overlying Peat, Hinuera Silts/Sands and Walton Subgroup Silts and Clays.

The topography is generally level between RL38 and 41. However, there are large uncontrolled fill and excavations onsite ranging between RL36 m up to RL61. The regional groundwater table is generally within 3 metres of the surface.

We consider that the Industrial Area is suitable for the proposed Industrial Development from a geotechnical /geohazards perspective, subject to the adoption of the geohazard and geotechnical recommendations summarised below, which we understand have been incorporated into the Project design and resource consent conditions as appropriate. In other words, we consider: the construction of the Industrial Development is feasible from a geotechnical perspective; there are satisfactory techniques available to address the geotechnical constraints identified within the Industrial Area; and that any potential geotechnical/geohazard effects/risks can be appropriately managed.

Key recommendations from our geotechnical investigations and geohazards assessment include:

- (1) The site is to be filled either with imported fill or by reusing suitable material from the onsite stockpiled material, to form level building sites with maximum proposed cut levels of up to 19m (for the existing uncontrolled fill stockpile areas) and proposed fill levels of up to 4m.
- (2) Contaminant testing by Swiftsure of the uncontrolled fill stockpile in the north-east corner of the site known as 'Latham's Mountain' indicated the presence of contaminants such as Asbestos Containing Material (ACM), zinc and Polycyclic Aromatic Hydrocarbons (PAH), at low but measurable concentrations. Further assessment is therefore proposed during detailed design. Materials with safe levels of contaminants will be screened and the remaining soil may be utilised onsite as engineer certified fill. Unsuitable materials will either be buried onsite under a clean fill cap or disposed offsite at a specialist waste facility.
- (3) Without mitigation, fill induced ground settlements within the swamp deposits are anticipated to be excessive (50-1900mm) over the design life of the structures. Geotechnical mitigation is therefore proposed for the Industrial Development.
 - a. For the proposed Road Corridors - A preliminary analysis for surcharging road corridors to reduce postconstruction settlements to within acceptable limits indicates that road corridors will require 1.2 to 7.3m thick preload to reduce settlements for the 100-year design life of services within the road corridor to within acceptable levels.
 - b. The lots will be filled to design level during earthworks. Then three foundation options have been provided for future lot owners to suit their specific requirements:
 - i. Option A – No surcharge, specific foundation design will be left to the lot owner (refer point 5 below for possible foundation design solutions).
 - ii. Option B – Nominal 20kPa (1.3m high) surcharge on the lots to reduce total consolidation settlement experienced by the structure. For example, this option may be selected if the landowner was to pile the building foundations and preload the carpark/yard areas to reduce settlements to acceptable levels.
 - iii. Option C – Surcharge lots (1.8 to 12.7m high) to achieve <25mm long-term settlement with no building load.

- (4) Surcharge durations will be between 3 to 15 months. However, further laboratory testing and analysis will be required at detailed design stage.
- (5) Specific foundation design will be required for the industrial buildings on a case-by-case basis. Options include surcharging the lot to reduce long term secondary creep settlement to acceptable limits, utilising ground improvement such as stone columns, grouted RAPs or rigid inclusions to transfer the building load to the underlying dense sands, or using deep pile foundations.
- (6) Preliminary stormwater artificial wetland batter slopes of 1V:2H were assessed for stability. Two scenarios were assessed for above ground and in-ground wetlands. Results indicate above-ground wetlands are stable, but the in-ground wetland batters are unstable following a ULS¹ seismic event where we predict 180mm of lateral displacement. If in-ground wetlands are adopted, building setbacks, more gentle batter slopes, or ground improvement will be required to support the wetland batters.
- (7) The potential ULS liquefaction risk is high for the peat soils and the upper 4m of Hinuera Formation soils. These effects will be reduced through the proposed preloading and capping with engineer certified fill. Confirmation of the reduction will be made during detailed design.
- (8) A preliminary geotechnical ultimate bearing pressure of 200kPa should be available in the static case for shallow strip and pad foundations constructed within both the natural cut ground and engineered fill areas.



Figure 1: Industrial North Overall Masterplan prepared by Barkers and Associates, dated June 2026

¹ Ultimate limit state.

STATEMENT OF QUALIFICATIONS AND EXPERIENCE

Mark McVinnie

I am an Engineering Geologist at CMW Geotechnical NZ Limited, trading as CMW Geosciences. CMW Geosciences is a specialist geotechnical engineering and geological services consultancy with offices in New Zealand and Australia. I have been employed at CMW Geosciences since March 2022.

I hold the qualifications of BSc(Tech) in Earth Sciences from the University of Waikato, which I completed in 2016.

I have 8 years of professional experience in geotechnical engineering in the Waikato and Auckland regions. My experience has been primarily in land development, infrastructure, and landslide mitigation, and for the last several years have been focused on projects in the Waikato.

David MacPherson

I am an Associate Geotechnical Engineer at CMW Geotechnical NZ Limited, trading as CMW Geosciences. CMW Geosciences is a specialist geotechnical engineering and geological services consultancy with offices in New Zealand and Australia. I have been employed at CMW Geosciences since August 2017.

I hold the qualifications of MEngSc from The University of New South Wales, which I completed in 2022 as well as an MSc from the University of Waikato, which I completed in 2016. I am a Chartered Professional Engineer and a Chartered Member of Engineering New Zealand.

With over 8 years of experience, I am a seasoned geotechnical engineer skilled in project management, design solutions, and team leadership. My background includes extensive work in infrastructure projects. My experience includes managing and participating in site investigations, instrumentation and monitoring, slopes and geotechnical hazard assessments, and slope stability design for major infrastructure projects such as the Peacockes Bridge in Hamilton.

Dave Sullivan

I am a Principal Engineering Geologist at CMW Geotechnical NZ Limited, trading as CMW Geosciences. CMW Geosciences is a specialist geotechnical engineering and geological sciences services consultancy with offices in New Zealand and Australia. I have been employed at CMW Geosciences since May 2025.

I hold the qualifications of BSc in Geological Engineering from Mackay School of Mines at University of Nevada, which I completed in 1997, and a Master of Business Administration from the University of Phoenix, which I completed in 2023. I am a Chartered Professional Engineer, and Chartered Member of Engineering New Zealand. As well as a Category 1 Geoprofessional (TCC), and a Licensed Civil Engineer in California and Nevada, USA.

I have over 30 years of professional experience in engineering geology and geotechnical engineering. My experience has been primarily in land and building development, disaster relief (Christchurch rebuild) and linear infrastructure (Waikato Expressway). The years in New Zealand have been focused on the Christchurch rebuild, the Waikato Expressway, and land development in the Waikato and Bay of Plenty.

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1.0 INTRODUCTION

CMW Geosciences (CMW) was engaged to carry out a geotechnical investigation of a 61Ha site located on the southern boundary of Hamilton City limits at Hamilton, which is being considered for industrial subdivision (the Industrial Development) within the wider Southern Links 1 development (the Project).

The purpose of this investigation is to carry out field investigation and prepare a Geotechnical Interpretative Report to support the substantive application for the Project under the FTAA (Fast-Track Approvals Act 2024).

1.1 Project Description

The SL1 Industrial North development forms part of the industrial component of Southern Links 1, a listed project under FTAA 2024. The Southern Links 1 project is a combined 106ha development, including residential and industrial activities. The industrial component comprises circa 60ha and is in joint landownership and is currently used for a mix of rural and related activities. The landholding includes four parcels Lot 2 Deposited Plan 603134, Lot 1 Deposited Plan 603134 both within Southpark Agri Development Limited landholdings (circa 50ha), and Lot 1 Deposited Plan 584573 Indusre Property Tahī Limited and S&S Property Investments Limited (7.8ha) and Lot 1 Deposited Plan 396081 and Lot 1 Deposited Plan 486522 within Indusre Property Tahī Limited landholding (2.1ha).

The proposal seeks approval for a Substantive Application under the Fast-Track Approvals Act 2024 for a comprehensively master planned and staged industrial development. This includes all necessary resource consents under the Resource Management Act 1991, including regional, district approvals where required.

The industrial component provides for the establishment of 60ha gross developable industrial land, together with associated infrastructure and services, including roading and associated stormwater ponds. The proposed industrial development is underpinned by a series of design principles, with a focus on creating a functional, efficient and contiguous industrial development that underscores the locational attributes of SL1 Growth Cell, the Hamilton Southern Links designation and future industrial growth. An integrated roading network which connects to both existing and planned infrastructure underpins this approach.

The 22 serviced industrial lots vary in size and attributes to support a range of industrial uses, none of which are heavy or noxious. A truck and logistics activity is proposed on lot 7 to facilitate the landowners existing national trucking operation. All other industrial lots are sized to support a range of anticipated and provided for industrial activities, underpinned by bulk and location controls.

The proposal includes substantial enabling works to establish developable land, including bulk earthworks across the site and necessary pre-load to manage ground conditions. An integrated stormwater management system including wetlands, detention/attenuation devices and treatment ponds supports the development.

Access and integration with the surrounding Hamilton area is provided through connected transport framework, including the establishment of intersections, roading, and walking and cycle networks, along with upgrades to existing transport infrastructure. The industrial development is proposed to be serviced via a coordinated strategy, including the extension of public networks, upgrades, and installation of appropriate infrastructure.

2.0 PREVIOUS REPORTING

Previous geotechnical reporting completed on this site include:

- CMW Geotechnical Desktop Assessment²

² Ref: HAM2024-0017AD Rev0, dated 15 March 2024

3.0 SITE DETAILS

3.1 Site Description

The existing Industrial Area site comprises several primarily industrial properties under different ownership with an area of approximately 61ha. The Site is located southwest of Kahikatea Drive and SH1C, as shown in **Figure 2**.

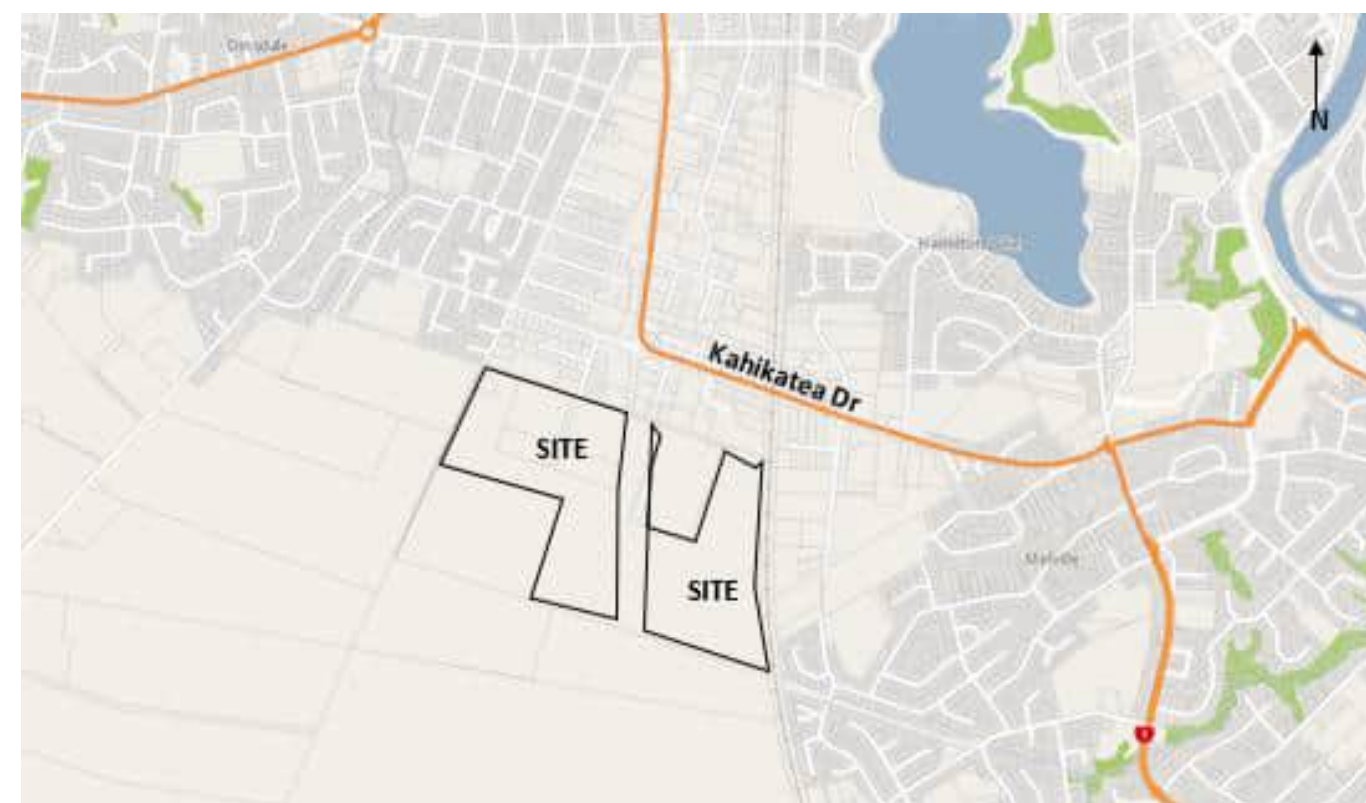


Figure 2: Site Location Plan (Source: Open Street Maps)

The site is bound to the north by industrial sites off Kahikatea Drive, to the south by farming pasture, to the east by the North Island Main Trunk (NIMT) rail line, and to the west by the G.A. Rogerson Racing Stables. At the time of our 2024 desktop investigation, there were several industrial sites and some farming paddocks within the site. The land is primarily utilised for farming with crop pastures, various industrial uses including storage yards, and waste dumps for green and construction materials.

3.2 Landform

The general landform of the project area is low-lying peatland. The site ranges in elevation between RL36m and RL61m. Pre-development, the site would have been near level, grading gently from RL39m in the north to RL41m to the south. This profile can still be seen in the undeveloped southern half of the site. It is noted the southern crop pasture portion of the site has had recontouring for drainage purposes.

The northern half of the site has been extensively developed with gravel yards covering most of the central area (RL36 to 42m). Additionally, there are three sites along the northern boundary which contain large material stockpiles: Hamilton Organic (Hamilton City Council Owned) has stockpiles up to 4m high (between RL38m and RL42m). The adjacent landscape supply is outside the Project/Industrial Area site boundary and has stockpiles up to

14m high (between RL40 and RL54m). The construction waste site in the north-eastern corner of the site has the largest stockpiles up to 21m high (between RL40m and RL61m) with slope angles up to 60 degrees to the horizontal. The current general landform, together with associated features located within and adjacent to the site is presented in the attached Geomorphology Plan, **Drawing 02** in **Appendix A**.

3.3 Historical Aerial Imagery

Historical aerial photographs dating back to 1943 show that the site was initially a swamp. The aerial imagery is presented in **Appendix B** and summarised below:

- 1943: The site is overgrown and situated within a swamp. The NIMT Railway line is located to the east of the site.
- 1971: The northern two thirds of the site had been drained and converted into pasture. Additionally, the Rogerson's Racetrack was developed west of the site during the 1970's.
- 1990: Fill was placed to form a platform in the northeastern corner of the site. In the following years this site would be used to store construction waste.
- 2008: Between 2002 and 2008 a green waste dump opened along the northern boundary. The Les Harrison gravel yard area was developed during the same period although this is outside the Project/Industrial Area site.
- 2016: The Les Harrisons gravel yard was extended southward.

