

TECHNICAL MEMORANDUM

WETLAND A DESIGN REVIEW

To	Southern Links 1 project team	From	Mark McVinnie, David MacPherson
Attention	c/o Dillon Kelliher (Colliers Project Leaders)	Date	28 August 2026
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Subject	Wetland A Design Review		

1.0 INTRODUCTION

The purpose of this memorandum is to analyse the effects of the confined aquifer on the stormwater design and support Maven's engineering designs.

This memorandum outlines the analysis results and provides preliminary design options for pond stability.

2.0 PROPOSED DEVELOPMENT

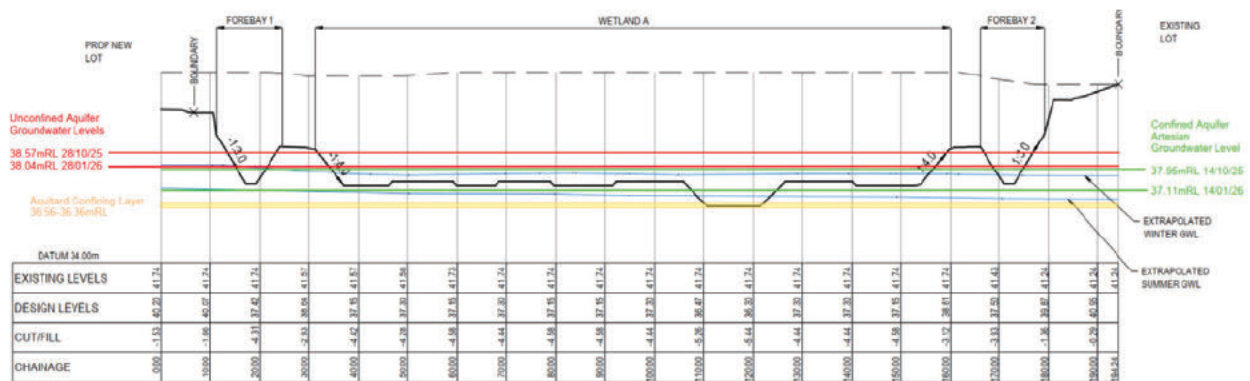
The proposed development for the Southern Links 1 Residential Development includes constructing approximately 938 lots with associated infrastructure across the 46Ha site. Original landform is to be modified with maximum proposed cut levels of 5m and proposed fill levels of up to 2m.

Six stormwater wetlands are proposed across the development. It is understood that they will be up to 4.8m deep with maximum batter slopes of 1V:3H. Plans provided by Maven Associates are attached in *Error! Reference source not found.A*.

3.0 HYDROGEOLOGY

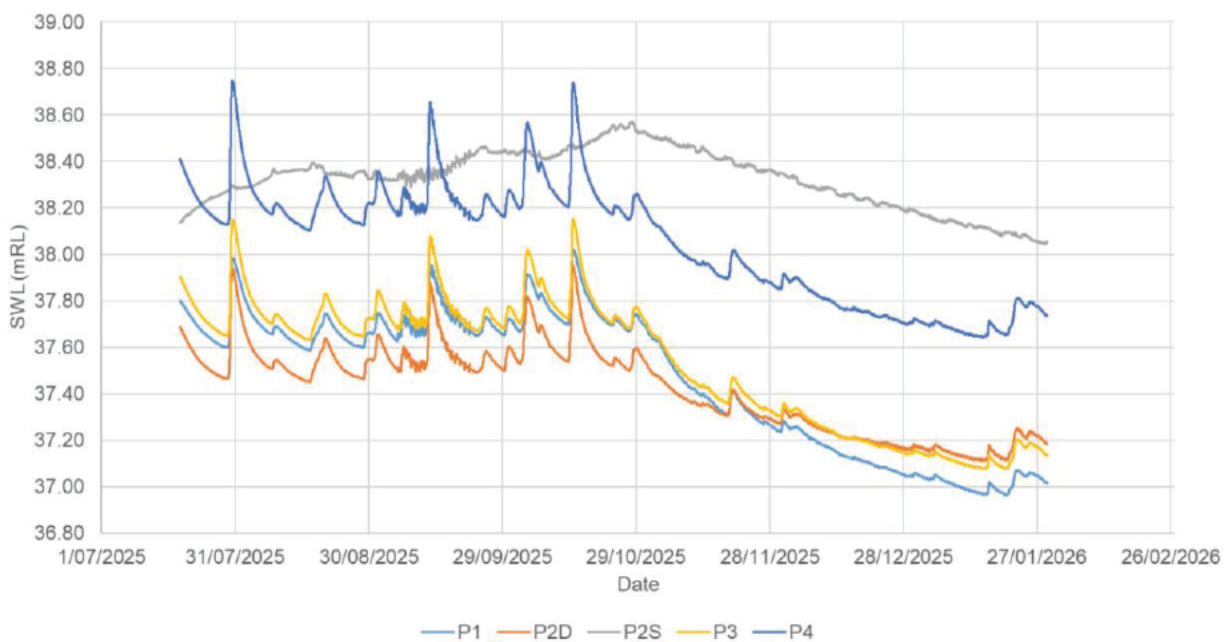
The WGA Hydrogeological Effects Assessment identified a low-permeability aquitard layer running beneath the site, which has a confining effect on the underlying deep aquifer, resulting in a measured pressure head of up to 1.35m above the deep aquifer. WGA carried out investigations, testing, and monitoring across the site at locations shown in **Appendix B**. This thin confining silt layer has been encountered across the site in investigations by CMW and WGA. The overall groundwater regime is shown in **Figure 1**.

Figure 1: Wetland A Hydrogeology. Source: WGA



Groundwater monitoring data is presented in **Figure 2**.

Figure 2: Groundwater Monitoring Data. P2 is within Wetland A. Source: WGA



Further discussion of the overall site hydrogeological regime is presented in the WGA Hydrological Effects Assessment.

Excavating the proposed stormwater ponds will remove the existing soil overburden pressure; and, without implementation of mitigation options (as later outlined in this memorandum), the uplift pressure from the confined aquifer could rupture the aquitard layer and destabilise the ponds.

The results of analyses of the field data have been utilised in the development of a 2D cross section model. Detailed descriptions of the hydrogeology and hydraulic testing of the site used to build the model are provided in the Hydrological Effects Assessment.

4.0 UPLIFT ASSESSMENT

An effective stress assessment was undertaken to assess the minimum thickness of overburden pressure that would need to be left in place when constructing the wetland. 13.9kPa uplift pressure is acting on the silt (using the measured maximum groundwater level). Adopting a factor of safety against uplift of 1.2, a minimum of 16.7kPa of overburden pressure would need to be maintained above the top of the silt layer. Using a unit weight of 16kN/m³, this equates to a minimum overburden thickness of 1.05m or an RL of 37.61m.

