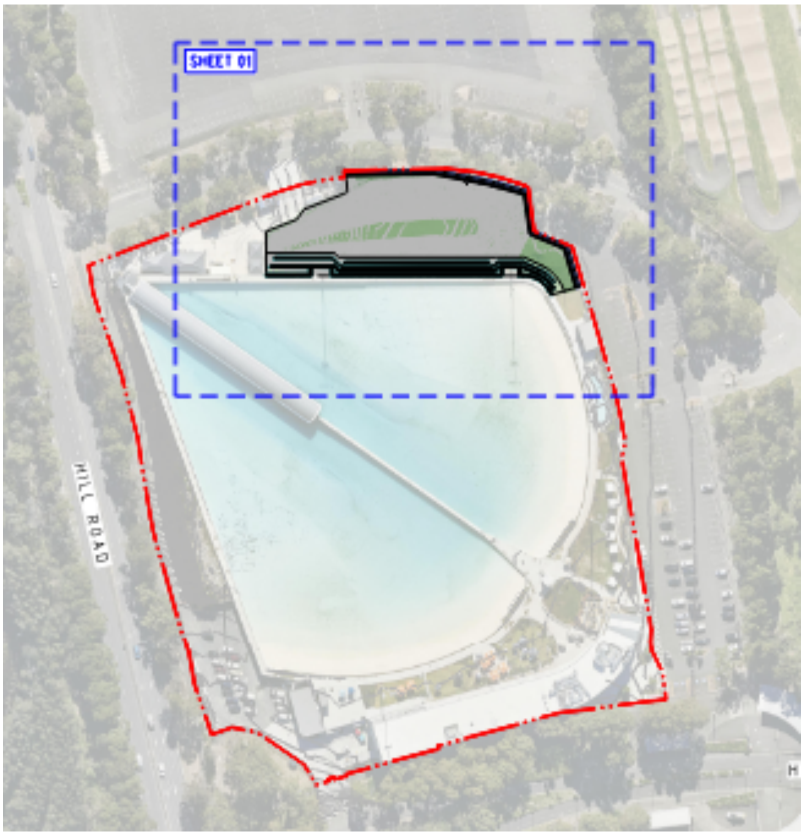
REPORT ON SLOPE STABILITY & SOIL BEARING CAPACITY OF LANDSCAPE MOUND BATTER AT URBNSURF SYDNEY, SYDNEY OLYMPIC PARK, NSW

Client:
URBNSURF GROUP PTY LTD

Project:
Report No. GT1019-05N, Rev. 04

Date:
02 JULY 2026



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To: 02 July 2026

Mr. Jerome DEANE
HEAD OF FACILITIES & ENGINEERING
URBNSURF GROUP PTY LTD
Level 1, 204 Clarence Street
Sydney NSW 2000
Email: jerome@urbnsurf.com
Phone: 0417 323 795

Project: **Slope Stability & Soil Bearing Capacity Assessment of Landscape Mound Batter at Urbnsurf Sydney, Sydney Olympic Park, NSW**

Dear Jerome,

Urbnsurf Group Pty Ltd have engaged Oz Geos (AM & PU PTY LTD), Victoria, to carry out the Slope Stability Assessment & Soil Bearing Capacity of Landscape Mound Batter. The assessment was carried out to estimate the slope stability of the proposed batter and to assess the soil bearing capacity of certain area loaded by Sandstone Blocks.

The report details the assessment and advice on proposed configuration at the above-mentioned project located in Sydney Olympic Park, NSW 2127.

We appreciate the opportunity of providing our services for this project. If you have questions regarding this report or if we may be of further assistance, please contact the undersigned.

Sincerely,

For Oz Geos,

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Director – Operations
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REVISION HISTORY

Project	**GT1019-05N** **Slope Stability & Soil Bearing Capacity Assessment**			
Location	Car Park P5, Cnr Hill Rd and Holker Busway, Sydney Olympic Park, NSW 2127, Australia			
Rev	Description	Date	Prepared by	Approved by
0	Issued to Client	18 FEB 2026	P.U	A.M
1	Issued to Client	20 FEB 2026	P.U	A.M
2	Issued to Client	23 FEB 2026	P.U	A.M
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4	Issued to Client	02 JULY 2026	P.U	A.M

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EXECUTIVE SUMMARY

URBNSURF Group has engaged Oz Geos to conduct Slope Stability and Soil Bearing Capacity Assessment of the Proposed batter located in the Surf Park Complex situated in Sydney Olympic Park, NSW 2127 (the project).

Field works were conducted for this location to identify the geotechnical characteristics in order to ease the analysis process. The present material that formed the slope was the site won material containing asbestos (ACM). The present proposal states that a layer of orange geotextile marker will be placed on top of the site won material, and a capping layer of 500mm thickness will be placed on top of the orange geotextile marker.

The proposed design of this batter was prepared by Northrop Design Consultant. It was proposed that the batter to be constructed by 1(Vertical) : 3(Horizontal). The analyses carried out are to assess the proposed slope inclination, soil bearing capacity due to sandstone block load and the assessment on the proposed Gabion Wall at the toe of the slope.

The slope stability check was carried out using PLAXIS3D Finite Element Code. The results found that the proposed batter was found to be satisfactory and deemed as SAFE for both short- and long-term conditions with the capping layer modified to 350mm thickness and incorporation of 150mm thick top-soil (*Refer to Chapter 8.2*). Oz Geos do not recommend this topsoil layer to be increased, as this will impact the slope stability due to the potential of erosion particularly during heavy rainfall.

The gabion wall design proposal was re-analyzed and found to be safe and acceptable. The proposal of placing the Sandstone blocks on the fill material was analysed and found that the estimated settlement was found to be within tolerable settlement, hence the proposal was acceptable.

The installation of drainage structure such as AGI Pipe (100mm Dia.) connected to the legal point of discharge at the top of the batter embedded into a locally deepened channel of 200mm depth, wrapped with geotextile and filled with free draining aggregate will help in retaining the integrity of this proposed batter.

1. INTRODUCTION

Oz Geos (AM & PU PTY LTD) (OG) is submitting a slope stability and soil bearing capacity report for the landscape mound batter located at the North side of the Surf Park Lagoon. This batter was proposed to be constructed at mainly 1V:3H (Spans from North-South), whereas other section (Sec 01) was proposed to be around 1V:2.5H (Spans from West to East). Certain area at the slope toe at this proposed site, to be retained by the Gabion Walls to create the level differences and also some areas facing the surf lagoon were to be loaded by the blocks of Sandstone in which the soil underneath needs to be assessed in terms of its soil bearing capacity against the load of the Sandstone Blocks. These Sandstone blocks were designed as the tiered seats of the viewers to watch people doing surfing inside the lagoon.

Fig. 01, shows the location of slope/batter analysed. Detailed analysis of this slope/batter is provided in the following sections.

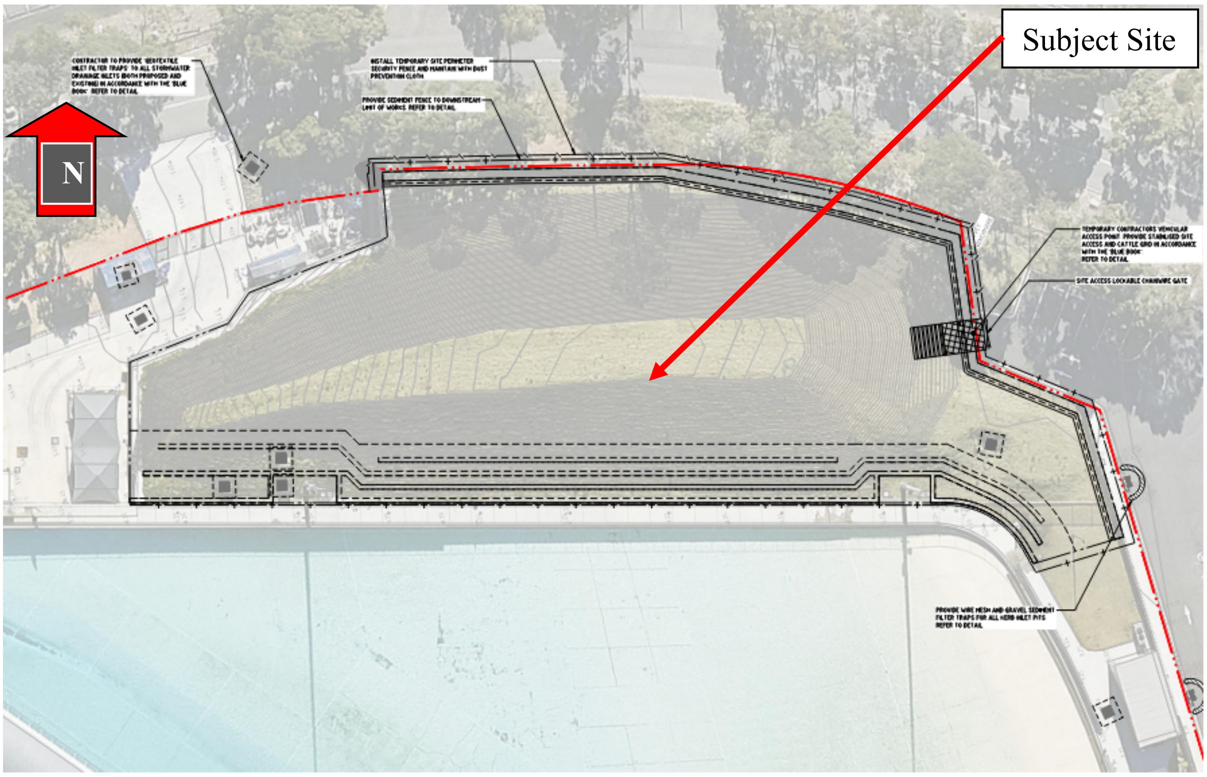

Figure 1. Site plan Showing the Batter Location (NORTHROP, 2026)

2. THE EXISTING SITE CONDITIONS

The proposed site is part of the large car park which was managed by Sydney Olympic Park Authority (SOPA). The Car Park P5 has been allocated by SOPA to be used by USS for the development of an artificial surf park.

3. GEOLOGY

The Geology under the main Lagoon of the Surf Park belongs to the Triassic Age Hawkesbury Sandstone and/or possibly the lower beds of the Ashfield Shale. The site is underlain by fill derived from dredged estuarine sand and mud, demolition rubble, industrial and household waste. The underlying natural undisturbed soils are likely to be Quaternary Age alluvium and estuarine sediments. The alluvium typically comprising silty to peaty quartz sand, silt and clay.

The geology of the crest area of the proposed batter was mainly uncontrolled FILL from the excavation carried out at the lagoon floor formation in the past containing Asbestos Containing Material (ACM). Currently, the existing batter consists of sandy CLAYS from top layer with different consistencies as the layer going into deeper stratum, the very top layer is ranging from firm to stiff, followed by Very Stiff and Hard Sandy CLAYS respectively. At the location of the toe, the top layer was dominated by firm to stiff Sandy CLAYS followed by Silty SAND layer in Medium Dense condition.