4.0 PROPOSED DEVELOPMENT

The proposed Industrial Development master scheme plan, proposed wetland section plan and earthworks cut/fill plan are presented in **Appendix C**.

Brief details for the development are as follows:

- Combined Indicative Master Plan provided by Barkers (June 2026) shows the site is split into industrial lots ranging from 800m² to 4000m² (20m to 50m wide by 40m to 80m deep), with associated stormwater and roading infrastructure.
- Minor cut and fill processes are proposed to create flat areas for construction of buildings and roading.
- Minor slopes are proposed across the Industrial Development site in the form of artificial stormwater wetlands. Proposed plans indicate these wetlands will have a maximum depth of 2m, with batter slope gradients of approximately 1V:2H.

5.0 INVESTIGATION

5.1 Field Investigation

Following a dial before you dig search and onsite service location, the site investigation was carried out between November 2025 and March 2026.

The scope of fieldwork completed is shown in **Table 1**: below:

Table 1: Investigation Summary

Test ID	Test Type	Depth (m bgl)	Methodology
TP25-01- TP25-10 (10 total)	Test Pit (TP)	0.8 (refusal) to 4.4m	1.2m (W) x 4m (L) holes dug by 19T excavator. Shear vanes completed within and DCP tests carried out alongside where appropriate.

³ New Zealand Geotechnical Database <https://nzgd.org.nz/>

Test ID	Test Type	Depth (m bgl)	Methodology
P1 to P4 (4 total)	Piezometer Borehole (P)	6.0m to 7.5m	90mm dia. Hand auger, slotted pipe installed, falling head test performed 3.7m to 6.0m screened piezometer
CPT25-01 to CPT25-20 (20 total)	Cone Penetrometer Test (CPT)	0.11m (early refusal) to 20m	GeoMil Panther 100 CPT rig with C10 Cone, drives probe rod to target 20m depth

The geotechnical investigation was carried out following a 'fill-in-the-gaps' methodology to supplement the existing historical data for the site.

Engineering logs for this recent investigation are presented in **Appendix D**. Logs from previous site investigations sourced from the publicly available New Zealand Geotechnical Database (NZGD)³ are presented in **Appendix E**.

The approximate locations of the respective investigation sites referred to above are shown on the Geotechnical Investigation Plan as **Drawing 01** attached in **Appendix A**.

5.2 Stormwater Soakage

A series of falling head permeability tests were conducted across the site to provide soakage rates. Results indicated low soakage rates ranging between 0.05 to 0.28m/day. Results of testing are presented in **Appendix J**.

6.0 GROUND MODEL

6.1 Published Geology

The published geological map for the area depicts the regional geology for the low-lying areas as comprising Holocene peat deposits. The peat is underlain by interbedded Silts and Sands of the Late Pleistocene alluvial deposits of the Hinuera formation, which is underlain by Early Pleistocene, fluvially reworked igneous silt/clay of the Walton Subgroup Formation which also form the base of the nearby hills outside the site.

An excerpt from the geological map⁴ showing the approximate distribution of the above units is presented in **Figure 3**.



Figure 3: Regional Geology Map Excerpt (GNS Web Map)

⁴ Edbrooke, S.W. (compiler) 2005: Geology of the Waikato area: scale 1:250,000, Map No4, Institute of Geological and Nuclear Sciences.

6.2 Stratigraphic Units

The ground conditions encountered during our investigation and inferred from previous investigations are generally consistent with the published geology, with the exception of the imported gravel fill in the yard areas, the uncontrolled fill in the existing green waste site and the construction waste site in the north-eastern corner of the site. Our investigation locations from our current investigation and approximate locations from previous investigations are presented in **Drawing 01**.

The distribution of the various units encountered is presented in the appended Geology Map **Drawing 02**, and Geological Sections **Drawing 03** and can be generalised to the units below:

Table 2: Summary of Strata Encountered

Geology	Unit Description	Top of Unit (mBGL)		Thickness (m)	
		Min	Max	Min	Max
Uncontrolled Fill (Gravel Yard / Latham's Mountain)	Stiff to very stiff Clayey Silt and medium dense Sand	0.0	0.2	0.5	3.2*
Peat (Swamp deposits)	Firm to very stiff fibrous organic Peat	0.1	1.0	0.3	12.0
Hinuera Formation	Loose to medium dense Sand with interbedded stiff to very stiff Silt, and lenses of Organic Silt	0.1	7.0	3.3	17.5
Walton Subgroup	Stiff to Hard Clay/Silt and dense to very dense Sand	5.2	19.8	0.2	>14.8
*Note: Unable to penetrate full depth, based on contours likely to be ~19m thick in the construction waste site					

6.3 Groundwater

During the investigation, completed in summer conditions (November 2025 - March 2026), groundwater was encountered at depths ranging from 0m to 3.7m below existing ground levels with a full summary presented below.

Table 3: Groundwater Depths Summary

Test Location	Depth to Groundwater (m bgl)	Test Location	Depth to Groundwater (m bgl)	Test Location	Depth to Groundwater (m bgl)
CPT25-01	2.4	CPT25-11	1.7	TP25-01	1.3
CPT25-02	2.9	CPT25-12	2.5	TP25-02	Not encountered
CPT25-03	0.1	CPT25-13	2.4	TP25-03	1
CPT25-04	1	CPT25-14	0.8	TP25-04	4.1
CPT25-05	1.8	CPT25-15	3	TP25-05	2
CPT25-06	0	CPT25-16	1.7	TP25-06	Not encountered
CPT25-07	0	CPT25-17	1.8	TP25-07	0.6
CPT25-08	0.1	CPT25-18	2	TP25-08	Not encountered
CPT25-09	3.4	CPT25-19	1.5	TP25-09	Not encountered
CPT25-10	1	CPT25-20	0.7	TP25-10	Not encountered

Additionally, ongoing groundwater monitoring carried out by WGA has produced results between 1.74 and 4.85m depth.

6.4 Recommended Geotechnical Parameters

The various geological strata encountered across the Industrial Development site have been rationalised based on geotechnical engineering properties and grouped into broad units to define a geotechnical model that has been used in subsequent assessment and analysis.

Parameters are based on results of the investigations, known correlations from literature, and our experience with similar soils in the Hamilton area such as the Temple view residential development a few kilometres away, with similar geology. The geotechnical design parameters adopted are presented in Table 4. It is noted that during the detailed design stage further laboratory testing should be conducted to confirm geotechnical parameters for fill materials determined to be utilised, discussed further in Section 7.12.

Table 4: Geotechnical Design Parameters

Unit Description	γ (kN/m ³)	c' (kPa)	ϕ' (deg)	S_u (kPa)
Imported Engineered Clay Fill	16	5	30	120
Insitu Uncontrolled Fill ('Lathams Mountain') - Stiff to very stiff Clayey Silt and medium dense Sand (Uncontrolled Fill)	16	2	26	32-40
Compacted Fill ('Lathams Mountain') - Stiff to very stiff Clayey Silt and medium dense Sand (Compacted Fill)	16	4	28	120
Firm to very stiff fibrous organic Peat (Swamp Deposits) – lower values represent disturbed values	12-16	0-1	16-26	20-40
Loose to medium dense Sand with interbedded stiff to very stiff Silt, and lenses of Organic Silt (Hinuera Formation)	17	1	34	-
Stiff to Hard Clay/Silt and dense to very dense Sand (Walton Subgroup)	17	5	30	140
Notes: γ = soil bulk unit weight (conservative value determined from CPT correlations / typical published values for similar soil types) c' = effective cohesion (conservative industry accepted value) ϕ' = effective friction angle (conservative industry accepted value/back analysis) S_u = undrained shear strength (lower bound value determined from vane shear testing / CPT correlations)				

7.0 GEOHAZARDS ASSESSMENT

7.1 Geohazard Risk Register

Section 106 of the Resource Management Act 1991 (RMA) requires an assessment of the risk from natural hazards to be carried out when considering the granting of a subdivision consent. S106 of the RMA specifically states that the assessment must consider the combined effect of the natural hazard likelihood and relevant material damage to land or structures (consequence).

Section 2 of the RMA defines natural hazards as any atmospheric or earth or water related occurrence (including earthquake, tsunami, erosion, volcanic and geothermal activity, landslip, subsidence, sedimentation, wind, drought, fire, or flooding) the action of which adversely affects or may adversely affect human life, property, or other aspects of the environment.

The National Policy Statement for Natural Hazards 2025 (NPS-NH) provides direction for assessing the risk associated with natural hazards for subdivision, use and development.

A summary of the natural hazard risk classification is provided in **Appendix F**. This is risk register of the geohazards at the site is a live document that will be updated as the Project progresses.

7.2 Seismicity

The Peak Ground Acceleration (PGA) for this development was calculated based on a 50-year design life in accordance with the New Zealand Building Code⁵ and importance level (IL) 2 and IL (3) structures. The seismic design and analysis for this site should consider two earthquake events in accordance with NZ1170.0, which are the serviceability limit state (SLS) and ultimate limit state (ULS) earthquakes. The calculated PGAs for these events are summarised in Table 5 below.

Table 5: Peak Ground Acceleration (PGA) for Two Limit States

Limit State	AEP	PGA (g) IL2	PGA(g) IL3	Magnitude _{eff}
SLS	1/25	0.06	0.06	5.9
ULS	1/1000	0.25	0.32	5.9

Notes: SLS = Serviceability Limit State; ULS = Ultimate Limit State; AEP = Annual Exceedance Probability

7.3 Site Subsoil Classification

Based on those ground conditions and specific geophysical data gathered by the University of Waikato and Auckland University, the typical site period for the site is expected to be about 3 seconds where the seismic site subsoil category is assessed as being Class D (deep soil site) in accordance with NZS1170.5:2004.⁶

7.4 Fault Rupture

As indicated on the GNS Community Fault Model, the nearest known active fault is the (Te Tatua o Wairere Fault, located 3km east of the site. The surface of the Holocene geology did not have observable evidence⁶ of fault rupture lineament in the Holocene geology at the site. Therefore, the risk of fault rupture or the presence of an active fault crossing the site is considered low. Additionally, the fault model states the recurrence interval of this fault to be >100,000 years which is outside the design life of any of the proposed structures, and need not be considered further.

7.5 Liquefaction

7.5.1 Methodology

In accordance with MBIE/NZGS guidance the liquefaction susceptibility of the soils at this site was assessed with respect to geological age and compositional (soil fabric and density) criteria, based on the following assumptions:

- Saturated soils below the assessed seasonal average groundwater level were modelled as being susceptible to liquefaction.
- In accordance with MBIE/NZGS guidance⁷ and in conjunction with site-specific shear wave velocity measurements, no aging / strength gain factor has been applied to the Pleistocene aged soils.
- Soils are also classified with respect to their grain size and plasticity to assess liquefaction susceptibility. For this Project, a cut-off threshold soil behaviour type index value (I_c) of 2.6 was used to distinguish between liquefiable ($I_c < 2.6$) and non-liquefiable ($I_c > 2.6$) soils. Published literature and our experience with the local soils have shown that Waikato soils typically have a lower I_c cutoff value, closer to 2.2. We have included an additional case using

this lower I_c value of 2.2 for the upper Hinuera Formation soils, however laboratory testing is required to determine if this is appropriate for the site.

- Specific liquefaction analyses were undertaken for IL2 structures, using the software package CLiq and the Boulanger and Idriss (2014) method. The cyclic stress ratio (CSR), being a function of the PGA corrected for the relevant magnitude for the design return period event, was compared to the cyclic resistance ratio (CRR), being a function of the CPT data.
- Free-field liquefaction induced settlements were determined in accordance with Zhang et al. (2002). With respect to liquefaction response, consideration was given to a 10m cut-off depth to estimate index settlements as per MBIE guidance⁶ (foundation technical categories). These were compared to liquefaction settlement estimates over the full depth range of the CPT's with a depth weighting factor (ev) applied ranging from 1 at the ground surface to 0 at 20m depth.

7.5.2 Liquefaction Analyses Results

A summary of liquefaction analyses results is presented below. A complete table of results (per CPT) is presented in **Appendix G**.

Table 6: Summary of Liquefaction Results

SLS Settlement (mm)	Total ULS Settlement (mm) IL2 (PGA 0.25)	Total ULS Settlement (mm) with reduced I_c (2.2) IL2 (PGA 0.25)	Total ULS Settlement (mm) IL3 (PGA 0.32)	Total ULS Settlement (mm) with reduced I_c (2.2) IL3 (PGA 0.32)	Depth to Liquefied Layer (m)
<5	10 - 165	10 - 60	10 - 170	10 - 60	0 – 4.3

Note: Settlements and depths based on existing ground profile.

The results show that liquefaction is not triggered in the SLS case. In the ULS case:

- Widespread liquefaction of the peat soils is predicted across the site, with effects from liquefaction assessed to be High⁶ with moderate to large differential settlement. In addition, the shallow depth to liquefaction presents a risk of surface manifestation, which may induce additional settlement not accounted for in the estimates in **Table 6**. Surface manifestation is difficult to predict and can result in larger differential settlements.
- Reducing the I_c cutoff to 2.2 reduces the amount of liquefaction anticipated due to the marginal Piako Subgroup silt/sands.
- Results indicate liquefaction is triggered at a seismic load of PGA 0.07, which is slightly above the 25-year return period PGA (SLS event) of 0.06 for the Hamilton area as per MBIE guidance.