This would result in lifting the pond base by 1.31m (above the current proposed level) to maintain a suitable factor of safety against uplift.

Recommendations are discussed in Section 6.0.

5.0 SLOPE STABILITY ASSESSMENT

5.1 Design Criteria

The stability of the ponds under a range of design conditions is expressed in terms of a factor of safety, which is defined as the ratio of the resisting forces to destabilising forces acting on a slope. Table 1 below presents the recommended performance standards¹ for slope stability assessment:

Table 1: 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 worst case conditions (elevated groundwater and pore water pressures)	1.3
Seismic (ULS) case*	1.0
Note: *Factor of safety < 1.0 acceptable where displacement-based approach is adopted.	

5.2 Changes to the WGA Model

The following changes were made to the WGA groundwater model to assist with our analysis:

- The model size was reduced to focus on the Wetland A area. This improved model computation time significantly with little difference in the modelled water levels in the area of interest.
- The boundary conditions were altered slightly to include a fixed water pressure head of RL38m beneath the silt layer to more accurately represent the pore-water pressures around the toe of the batter slope.

These changes resulted in similar modelled water levels to WGA.

5.3 Analysis

Slope stability analyses were undertaken using the Morgenstern-Price method of slices under both circular and translational failure mechanisms using the proprietary software Slide2. Groundwater pressures were

¹ CoLab, (October 2025). Regional Infrastructure Technical Specification (RITS) V2, Table 16: Proposed factors for safety for different slope stability scenarios

calculated using the Slide2 finite element groundwater module and subsequently used for stability assessment. Analyses were completed for a various conditions stated in **Table 1** based on the following:

- Static long-term conditions were analysed for the prevailing case with the groundwater table based on the modelled groundwater table.
- When analysing transient conditions, the water pressure heads on the model boundary conditions were increased by 0.5m and a vertical infiltration provided by WGA was introduced to the model.
- Seismic displacements were estimated based on a Newmark Sliding Block approach using 50th percentile correlations published in Ambraseys & Srbulov (1995)² and Jibson (2007)³ for thin, translational rigid sliding block cases, and Bray & Travararou (2007)⁴ and Bray and Macedo (2019)⁵ for flexible sliding mass cases.
- An additional static case was run modelling a 200mm wide gap in the confining silt layer to assess the affect on the hydraulic gradient.
- A 10 kPa dead load was used to model single-story residential dwellings at the crest of the slope.

The following slopes have been assessed:

- Section A: 4.8m deep artificial wetland (Wetland A – Section A2 on Maven Associates drawings) embankment with a maximum cut slope of 1V:3H.

5.4 Results

Selected stability printouts are attached in **Appendix C** with the factor of safety for each scenario outlined in **Table 1**

Table 2: Slope Stability Analyses Results

Location	Slope Stability Factor of Safety						Seismic Displacements (mm)	
	Prevailing	Transient	Seismic (ULS)	Prevailing with broken silt (during construction)	Yield Acceleration		Pond Batter	At Crest
					Pond batter	At crest		
Wetland A Section A2	2.8	1.8	<1.0	2.8	0.1	0.2	15 - 20	<5

The results show that the Section A seismic case do not meet the requisite factor of safety, and displacements of 15 to 20mm are estimated for the structures closest to the wetland. Recommendations are discussed in Section 6 below.

² Ambraseys, N. and Srbulov, M. (1995) Earthquake induced displacement of slopes, Journal of Soil Dynamics and Earthquake Engineering, Vol 14

³ Jibson, R.W., (2007) Regression models for estimating coseismic landslide displacement, Journal of Engineering Geology, Vol 91

⁴ Bray, J. D., & Travararou, T. (2007). Simplified procedure for estimating earthquake-induced deviatoric slope displacements. *Journal of geotechnical and geoenvironmental engineering*, 133(4), 381-392.

⁵ Bray, J.D., and Macedo, J. (2019) "Procedure for Estimating Shear-Induced Seismic Slope Displacement for Shallow Crustal Earthquakes," J. of Geotechnical and Geoenvironmental Engineering, ASCE, V. 145(12), doi: 10.1061/(ASCE)GT.1943-5606.0002143.

6.0 RECOMMENDATIONS

The following options have been considered and discussed with the wider project team to mitigate this hazard:

- Lifting the pond base by 1.31m to RL37.61m to leave a minimum of 1.05m overburden material above the confining silt layer. This option is not preferred due to the implications it has on the stormwater design.
- Using deep soil mixing to mix concrete into the soils to seal the aquifer off. Conceptually, this mixed layer would need to be 500mm to 1m thick. This, coupled with the construction challenges, mean this option is not preferred.
- Temporarily depressurisation of the confined aquifer to allow for jet grouting beneath the silt layer to seal it off beneath each wetland. This option would be very expensive.
- Installing sheet piles around the perimeter of the pond to isolate the pond from the surrounding aquifer. This option is discussed further in Section 6.1.1.
- Installing pressure relief valves at regular intervals across the ponds. These are discussed further in Section 6.2.1
- Temporary depressurisation of the confined aquifer through dewatering to allow construction of a continuous drainage blanket along the underside of the wetland, which would then be hydrologically separated by a Geosynthetic Clay Liner. This option is discussed further in Section 6.2.2.

6.1 Preferred Mitigation Option

In discussions with the project team, the following option is considered the most preferred option for mitigating uplift in the proposed artificial wetlands.

6.1.1 Sheet Piles

Sheet piles could be adopted as a permanent groundwater cut-off system around the perimeter of the wetland. The intent of the sheet pile wall would be to hydraulically isolate the wetland excavation from the surrounding confined aquifer, allowing the confining silt layer to be locally penetrated during construction while maintaining control of artesian pressures and reducing the risk of base heave or uplift instability.

The sheet piles would need to extend continuously around the wetland perimeter and be embedded into the underlying Walton Subgroup to provide an effective seepage cut-off. The required embedment depth would need to be confirmed during detailed design, based on the interpreted depth to the Walton Subgroup, design artesian head, seepage gradients, constructability, and the required factor of safety against uplift and piping. Particular attention would be required at wall corners, terminations, service penetrations, outlet structures, and any localised gaps where preferential seepage paths could develop.

Either steel or vinyl sheet piles may be feasible, subject to detailed design and constructability assessment. Steel sheet piles would generally provide higher stiffness and improved driveability where deeper embedment is required, but would need to be designed for durability over the adopted design life, including corrosion allowance and/or protective measures. Vinyl sheet piles may offer durability advantages in a wetland environment, but their ability to achieve the required embedment into the Walton Subgroup would need to be confirmed.