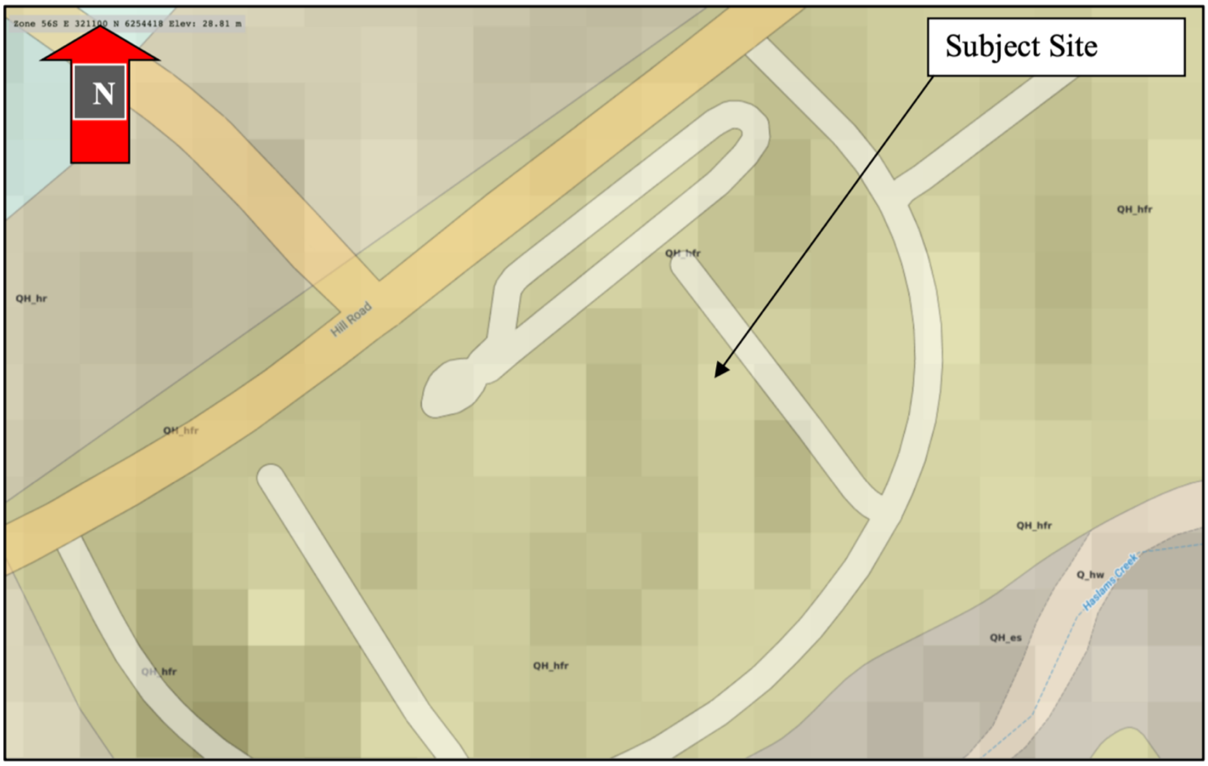

Figure 2. Geological condition of the proposed site (Source: Minview)

4. ENGINEERING PROPERTIES FOR FINITE ELEMENT ANALYSIS

Table 1. Estimated shear strength parameters of the materials at the Slope Crest (Adopted from Table 2 of Oz Geos Report) (Oz Geos, 2026)

Table 2. Inferred Geotechnical Design Parameters for in situ ground conditions at TOE of the Events Hill (*Based on BH1 to BH4 and BH8 to BH10*)

Depth m, bgl	Layer Description	Soil Parameters								
		ϕ' (degrees)	c' (kPa)	S_u (kPa)	γ (kN/m^3)	E_v (MPa)	E'_v (MPa)	E_h (MPa)	E'_h (MPa)	ν
0.2–1.0	Sandy CLAY – CI FILL (Firm to stiff)	26	1	50	19	16	4.8	7.15	3.65	0.3
1.0–2.0	Sandy CLAY – CI FILL (Very Stiff)	28	5	150	20	32	9.5	11	7.25	0.3

Notes: *ϕ'= Effective friction angle; c' = drained shear strength; S_u = undrained shear strength.*
γ = Bulk density; Ev = Modulus of Elasticity under axial loading; E'v = Effective Modulus of Elasticity under axial loading, E_h = Modulus of Elasticity under lateral loading, E_h = Modulus of Elasticity under lateral loading, E'_h = Effective Modulus of Elasticity under lateral loading and ν = Poisson's Ratio

Table 2. Estimated shear strength parameters of the materials at the Toe of the Slope (Adopted from Table 3 of Oz Geos Report) (Oz Geos, 2026)

Table 3. Inferred Geotechnical Design Parameters for in situ ground conditions at CREST of the Events Hill (*Based on BH5, BH6 and BH7*)

Depth m, bgl	Layer Description	Soil Parameters								
		ϕ' (degrees)	c' (kPa)	S_u (kPa)	γ (kN/m³)	E_v (MPa)	E'_v (MPa)	E_h (MPa)	E'_h (MPa)	ν
0.2–1.3	Sandy CLAY – CI FILL (Firm to stiff)	26	1	50	19	16	4.8	7.15	3.65	0.3
1.3–3.0	Sandy CLAY – CI FILL (Very Stiff)	28	5	150	20	32	9.5	11	7.25	0.3
3.0–4.0	Sandy CLAY – CI FILL (Hard)	29	10	200	22	64	19	17	14.5	0.3

Notes: *ϕ'= Effective friction angle; c' = drained shear strength; S_u = undrained shear strength. γ = Bulk density; Ev = Modulus of Elasticity under axial loading; E'v = Effective Modulus of Elasticity under axial loading, E_h = Modulus of Elasticity under lateral loading, E_h = Modulus of Elasticity under lateral loading, E'_h = Effective Modulus of Elasticity under lateral loading and ν = Poisson's Ratio*

5. FINITE ELEMENT ANALYSIS FOR THE PROPOSED SLOPE DESIGNS

5.1. Section 1 at Eastern Side - Slope Stability Analysis

In figure 3, at Section 01 there are 2 slopes to be analysed, one section located at the Eastern Side where at the toe, the slope was retained by the Gabion Wall. The proposed batter inclination is 1(Vertical):3(Horizontal). The proposed slope is to be backfilled by the site won material containing asbestos (ACM) as per section in **Fig 4**. The present proposal states that a layer of orange geotextile marker will be placed on top of the site won material and a top garden soil of 500mm thickness (350mm for capping material and 150mm for topsoil) will be placed on top of the orange geotextile marker.

The slope stability analyses were conducted in 2 scenarios simulating the short and long terms conditions, as follows:

1. Short Term Safety Factor (SF) Calculation/Analysis – UNDRAINED ANALYSIS.
2. Long Term Safety Factor (SF) Calculation/Analysis – DRAINED ANALYSIS.

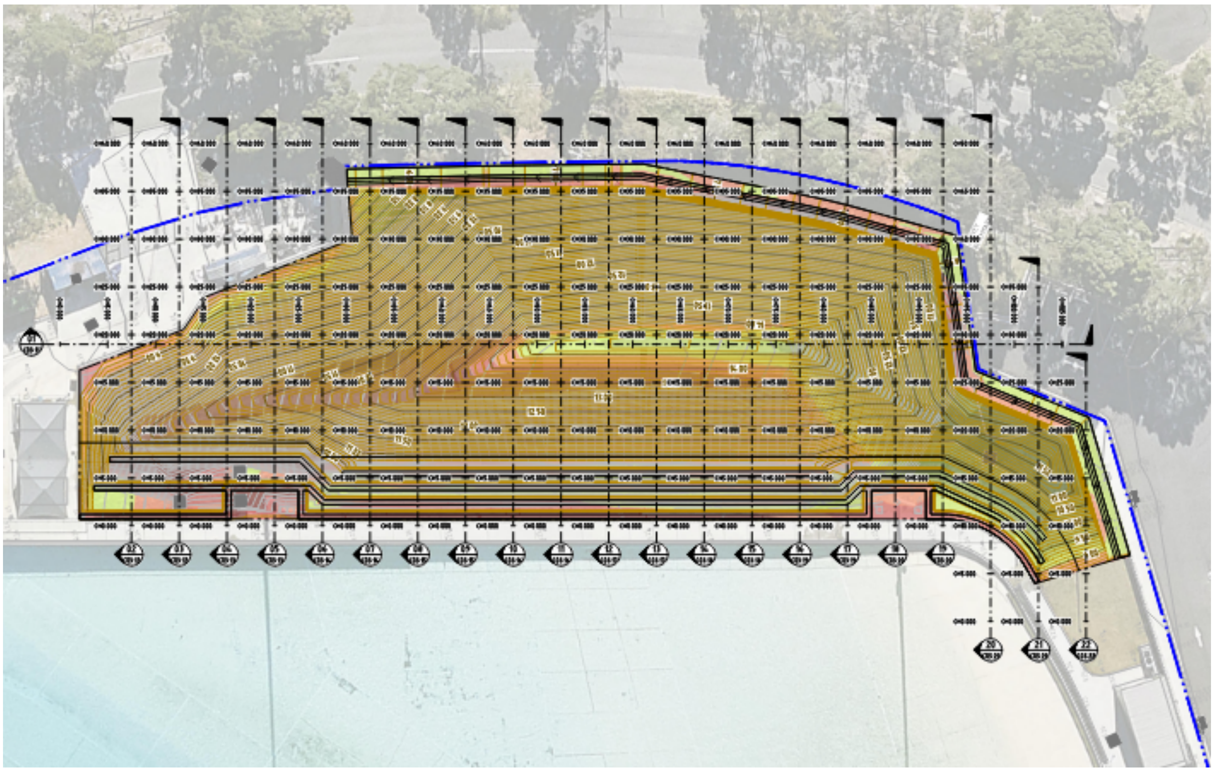

Figure 3. Site Plan Showing Different Sections (NORTHROP, 2026)

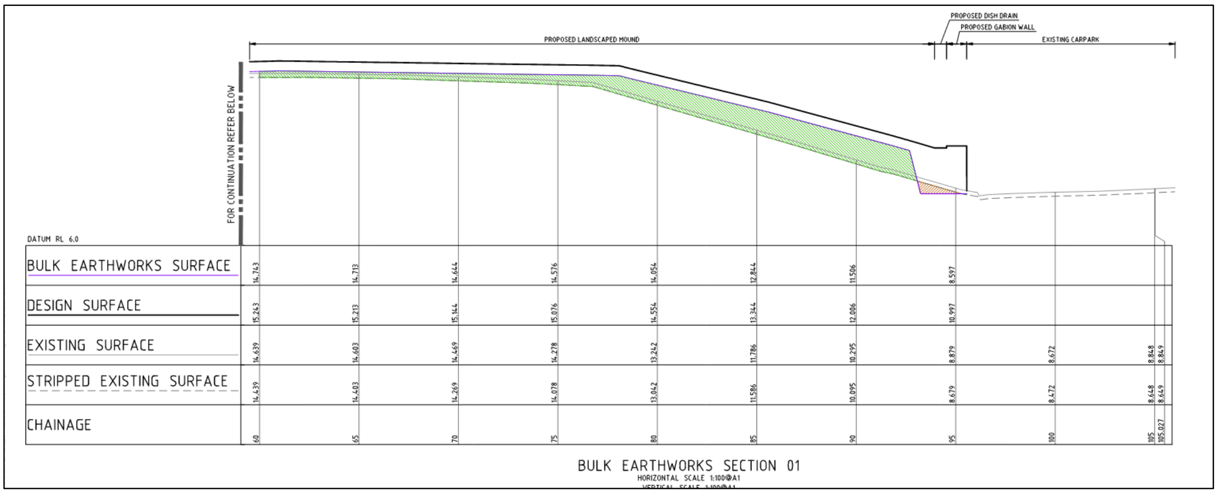

Figure 4. Slope Section of Section-01 At Eastern Side with Toe Retained by Gabion Wall (NORTHROP, 2026)

Table 3. Slope Stability Analysis Results for both short- and long-term conditions (Sec 01 – Eastern Side)

Case	Safety Factor	Remarks
Undrained (Short Term)	4.30	OK!
Drained (Long Term)	1.87	OK!