⁵ Earthquake Geotechnical Engineering Practice, Module 3: Identification, assessment and mitigation of liquefaction hazards", (November 2021)

⁶ NZS 1170.5: 2004 Structural Design Actions Part 5 Earthquake Actions – New Zealand

⁷ Earthquake Geotechnical Engineering Practice, Module 3: Identification, assessment and mitigation of liquefaction hazards", (November 2021)

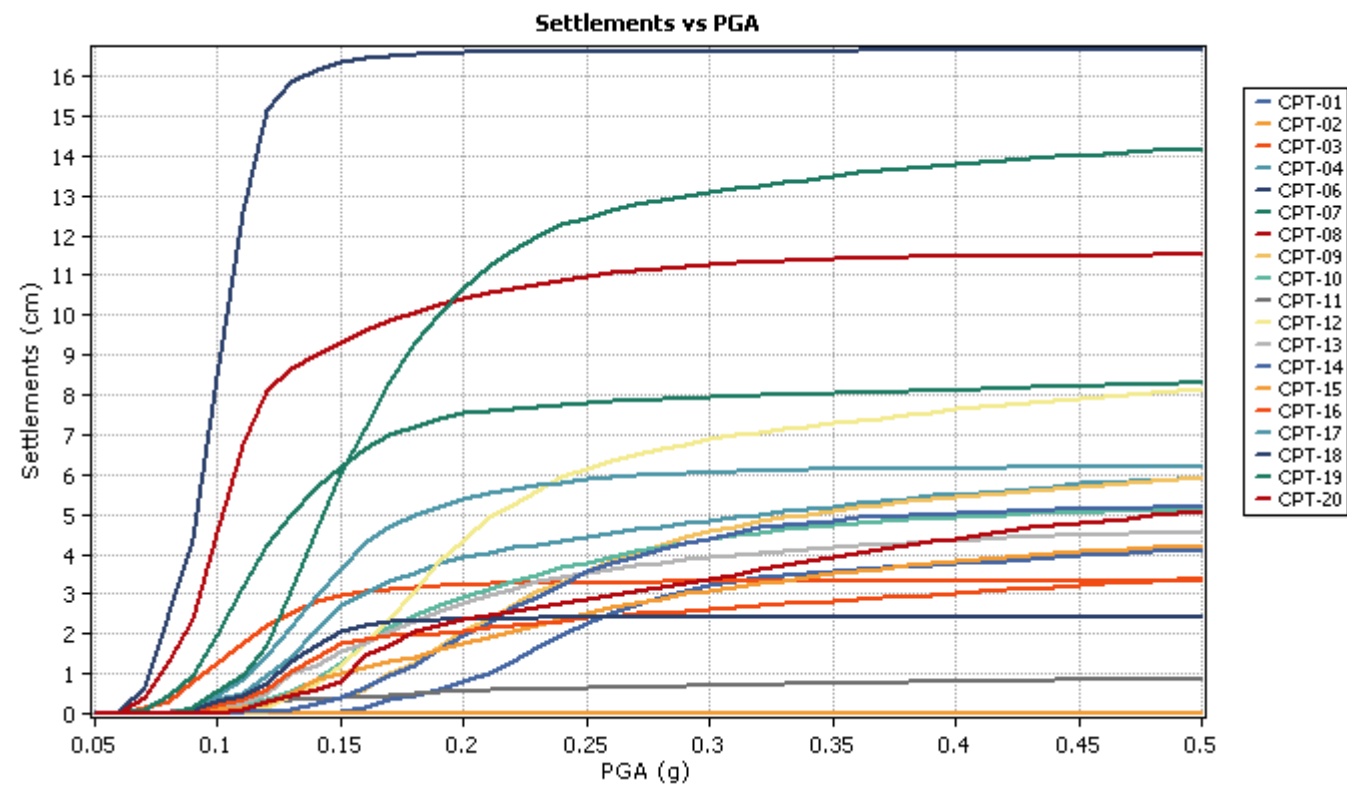


Figure 4: Parametric Analysis and Seismic Triggering (Ic=2.6)

Recommendations to mitigate effects of liquefaction induced settlement on the proposed development are provided in Section 8.7

7.6 Cyclic Softening

The fine-grained alluvium may be susceptible to strength loss, referred to as cyclic softening, during a ULS seismic event.

Cyclic softening analyses of those soils was carried out in accordance with Boulanger⁸ and Idriss⁹. This correlates earthquake magnitude to the estimated number of equivalent stress cycles (Figure 4) and then correlates number of cycles to a cyclic shear strength ratio (Figure 5).

Figure 5: Relationship between earthquake magnitude and mean number of uniform stress cycles

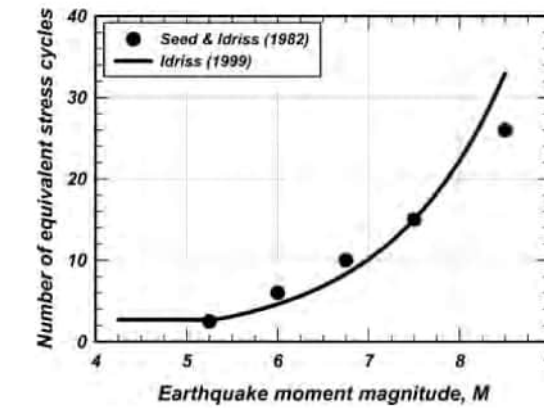
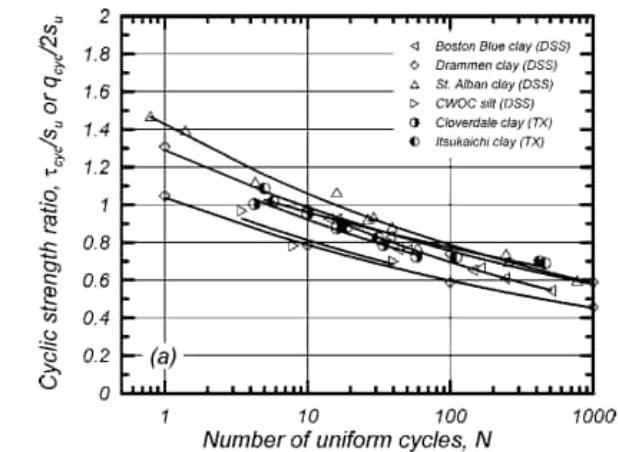


Figure 6: Relationship between cyclic strength ratio and number of uniform stress cycles



Cyclic softening is predicted to occur within the Hinuera Formation silts. Based on the above assessment, 6 stress cycles are estimated during the ULS M5.9 earthquake resulting in a worst-case cyclic shear strength of 85% of the peak shear strength. Reduced shear strengths have been considered for the slope stability analysis in Section 7.8 below.

7.7 Lateral Spread

Following the onset of liquefaction, the liquefied soils behave as a very weak undrained material, which can give rise to lateral spreading where a free face is present within the vicinity of the site or where proposed cut and fill batters are proposed over or within liquefied soils.

The potential for and estimated magnitude of lateral spread beneath the site has been assessed with respect to liquefied shear strengths ($S_{u(liq)}$) within the liquefied soil units using a Newmark Sliding Block approach as discussed in Section 8.3 below.

⁸ Boulanger, R.W. and Idriss, I. M. (2007) Evaluation of Cyclic Softening in Silts and Clays, Journal of Geotechnical and Environmental Engineering, Vol 133, Issue

⁹ Idriss, I. M. and Boulanger, R. W. (2008) Soil Liquefaction During Earthquakes. Monograph 12, Earthquake Engineering Research Institute.

7.8 Slope Stability

7.8.1 Design Criteria

The stability of the artificial wetland batters and proposed fill batters have been assessed under a range of design conditions is expressed in terms of a factor of safety, which is defined as ratio of the resisting forces to destabilising forces acting on a slope. The table below presents the performance standards¹⁰ recommended for slope stability assessment:

Table 7: Slope Stability Factor of Safety Criteria

Condition	Required Factor of Safety
Static long-term conditions (drained soil conditions, normal groundwater)	1.5
Transient short-term conditions (elevated groundwater and pore pressures)	1.3
Seismic (ULS) case	1.0*
Note: * = Factor of safety <1.0 acceptable where displacement-based approach is adopted.	

7.8.2 Slope Stability Analysis

The slope stability analysis was carried out using the Morgenstern-Price method of both circular and translational failure mechanisms using the proprietary software Slide2.

Analyses have been completed for the various conditions stated in **Table 8** based on the following:

- Drained soil conditions were analysed for the prevailing case, with the groundwater table based on the available groundwater data.
- When analysing transient/elevated groundwater conditions, an elevated pore pressure of $r_u = 0.3$ was applied to the soils above the groundwater table.
- Seismic displacements were estimated based on a Newmark Sliding Block approach using 50th percentile correlations published in Bray and Macedo (2019)¹¹, and Macedo et al. (2023)¹².
- For the seismic case, a liquefied shear strength ratio is predicted based on methods discussed by Olson & Stark (2002) and Idriss & Boulanger (2008), which takes into account overburden pressure above the liquefiable layers.

We have assessed the slope stability of the following typical sections from the supplied Maven Landform Drawings:

- Section A (Artificial Wetland 1): 1.95m deep wetland, cut into peat with a slope gradient of 1V:3H. Reduced effective stress parameters have been adopted here due to cutting into the peat. When cutting into peat, it damages the matrix and reduces its strength, effectively resulting in it having a residual strength remaining in the soil and being susceptible to “ooze”.
- Section B (Artificial Wetland 2): 1.75m deep wetland, cut into compacted fill and peat with a slope gradient of 1V:3H. Reduced peat strengths have also been adopted in this case.
- Section C (Artificial Wetland 3): 1.95m deep wetland, cut into in-situ uncontrolled fill (‘Lathams Mountain’) with a slope gradient of 1V:3H.
- Section D: 2.8m high compacted fill batter with a slope gradient of 1V:3.5H. This is the steepest proposed slope outside of the artificial wetlands. The same recommendations can conservatively be applied to similar slopes onsite, such as the fill batter adjacent to the NIMT Railway.
- Temporary Stability (Preload): Nominal 5m high preload in compacted fill at a maximum 1V:3H Slope.

¹⁰ CoLab, (October 2025). Regional Infrastructure Technical Specification (RITS) V2, Table 16: Proposed factors for safety for different slope stability scenarios

¹¹ Bray, J. D., & Macedo, J. (2019). Procedure for estimating shear-induced seismic slope displacement for shallow crustal earthquakes. *Journal of Geotechnical and Geoenvironmental engineering*, 145(12), 04019106.

Where the proposed slope gradient does not meet the requisite factors of safety, remedial measures have been proposed and modelled, and are discussed in Section 7.8.3.

7.8.3 Analysis Results

Selected stability printouts are attached in **Appendix H** with the factor of safety (FoS) for each scenario outlined below.

Table 8: Slope Stability Analyses Results

Condition	FoS: Section A Wetland 1	FoS: Section B Wetland 2	FoS: Section C Wetland 3	FoS: Section D NZTA Road Cut	Temporary Stability: 5m high preload at 1V:3H slope
Design Case - Static long-term conditions (drained soil conditions, normal groundwater)	1.2	2.4	2.3	2.5	1.9
Transient short-term conditions (elevated groundwater and pore pressures)	0.7	1.7	2.0	2.0	4.8*
Seismic (ULS) case	1.1	<1	<1	<1	-
Liquefied Flow Failure case	1.9	2.1	3.5	3.8	
Yield Acceleration (g)	0.137	0.064	0.178	0.269	-
Lateral displacements (mm) in worst case scenario – Liquefied Soil post (ULS) seismic event	10mm	70mm	10mm	<5mm	-
Notes: * Modelled using undrained parameters.					

Section A has a predicted Factor of Safety <1 in the transient short-term case which does not meet the requisite factor of safety.

Additionally, Seismic results have a Factor of Safety <1. Therefore the displacement based (Newmark sliding block) calculation for lateral spread was carried out. Results indicate <70mm predicted lateral displacement.

Recommendations for foundation design are outlined in Section 8.3.

7.9 Load Induced Settlement

Load-induced static settlements occur in the peat subsoils that are subject to static loading (e.g. by filling and / or structural loads) where the magnitude of settlement is governed by the soil stiffness and the load applied.

7.9.1 Design Criteria

7.9.1.1 Industrial Buildings

Foundation design should limit the probable maximum differential settlement over a horizontal distance of 6m to no more than 25mm under serviceability limit state load combinations of AS/NZS 1170¹³, unless the structure is specifically designed to prevent damage under greater settlement.

¹² Macedo, J., Bray, J. D., & Liu, C. (2023). Seismic slope displacement procedure for interface and intraslab subduction zone earthquakes. *Journal of Geotechnical and Geoenvironmental Engineering*, 149(11), 04023104.

¹³ AS/NZS 1170.0:2002, Structural Design Actions, Part 0: General Principles

7.9.1.2 Roads and Buried Gravity Services

To avoid damage to the pavements by groundwater, the pavements will need to be constructed above existing ground level and will therefore induce settlement in the underlying soil, as will any subgrade improvement where the existing soils are undercut and replaced with heavier material. For preliminary design we have assumed a maximum allowable settlement of 25mm at the ground surface. Tighter differential settlement tolerances may be required for gravity lines to achieve the minimum gradients for self-cleaning at the end of the utilities design life.

7.9.2 Preliminary Settlement Analysis

The underlying peat soils will undergo settlement following the application of a fill or building load. Preliminary settlement analysis has been undertaken to assess the magnitude of primary and secondary consolidation settlement induced by the construction of the earthfill embankments and the long-term static building loads proposed.

7.9.2.1 Design Parameters

The peat soils on this site are near-normally consolidated meaning that for the proposed fill embankment and foundation pressures, these soils will experience significant consolidation settlement following the virgin compression (c_c) line.

In lieu of oedometer testing, this assessment has used the consolidation parameters outlined in Table . These values give a normalised compression index of 0.34, which is considered upper-bound when compared against testing on other peats in Hamilton, and is considered appropriate for preliminary design and the resource consent application stage.

Table 9: Surcharge Design Consolidation Parameters

Parameter	Value
Compression Index (c_c)	1.0
Recompression Index (c_r)	0.05
Initial Void Ratio (e_0)	1.9
Secondary / Primary Compression Index Ratio (c_α/c_c)	0.05
Coefficient of vertical consolidation ($c_v - m^2/year$)	10

7.9.2.2 Methodology

We assessed static settlements for a range of surcharge design scenarios using the following methods:

- CMW's in-house 1-dimensional consolidation settlement spreadsheet was used to assess the settlements under future building and fill loads at each CPT location. This spreadsheet uses the Terzaghi's 1-dimensional consolidation theory for normally consolidated soils, where the primary consolidation settlement is given by:

$$S_{consol} = \frac{C_c}{1 + e_0} \cdot H \cdot \log\left(\frac{\sigma'_v + \Delta\sigma_v}{\sigma'_v}\right)$$

- Post-construction (secondary or creep) settlements were then calculated based on the method described in Mesri et al (1994):

$$S_{creep} = \frac{C_\alpha}{1 + e_0} \cdot H \cdot \log\left(\frac{t}{t_l}\right)$$

We have assessed predicted settlements over a building design life of 50 years and infrastructure design life of 100-years with our surcharge design assuming the following:

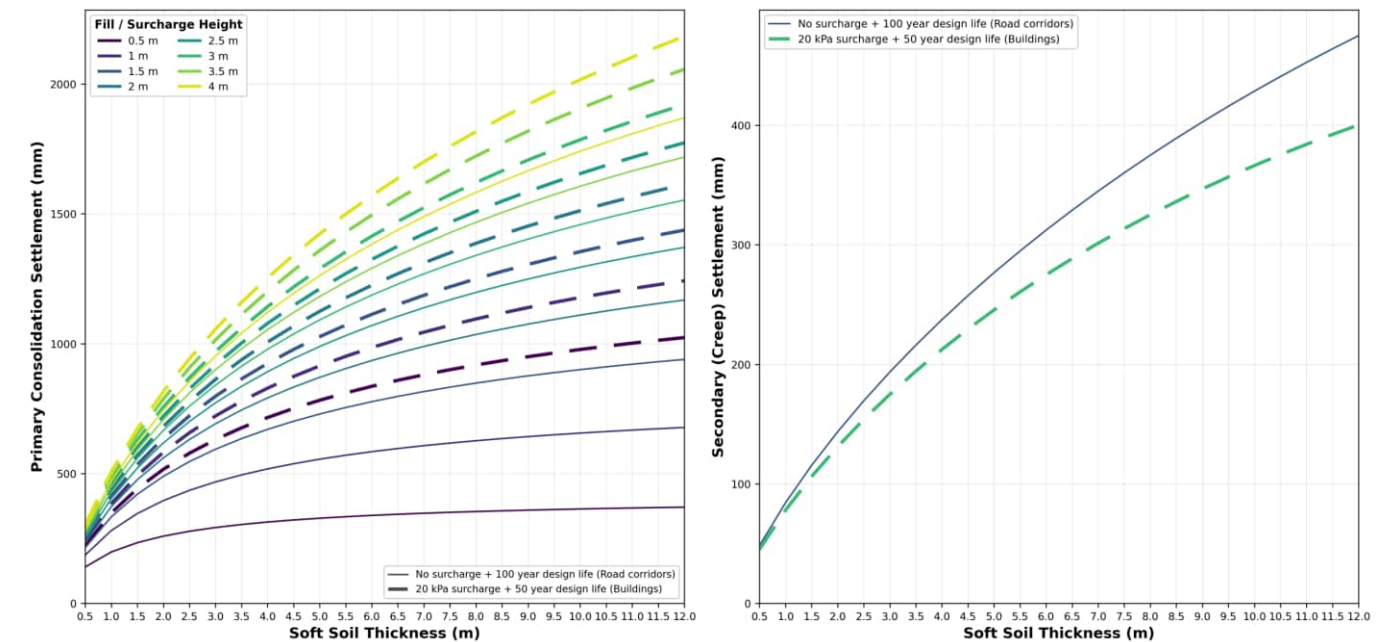
- Surcharges will be constructed using site-won materials (excavated from proposed stormwater basins and uncontrolled fill material stockpiled on site).