Construction considerations would include potential disturbance or puncturing of the aquitard, the presence of obstructions, installation tolerances, and the need for groundwater monitoring during sheet pile installation and excavation. Monitoring should include piezometric levels inside and outside the sheet pile

perimeter, visual checks for seepage or boiling, and contingency measures if artesian pressures or seepage rates exceed design assumptions.

Additionally, vibration and noise effects will need to be considered during sheet pile installation (which could take several months) as sheet piling may exceed noise limits for neighbouring dwellings. Construction noise management could include; commencing sheet pile installation away from neighbouring buildings, and if noise is excessive, utilising smaller quieter plant to install the sheet piles, or alternatively by adopting another mitigation option that generates less construction noise.

Sheet piles have been adopted in the permanent design in several other projects across Hamilton, including the Whatukooruru Drive Stage 3 bridge, and the Peacocks Waikato River Bridge stormwater outfall. In both these cases, steel sheet piles were designed to withstand corrosion over their 100-year design life. This was achieved by oversizing the sheet pile thickness to allow for corrosion over the design life.

Alternative cutoff wall options could also be considered, such as a secant bored pile wall, a bentonite slurry wall, or a cutter soil mix wall.

6.2 Alternative Mitigation Options

In discussions with the project team, the following options were considered less favourable than the sheet pile option and are presented in this report as documentation of alternatives considered.

6.2.1 Pressure Relief Valve

Pressure relief valves or controlled relief wells through the aquitard could be installed to allow excess porewater pressure in the confined aquifer to dissipate in a controlled manner. The intent of this system would be to reduce the uplift pressure acting on the underside of the aquitard and maintain an acceptable factor of safety against base heave and uplift instability.

The pressure relief system would likely comprise a series of small-diameter wells or valves installed within the base of the wetland, extending through the aquitard and into the confined aquifer. These would discharge into the wetland or into a controlled drainage layer, subject to civil and stormwater design requirements. The spacing, diameter, discharge capacity, filter requirements, and long-term maintenance provisions would need to be confirmed by detailed hydrogeological design.

This option has the advantage of directly reducing the artesian pressure beneath the wetland, rather than relying solely on additional structural resistance. However, it would introduce a permanent hydraulic connection between the confined aquifer and the wetland.

In artificial wetlands A-C, and possibly D and E in winter conditions, the head pressure in the wells (RL38m) is higher than the outlet levels (presented in Table 3 below), so the water level in these ponds will remain at the low flow outlet level and there will be constant discharge flow using this mitigation method. This will reduce the total stormwater capacity of the artificial wetlands.

Table 3: Wetland Low Flow Outlet Levels provided by Maven Associates

Wetland	Low Flow Outlet (RLm)	Wetland	Low Flow Outlet (RLm)
A	37.77	D	38.1
B	37.53	E	38.13
C	37.91	F	39.37
Note: WGA calculate the head pressure of the confined aquifer to be RL38m, which is higher than the outflows in wetlands A to C, and possibly D and E in extremely wet conditions.			

For this option, the following would need to be considered:

- Confirmation of the design artesian head and required pressure reduction;
- Hydraulic assessment of expected discharge rates;
- Filter design to manage piping/internal erosion risk;
- Assessment of groundwater drawdown effects;
- Allowance for inspection, maintenance, and replacement of valves or wells; and
- Coordination with the hydrologist and stormwater designer to ensure the discharge arrangement is compatible with the wetland design.

6.2.2 Basal Drainage with Geosynthetic Clay Liner (GCL) Separation Layer

A basal drainage system could be constructed beneath the wetland to provide a permanent controlled drainage pathway for artesian groundwater pressures below the wetland base. This would likely require temporary depressurisation of the confined aquifer during construction to allow excavation, placement of the drainage blanket, and installation of the separation/lining system in a stable condition. Subsoil drains would be installed at regular intervals within the drainage layer to provide outlets for the underlying water pressure.

The drainage layer would need to be designed to collect and convey groundwater pressures to a controlled discharge point without allowing uncontrolled seepage, piping, or uplift of the wetland base. A geosynthetic clay liner could then be installed above the drainage layer to hydraulically separate the wetland from the underlying drainage system and reduce the potential for interaction between stormwater and groundwater.

For this option, the following would need to be considered:

- drainage capacity of the GCL
- discharge arrangements of the subsoil drains
- liner performance for constructability
- potential effects on surrounding groundwater levels and wetland water balance.

Long term, this system would perform similarly to the pressure relief valve option, where the groundwater would need to be discharged to a suitable location such as the wetland, which would reduce capacity.

7.0 CONCLUSION

Several options are capable of addressing the potential instability risk for the proposed stormwater ponds from artesian water pressures in the underlying confined aquifer below the subject site. Advantages and disadvantages for the three most preferable options are summarised below:

7.1 Sheet Piles

This option has the advantage of providing a passive, permanent cut-off system without relying on ongoing pressure relief infrastructure. However, it may introduce higher capital cost, constructability risk, and detailing complexity at penetrations and interfaces with the stormwater design.

7.2 Pressure Relief Valve

This option is likely the most economical solution of those presented as it avoids the need to reinforce the walls or base of the pond, however flows from the wells will require constant monitoring during construction and if the groundwater needs to be raised or lowered during construction it will need to be carried out slowly to maintain pond stability. The ongoing pressure relief into the wetland would reduce the stormwater capacity of the ponds.

7.3 Basal Drainage with GCL Layer

This option has the advantage of providing pressure relief beneath the wetland without requiring direct discharge through relief valves into the wetland. However, it would require careful detailing of the drainage outlet, filter compatibility, liner continuity, construction sequencing, and long-term inspection and maintenance. This option would also require extensive dewatering during construction, and additional testing such as pumping tests may be required to assess aquifer recharge and overall viability. Like the pressure relief valve option, the water discharged through this option would reduce pond capacity.

7.4 Construction Monitoring

All of these options will require settlement and groundwater level monitoring in the vicinity of the wetlands during construction to ensure stability is maintained. Requirements including alert levels to be specified during detailed design.