5.2. Section 1 at Western Side - Slope Stability Analysis

The proposed slope at the location of Western Side consist of a gentler slope in which the sexisting slope would be in cut condition.

The following **Fig 5** depicts the slope section at the Western Side of the mound.

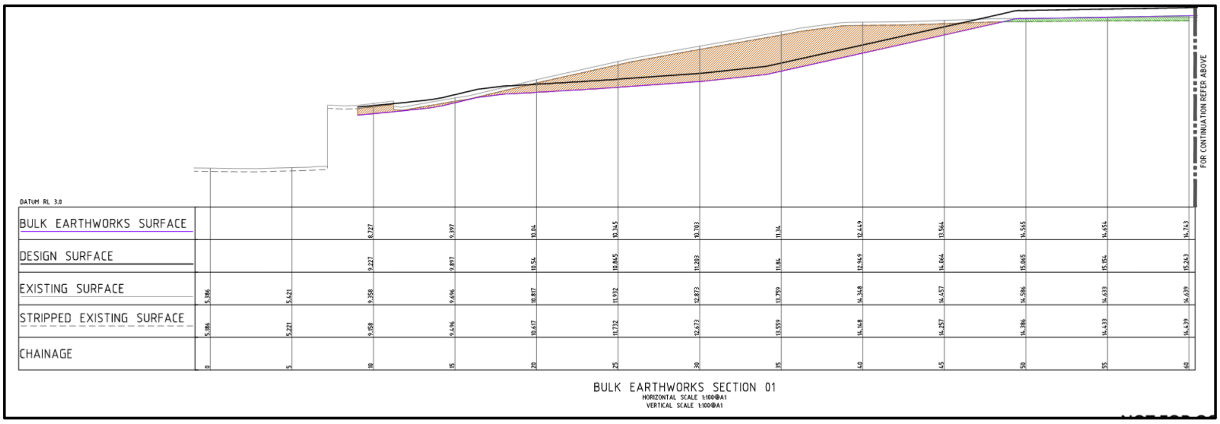

Figure 5. Slope Section of Section-01 At West Side (NORTHROP, 2026)

Table 4. Slope Stability Analysis Results for both short- and long-term conditions (Sec 01 – West Side)

Case	Safety Factor	Remarks
Undrained (Short Term)	7.50	OK!
Drained (Long Term)	3.60	OK!

5.3. Section 09 at Northern Side - Slope Stability Analysis

The batter that spans from North to South has got 2 slopes. One of the extreme one is depicted at Section 09 based on the site plan in **Fig 3**.

The northern side of the mound is shown as per **Fig 6** below.

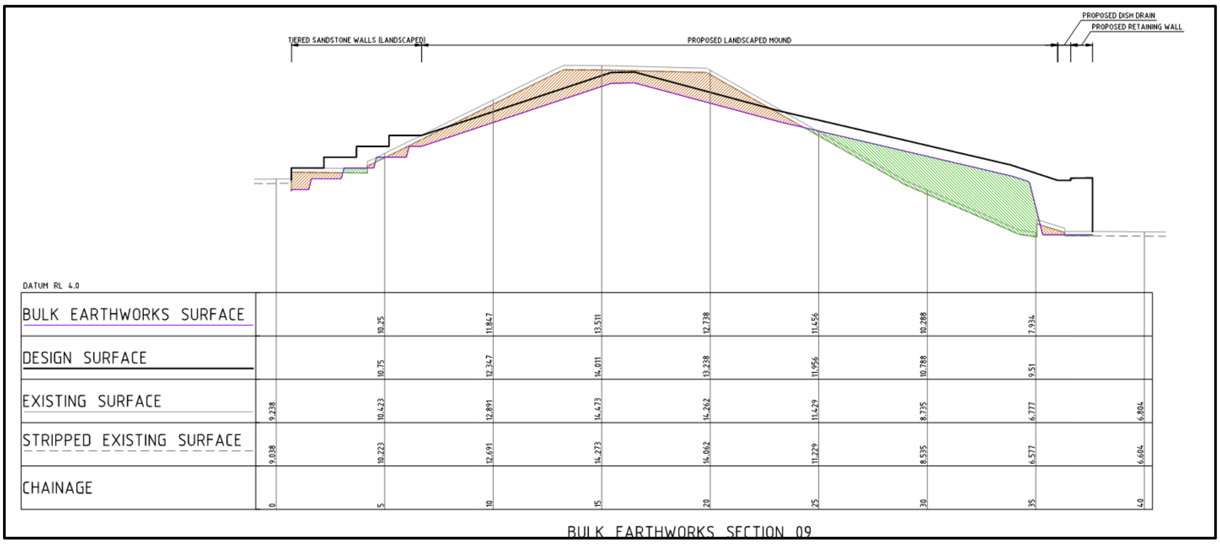

Figure 6. Slope Section of Section-09 Spans from North to South (NORTHROP, 2026)

Table 5. Slope Stability Analysis Results for both short- and long-term conditions (Sec 09 – North Side)

Case	Safety Factor	Remarks
Undrained (Short Term)	6.70	OK!
Drained (Long Term)	2.20	OK!

5.4. Section 09 at Southern Side - Slope Stability Analysis

Table 6. Slope Stability Analysis Results for both short- and long-term conditions (Sec 09 – South Side)

Case	Safety Factor	Remarks
Undrained (Short Term)	9.50	OK!
Drained (Long Term)	2.65	OK!

6. GABION WALL DESIGN PROPOSAL CHECK

The proposed Gabion Wall design is depicted in the following **Fig 7**.

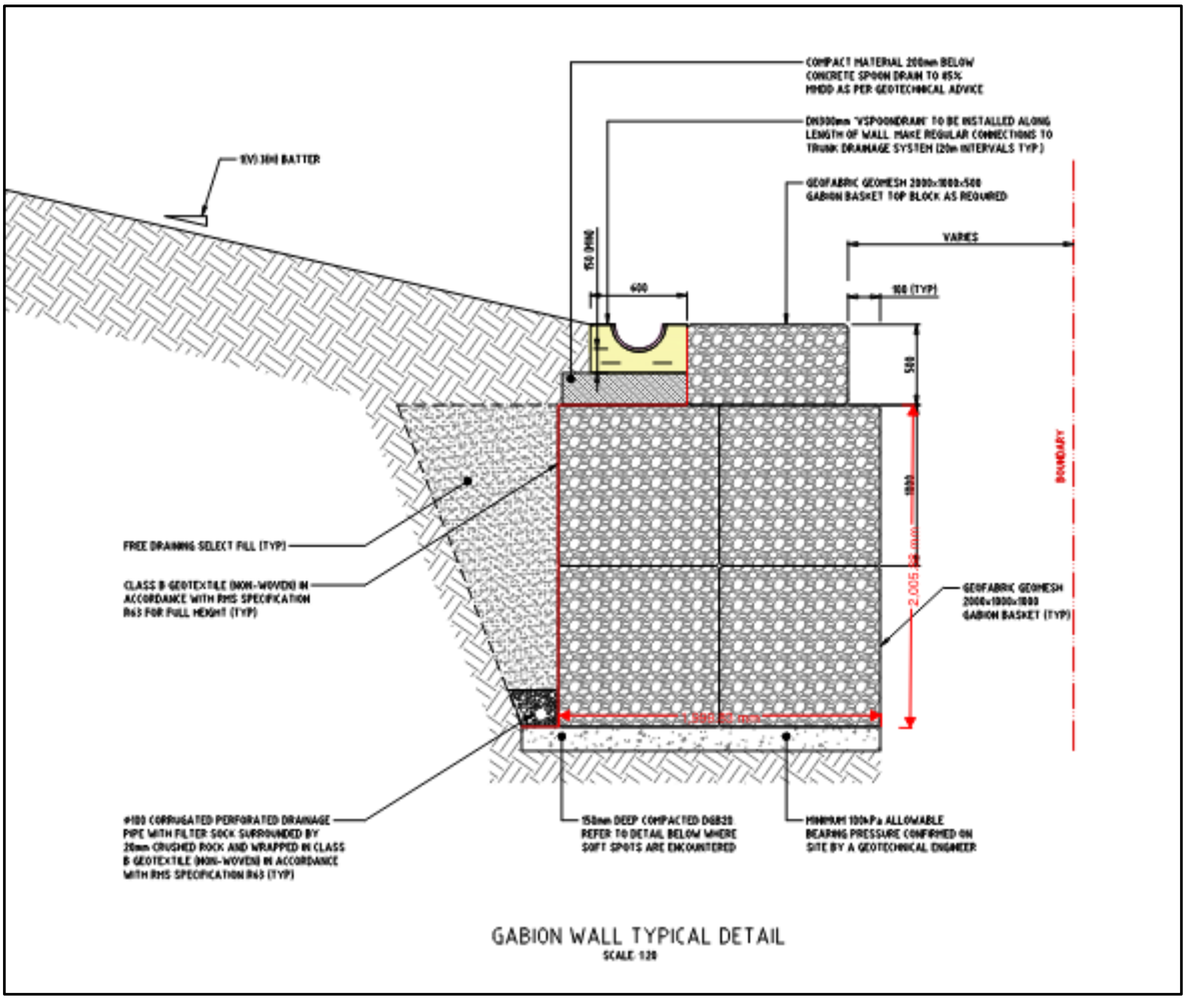

Figure 7. Gabion Wall Proposed Design (NORTHROP, 2026)

Design summary

Combination 1

Action	Resistance	Force	FoS	Allowable FoS	Status
Overturning, sliding and bearing at base level					
Overturning (kNm/m)	87.7	29.7	2.956	1.000	PASS
Sliding (kN/m)	46.1	36.6	1.260	1.000	PASS
Bearing (kN/m²)	100.0	56.9	1.758	1.000	PASS
Eccentricity (mm)	Reaction acts within the middle third of base				PASS
Overturning and sliding between courses 1 and 2					
Overturning (kNm/m)	40.7	5.7	7.144	1.000	PASS
Sliding (kN/m)	26.8	13.1	2.047	1.000	PASS

7. SOIL BEARING CAPACITY CHECK AT THE LOCATION OF SANDSTONE BLOCKS

The Sandstone block is proposed to be seated on the ACM fill, where the consistency is described as Hard silty CLAY. It was analysed using PLAXIS3D and found the settlement is estimated to be 3mm, which is considered safe.