- The surcharge materials and any engineered fill placed beneath the surcharge have a unit weight of 17kN/m³.
- No surcharge and a 100-year design life for road corridors, and 20kPa widespread load and a 50-year design life for industrial buildings.
- Site level raised by between 0.5m and 4.0m.

7.9.2.3 Results

The magnitude of settlement observed is a function of the load imposed and the thickness of the soft soils. Primary settlement in the order of 140mm to 1900mm (Road) and 220mm – 2180mm (Building), and secondary (creep) settlement in the order of 50mm – 465mm (Road) and 45mm – 400mm (Buildings) has been estimated. These results are visualised in **Figure 7**. Placing additional fill material over the existing ground would increase the loading and magnitude of settlement. This magnitude of settlement exceeds the design criteria. Recommendations regarding mitigation is therefore discussed in section 8.2.

Figure 7: Load Induced Settlement Assessment



7.9.2.4 Time Rate of Settlement

In near normally consolidated soils, the rate of consolidation settlement is dictated by the rate in which excess pore water pressures can dissipate from within the peat, which is in turn dictated by its permeability and thickness. The ground model presents a maximum compressible peat layer thickness of 12m. Two-way drainage has been assumed due to the underlying sandy subsoils and the need for a granular starter layer to place fill on top of.

Time rates of settlement are difficult to estimate due to their depositional environment where there is inherent material composition lateral and depth variability and presence of intermediate sand lenses. Reference to coefficient of vertical consolidation (C_v) laboratory data shows a wide scatter of data although a value of 25m²/year is assessed to provide a reasonable lower bound estimate for preliminary design purposes. Based on this value, 3 – 15 month settlement timeframes are estimated.

7.10 Bearing Capacity

We anticipate a preliminary geotechnical ultimate bearing pressure of 200kPa where fill is at least 1.5m thick overlying the low strength peat soils, subject to the short axis of those footings being no wider than 1.0m.

Where fill is less than 1.5m thick, specific assessment will be required at the design stage once building loads are known.

There may be areas where localised variations in shear strength within the natural cut ground occur, particularly where the depth of cut varies across the building platforms.

7.11 Expansive Site Classification

The expansive site class/s, as defined in AS 2870:2011 (Residential Slabs and Footings – Construction), was assessed based on laboratory test results, visual-tactile assessment of the site soils, and BRANZ Report SR120A.

The assessed expansive site class with respect to AS 2870:2011 is P. Class P sites include soft or unstable foundations such as soft clay or silt or loose sands, landslip, mine subsidence, collapsing soils and soils subject to erosion, reactive sites subject to abnormal moisture conditions.

Measures are required to mitigate against the possible negative effects of seasonal shrinkage and swelling of expansive soils on structures, including their foundations, floors and superstructure.

The November 2019 update to the NZ Building Code, B1/AS1, includes acceptable foundation solutions for a range of expansive soils classifications. However, these recommendations only cover a limited range of dwelling sizes and shapes, and only to conventional embedded strip footing systems, which for Class P soils are recommended to be founded a minimum of 550 mm below finished ground level. For the design of such footing systems, we recommend the designer refer to B1/AS1 for further advice.

We recommend that specific engineering design of foundations is undertaken by a suitably qualified engineer experienced in the design of footing systems. The provisions of AS2870:2011 may be utilised, and Clauses 5.6 and 6.6 should be considered with respect to the additional detailing and construction requirements for Class M sites.

Waffle/pod/raft floor systems are considered a suitable foundation type to address the risk posed by expansive soils this site. This is due to their ability to bridge shrinkable soils and withstand moderately expansive soils.

Foundation contractors should maintain the moisture content of exposed building platforms during foundation preparation, at all proposed building platform locations. The purpose of this is to prevent excessive drying of exposed clay soils and reduce the risk of soil strength reduction upon re-wetting. Appropriate measures include protecting the platform with polythene; a hardfill layer; pouring a site concrete blinding layer in footing bases; and wetting of the building platform with sprinklers, whilst being careful to avoid over-wetting and softening the soil.

7.12 Assessment of Onsite Fill

To determine the if the construction waste fill stockpile known as ‘Latham’s Mountain’ is safe for reuse, Swiftsure carried out preliminary contaminant testing on the surface of ‘Latham’s Mountain’ as part of their Detailed Site Investigation (DSI) Report¹⁴. While the detailed assessment of these matters is contained in in Swiftsure’s DSI Report, key aspects from that report are summarised below. Results indicated the presence of contaminants such as asbestos containing material (ACM), zinc and Polycyclic aromatic hydrocarbons (PAH), at low but measurable concentrations.

Overall, the results indicate that soils at the site are unlikely to represent a significant contamination source capable of generating ongoing contaminant discharges within the site or beyond the site boundary. However, due to the heterogeneous nature of the fill materials within ‘Latham’s Mountain’, the full composition of the fill mass cannot be completely characterised and the presence of additional contamination cannot be ruled out. From a human health perspective, the primary risk relates to the disturbance of soils containing ACM, which may result in the release of airborne asbestos fibres during activities such as excavation, piling, landscaping, or redevelopment. Appropriate management measures for any future ground disturbance within the affected areas are recommended in the DSI Report).

¹⁴ Swiftsure Environmental Consultants, Detailed Site Investigation (DSI) Southern Links FTA: Stage 1 Industrial (Northern Block), dated June 2026

¹⁵ Standards New Zealand (2022) Engineered fill construction for lightweight structures, NZS 4431:2022

Therefore, the suitability of this fill will be subject to further testing and reporting by Swiftsure as well as ongoing monitoring during construction. Preliminary requirements for use and testing of this material discussed in Section 8.1.1.

8.0 RECOMMENDATIONS

8.1 Earthworks

All earthwork activities must be carried out in general accordance with the requirements of NZS 4431:2022¹⁵ and the requirements of the Hamilton City Council WLASS R.I.T.S¹⁶ and under the guidance of a Chartered Geo-professional.

The subgrade of the site will predominantly comprise either reusing onsite fill from the ‘Latham’s Mountain’ stockpile or imported clay or gravel aggregate. The proportion of onsite versus imported material to be utilised will be determined in accordance with the process and controls outlined below.

8.1.1 Reuse of Onsite Fill

Initial contamination testing indicates that much of the surficial fill from ‘Latham’s Mountain’ may be suitable for reuse, provided appropriate management controls are implemented. These controls include visual monitoring of the excavation face by a suitably qualified land contamination specialist, regular laboratory testing, and air monitoring for airborne asbestos fibres, as recommended by Swiftsure.

The preliminary plan for the excavation of ‘Latham’s Mountain’ involves excavation being carried out in stages. Approximately 1,000m³ material would be excavated at a time, then screened to remove large waste (e.g. concrete/steel), and the remaining soil set aside in separate stockpiles. Stockpiles would be assessed for contaminants.

Any material found to contain contaminants above acceptable levels would need to be either disposed of offsite at a specialist landfill facility or, where approved by a suitably qualified land contamination specialist, managed onsite beneath an adequate capping layer of clean imported material. Swiftsure has indicated that a minimum 300mm thick capping layer underlain by geocloth may be suitable. We recommend increasing the thickness of this layer to allow lot owners to construct service trenches without interacting with the contaminated material below.

Once a stockpile has been confirmed to contain contaminants at acceptable levels, geotechnical laboratory testing will be carried out to determine suitability and parameters. If suitable, the material may be reused onsite in accordance with the recommendations in Section 8.1.2. Further earthworks recommendations and detailed specifications should be confirmed during detailed design.

8.1.2 Preliminary Compaction and Testing Requirements

The preliminary test criteria and frequency for subgrade materials are set out in Table 10 below, to be confirmed at the detailed design stage.

¹⁶ WLASS Regional-Infrastructure-Technical-Specification-V2.0, dated October 2025

Table 10: Preliminary Compaction and Testing Requirements

Material Type	Test and Method	Acceptance Requirement	Minimum Frequency
Cohesive (Clay/Silt)	Field water content and density (NZS4402 test 2.1 and NZS4407 4.1, 4.2 or 4.3)	≥ 95% Maximum dry density < 10% air voids with single <12% maximum result over 10 tests	1 per 1,000m ³ (minimum 2 per lift)
	Shear strength (NZGS guideline for handheld shear vane)	Lowest value > 120kPa with single minimum 110kPa result over 10 tests	4 per 1,000m ³ (minimum 8 per lift)
Granular (Sand)	Field water content and density (NZS4402.2.1 and NZS4407 test methods 4.1, 4.2 and 4.3)	≥ 95% Maximum dry density < 10% air voids	1 per 1,000m ³ (minimum 2 per lift)
	Dynamic Cone Penetrometer	≥4 blows per 100mm	1 per 1,000m ³ for full depth of lift (minimum 2 per lift)
Granular (aggregate GAP65 or similar)	Clegg Impact Hammer	≥25 Clegg Impact Value (CIV)	4 per 1,000m ³ (minimum 8 per lift)

If subgrade materials are unable to achieve the above criteria they should be undercut and may be used as landscape fill in reserve areas or be removed from site. This may include the geotechnical test pit excavation locations.

The natural subgrade soils can be sensitive to remoulding and moisture ingress. Care will be required to avoid over working and trafficking of these materials, and to protect them from moisture ingress. Pumping and yielding conditions may be encountered near the groundwater level or during particularly rainy periods.

Other areas of unsuitable materials such as variable strength uncontrolled fill present within road or building areas should be undercut and may be used as landscape fill in reserve areas or be removed from site. This may include the geotechnical test pit excavation locations. Further specific earthworks recommendations and specifications should be confirmed at the detailed design stage.

8.2 Ground Improvement for Static Settlement

8.2.1 Ground Improvement Options

To minimise post construction static ground settlements, there are a range of options, including the following:

- Construction of a temporary surcharge or pre-load fill embankment above design finished, with or without wick drains, to over-consolidate the compressible soils and minimise post construction embankment settlements;
- Use of lightweight geofoam, such as EPS-block materials for embankment construction to keep embankment pressures below pre-consolidation pressures within the compressible soil unit thereby reducing consolidation settlements;
- Undertake ground improvement beneath the embankment footprint, such as stone columns, soil mixed columns, CFA piles, Rammed Aggregate Piers (RAP's) or similar rigid inclusions to transfer loads from the embankment to more competent underlying soils at depth.

Consideration has been given to these options, however due to the widespread compressible soils across the site, peat thickness of up to 12m, and shallow groundwater depth, temporary surcharge filling during construction is the most practicable option.

8.2.2 Preliminary Ground Improvement Design

Preliminary surcharge design was carried out using the parameters discussed in Section 7.9.1 to assess the temporary surcharge height required to minimise post-construction embankment settlements to acceptable limits.

The surcharge design has assumed that engineered fill and surcharge material will be compacted to a bulk density of 17kN/m³.

The design calculations account for the change in effective stress resulting from the settlement of the engineered fill during the surcharge hold period.

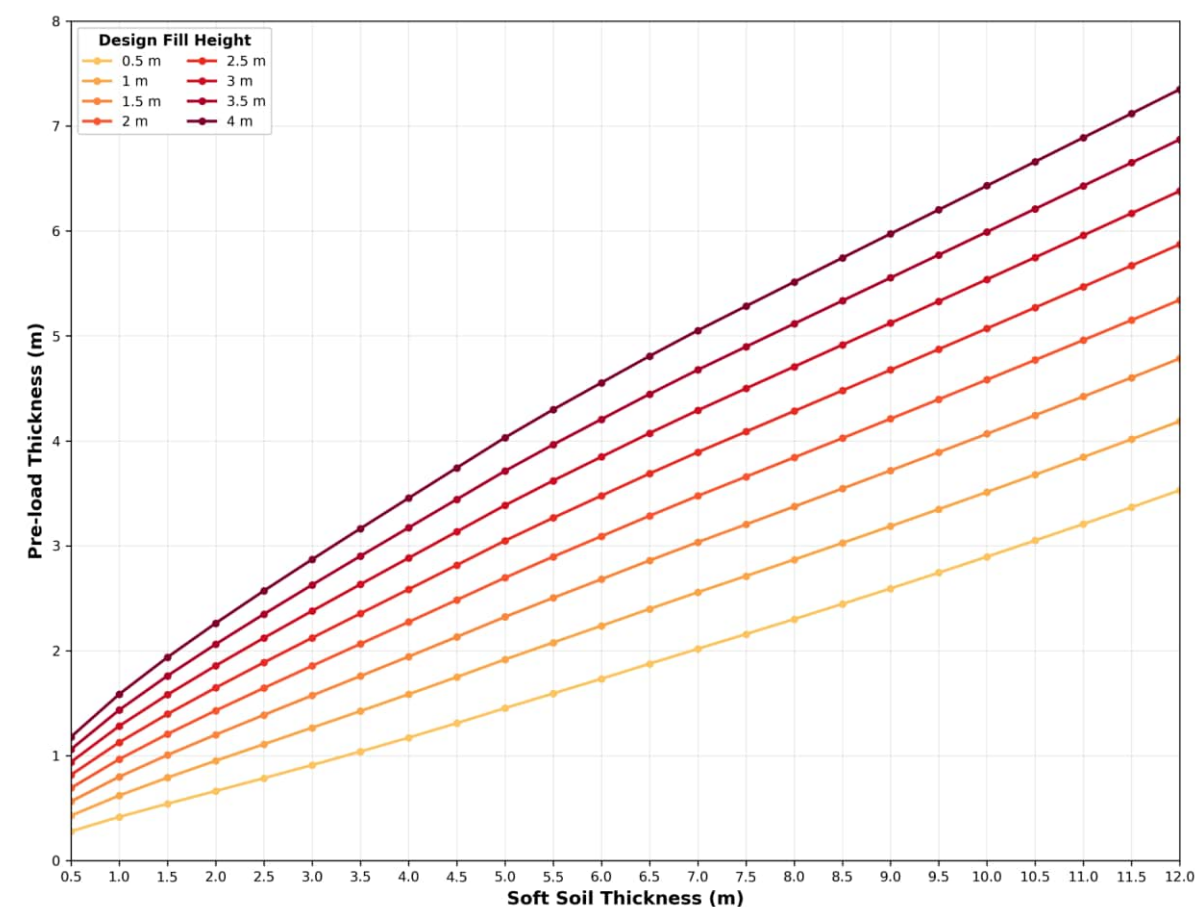
8.2.2.1 Case A: Roads

Surcharge design has been completed for the roads using the following parameters:

- Design life = 100 years
- Widespread deadload surcharge following temporary surcharge removal = 0kPa
- Target secondary creep settlement = 25mm

The resulting preliminary design surcharge heights are presented in **Figure 8**.