For and on behalf of CMW Geosciences

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Authorised by:



David Sullivan
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CMEng, CPEng

Attachments: Appendix A: Development Plans
Appendix B: WGA Piezometer Location Plan
Appendix C: Slope Stability Results



APPENDIX A

Maven Associates Design Drawings

NOTES

- ALL WORKS TO BE IN ACCORDANCE WITH WAIKATO REGIONAL INFRASTRUCTURE TECHNICAL SPECIFICATIONS.
- COORDINATES IN TERMS OF NZ GEODETIC DATUM MT EGM 2000. LEVELS IN TERMS OF THE NEW ZEALAND VERTICAL DATUM (NZVD) SHALL BE TO MOTU (1985) CAN BE ACHIEVED BY ADDING 778.1m.
- IT IS THE CONTRACTOR'S RESPONSIBILITY TO LOCATE ALL SERVICES THAT MAY BE AFFECTED BY HIS OPERATIONS.
- ALL CONCRETE SW PREP TO BE INSTALLED IN ACCORDANCE WITH WAIKATO REGIONAL INFRASTRUCTURE TECHNICAL SPECIFICATIONS AND NZS 3101:2009 FOR PRECAST CONCRETE PIPES (PIRE SQUARE AND NON-PRECAST AS SUPPLIED IN THE BILLS OF MATERIALS).
- APPROVED HARDFILL IS TO BE USED IN BACKFILLING OF ALL STORMWATER LINES WITHIN THE ROAD FOREBAY.
- HEAVY DUTY MANHOLELS AND FRAMES TO BE USED IN TRAFFICED AREAS.
- ALL CESSPIT LIDS SHALL HAVE MIN COVER 0.8m. ALL LINES ARE TO BE CLASS A R400 UNLESS SHOWN OTHERWISE.
- ALL LINES TO BE ABANDONED SHALL BE SEARATED BY ENDING. TIMING OF ALL WORKING TO BE COORDINATED WITH COUNCIL STAFF.

LEGEND

--- EX BOUNDARY

--- PROP BOUNDARY

Rev	Description	By	Date
C	DRAFT	M.A.P.	02/02/20
B	DRAFT	M.A.P.	02/02/20
A	DRAFT	M.A.P.	11/02/20

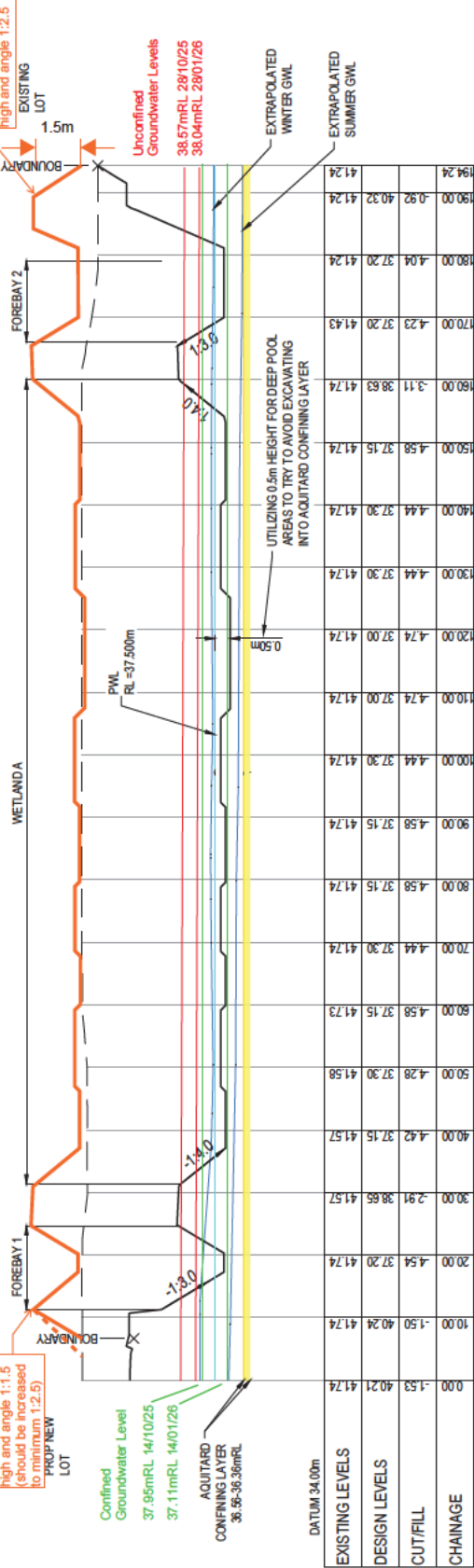
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Design	07/02/20
Drawn	11/02/20
Checked	11/02/20

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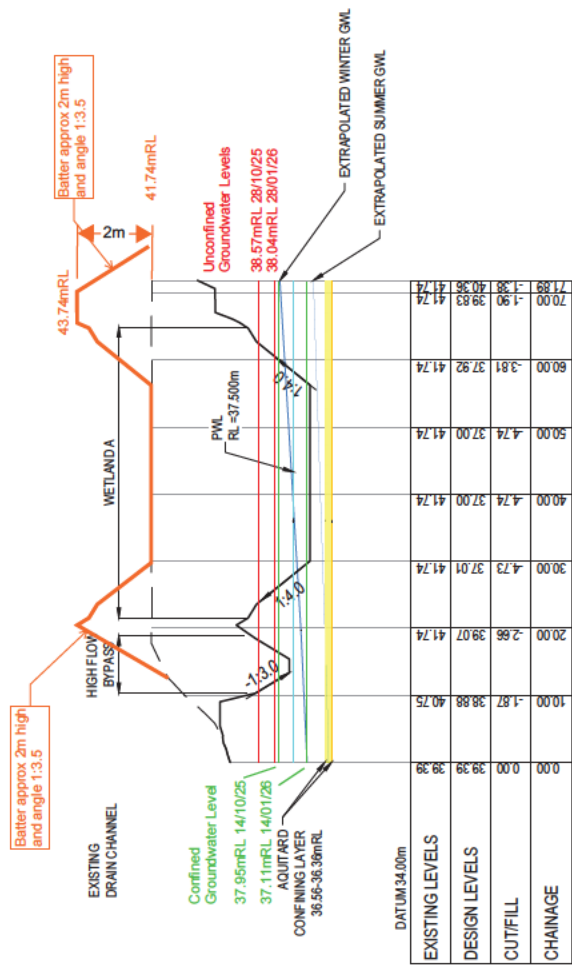
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Batter approx 1.5m high and angle 1:1.5 (should be increased to minimum 1:2.5) PROPOSED LOT

Confined Groundwater Level
37.95mRL 14/10/25
37.11mRL 14/01/26
AQUITARD CONFINING LAYER
36.56-36.36mRL



PROPOSED - Section - A1 CH 0+00.00 TO CH 1+94.24
SCALE: 1:750 H / 1:150 V



PROPOSED - Section - A2 CH 0+00.00 TO CH 0+71.89
SCALE: 1:750 H / 1:150 V

Option 1: Raised SW Bunds
CMW draft markup for discussion
(shown in orange)