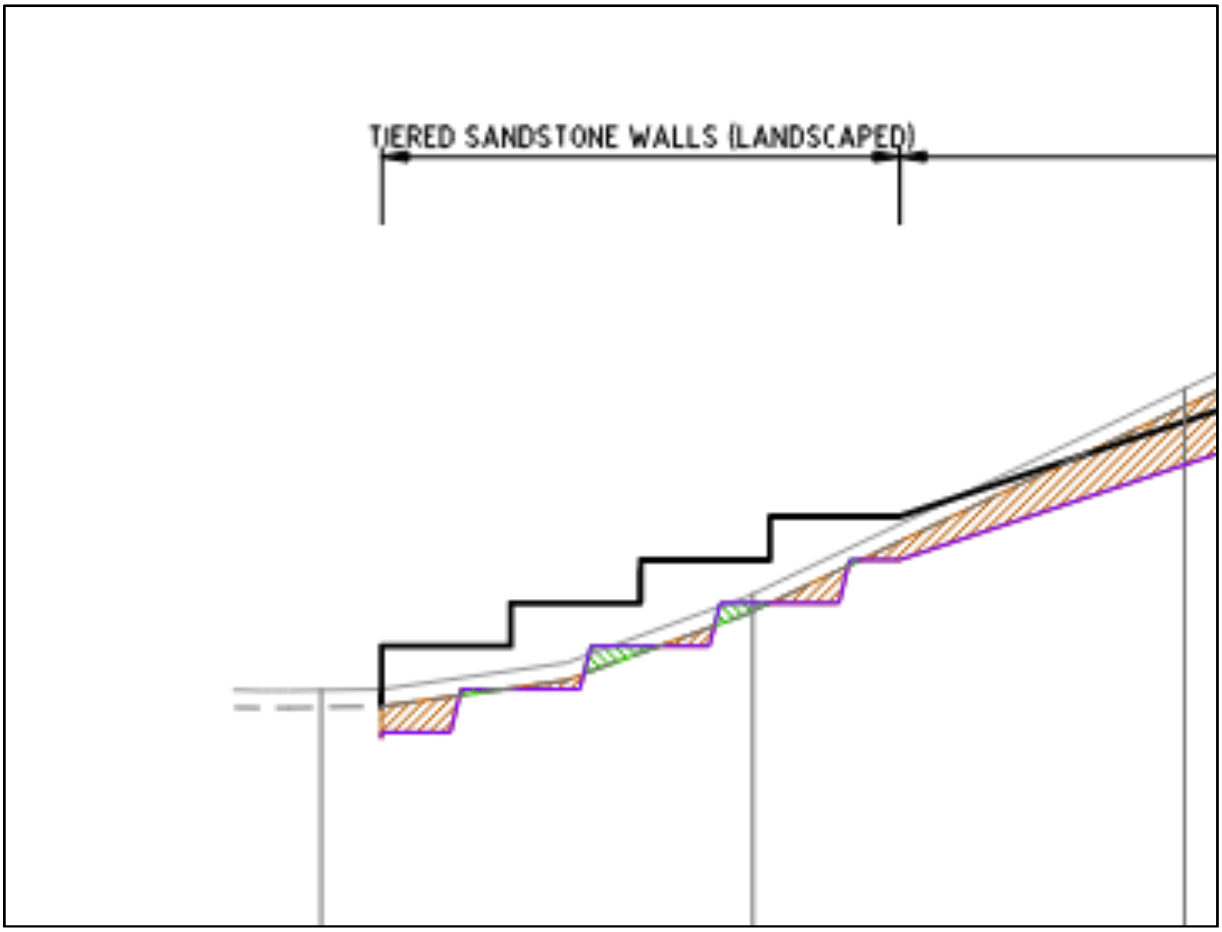

Figure 8. Sandstone Block Seated on the ACM fill (NORTHROP, 2026)

8. CONCLUSIONS AND RECOMMENDATIONS

8.1. CONCLUSION

Based on the analysis performed in this report, the following conclusions are drawn:

1. The slope stability analysis has been carried out for the proposed batter at this particular location. The proposed slope with 1 (vertical) to 3 (horizontal) has been simulated in finite element modelling. The proposed slope geometry is found to be **ACCEPTABLE** and considered to be a **SAFE** slope design for both short- and long-term conditions.
2. The proposed Gabion wall design has been re-analysed and confirmed to be satisfactory.
3. The soil bearing capacity analysis thru settlement analysis against the Sandstone Block placement on the ACM found to be safe with insignificant settlement.

8.2. RECOMMENDATIONS

1. The slope needs to be monitored and prevented from being saturated from the rain water or any source of water. The saturated clay will tend to be weaker and could cause the slope failure. It is recommended to implement the drainage as per the Drainage Plan (*Refer Civil Drawings issued by Northrop (Ref.: SY242357, dated 07 Jan 2026*)) and place AGI Pipe at the crest of the batter and embedded in the clean fill material.
2. The 1V:3H slope should not be loaded with any vehicular loading, other than grass mowing machines also no surcharge load due to vehicle movement is applied on top of the event landscape batter.
3. The entire events landscape to be inspected once a month or after any significant storm event (with rainfall above 20mm). The batter slopes to be inspected for any physical deformation, scouring due to surface run-offs, identification of scarp (erosion) and de-vegetation.
4. The existing Top-Soil can be re-purposed at the proposed events landscape.
5. The Sandstone Blocks and the Gabion retaining wall to be seated on 150mm thick compacted Crushed Rock. Compaction to be at least 98% (Standard compactive effort).
6. The Capping layer make up should comprise of the following:
   a) Place a layer of polypropylene triaxial geogrid above the orange geotextile marker layer.
   b) Lay 150mm of compacted crushed rock (VENM) with 95% standard compaction.
   c) Place a geotextile layer (BIDIM Green A34GA40 or similar).
   d) Place a MAX layer of 200mm thick Sandy, gravelly CLAY (VENM) with 85% standard compaction.
   Note: The above-mentioned layer of crushed rock (VENM) overlain by Sandy, gravelly CLAY (VENM) forms the 350mm thick Capping Layer.
   e) **Place a MAX layer of 150mm thick Top-Soil (VENM)**
   Note: Soil Mix for Top-Soil to be advised by landscaping Consultant.
   f) Place a layer of jute mat or Erosion Control 3D Geomat (preferred)

9. LIMITATIONS AND ADDITIONAL SERVICES

9.1. Report Limitations

The depicted subsurface conditions of this site are solely based on the drilled boreholes which are specific to their locations and provide information about a relatively small column of the soils and the possibility of actual conditions differing must be recognized.

9.2. Additional Services

Variations in soil types and conditions are possible and may be encountered during construction. To permit correlation between the soil data obtained during this investigation and the actual soil conditions encountered during construction, we recommend that Oz Geos to be retained to provide observation and testing services during site earthwork and foundation construction. This will allow us the opportunity to compare actual conditions exposed during construction with those encountered in our investigation and to provide supplementary recommendations if warranted by the exposed conditions. We also provide consultation during construction to assist in the observation of how key parts of the design are implemented, answering questions from the designers or contractors, and looking for subsurface conditions that might differ from the design assumptions or that might require modification of the design.

10. REFERENCES

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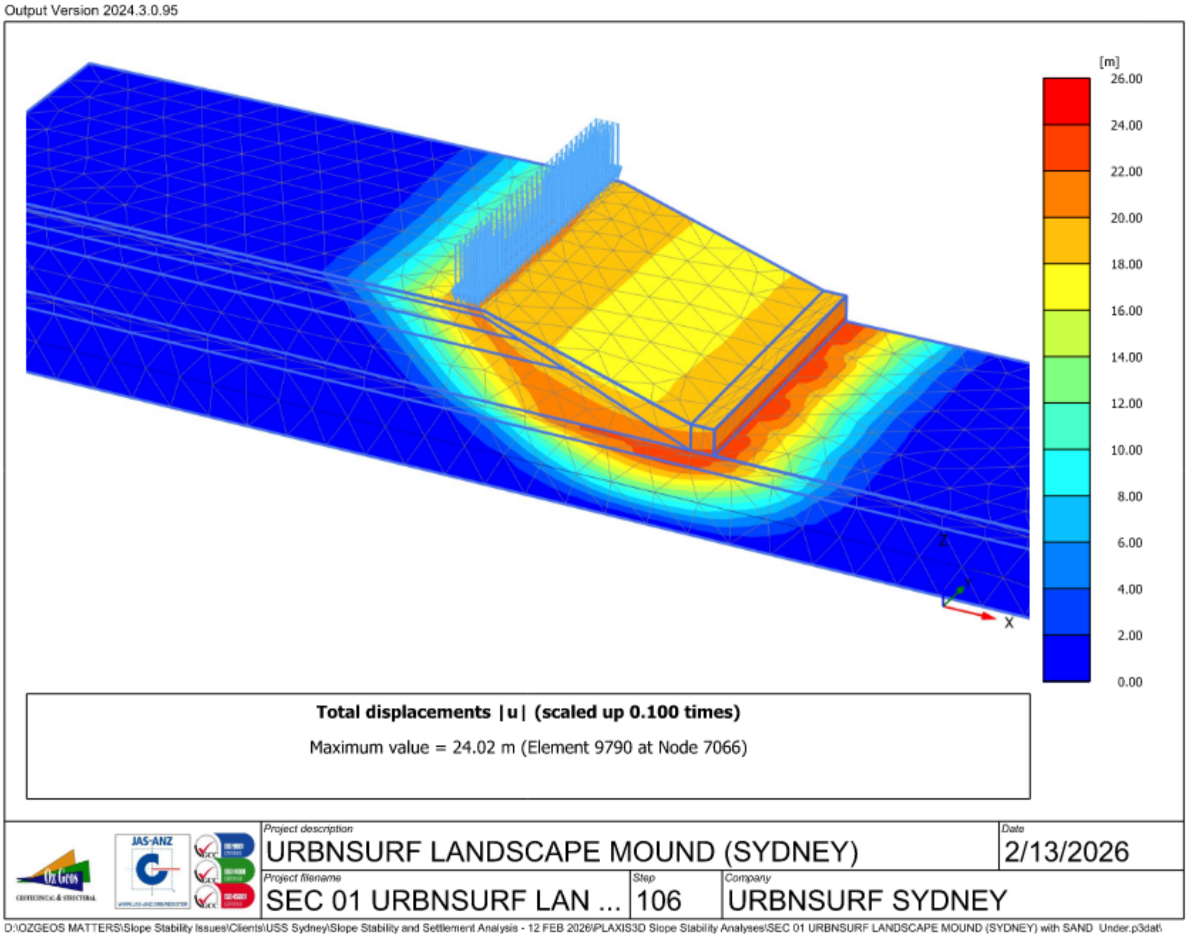
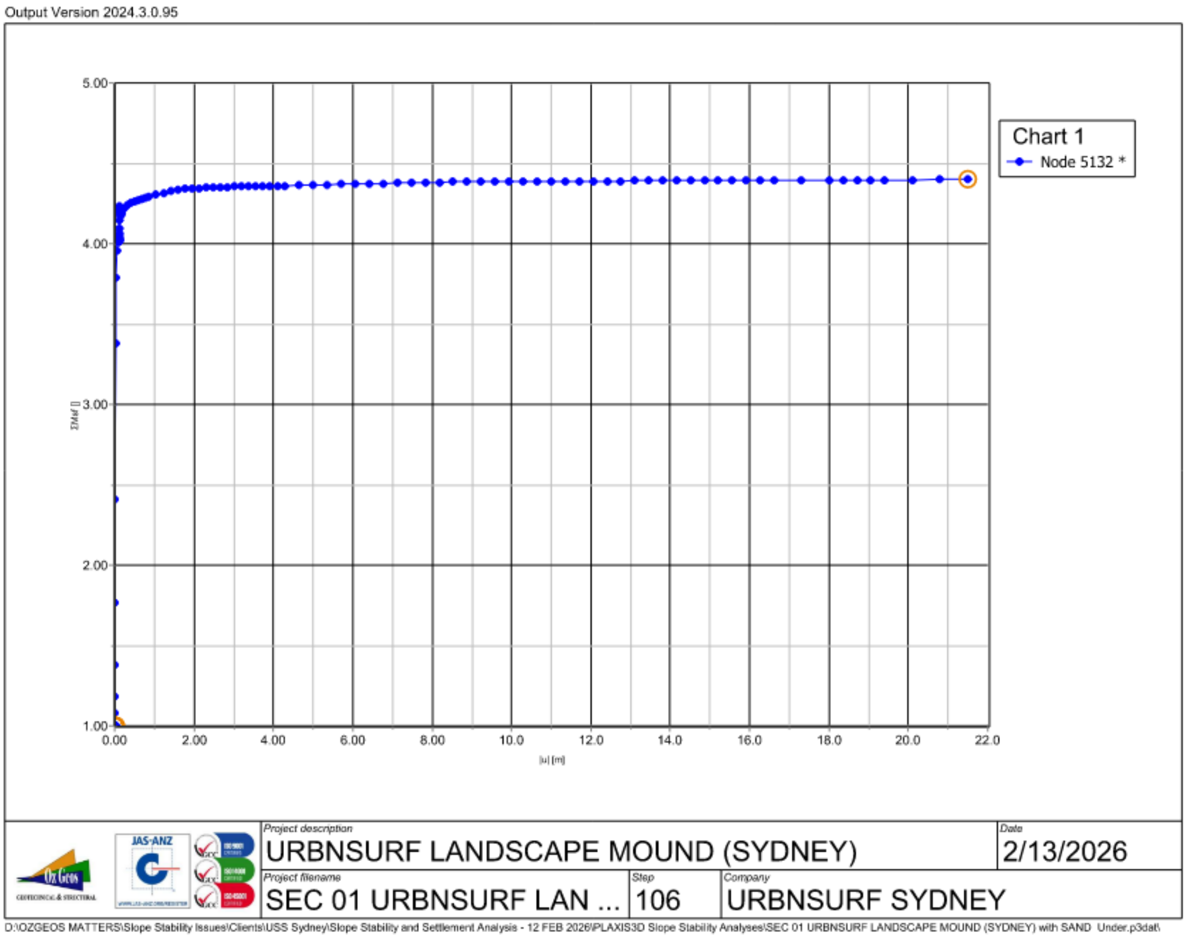
APPENDIX A: FEM FOR UNDRAINED ANALYSIS (Section 01 – Eastern Side)