Figure 8: Preliminary surcharge height required to achieve 25mm secondary creep settlement over 100-year design life



Nominally, the surcharge embankment would need to extend the depth of the soft soils beyond the road corridor extent to ensure the underlying subsoils are adequately surcharged within the zone of influence of the road.

8.2.2.2 Buildings

For buildings, consideration has been given to the following cases:

- No surcharge placed on lots. Lots will settle as per Section 7.9.2.3. Specific ground improvement design would be left to the future lot-owner.
- Place a nominal 20kPa surcharge on the lots. This reduces the long-term secondary creep settlement over the design life of the lots. Specific ground improvement required for the building platform would be left to the lot owner.
- Surcharge lots to achieve 25mm settlement with no building load. Specific design will be required for the building foundations.

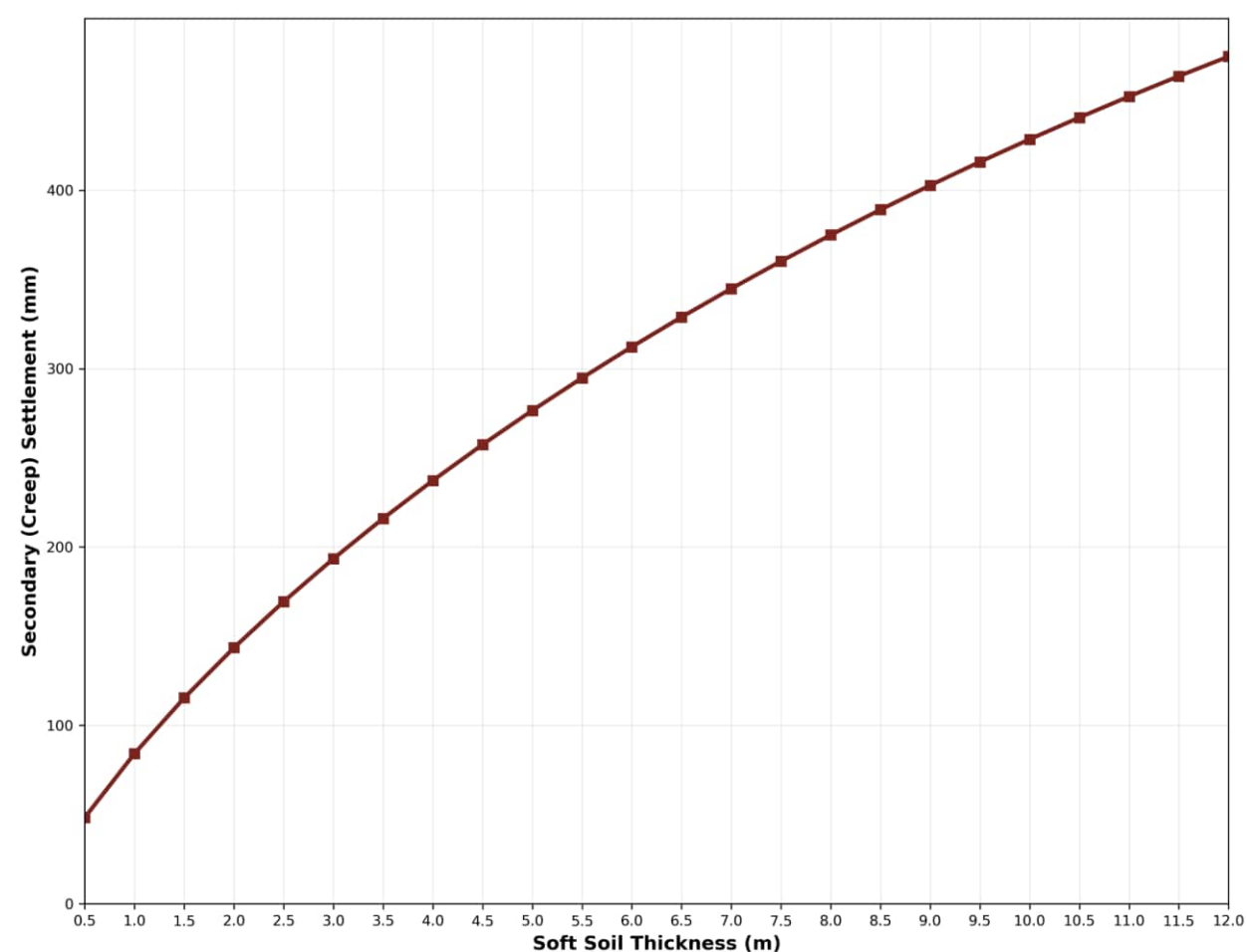
Where lots are not preloaded to meet the nominated building settlement criteria, future lot owners should be advised that site-specific geotechnical investigation, settlement assessment and foundation/ground improvement design will be required prior to building consent.

In options a) and b), specific ground improvement design would be left to the lot owner and could be built into the foundations (e.g. deep piles, rigid inclusions, or similar mechanism to transfer the building load to the underlying dense soils). However, in these scenarios there would still be differential settlement between the buildings and the road that would need to be designed for. For buried services, this should include flexible connections.

8.2.2.2.1 Case B: 20kPa Temporary Surcharge

A 20kPa temporary surcharge case has been modelled using a temporary earthfill surcharge placed at 17kN/m³ (1.3m high). In this case, secondary creep settlement following the removal of the 20kPa surcharge is reduced to 50mm – 275mm, as shown in **Figure 9**.

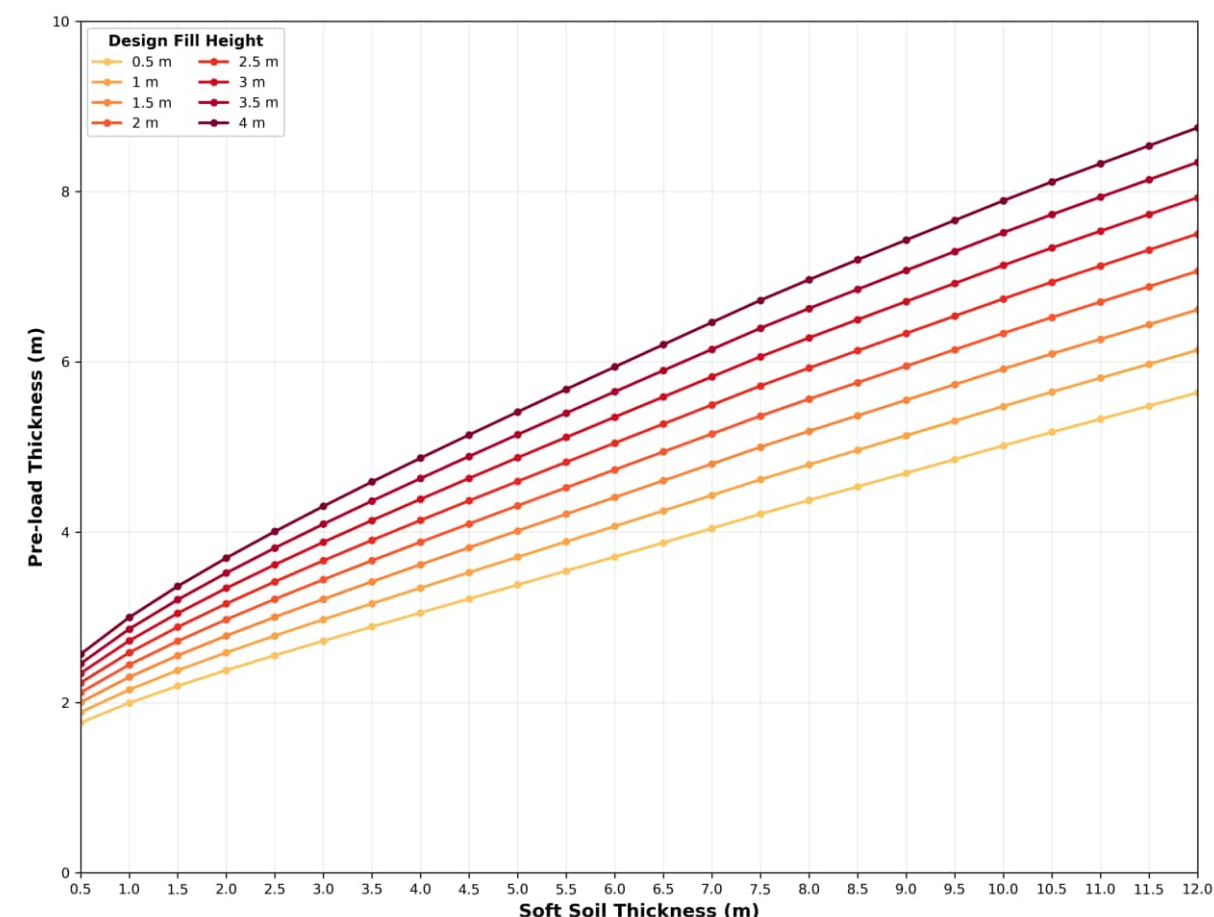
Figure 9: Secondary Creep Settlement in the 20kPa Temporary Surcharge Case



8.2.2.2.2 Case C: Surcharge lots to achieve 25mm Secondary Settlement

In this case, preliminary design was completed to assess the height of the surcharge that could be required to reduce the secondary creep settlement to no more than 25mm, assuming a 20kPa building surcharge load with a 50-year design life. In this case, the building load would still require specific design, which could include targeted surcharging, deep piles, or ground improvement. The height of the surcharge required is a function of the soft soil thickness and the dead load applied. The required surcharge height ranges from 1.8m to 12.7m.

Figure 10: Preliminary surcharge height required to achieve 25mm secondary creep settlement over 50-year design life with 20kPa widespread surcharge building load



8.2.2.3 Gravity Lines

Gravity wastewater lines are proposed along the road corridors between pump stations. These lines are anticipated to have very flat grades that need to achieve the design cleaning gradient requirements outlined in RITS. These lines will be located within the road corridors which are being designed to a total of 25mm settlement over the design life of the road. The critical case for these lines will be the differential settlement between manholes to ensure the lines remain serviceable. To achieve this, assessment will be undertaken during detailed design to analyse the differential settlement along the line, and resize the surcharge as required to achieve the performance requirement for the line.

8.2.2.4 Settlement Monitoring

Detailed settlement monitoring will be required during construction.¹⁷ As there will inevitably be some variation in soil composition and resulting settlement profiles from one location to the next, it is imperative that settlement monitoring is undertaken during construction to back analyse consolidation settlement parameters and progressively refine the pre-load design. It is expected that additional surcharge heights will be required at transition points from cut to fill.

Due to material composition and depth variability, regular monitoring points would be required, such as for example, at 50m longitudinal intervals along the road centreline to demonstrate that differential settlement compliance can be achieved.

We recommend the following monitoring is carried out while surcharging the road:

- Surface settlement plates, placed over the ground surface prior to filling, to assess total settlement magnitudes, which provide a cost-effective robust monitoring technique;
- Hydraulic profilometers installed beneath the embankment footprint to more accurately define cross sectional differential settlements;
- Vibrating wire piezometers installed at multiple depths within the peat unit to assess pore water pressure dissipation profiles for settlement and stability purposes.

Settlement plates and piezometers will also be required to monitor settlement during construction of the artificial wetlands, with spacing and depths to be confirmed during detailed design.

8.2.2.5 Peat Degradation

In addition to consolidation through settlement, peats can lose further volume through degradation and decomposition over long periods of time. This risk is considered low for this development due to the following:

- Filling across the site is required to raise the site, which will induce settlement within the peat and result in the peat remaining below the groundwater (saturated) over the design life of the structures.
- We have adopted a conservative c_α in the preliminary surcharge design of 0.06 to account for some long term degradation of the organic soils.

8.2.3 Cross Boundary Effects

Cross boundary settlement caused by proposed earthworks is expected to be negligible given the offsets of proposed fills;

- >10m offset to the nearest neighbouring building beyond the north-western boundary.
- 20m offset to the NIMT railway line.

The development proposal includes provision to surcharge the Southern Links road corridor. This will cause settlement within the proposed lots, therefore the road surcharge will need to occur prior to any buildings being constructed on the lots.

For lots within the site, if future lot owners choose to surcharge their lots, cross boundary settlement may occur. Any loading to the lots is not allowed within 5m of any lot boundary without detailed design.

Based on the current landform design, we are of the opinion that there are suitable setbacks from neighbouring structures outside the site and cross boundary settlement will be negligible, subject to review at detailed design.

¹⁷ The above settlement magnitude and time rate estimates are preliminary only, based on a limited amount of testing data, and have been averaged across the site.

8.3 Slope Stability

Proposed slopes for the artificial wetland and proposed fill batters meet the requisite factors of safety where filling is proposed above peat. We recommend maximum gradients of up to 1V:3H for the fill slopes are adopted for design. Steeper or higher slopes will require additional retention measures such as geogrid reinforcement, retaining walls or soil nails.

Where cutting into the peat is proposed, wetland gradients of 1V:3H do not meet the requisite factors of safety. Mitigation options include:

- Constructing a rock buttress (nominally 3 times H) at the toe of the slope to increase the resisting force against the peat.
- Reducing the cut batter slope within the peat. Nominally, a batter gradient of 1V:4H would be required.

Regarding lateral spread, preliminary results of the liquefied slope stability analyses estimate up to 70mm lateral displacement in liquefied soils for below ground wetland designs following ULS seismic event.

Buildings within 50m of the wetlands will require specific foundation design to withstand the lateral displacement. This requirement could be waived during detailed design, provided one of the following options are adopted to mitigate the effects of lateral spread:

- Building setback from stormwater wetlands;
- Wetland batter reinforcement with clay lining and geogrid reinforcing;
- Buttrressing wetland batters with aggregate material.
- Reinforcing wetland batters with sheet piles.

8.4 Road Subgrades

These equate to an estimated California Bearing Ratio (CBR) of 2% to 3% for the clay fill or CBR >10% could be expected on the gravel fill subject to fill thickness. Where there is at least 1m of fill after preloading these subgrade strengths will be lower and will require specific pavement design at the design stage.

8.5 Trench and Open Drain Backfilling

All services trenches and open drains beneath road carriageways and within a 45-degree zone of influence of any building foundation should be backfilled with compacted trench spoil or imported pit-sand to meet engineered fill standards.

8.6 Culverts and Bridges

Culverts will be required around the development where roads cross swales. Constructing these in peat is feasible from a geotechnical perspective but will require careful geotechnical assessment and construction staging. Concept design philosophy is outlined below:

- Mitigate settlement through either temporary surcharging or piled embankment.
- Mitigate differential settlement between approach embankment and culverts with detailed soil improvement design and geogrids.
- Slope batter stability to be accounted for in geotechnical design. Geogrids and/or a rock buttress could be required depending on final batter configuration.