NOTES

- ALL WORKS TO BE IN ACCORDANCE WITH WAIKATO REGIONAL INFRASTRUCTURE TECHNICAL SPECIFICATIONS.
- COORDINATES IN TERMS OF NZ GEODETIC DATUM MT ED88 2000. LEVELS IN TERMS OF THE NEW ZEALAND MEAN SEA LEVEL DATUM 1984 TO WHICH THE 1984 DATUM CAN BE ADJUSTED BY ADDING 77.0m.
- IT IS THE CONTRACTOR'S RESPONSIBILITY TO LOCATE ALL SERVICES THAT MAY BE AFFECTED BY HIS OPERATIONS.
- ALL CONCRETE SW PIPE TO BE INSTALLED IN ACCORDANCE WITH WAIKATO REGIONAL INFRASTRUCTURE TECHNICAL SPECIFICATIONS FOR PRECAST CONCRETE AND AS 3600:2007 FOR PRECAST CONCRETE PIPES (PRE-SHAPE AND NON-PRE-SHAPE) AS 8 PULLED IN THE RITS 2016.
- APPROVED HARDFILL IS TO BE USED IN BACKFILLING OF ALL STORMWATER LINES WITHIN THE ROAD.
- HEAVY DUTY MANHOLEDS AND FRAMES TO BE USED IN TRAFFICED AREAS.
- ALL CESSPIT LEADS SHALL HAVE MIN COVER 0.9m. ALL LINES ARE TO BE CLASS 4 R UNLESS SHOWN OTHERWISE.
- ALL LINES TO BE ADJACENT SHALL BE SEALED AT EACH END. TIMING OF ALL WORKS TO BE CO-ORDINATED WITH COUNCIL STAFF.

Legend

- EX GROUND LEVEL
- PROP GROUND LEVEL
- 100YR - 10YR LEVEL
- GROUNDWATER LEVEL

C	DRAFT	M/A	02/2026
B	DRAFT	M/A	02/2026
A	DRAFT	M/A	11/2025

Rev	Description	By	Date
1	Survey	WVH	09/2025
2	Design	WKS	07/2025
3	Drawn	M/A	11/2025
4	Checked	DM	11/2025

Maven Waikato
07 342 0001
maven@waikato.govt.nz
Level 1, 130 Years Road, Hamilton
3900 130 Years Road

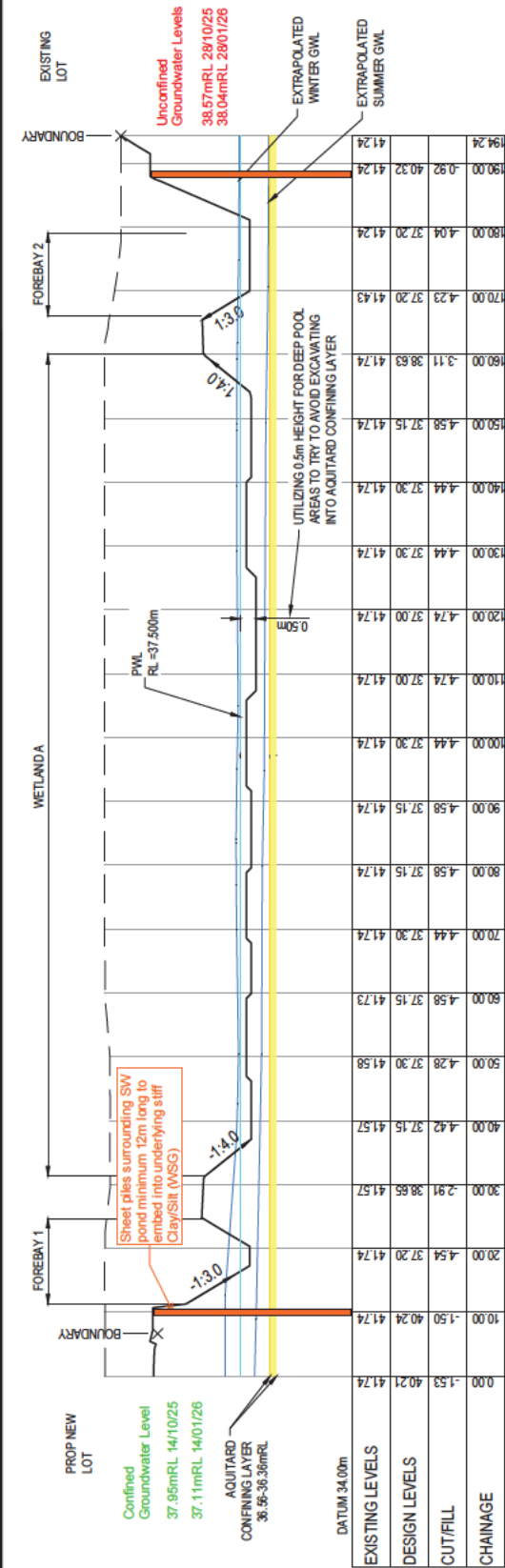
Project
**SOUTHERN LINKS 1
RESIDENTIAL STAGE 1
HAMILTON, WAIKATO
FOR
SL1 CONSORTIUM GROUP**

Title
**PROPOSED STORMWATER
WETLAND A
SECTIONS**

Project no.	298001-RT SL1
Scale	AS SHOWN
Client	298001-RT-C440 SW WETLAND/DWG
Drawing no.	RT-C440-2
Rev	C