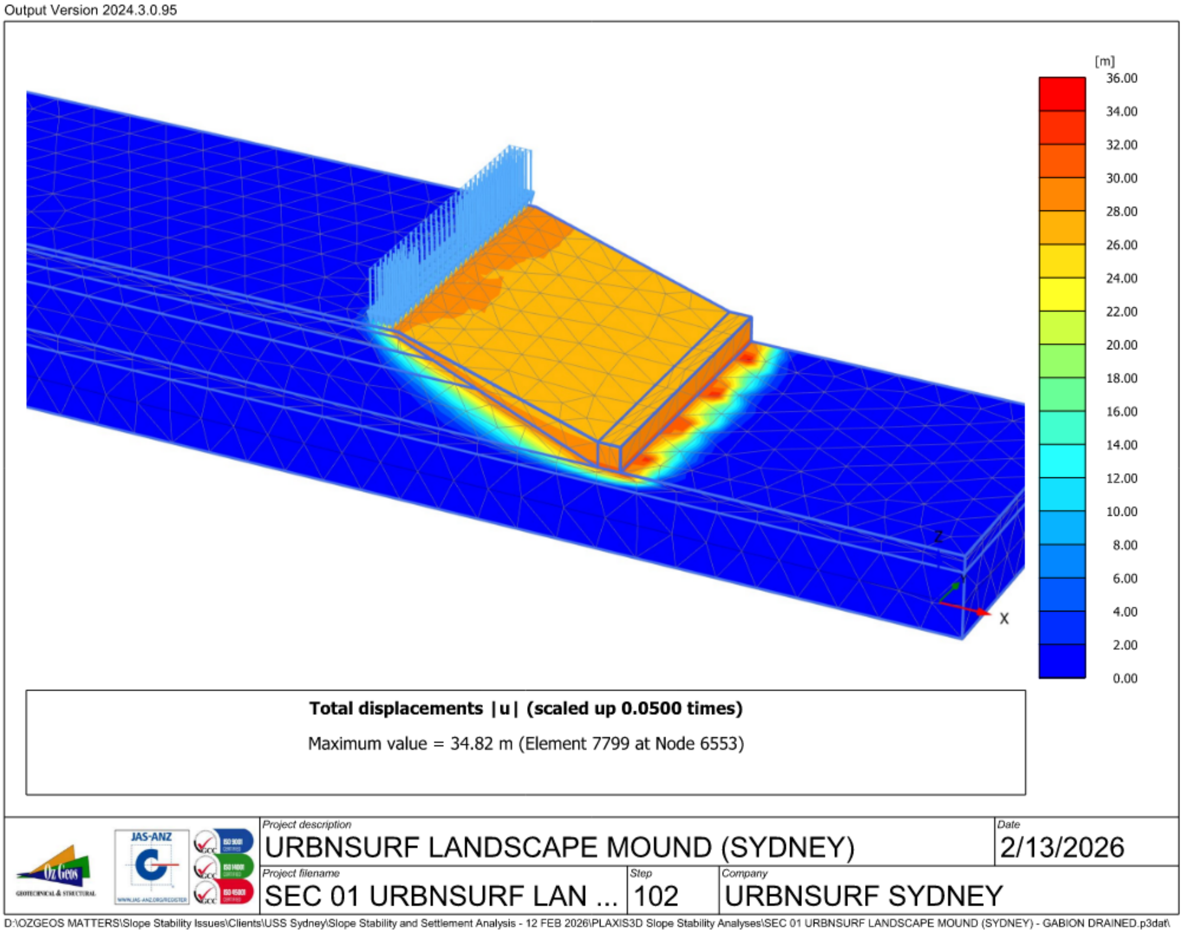


Slope Failure Plane – Sec 01 – Gabion Side - UNDRAINED

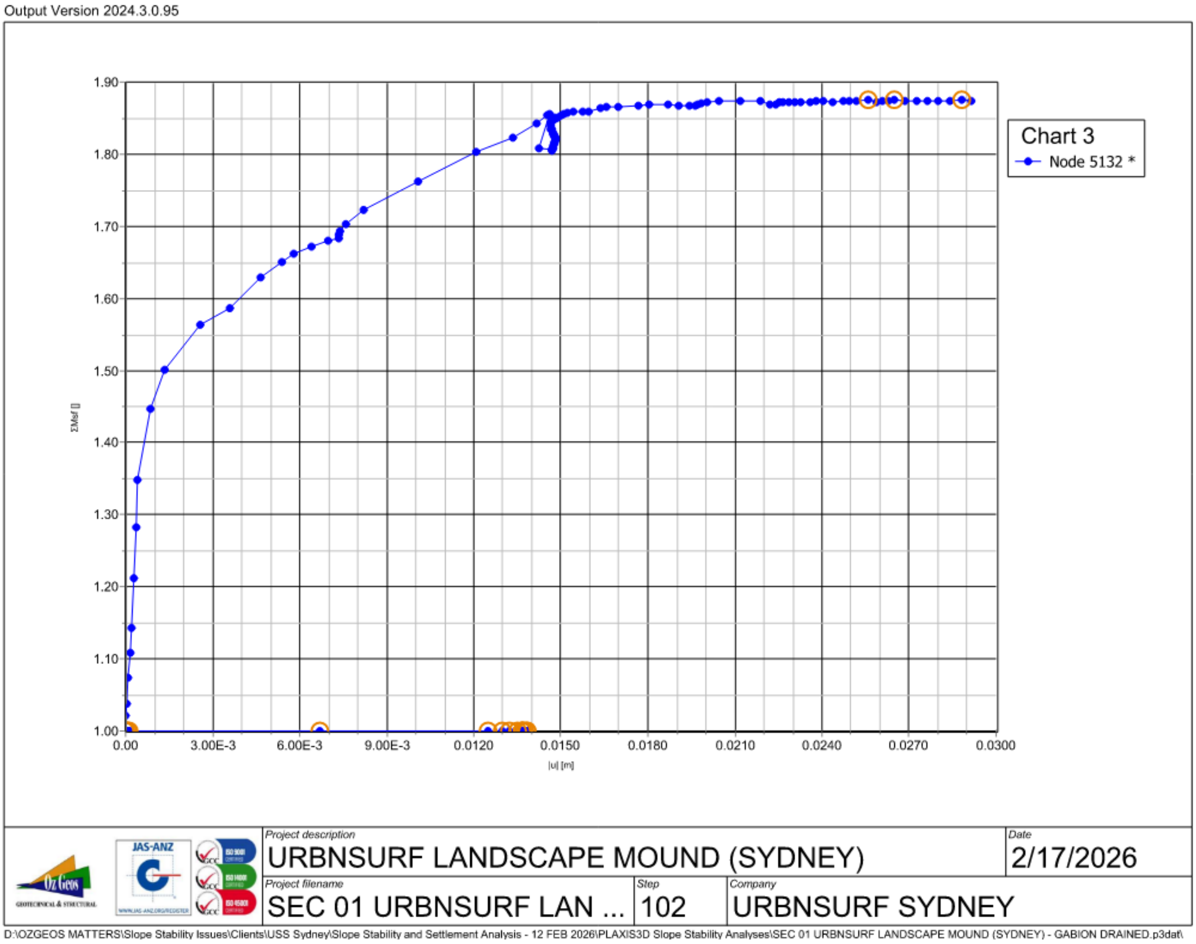


Safety Factor of Sec 01 – Gabion Side - UNDRAINED

APPENDIX B: FEM FOR DRAINED ANALYSIS (Section 01 – Eastern Side)