- Culverts will need to be constructed on top of a geogrid reinforced gravel raft to mitigate against differential settlement and bearing capacity risks. This will be required to be wrapped in geotextile and geogrid to mitigate against the loss of the raft into the peat.

Construction of culverts will need to be staged as follows:

- Surcharging:
 - Build embankment and surcharge for the required duration. Surcharge will be required along the full length of the culvert.
 - Remove surcharge and construct gravel raft for culvert.
 - Construct culvert/bridge. Bridge to be installed post-surcharging to mitigate against down drag risk.
- Piled Embankment:
 - Construct working platform, install rigid inclusions. Inclusions would be required along the full length of the culvert.
 - Construct LTP and fill embankment.
 - Construct culvert/bridge. Bridge can be constructed concurrently with the fill embankment.

8.7 Foundations

It is our understanding that the subdivision will consist of predominantly industrial warehouses constructed with concrete tilt panels and/or lightweight cladding.

As discussed in Section 7.5, there is potential for liquefaction-induced ground deformation to occur at the site, and differential settlement is likely to occur following a ULS earthquake that will exceed the limits of typical foundations. These effects can be managed provided building sites are sufficiently preloaded and capped with engineer certified clay or gravel fill as per the recommendations outlined in Section 7.10. Specific assessment of the SLS and ULS liquefaction settlements will be required during detailed design once design ground levels are finalised.

Buildings with increased loads will require a specific design to mitigate the long-term settlement risks. This could include designing the foundation to distribute the load across a wider area, or transferring the load to more competent subsoils using piles or ground improvement such as site-specific temporary surcharges, rigid inclusions or stone columns.

A preliminary geotechnical ultimate bearing pressure of 200kPa should be available in the static case for shallow strip and pad foundations constructed within the engineered fill areas after preload. For clay materials, it is important to maintain the moisture content of exposed building platforms

Once building designs have been finalised, specific site investigation and foundation suitability assessments including bearing capacity and static settlement should be carried out prior to Building Consent application.

Table 11: Anticipated Foundation Details

Item	Methodology	Value	Comment
Geotechnical Ultimate Bearing Capacity (GUBC) for shallow foundations		200 kPa (static case)	Available within engineered fill areas. TBC at GCR stage
Expansive Soil	AS2870	M (moderate)	Anticipated characteristic surface movement of up to 40mm (M)

¹⁸ Ministry of Business, Innovation and Employment (2025) *B1 Structure, Verification Method B1/VM2*, Geotechnical design of foundations

Item	Methodology	Value	Comment
Strength reduction factors	B1/VM2 ¹⁸	0.8	Load combinations involving earthquake overstrength
		0.5	All other load combinations
Seismic Site Class(es)	NZS1170	D	

8.8 Certification

At the completion of the works, a Geotechnical Completion Report (GCR) will be prepared. The GCR will:

- report on the works undertaken
- confirm foundation design parameters
- describe future building and/ or earthworks limitations

8.9 Additional recommendations

The following are additional recommendations to be adopted as the project progresses:

- Prior to construction a Geotechnical Earthworks Specification should be prepared, outlining fill specifications and testing required during construction.
- Additionally, in site won materials that are to be used as fill, further laboratory testing should be carried out to optimise the design, subject to recommendations in Section 8.1.1. Atterberg/fines content testing should be carried out to optimise liquefaction and lateral spread modelling, compaction curve and solid density testing to determine potential fill quality, and oedometer testing should be considered to optimise preload design
- At the completion of the works, a Geotechnical Completion Report (GCR) will be prepared by a Chartered Professional Engineer outlining the works completed and all recommendations and restrictions for future development on each lot.

9.0 SUITABILITY STATEMENT AND CONCLUSIONS

Based on the results of previous investigations at the site and subject to the recommendations stated above, we consider that the site is suitable for the proposed Industrial Development. The proposed plan to enable industrial development, with associated roading infrastructure and stormwater management, is considered appropriate from a geotechnical perspective provided the recommendations in this report are adopted.

10.0 CLOSURE

Additional important information regarding the use of this CMW report is provided in the 'Using your CMW Report' document attached to this report.

This report has been prepared for use by the client in relation to the the substantive application for the Project under the FTAA in accordance with the scope, proposed uses and limitations described in the report.

APPENDIX A

Drawings



LEGEND:

- SOUTH PARK AGRI

HCC LAND

INDUSTRE PROPERTY TAHI

PIEZOMETERS (WGA 2025)

CPT (CMW 2025)

DCP (NZGD)

HA (NZGD)

SOAKAGE TEST (2025)

TEST PIT (NZGD)

BOREHOLE (NZGD)

TP (CMW 2025)

CPT (NZGD)

SECTION 0

CONTOUR
- NOTES:
1. BASE AERIAL PLAN SOURCED FROM GOOGLE EARTH (2025 IMAGE)
2. CONTOURS AT 1M INTERVALS ADAPTED FROM MAVEN ASSOCIATES LIDAR DRONE SURVEY (REF: EX-GL FINAL)
3. TEST LOCATIONS ARE INDICATIVE ONLY

0

100

200

300

400 m
-
- | | | | | | | |
|----------|-----------------------------------|--|-----------|------------|----------|--------------|
| CLIENT: | COLLIERS PROJECT LEADERS | | DRAWN: | AN | PROJECT: | HAM2024-0120 |
| PROJECT: | STAGE 1 INDUSTRIAL SOUTHERN LINKS | | CHECKED: | MM | DRAWING: | DWG25-01 |
| TITLE: | SITE INVESTIGATION PLAN | | REVISION: | 0 | SCALE: | 1:5,000 |
| | | | DATE: | 11/03/2025 | SHEET: | A3L |



- PEAT THICKNESS CONTOURS (1M INTERVALS)
- SOUTH PARK AGRI
- HCC LAND
- INDUSTRE PROPERTY TAHI
- UNCONTROLLED FILL
- PEAT

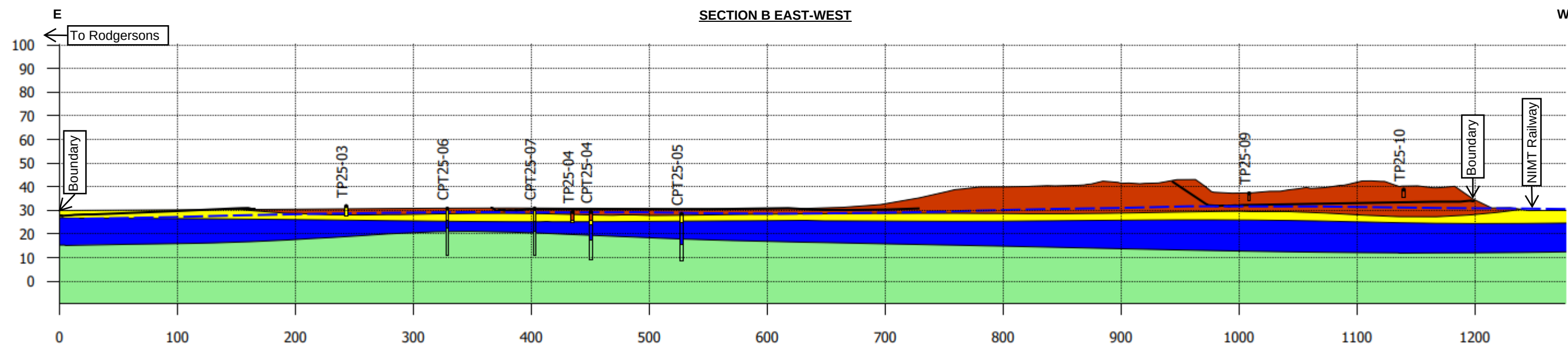
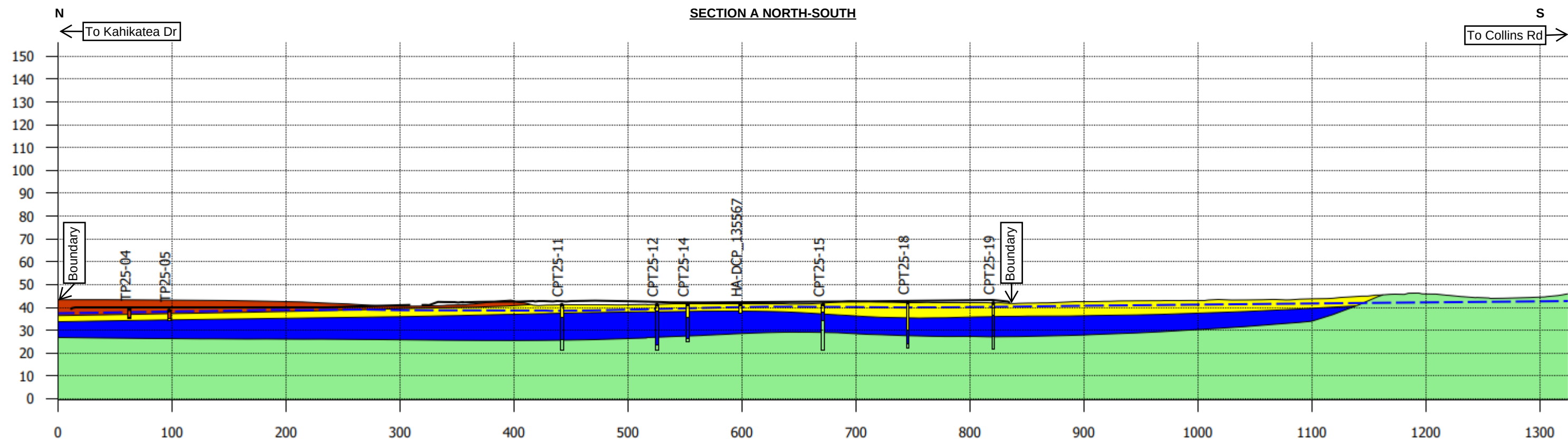
NOTES:
1. BASE AERIAL PLAN SOURCED FROM GOOGLE EARTH (2025 IMAGE)
2. GEOLOGY UNITS ARE INDICATIVE ONLY.
3. PEAT THICKNESS CONTOURS WERE GENERATED BY 3D LEAPFROG MODEL BASED ON LIMITED TEST DATA AND ARE INDICATIVE ONLY.





Great People | Practical Solutions

CLIENT:	COLLIERS PROJECT LEADERS	
PROJECT:	STAGE 1 INDUSTRIAL SOUTHERN LINKS	
TITLE:	GEOLOGY PLAN	
CLIENT:	DRAWN: AN	PROJECT: HAM2024-0120
PROJECT:	CHECKED: MM	DRAWING: DWG25-02
TITLE:	REVISION: 0	SCALE: 1:5,000
	DATE: 11/03/2025	SHEET: A3L



Scale: 1:3,500
Vertical exaggeration: 2x

Legend

GM

 Fill	 Int Sand/Silt
 peat	 Clay/Silt

Surfaces

Preliminary Design Surface Groundwater

NOTES:

1. PRELIMINARY DESIGN SURFACE PROVIDED BY MAVEN ASSOCIATES REF: 260218 DRAFT PGL SURFACE
2. STRATIGRAPHY PRODUCED IN LEAPFROG 3D MODELLING SOFTWARE BASED ON LIMITED TEST DATA AND INDICATIVE ONLY



CLIENT:	COLLIERS PROJECT LEADERS		DRAWN:	MM	PROJECT:	HAM2024-0120
PROJECT:	STAGE 1 INDUSTRIAL SOUTHERN LINKS		CHECKED:	DMM	DRAWING:	03
TITLE:	CROSS SECTIONS A & B		REVISION:	0	SCALE:	1:3,500
			DATE:	11/03/2026	SHEET:	A3

APPENDIX B

Historical Aerial Images



LEGEND:



SITE BOUNDARY

NOTES:

BASE IMAGE SOURCED FROM RETROLENS



CLIENT:	COLLIERS PROJECT LEADERS LTD	DRAWN:	LM	PROJECT:	HAM2024-0120
PROJECT:	SOUTHERN LINKS STAGE 1 INDUSTRIAL HISTORICAL PHOTOS	CHECKED:	MM	DRAWING:	A1
		REVISION:	0	SCALE:	NTS
TITLE:	1943	DATE:	30/01/2026	SHEET:	A3



LEGEND:
SITE BOUNDARY

NOTES:
BASE IMAGE SOURCED FROM RETROLENS

	CLIENT:	COLLIERS PROJECT LEADERS LTD		DRAWN:	LM	PROJECT:	HAM2024-0120
	PROJECT:	SOUTHERN LINKS STAGE 1 INDUSTRIAL HISTORICAL PHOTOS		CHECKED:	MM	DRAWING:	A2
				REVISION:	0	SCALE:	NTS
	TITLE:	1971		DATE:	30/01/2026	SHEET:	A3



LEGEND:

 SITE BOUNDARY

NOTES:

BASE IMAGE SOURCED FROM RETROLENS

 Great People Practical Solutions	CLIENT:	COLLIERS PROJECT LEADERS LTD		DRAWN:	LM	PROJECT:	HAM2024-0120
	PROJECT:	SOUTHERN LINKS STAGE 1 INDUSTRIAL HISTORICAL PHOTOS		CHECKED:	MM	DRAWING:	A3
	TITLE:	1990		REVISION:	0	SCALE:	NTS
				DATE:	30/01/2026	SHEET:	A3



LEGEND:

 SITE BOUNDARY

NOTES:

BASE IMAGE SOURCED FROM GOOGLE EARTH

 Great People Practical Solutions	CLIENT:	COLLIERS PROJECT LEADERS LTD		DRAWN:	LM	PROJECT:	HAM2024-0120
	PROJECT:	SOUTHERN LINKS STAGE 1 INDUSTRIAL HISTORICAL PHOTOS		CHECKED:	MM	DRAWING:	A4
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				DATE:	03/02/2026	SHEET:	A3



LEGEND:

— SITE BOUNDARY

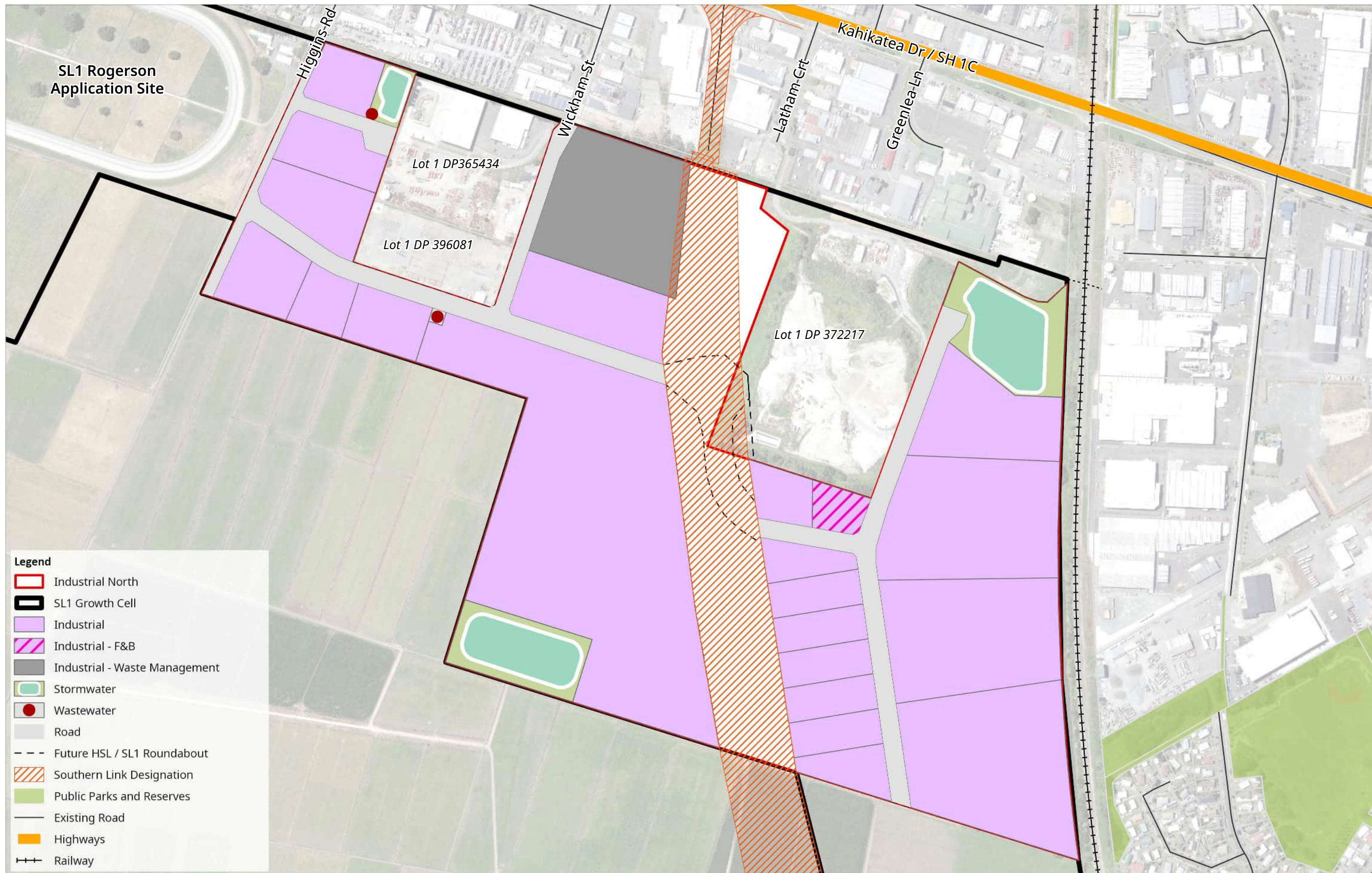
NOTES:

BASE IMAGE SOURCED FROM GOOGLE EARTH

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	TITLE:	2016		REVISION:	0	SCALE:	NTS
				DATE:	02/03/2026	SHEET:	A3

APPENDIX C

Development Plans



Client: Saxby Rd JV Ltd, Industrie Property Tahi & South Park Agri Development Ltd

Job Number: 25546 & 26111

Drawing: Industrial North Proposed Masterplan

Version: Draft

Date: 30/06/2026

Scale: 1:5,000 @ A3

Sheet: 1 of 1



SL1 Residential South & Industrial North Substantive Application

49A and 49B Saxbys Road, Saxbys Road, 18, 20 & 24 Wickham Street, 160 & 162 Higgins Road, Waikato

B&A
Urban & Environmental



- NOTES
1. ALL WORKS TO BE IN ACCORDANCE WITH WAIKATO REGIONAL COUNCIL EROSION AND SEDIMENT CONTROL: GUIDELINES FOR SOIL DISTURBING ACTIVITIES TR 2009/02.
 2. COORDINATES IN TERMS OF NZ GEODETIC DATUM MT EDEN 2000
 3. LEVELS IN TERMS OF THE NEW ZEALAND VERTICAL DATUM 2016.
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 6. THE CONTRACTOR SHALL OBTAIN ALL NECESSARY APPROVAL FROM UTILITY OPERATORS BEFORE COMMENCING WORK UNDER OR NEAR THEIR SERVICES.
 7. CONTRACTOR SHALL PROVIDE AS-BUILT OF WORKING SEDIMENT CONTROL DEVICES AND CONFIRMATION OF POND/DECENT VOLUMES TO ENGINEER.

LEGEND

---	EX BDY
---	PROP BDY
---	PR MAJOR CONTOUR
---	PR MINOR CONTOUR

B	DRAFT	MKS	06/2026
A	DRAFT	MKS	06/2026
Rev	Description	By	Date
Survey	MAVEN		05/2025
Design	MKS		06/2026
Drawn	MKS		06/2026
Checked	SM		06/2026



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Project
**SOUTHERN LINKS 1
INDUSTRIAL STAGE 1
HAMILTON, WAIKATO
FOR
SL1 CONSORTIUM GROUP**

Title
**PROPOSED
CONTOUR
OVERVIEW PLAN**

Project no.	298002		
Scale	1:5000 @ A3		
Cad file	298002-11-C200 EARTHWORKS.DWG		
Drawing no.	C200	Rev	B

DRAFT FOR REVIEW



HIGGINS ROAD

WICKHAM STREET

ARTIFICIAL
STORMWATER
WETLAND 1

- NOTES
1. ALL WORKS TO BE IN ACCORDANCE WITH WAIKATO REGIONAL COUNCIL EROSION AND SEDIMENT CONTROL: GUIDELINES FOR SOIL DISTURBING ACTIVITIES TR 2009/02.
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LEGEND

---	EX BDY
---	PROP BDY
---	PR MAJOR CONTOUR
---	PR MINOR CONTOUR

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A	DRAFT	MKS	06/2026
Rev	Description	By	Date
		By	Date
Survey	MAVEN	05/2025	
Design	MKS	06/2026	
Drawn	MKS	06/2026	
Checked	SM	06/2026	



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Project
**SOUTHERN LINKS 1
INDUSTRIAL STAGE 1
HAMILTON, WAIKATO
FOR
SL1 CONSORTIUM GROUP**

Title
**PROPOSED
CONTOUR PLAN
STAGE 1A**

Project no.	298002		
Scale	1:2000 @ A3		
Cad file	298002-11-C200 EARTHWORKS.DWG		
Drawing no.	C200-1	Rev	B

LOT 1 DPS 51437

DRAFT FOR REVIEW



LOT 1 DPS 51437

PART LOT 2 DPS 4764

LOT 2 DPS 51437

ARTIFICIAL STORMWATER WETLAND 2

- NOTES
1. ALL WORKS TO BE IN ACCORDANCE WITH WAIKATO REGIONAL COUNCIL EROSION AND SEDIMENT CONTROL: GUIDELINES FOR SOIL DISTURBING ACTIVITIES TR 2009/02.
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LEGEND

---	EX BDY
---	PROP BDY
---	PR MAJOR CONTOUR
---	PR MINOR CONTOUR

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A	DRAFT	MKS	06/2026
Rev	Description	By	Date
		By	Date
Survey	MAVEN		05/2025
Design	MKS		06/2026
Drawn	MKS		06/2026
Checked	SM		06/2026



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Project

**SOUTHERN LINKS 1
INDUSTRIAL STAGE 1
HAMILTON, WAIKATO
FOR
SL1 CONSORTIUM GROUP**

Title

**PROPOSED
CONTOUR PLAN
STAGE 1B**

Project no.	298002		
Scale	1:2500 @ A3		
Cad file	298002-11-C200 EARTHWORKS.DWG		
Drawing no.	C200-2	Rev	B

DRAFT FOR REVIEW



- NOTES
1. ALL WORKS TO BE IN ACCORDANCE WITH WAIKATO REGIONAL COUNCIL EROSION AND SEDIMENT CONTROL: GUIDELINES FOR SOIL DISTURBING ACTIVITIES TR 2009/02.
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LEGEND

	EX BDY
	PROP BDY
	PR MAJOR CONTOUR
	PR MINOR CONTOUR

B	DRAFT	MKS	06/2026
A	DRAFT	MKS	06/2026
Rev	Description	By	Date
		By	Date
Survey	MAVEN		05/2025
Design	MKS		06/2026
Drawn	MKS		06/2026
Checked	SM		06/2026

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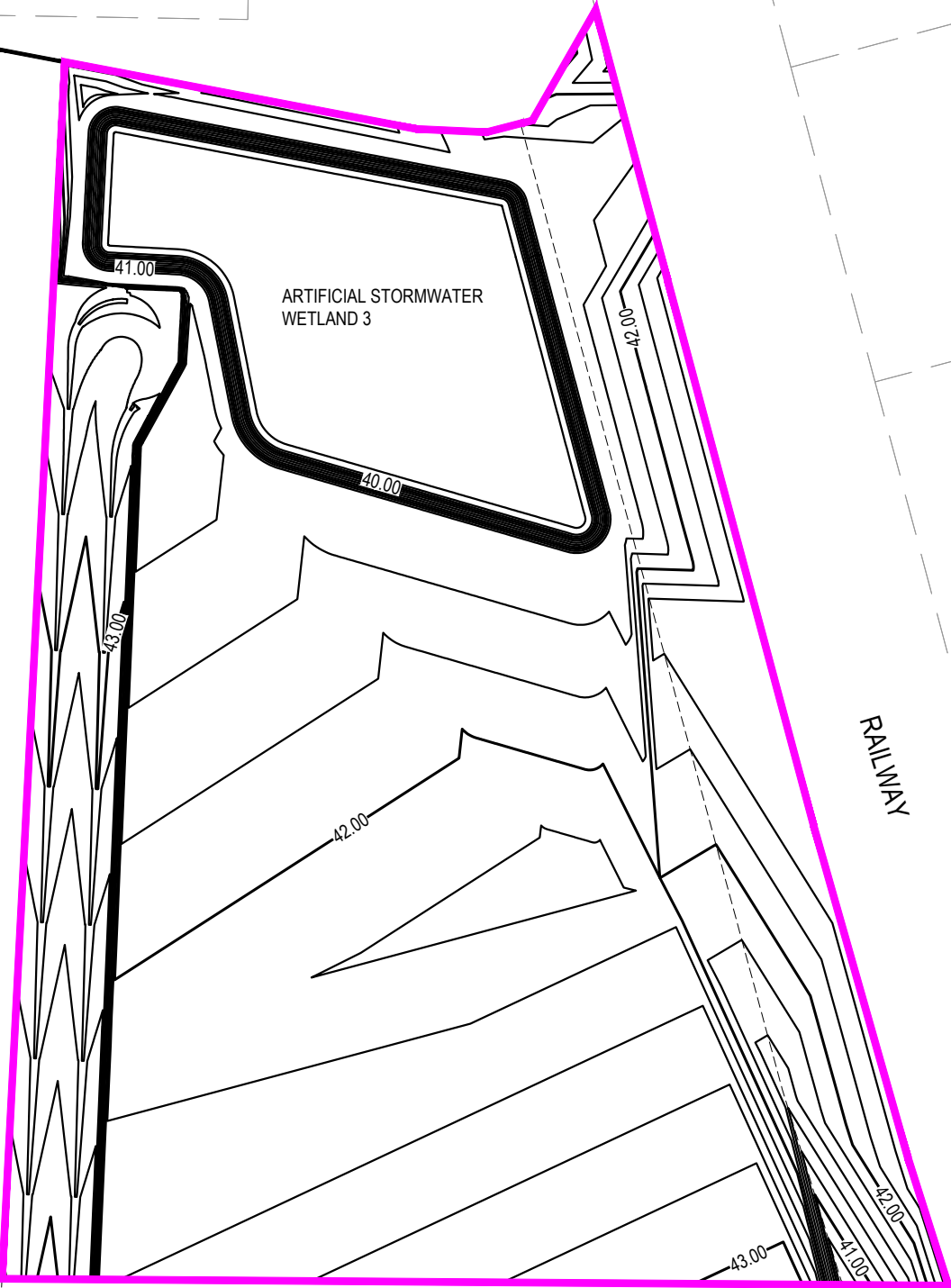
Project
**SOUTHERN LINKS 1
INDUSTRIAL STAGE 1
HAMILTON, WAIKATO
FOR
SL1 CONSORTIUM GROUP**

Title
**PROPOSED
CONTOUR PLAN
STAGE 2**

Project no.	298002		
Scale	1:2500 @ A3		
Cad file	298002-11-C200 EARTHWORKS.DWG		
Drawing no.	C200-3	Rev	B

LOT 2 DPS 51437

DRAFT FOR REVIEW



- NOTES
1. ALL WORKS TO BE IN ACCORDANCE WITH WAIKATO REGIONAL COUNCIL EROSION AND SEDIMENT CONTROL: GUIDELINES FOR SOIL DISTURBING ACTIVITIES TR 2009/02.
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LEGEND

---	EX BDY
---	PROP BDY
---	PR MAJOR CONTOUR
---	PR MINOR CONTOUR

B	DRAFT	MKS	06/2026
A	DRAFT	MKS	06/2026
Rev	Description	By	Date
		By	Date
Survey	MAVEN	05/2025	
Design	MKS	06/2026	
Drawn	MKS	06/2026	
Checked	SM	06/2026	



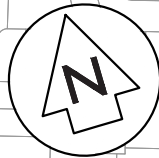
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Project
**SOUTHERN LINKS 1
INDUSTRIAL STAGE 1
HAMILTON, WAIKATO
FOR
SL1 CONSORTIUM GROUP**

Title
**PROPOSED
CONTOUR PLAN
STAGE 3**

Project no.	298002		
Scale	1:2000 @ A3		
Cad file	298002-11-C200 EARTHWORKS.DWG		
Drawing no.	C200-4	Rev	B

DRAFT FOR REVIEW



KAHIKATEA DRIVE

HIGGINS ROAD

WICKHAM STREET

LATHAM COURT

NZTA
ROAD
CORRIDOR

RAILWAY

STAGE 1A

STAGE 3

STAGE 1B

STAGE 2

NZTA
ROAD
CORRIDOR

LOT 2 DPS 51437

PART LOT 2 DPS 4764

OVERALL EARTHWORKS CUT/FILL

(FROM FINISHED SUBGRADE
TO EXISTING SUBGRADE I.E EXCLUDES TOPSOIL)

CUT VOLUME (IN-SITU) 739,445m³
FILL VOLUME (IN-SITU) 851,071m³
FILL VOLUME (FACTOR 1.2) 1,021,285m³
SHORTAGE OF FILL 284,321m³

TOPSOIL STRIPPED (300mm) = 156,253m³
EARTHWORKS AREA = 52.08ha

NOTE: NO ALLOWANCE FOR WETLAND POND PLANTING MEDIAN,
PAVEMENT CONSTRUCTION, SERVICES TRENCHES,
OR DRAINAGE TRENCHES

NOTES

1. ALL WORKS TO BE IN ACCORDANCE WITH WAIKATO REGIONAL COUNCIL EROSION AND SEDIMENT CONTROL: GUIDELINES FOR SOIL DISTURBING ACTIVITIES TR 2009/02.
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Cut/Fill Table