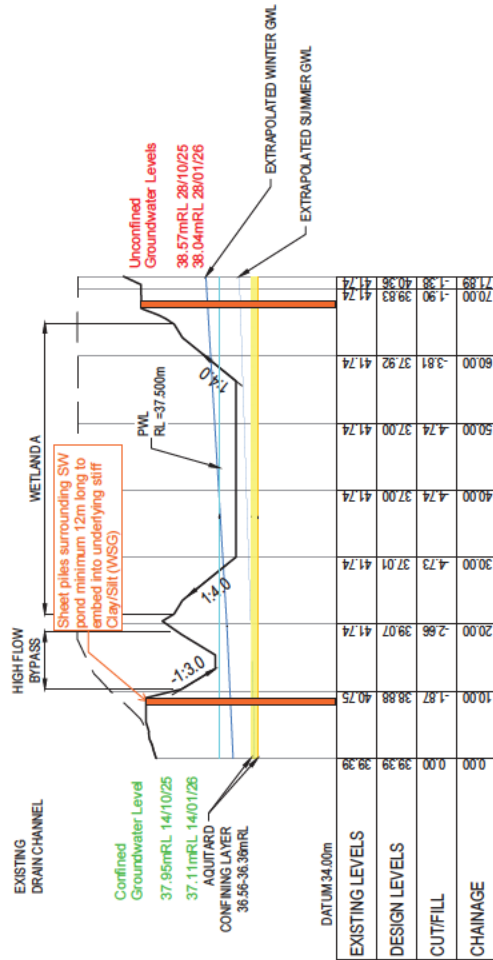
DRAFT FOR REVIEW

NOTES	1. ALL WORKS TO BE IN ACCORDANCE WITH WAIKATO REGIONAL INFRASTRUCTURE TECHNICAL SPECIFICATIONS.
	2. CO-ORDINATES IN TERMS OF NZ GEODETIC DATUM MTDEN 2000. LEVELS IN TERMS OF THE NEW ZEALAND MEAN SEA LEVEL DATUM (NZMSL) TO MTDEN 1980 CAN BE ACHIEVED BY ADDING 778m.
	3. IT IS THE CONTRACTOR'S RESPONSIBILITY TO LOCATE ALL SERVICES THAT MAY BE AFFECTED BY HIS OPERATIONS.
	4. ALL CONCRETE SW PREP TO BE INSTALLED IN ACCORDANCE WITH NZS 3101:2007 FOR PRECAST CONCRETE PILES (PRE-SHAPE AND NON-PRE-SHAPE) AS SUPPLIED IN THE RITS 2016. APPROVED HARDFOILL IS TO BE USED IN BACKFILLING OF ALL STORMWATER LINES WITHIN THE ROAD.
	5. HEAVY DUTY MANHOLEDS AND FRAMES TO BE USED IN TRAFFICED AREAS.
	7. ALL ACCESS POINT LEADS SHALL HAVE MIN COVER 0.9m. ALL LINES ARE TO BE CLASS 4 ROR UNLESS SHOWN OTHERWISE.
	8. ALL LINES TO BE ADOPTED SHALL BE SEALED AT BACKEND. TIMING OF ALL SERVICES TO BE CO-ORDINATED WITH COUNCIL STAFF.
Legend	EX GROUND LEVEL PROP GROUND LEVEL 100YR - 10YR LEVEL GROUNDWATER LEVEL
C	DRAFT M.A.R. 02/2026
B	DRAFT M.A.R. 02/2026
A	DRAFT M.A.R. 11/2025
Rev	Description By Date
Survey	By WWHN 09/2025
Design	By MKS 07/2025
Drawn	By M.A.R. 11/2025
Checked	By J.M. 11/2025
 Maven Waikato 07 342 0001 maven@waikato.co.nz Level 1, 130 Years Road, Hamilton 390001-RT-SL1	
SOUTHERN LINKS 1 RESIDENTIAL STAGE 1 HAMILTON, WAIKATO FOR SL1 CONSORTIUM GROUP	
PROPOSED STORMWATER WETLAND A SECTIONS	
Project no.	298001-RT-SL1
Scale	AS SHOWN
Cell file	298001-RT-SL1-SW WETLAND.DWG
Drawing no.	RT-C440-2
Rev	C



PROPOSED - Section - A1 CH 0+00.00 TO CH 1+94.24
SCALE: 1:750 H / 1:150 V

ROAD 13



PROPOSED - Section - A2 CH 0+00.00 TO CH 0+71.89
SCALE: 1:750 H / 1:150 V

Option 3: Sheet Piles
CMW draft markup for discussion
(shown in orange)

DRAFT FOR REVIEW

APPENDIX B

WGA Piezomometer Location Plan

WGA Piezometer – Southern Block Locations

Maven Design Rev A

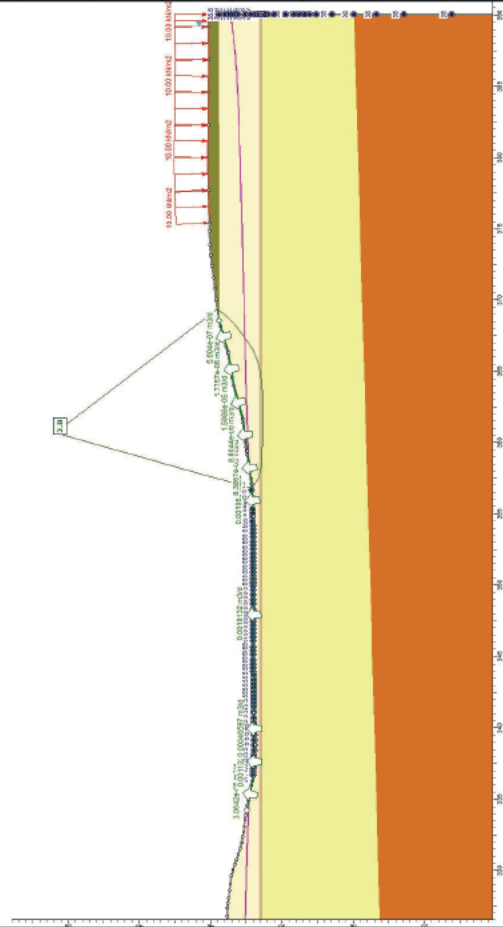


WGA

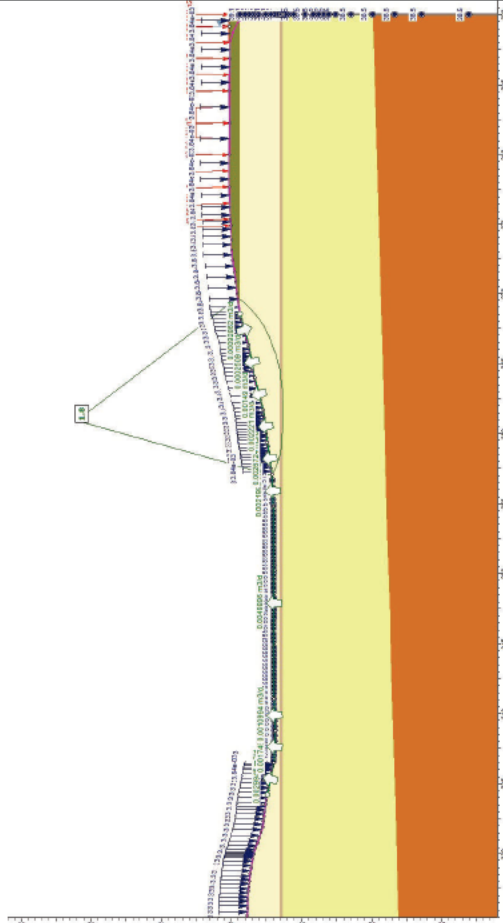
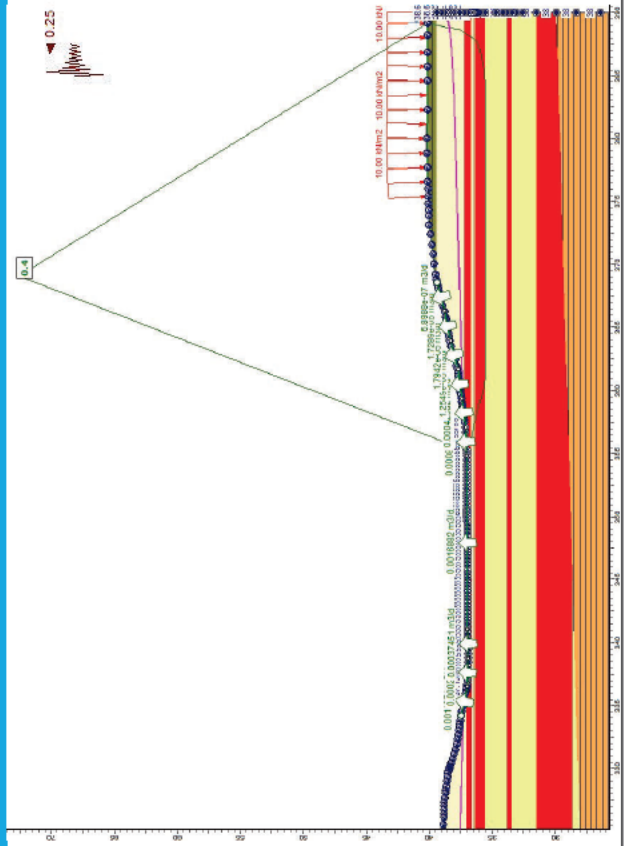
APPENDIX C