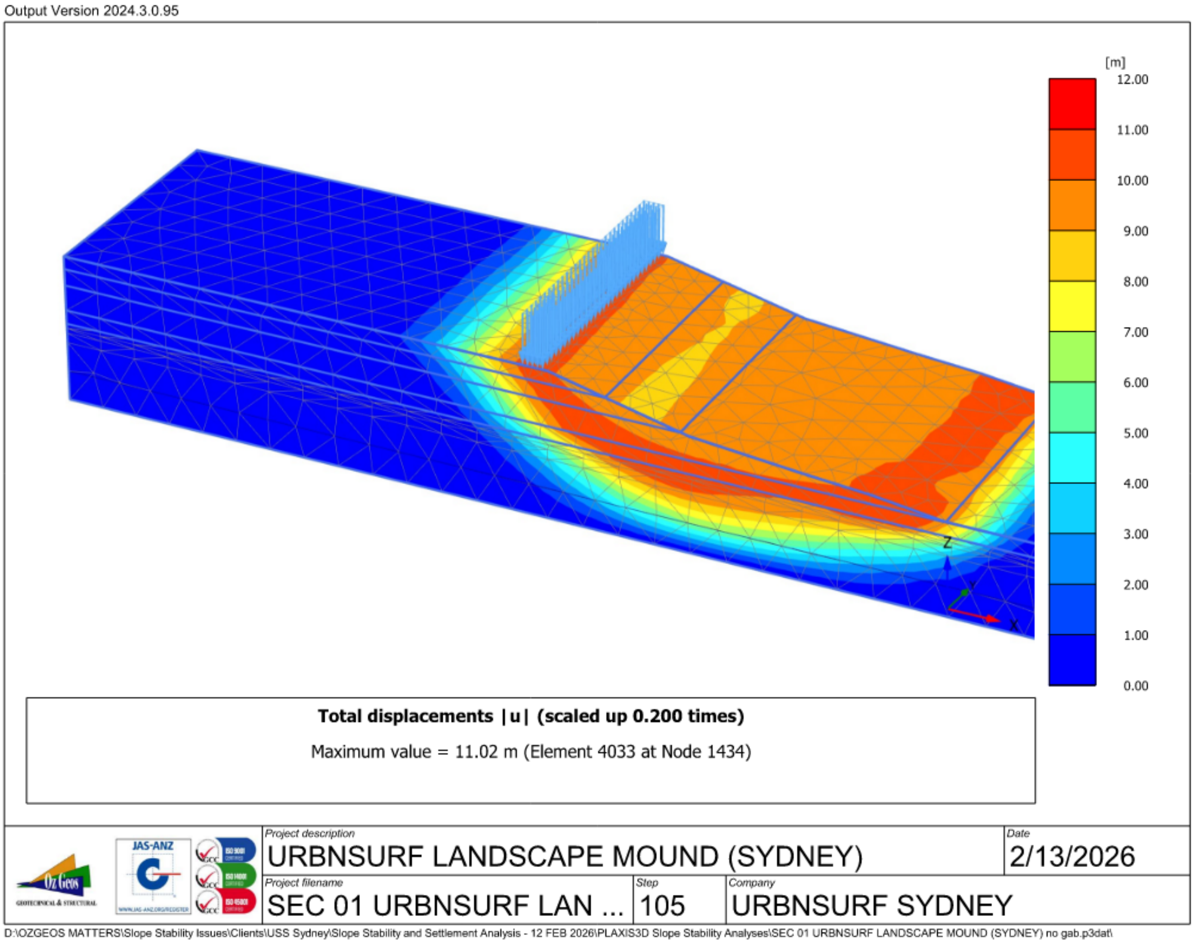


Slope Failure Plane – Sec 01 – Gabion Side - DRAINED

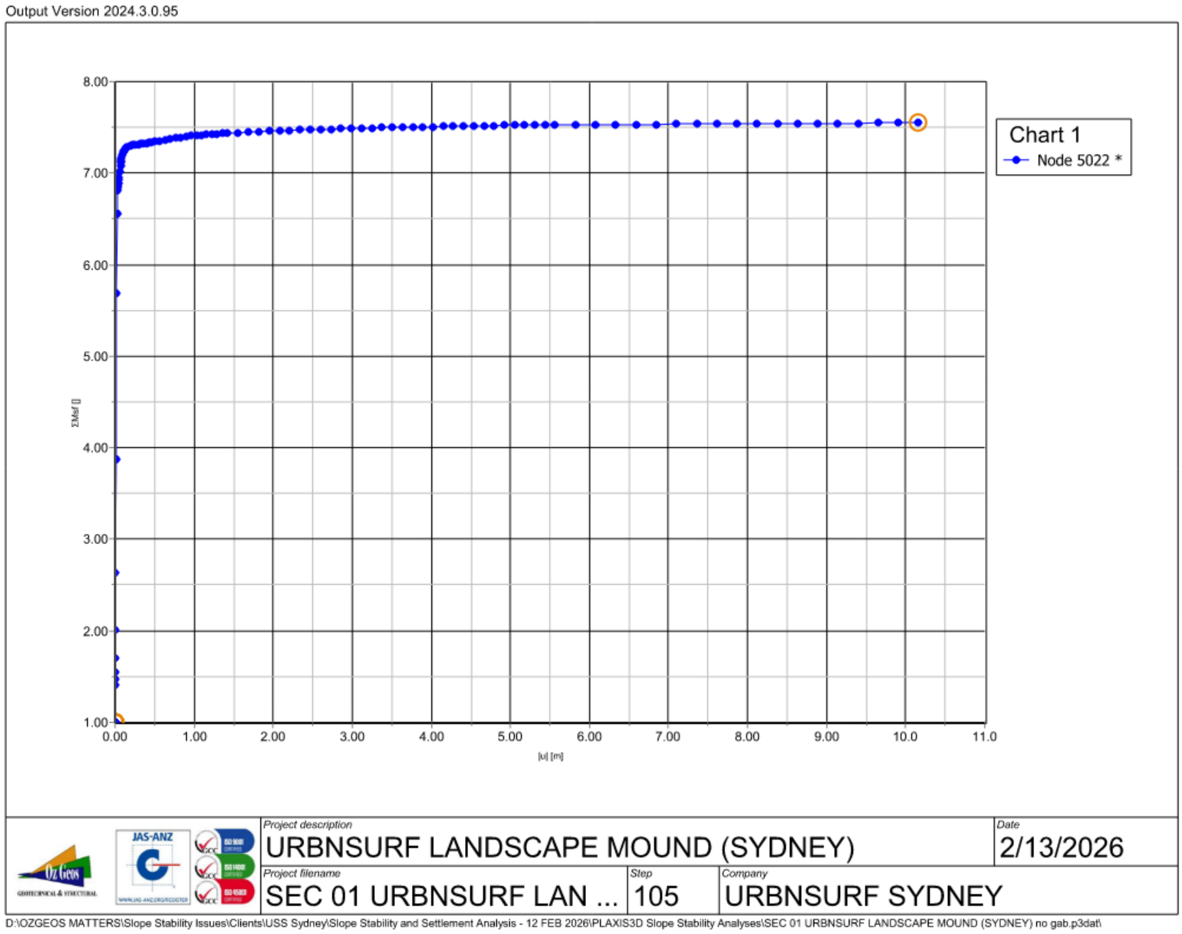


Safety Factor of Sec 01 – Gabion Side - DRAINED

APPENDIX C: FEM FOR UNDRAINED ANALYSIS (Section 01 – Western Side)

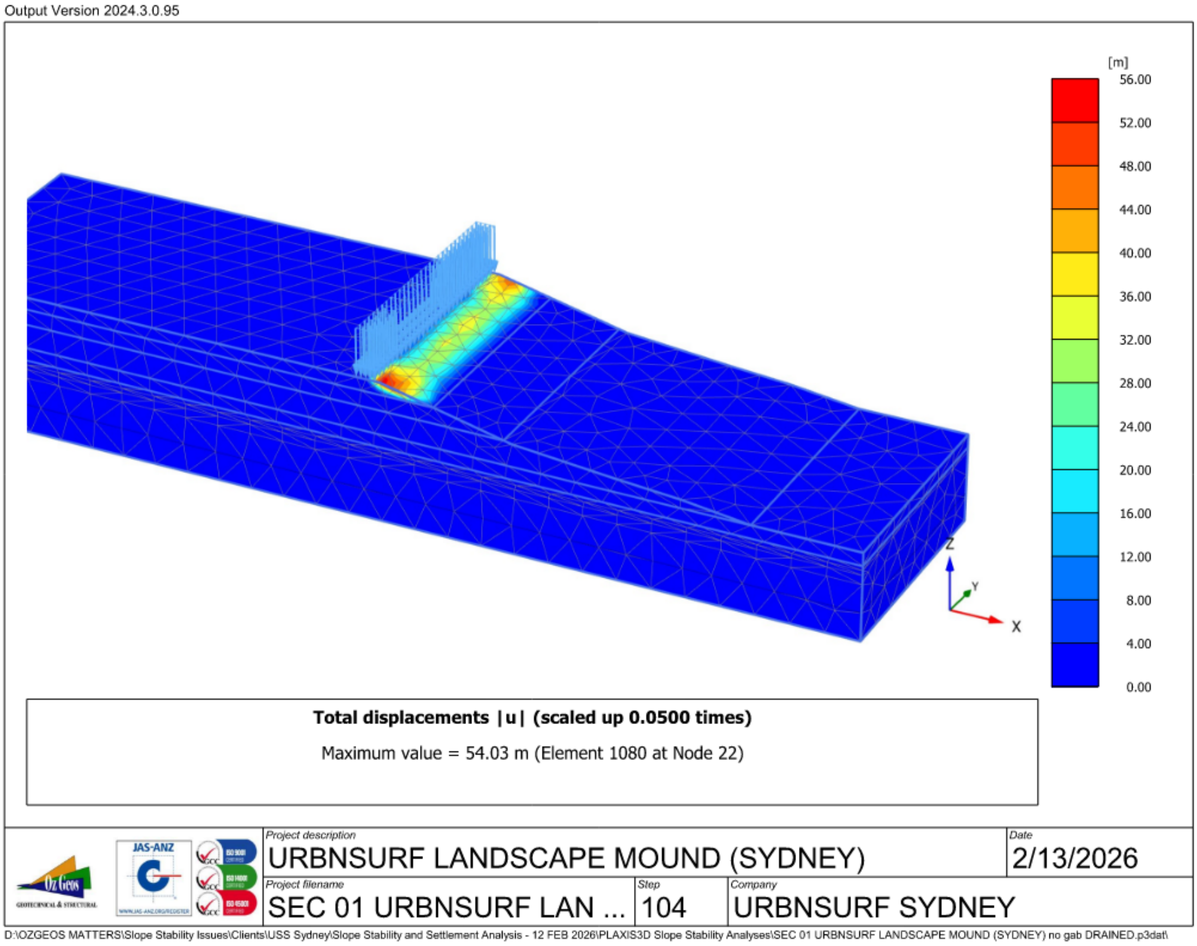


Slope Failure Plane – Sec 01 – Western Side – UNDRAINED

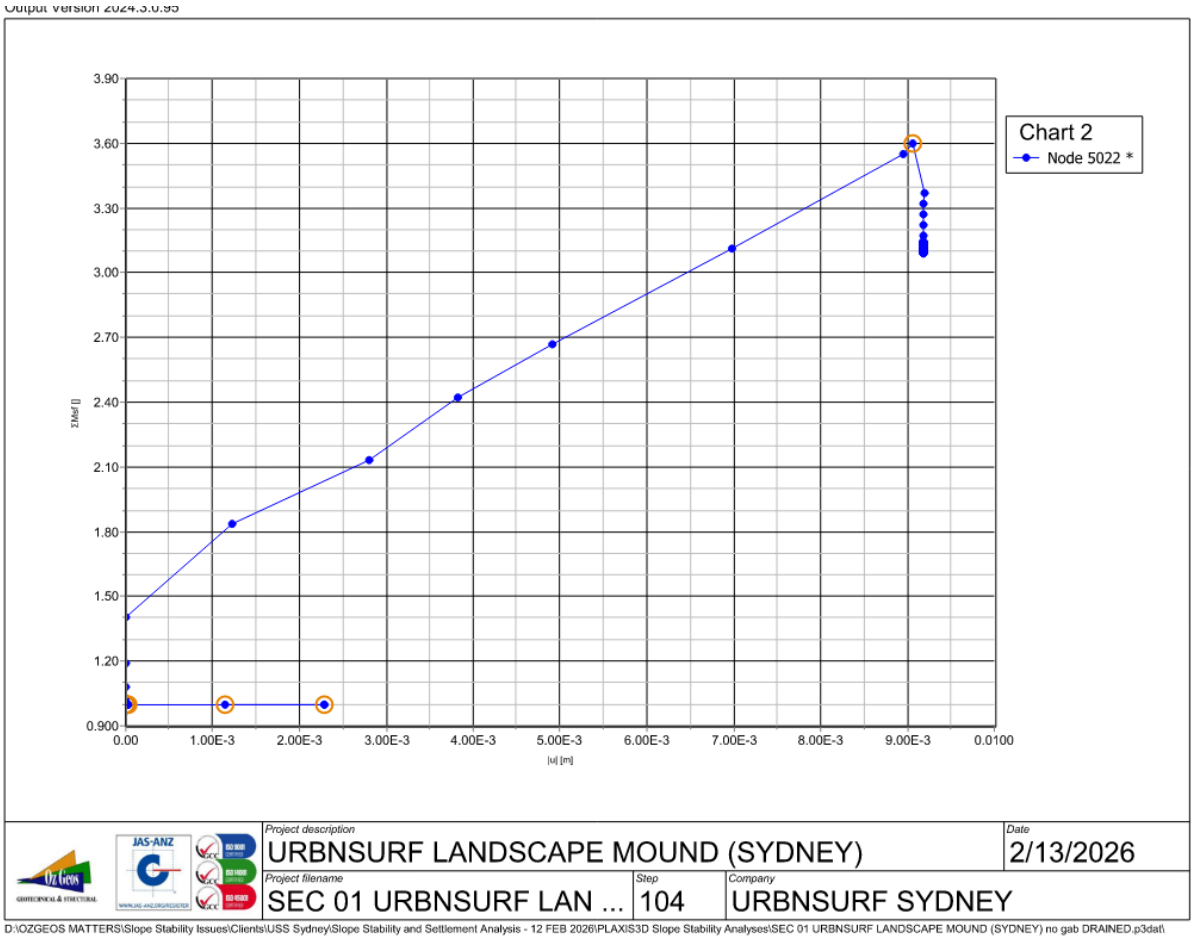


Safety Factor of Sec 01 – Western Side – UNDRAINED

APPENDIX D: FEM FOR DRAINED ANALYSIS (Section 01 – Western Side)