Number #	Minimum Elevation	Maximum Elevation	Color
1	-22.000	-20.000	
2	-20.000	-18.000	
3	-18.000	-16.000	
4	-16.000	-14.000	
5	-14.000	-12.000	
6	-12.000	-10.000	
7	-10.000	-8.000	
8	-8.000	-6.000	
9	-6.000	-4.000	
10	-4.000	-2.000	
11	-2.000	0.000	
12	0.000	2.000	
13	2.000	4.000	
14	4.000	6.000	

B	DRAFT	MKS	06/2026
A	DRAFT	MKS	06/2026
Rev	Description	By	Date
		By	Date
Survey	MAVEN		05/2025
Design	MKS		06/2026
Drawn	MKS		06/2026
Checked	SM		06/2026

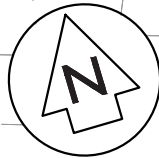


Project
**SOUTHERN LINKS 1
INDUSTRIAL STAGE 1
HAMILTON, WAIKATO
FOR
SL1 CONSORTIUM GROUP**

Title
**PROPOSED
CUT/FILL
OVERVIEW PLAN**

Project no.	298002		
Scale	1:5000 @ A3		
Cad file	298002-11-C220 EW CUT&FILL.DWG		
Drawing no.	C220	Rev	B

DRAFT FOR REVIEW



HIGGINS ROAD

WICKHAM STREET



STAGE 1A - EARTHWORKS CUT/FILL
(FROM FINISHED SUBGRADE
TO EXISTING SUBGRADE I.E EXCLUDES TOPSOIL)

CUT VOLUME (IN-SITU)	45,825m³
FILL VOLUME (IN-SITU)	41,167m³
FILL VOLUME (FACTOR 1.2)	49,400m³
SHORTAGE OF FILL	3,575m³

TOPSOIL STRIPPED (300mm) =	27,075m³
EARTHWORKS AREA =	9.02ha

NOTE: NO ALLOWANCE FOR WETLAND POND PLANTING MEDIAN,
PAVEMENT CONSTRUCTION, SERVICES TRENCHES,
OR DRAINAGE TRENCHES

LOT 1 DPS 51437

- NOTES
1. ALL WORKS TO BE IN ACCORDANCE WITH WAIKATO REGIONAL COUNCIL EROSION AND SEDIMENT CONTROL: GUIDELINES FOR SOIL DISTURBING ACTIVITIES TR 2009/02.
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Cut/Fill Table			
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5	-14.000	-12.000	
6	-12.000	-10.000	
7	-10.000	-8.000	
8	-8.000	-6.000	
9	-6.000	-4.000	
10	-4.000	-2.000	
11	-2.000	0.000	
12	0.000	2.000	
13	2.000	4.000	
14	4.000	6.000	

B	DRAFT	MKS	06/2026
A	DRAFT	MKS	06/2026
Rev	Description	By	Date
		By	Date
Survey	MAVEN		05/2025
Design	MKS		06/2026
Drawn	MKS		06/2026
Checked	SM		06/2026



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Project
**SOUTHERN LINKS 1
INDUSTRIAL STAGE 1
HAMILTON, WAIKATO
FOR
SL1 CONSORTIUM GROUP**

Title
**PROPOSED
CUT/FILL PLAN
STAGE 1A**

Project no.	298002		
Scale	1:2000 @ A3		
Cad file	298002-11-C220 EW CUT&FILL.DWG		
Drawing no.	C220-1	Rev	B

DRAFT FOR REVIEW



LOT 1 DPS 51437

PART LOT 2 DPS 4764

LOT 2 DPS 51437

STAGE 1B - EARTHWORKS CUT/FILL
(FROM FINISHED SUBGRADE
TO EXISTING SUBGRADE I.E EXCLUDES TOPSOIL)

CUT VOLUME (IN-SITU) 9,243m³
FILL VOLUME (IN-SITU) 325,570m³
FILL VOLUME (FACTOR 1.2) 390,684m³
SHORTAGE OF FILL 381,441m³

TOPSOIL STRIPPED (300mm) = 54,745m³
EARTHWORKS AREA = 18.25ha

NOTE: NO ALLOWANCE FOR WETLAND POND PLANTING MEDIAN,
PAVEMENT CONSTRUCTION, SERVICES TRENCHES,
OR DRAINAGE TRENCHES

- NOTES
1. ALL WORKS TO BE IN ACCORDANCE WITH WAIKATO REGIONAL COUNCIL EROSION AND SEDIMENT CONTROL: GUIDELINES FOR SOIL DISTURBING ACTIVITIES TR 2009/02.
 2. COORDINATES IN TERMS OF NZ GEODETIC DATUM MT EDEN 2000
 3. LEVELS IN TERMS OF THE NEW ZEALAND VERTICAL DATUM 2016.
 4. IT IS THE CONTRACTORS RESPONSIBILITY TO LOCATE ALL SERVICES THAT MAY BE AFFECTED BY THEIR OPERATIONS.
 5. THE CONTRACTOR SHALL COMPLY WITH ALL RELEVANT HEALTH AND SAFETY REQUIREMENTS.
 6. THE CONTRACTOR SHALL OBTAIN ALL NECESSARY APPROVAL FROM UTILITY OPERATORS BEFORE COMMENCING WORK UNDER OR NEAR THEIR SERVICES.

Cut/Fill Table			
Number #	Minimum Elevation	Maximum Elevation	Color
1	-22.000	-20.000	
2	-20.000	-18.000	
3	-18.000	-16.000	
4	-16.000	-14.000	
5	-14.000	-12.000	
6	-12.000	-10.000	
7	-10.000	-8.000	
8	-8.000	-6.000	
9	-6.000	-4.000	
10	-4.000	-2.000	
11	-2.000	0.000	
12	0.000	2.000	
13	2.000	4.000	
14	4.000	6.000	

B	DRAFT	MKS	06/2026
A	DRAFT	MKS	06/2026
Rev	Description	By	Date
		By	Date
Survey	MAVEN		05/2025
Design	MKS		06/2026
Drawn	MKS		06/2026
Checked	SM		06/2026



Project
**SOUTHERN LINKS 1
INDUSTRIAL STAGE 1
HAMILTON, WAIKATO
FOR
SL1 CONSORTIUM GROUP**

Title
**PROPOSED
CUT/FILL PLAN
STAGE 1B**

Project no.	298002		
Scale	1:2500 @ A3		
Cad file	298002-11-C220 EW CUT&FILL.DWG		
Drawing no.	C220-2	Rev	B

DRAFT FOR REVIEW



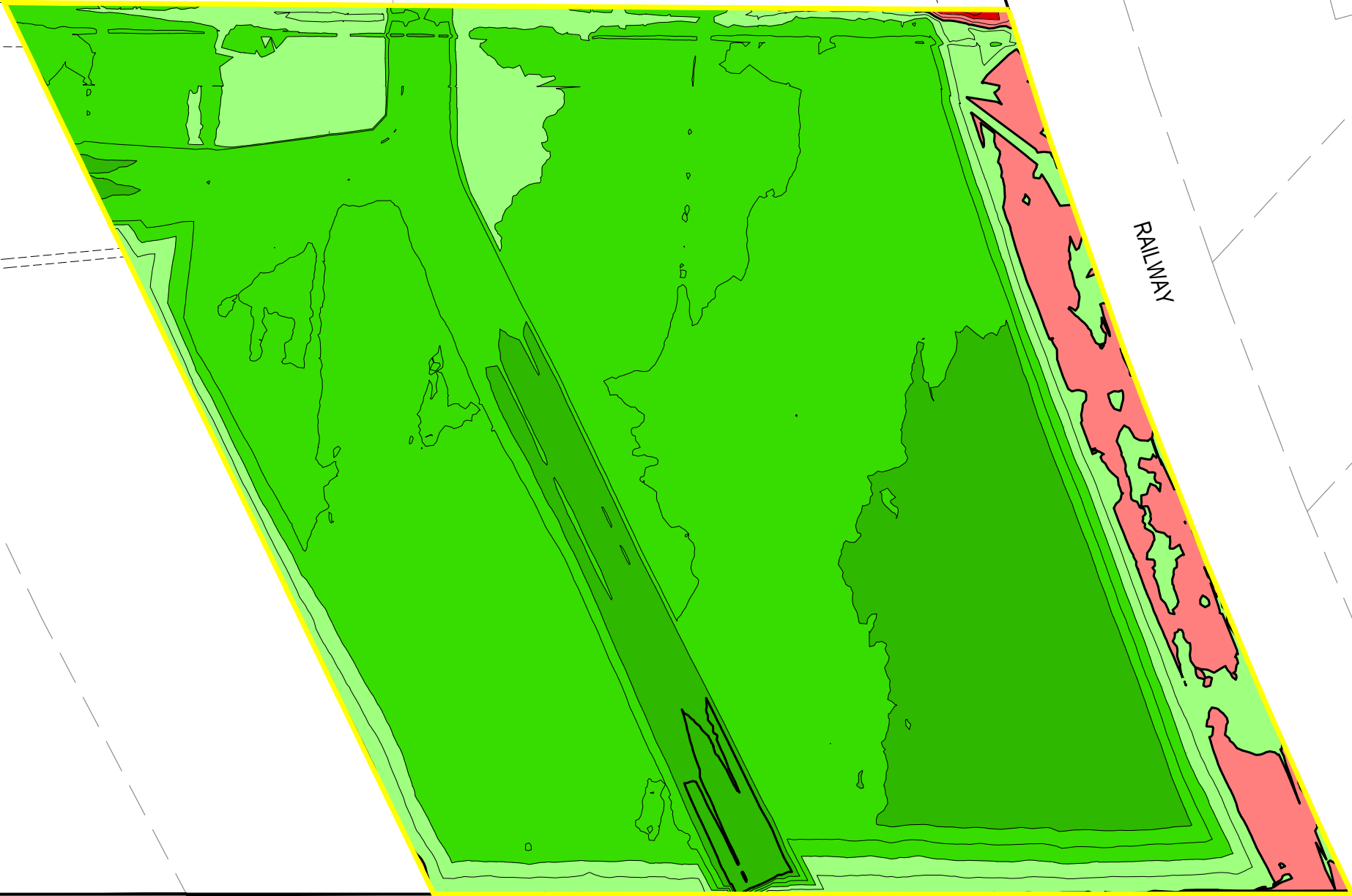
STAGE 2 - EARTHWORKS CUT/FILL

(FROM FINISHED SUBGRADE
TO EXISTING SUBGRADE I.E EXCLUDES TOPSOIL)

CUT VOLUME (IN-SITU) 1,052m³
FILL VOLUME (IN-SITU) 483,425m³
FILL VOLUME (FACTOR 1.2) 580,110m³
SHORTAGE OF FILL 579,058m³

TOPSOIL STRIPPED (300mm) = 51,542m³
EARTHWORKS AREA = 17.18ha

NOTE: NO ALLOWANCE FOR WETLAND POND PLANTING MEDIAN,
PAVEMENT CONSTRUCTION, SERVICES TRENCHES,
OR DRAINAGE TRENCHES



LOT 2 DPS 51437

- NOTES
1. ALL WORKS TO BE IN ACCORDANCE WITH WAIKATO REGIONAL COUNCIL EROSION AND SEDIMENT CONTROL: GUIDELINES FOR SOIL DISTURBING ACTIVITIES TR 2009/02.
 2. COORDINATES IN TERMS OF NZ GEODETIC DATUM MT EDEN 2000
 3. LEVELS IN TERMS OF THE NEW ZEALAND VERTICAL DATUM 2016.
 4. IT IS THE CONTRACTORS RESPONSIBILITY TO LOCATE ALL SERVICES THAT MAY BE AFFECTED BY THEIR OPERATIONS.
 5. THE CONTRACTOR SHALL COMPLY WITH ALL RELEVANT HEALTH AND SAFETY REQUIREMENTS.
 6. THE CONTRACTOR SHALL OBTAIN ALL NECESSARY APPROVAL FROM UTILITY OPERATORS BEFORE COMMENCING WORK UNDER OR NEAR THEIR SERVICES.

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3	-18.000	-16.000	
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9	-6.000	-4.000	
10	-4.000	-2.000	
11	-2.000	0.000	
12	0.000	2.000	
13	2.000	4.000	
14	4.000	6.000	

B	DRAFT		MKS 06/2026
A	DRAFT		MKS 06/2026
Rev	Description	By	Date
Survey	MAVEN	05/2025	
Design	MKS	06/2026	
Drawn	MKS	06/2026	
Checked	SM	06/2026	



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Project
**SOUTHERN LINKS 1
INDUSTRIAL STAGE 1
HAMILTON, WAIKATO
FOR
SL1 CONSORTIUM GROUP**

Title
**PROPOSED
CUT/FILL PLAN
STAGE 2**

Project no.	298002		
Scale	1:2500 @ A3		
Cad file	298002-11-C220 EW CUT&FILL.DWG		
Drawing no.	C220-3	Rev	B

DRAFT FOR REVIEW



STAGE 3 - EARTHWORKS CUT/FILL

(FROM FINISHED SUBGRADE
TO EXISTING SUBGRADE I.E EXCLUDES TOPSOIL)

CUT VOLUME (IN-SITU)	683,325m³
FILL VOLUME (IN-SITU)	909m³
FILL VOLUME (FACTOR 1.2)	1,091m³
SURPLUS OF CUT	682,234m³

TOPSOIL STRIPPED (300mm) =	22,891m³
EARTHWORKS AREA =	7.63ha

NOTE: NO ALLOWANCE FOR WETLAND POND PLANTING MEDIAN,
PAVEMENT CONSTRUCTION, SERVICES TRENCHES,
OR DRAINAGE TRENCHES



- NOTES
1. ALL WORKS TO BE IN ACCORDANCE WITH WAIKATO REGIONAL COUNCIL EROSION AND SEDIMENT CONTROL: GUIDELINES FOR SOIL DISTURBING ACTIVITIES TR 2009/02.
 2. COORDINATES IN TERMS OF NZ GEODETIC DATUM MT EDEN 2000
 3. LEVELS IN TERMS OF THE NEW ZEALAND VERTICAL DATUM 2016.
 4. IT IS THE CONTRACTORS RESPONSIBILITY TO LOCATE ALL SERVICES THAT MAY BE AFFECTED BY THEIR OPERATIONS.
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Cut/Fill Table			
Number #	Minimum Elevation	Maximum Elevation	Color
1	-22.000	-20.000	
2	-20.000	-18.000	
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6	-12.000	-10.000	
7	-10.000	-8.000	
8	-8.000	-6.000	
9	-6.000	-4.000	
10	-4.000	-2.000	
11	-2.000	0.000	
12	0.000	2.000	
13	2.000	4.000	
14	4.000	6.000	

B	DRAFT	MKS	06/2026
A	DRAFT	MKS	06/2026
Rev	Description	By	Date
	By	Date	
Survey	MAVEN	05/2025	
Design	MKS	06/2026	
Drawn	MKS	06/2026	
Checked	SM	06/2026	

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A

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New Zealand

Project
**SOUTHERN LINKS 1
INDUSTRIAL STAGE 1
HAMILTON, WAIKATO
FOR
SL1 CONSORTIUM GROUP**

Title
**PROPOSED
CUT/FILL PLAN
STAGE 3**

Project no.	298002		
Scale	1:2000 @ A3		
Cad file	298002-11-C220 EW CUT&FILL.DWG		
Drawing no.	C220-4	Rev	B

DRAFT FOR REVIEW