Slope Stability Assessment

Proposed Profile - Normal Groundwater Conditions. Failures Filtered to Fo≤1.5



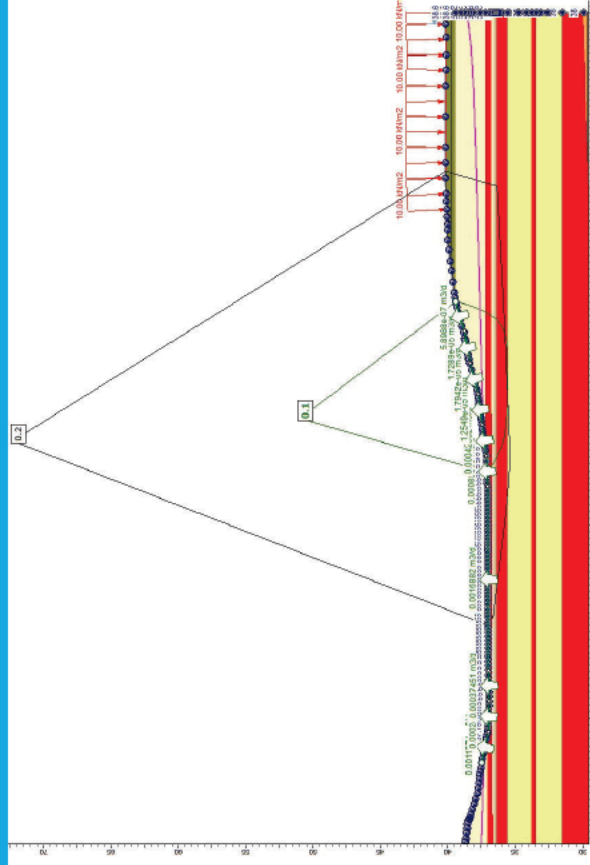
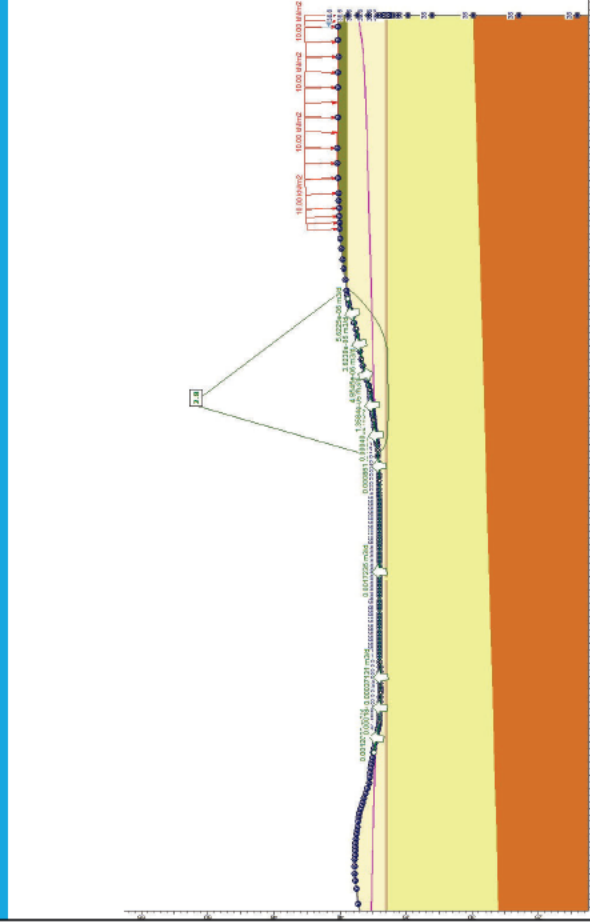
Proposed Profile - Worst Case Groundwater Conditions (+0.5m GWT increase and ground infiltration). Failures Filtered to FoS $\geq$ 1.3

Proposed Profile - ULS Seismic Event. Failures Filtered to $Fo \leq 1.0$ 

Parameters

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Phi b (°)	Air Entry Value (kPa)	Tensile Strength (kPa)
Top Silt		17	Mohr-Coulomb	2	28	0	0	80
Shallow Aquifer		17	Mohr-Coulomb	1	35	0	0	
Aquitard		17	Mohr-Coulomb	2	28	0	0	80
Deep Aquifer		20	Mohr-Coulomb	1	35	0	0	
Walton Clay/ Silt		17	Mohr-Coulomb	5	30	0	0	140

Material Name	Color	Unit Weight [kg/m ³]	Strength [MPa] Moisture Content	Collection Type	PH	Vertical Stress Ratio	Minimum Shear Strength [kPa]	PH	Air Entry Value [kPa]	Penetration Resistance [MPa]
Shallow Aquifer		17	Moisture Content	1	5.8		0	0	0	
Aquifer		17	Moisture Content	2	2.8		0	0	0	80
Deep Aquifer		20	Moisture Content	1	3.9		0	0	0	
Unconsolidated		17	Unconsolidated	50	Constant		0	0	0	80
Wilton Clay/Silt - Undrained		17	Undrained	340	Constant		0	0	0	1400
Shallow Aquifer - Liquefied		17	Vertical Stress Ratio			0.1	5	0	0	
Deep Aquifer -		17	Vertical			0.1	5	0	0	



Parameters

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Phi b (°)	Air Entry Value (kPa)	Tensile Strength (kPa)
Top Silt		17	Mohr-Coulomb	2	28	0	0	80
Shallow Aquifer		17	Mohr-Coulomb	1	35	0	0	
Aquitard		17	Mohr-Coulomb	2	28	0	0	80
Deep Aquifer		20	Mohr-Coulomb	1	35	0	0	
Walton Clay/Silt		17	Mohr-Coulomb	5	30	0	0	140

Material Name	Color	Model	KS (m/s)	K2/K1	K1 Angle (deg)	Soil Type
Top Silt		Simple	1e-07	1	0	General
Shallow Aquifer		Simple	1e-07	1	0	General
Aquitard		Simple	2e-08	1	0	General
Deep Aquifer		Simple	1e-07	1	0	General
Walton Clay/Silt		Simple	1e-08	1	0	General

	CLIENT:	Saxby Road JV	DESIGNER:	PM
	PROJECT:	Mexted Block - Southern Link Development	CHECKED:	DMM
	TITLE:	COSEISMIC DISPLACEMENTS - FLEXIBLE SLIDING MASS	REVISION:	8
			DATE:	7/05/2026
			PROJECT:	HAM2025-0056AK

WHEN TO USE THIS SHEET:
 Use this sheet when calculating the coseismic displacements of a block assuming a flexible sliding mass mechanism.