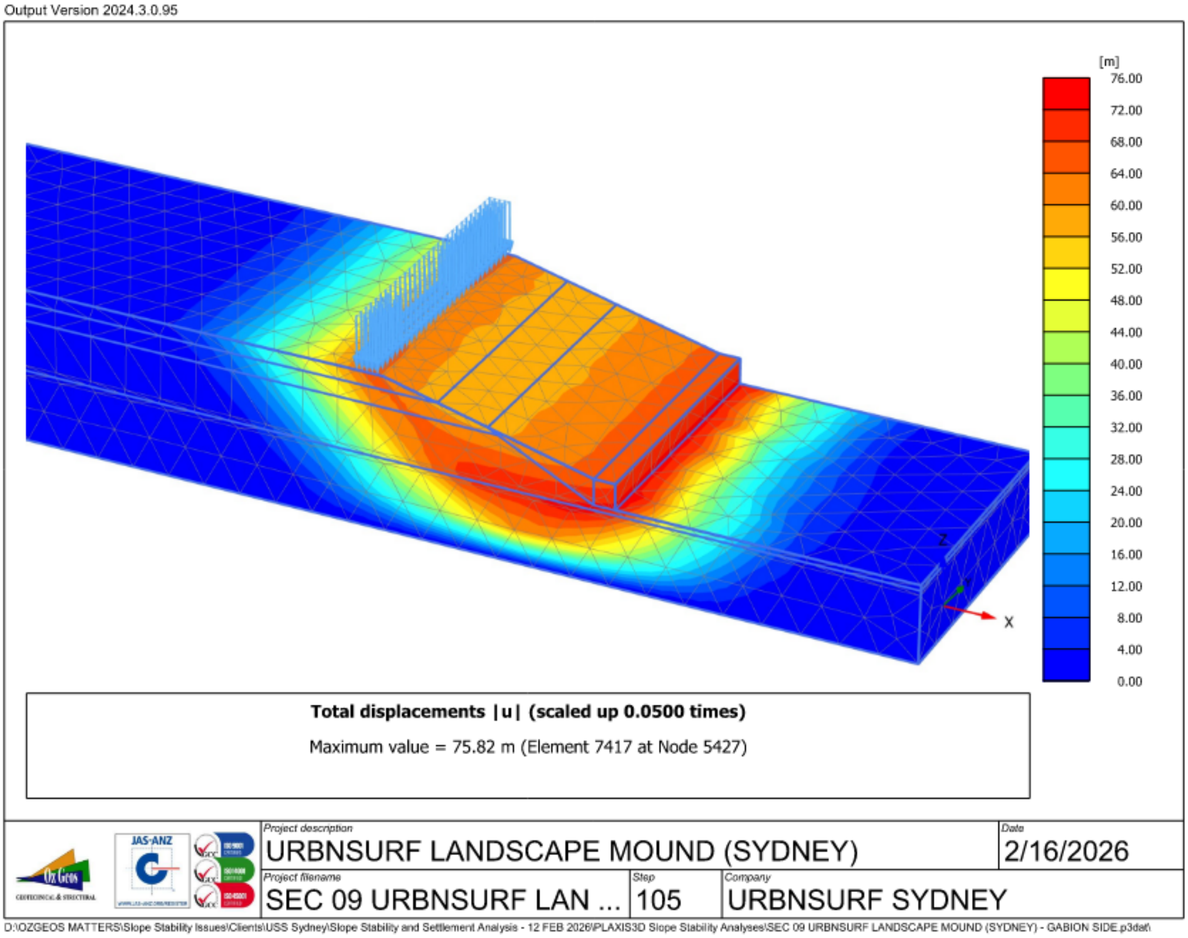


Slope Failure Plane – Sec 01 – Western Side – DRAINED



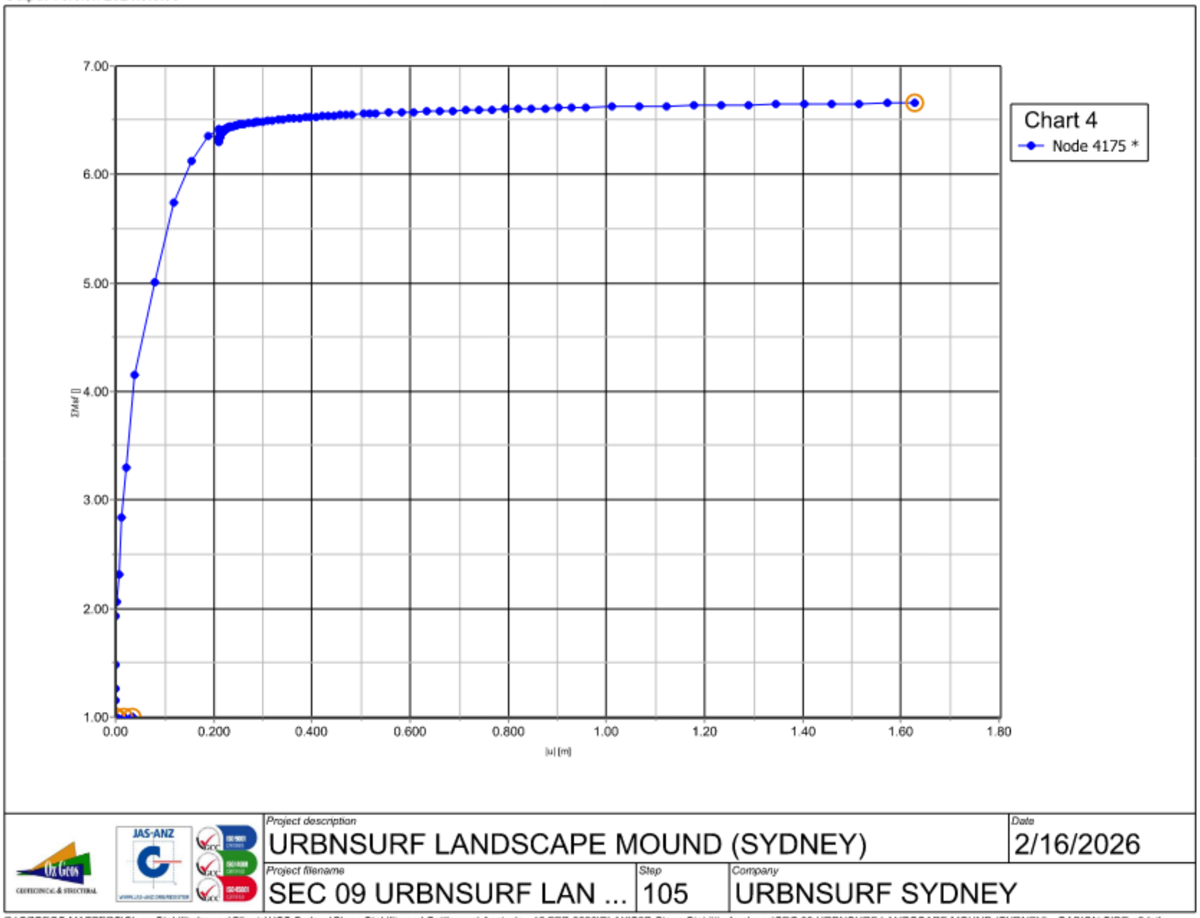
Safety Factor of Sec 01 – Western Side – DRAINED

APPENDIX E: FEM FOR UNDRAINED ANALYSIS (Section 09 – Northern Side)



Slope Failure Plane – Section 09 – Northern Side - UNDRAINED

Output Version 2024.3.0.95



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Safety Factor of Section 09 – Northern Side – UNDRAINED

APPENDIX F: FEM FOR DRAINED ANALYSIS (Section 09 – Northern Side)

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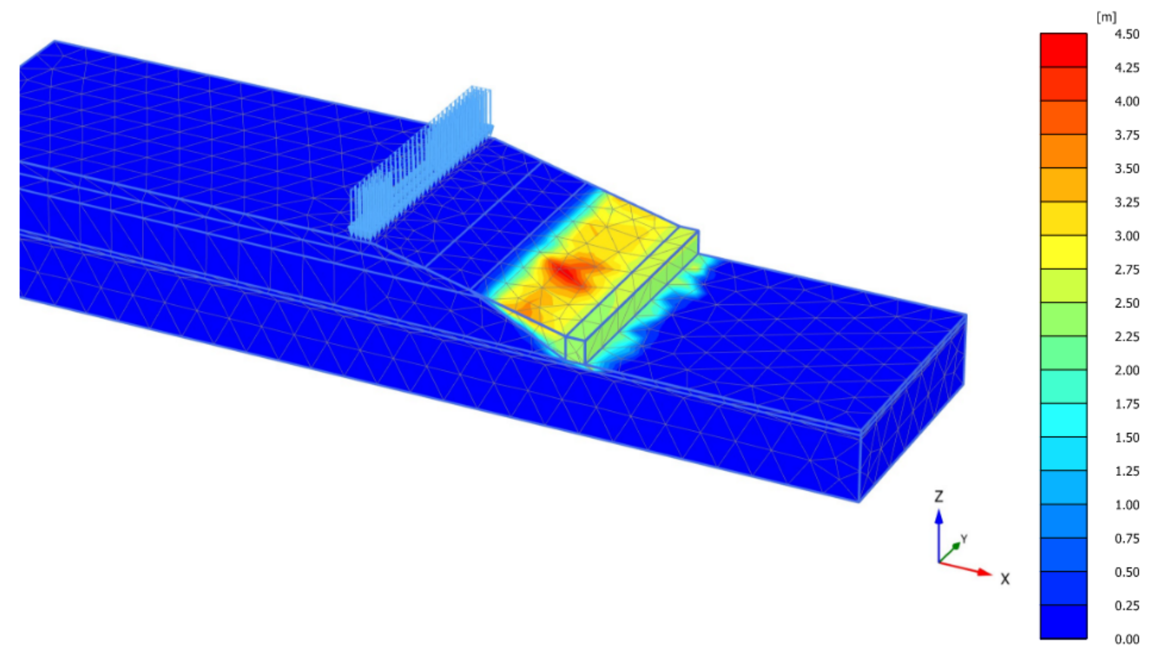