1. CALCULATE THE PERIOD OF THE SLIDING MASS (T_s)

1D or 2D?	1D	Generally 1D, but see guidance tab for details
H	3.7 m	Maximum height of the sliding mass, measured from within the sliding mass
V _s	150 m/s	Average shear wave velocity of the sliding mass
T _s	0.0987 s	

2. SEISMIC PARAMETERS

Case	Ordinary Ground Motions	Only applicable to Bray and Macedo (2019). Almost always ordinary ground motions, see guidance tab re pulses
Yield Coefficient (a _y)	0.1 g	
Magnitude (M _w)	5.9	
PGA	0.25 g	
PGA	0.25 g	
S _a (1.3T _s)	0.43 g	Required for Bray (2019) and Macedo (2023)
S _a (1.5T _s)	0.43 g	Required for Bray (2007)
PGV	23.33 cm/s	Required for Rathje & Antonakos (2011) and Bray (2019) pulses.
T _m	0.53	Only required for Rathje & Antonakos (2011)
CALS?	☐	Multiplies S _a (1.3T _s), S _a (1.5T _s) and PGA by 1.5, in accordance with BM v3.4

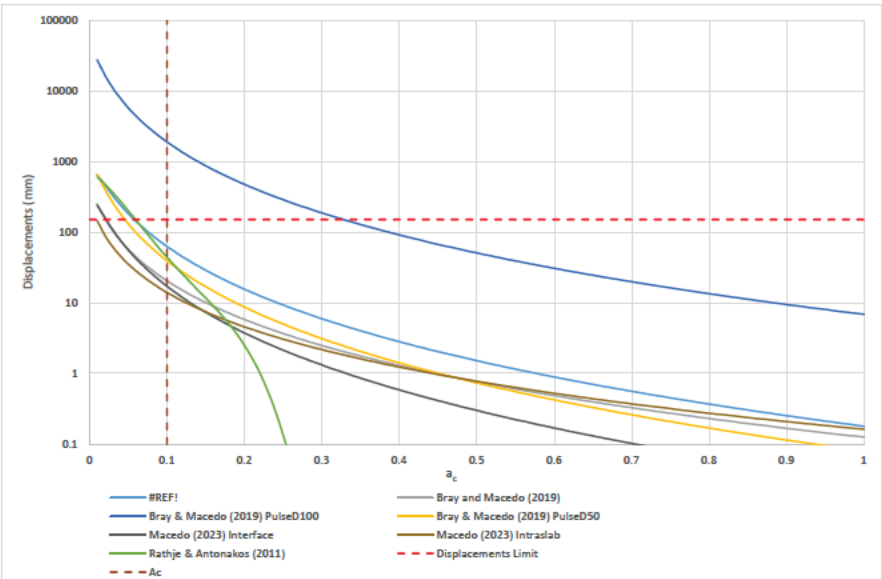
Seismic Demand Parameters:
 Hamilton
 Class D
 1/500

3. RESULTS

Probability of Exceedence:	Bray and Macedo (2019)	Macedo et al. (2023)	Macedo et al. (2023)
Probability of Negligible Displacement	5%	3%	4%
50%	20 mm	17 mm	14 mm
16%	41 mm	37 mm	26 mm
5%	66 mm	60 mm	40 mm
2.5%	83 mm	76 mm	48 mm
0.5%	129 mm	121 mm	71 mm
Applicable Seismic Source	Shallow Crustal	Subduction Interface	Subduction Intraslab

4. CHART OPTIONS

Displacements Limit 150



	CLIENT:	Saxby Road JV	DESIGNER:	PM
	PROJECT:	Mexted Block - Southern Link Development	CHECKED:	DMM
	TITLE:	COSEISMIC DISPLACEMENTS - FLEXIBLE SLIDING MASS	REVISION:	8
			DATE:	7/05/2026
			PROJECT:	HAM2025-0056AK

WHEN TO USE THIS SHEET:

Use this sheet when calculating the coseismic displacements of a block assuming a flexible sliding mass mechanism.

1. CALCULATE THE PERIOD OF THE SLIDING MASS (T_s)

1D or 2D?	1D	Generally 1D, but see guidance tab for details
H	3.7 m	Maximum height of the sliding mass, measured from within the sliding mass
V_s	150 m/s	Average shear wave velocity of the sliding mass
T_s	0.0987 s	

2. SEISMIC PARAMETERS

Case	Ordinary Ground Motions	Only applicable to Bray and Macedo (2019). Almost always ordinary ground motions, see guidance tab re pulses
Yield Coefficient (a_y)	0.2 g	
Magnitude (M_w)	5.9	
PGA	0.25 g	
PGA	0.25 g	
$S_a(1.3T_s)$	0.43 g	Required for Bray (2019) and Macedo (2023)
$S_a(1.5T_s)$	0.43 g	Required for Bray (2007)
PGV	23.33 cm/s	Required for Rathje & Antonakos (2011) and Bray (2019) pulses.
T_m	0.53	Only required for Rathje & Antonakos (2011)
CALS?	☐	Multiplies $S_a(1.3T_s)$, $S_a(1.5T_s)$ and PGA by 1.5, in accordance with BM v3.4

Seismic Demand Parameters:

Hamilton
Class D
1/500

3. RESULTS

Probability of Exceedence:	Bray and Macedo (2019)	Macedo et al. (2023)	Macedo et al. (2023)
Probability of Negligible Displacement	54%	44%	62%
50%	<5 mm	2 mm	<5 mm
16%	8 mm	6 mm	5 mm
5%	14 mm	11 mm	9 mm
2.5%	18 mm	14 mm	12 mm
0.5%	30 mm	23 mm	19 mm
Applicable Seismic Source	Shallow Crustal	Subduction Interface	Subduction Intraslab

4. CHART OPTIONS

Displacements Limit 150