Total displacements |u| (scaled up 0.500 times)

Maximum value = 4.482 m (Element 3448 at Node 9014)

Project description		Date
URBNSURF LANDSCAPE MOUND (SYDNEY)		2/16/2026
Project filename: SEC 09 URBNSURF LAN ...	Step: 102	Company: URBNSURF SYDNEY

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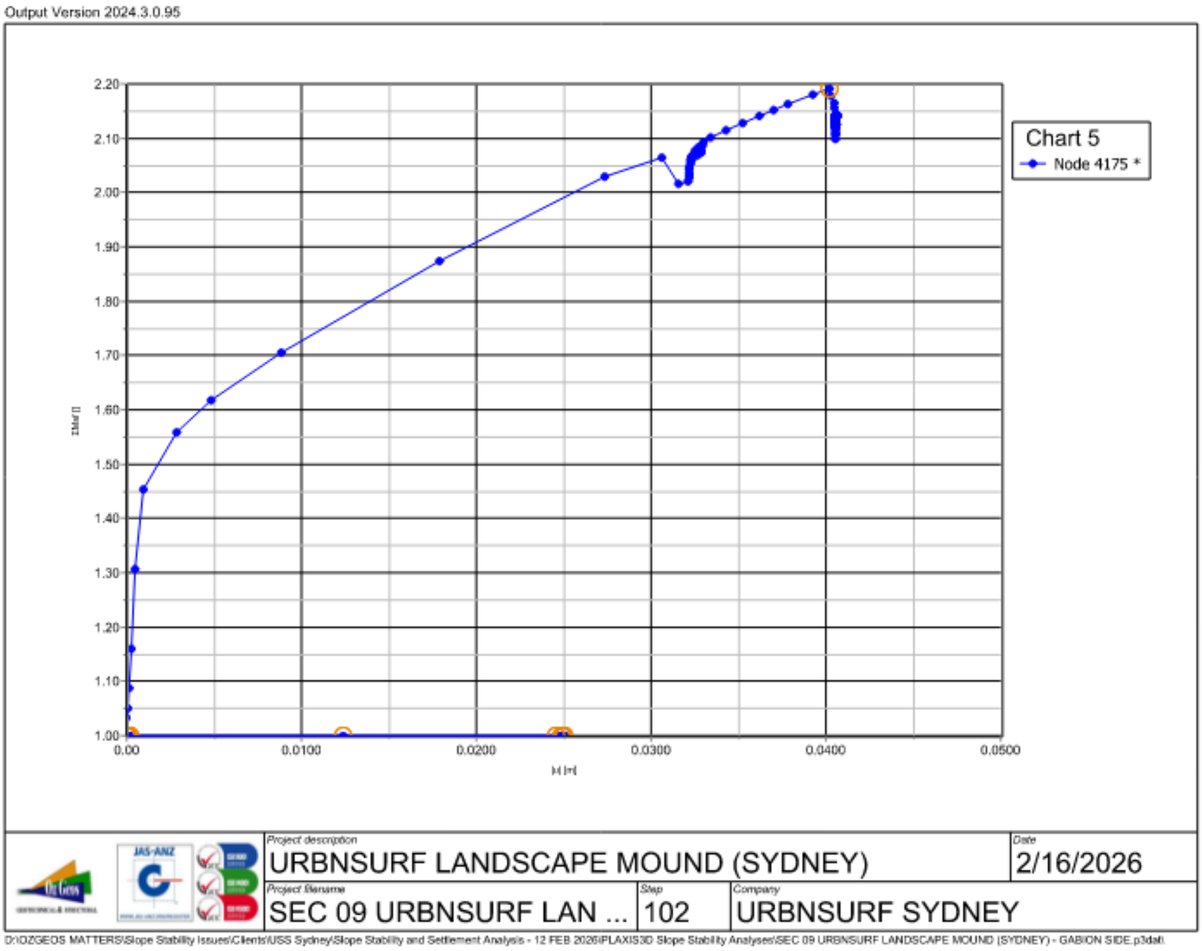
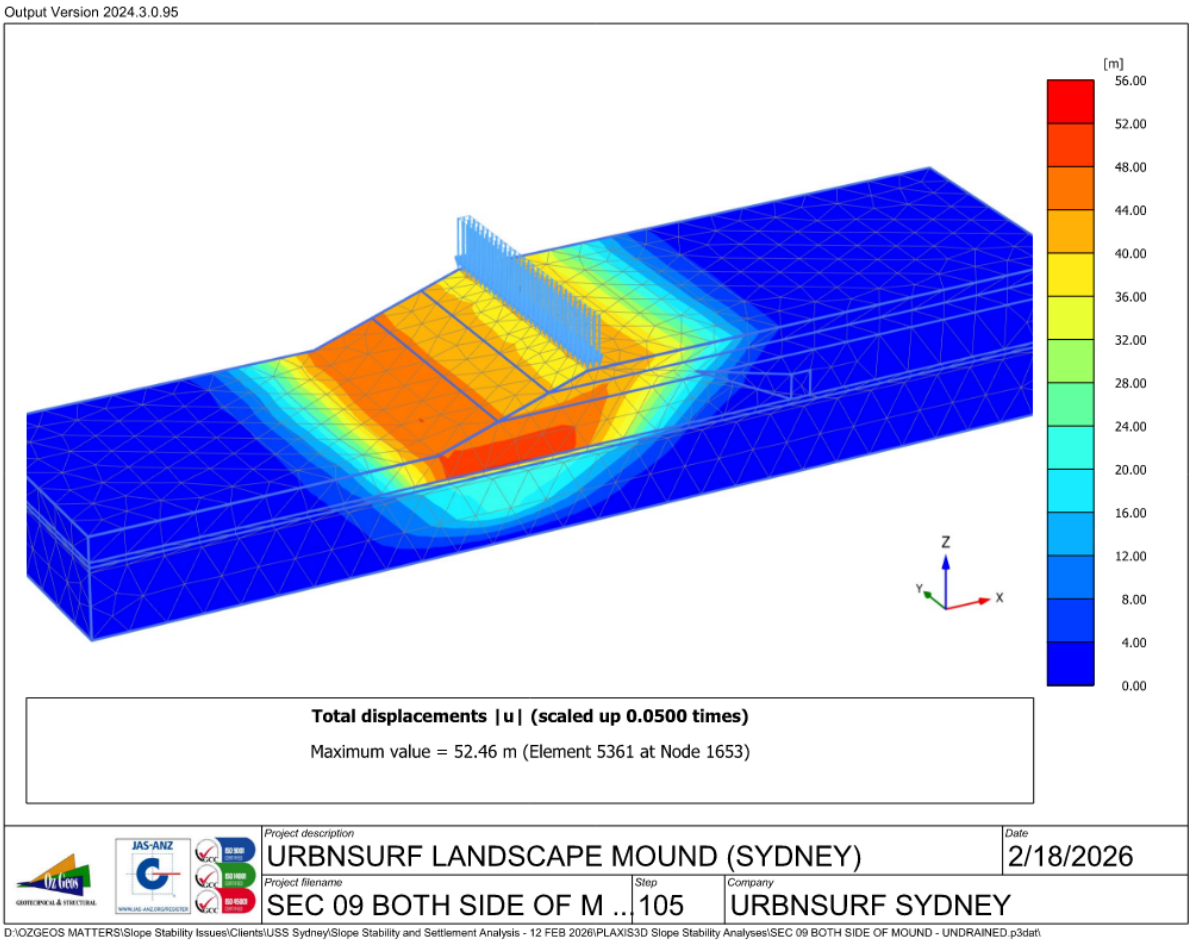
Slope Failure Plane – Section 09 – Northern Side – DRAINED

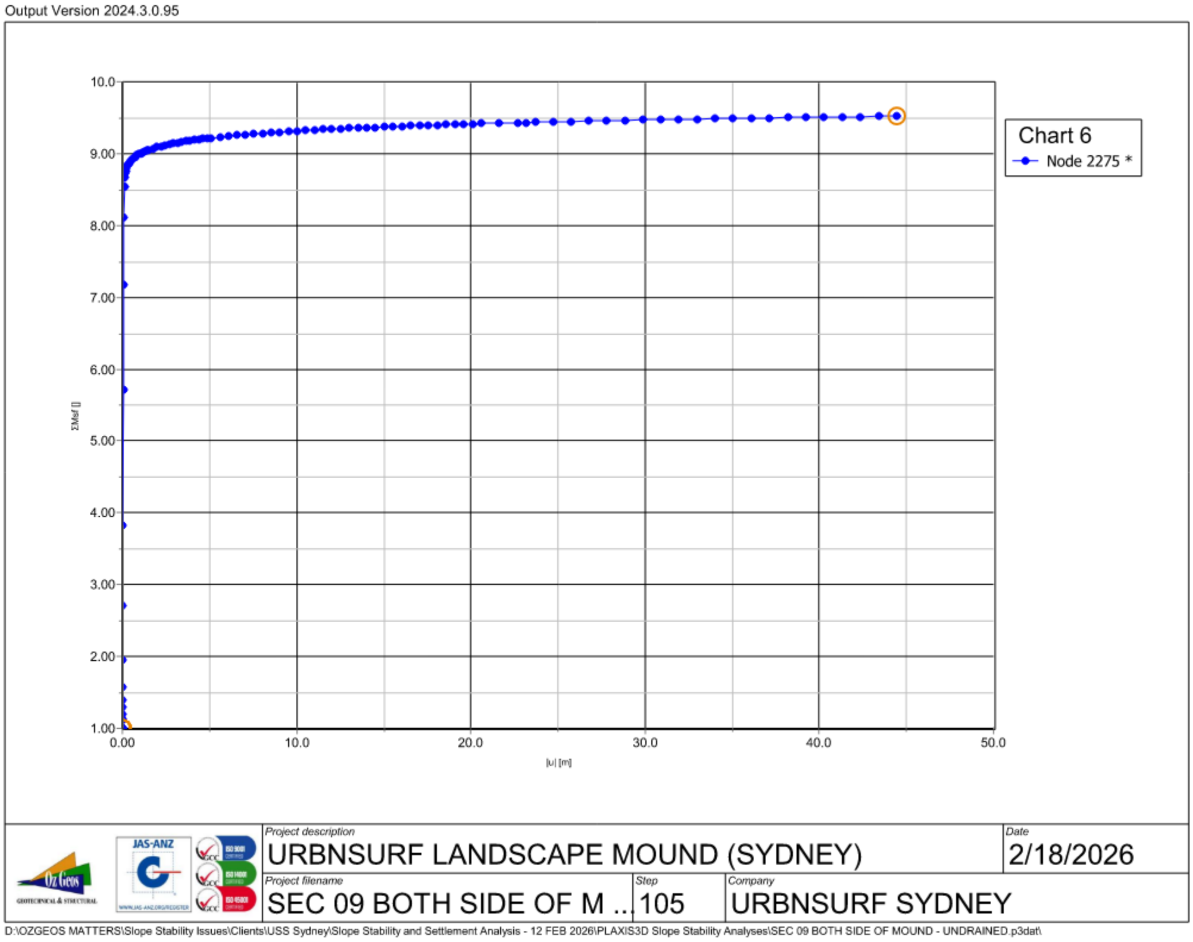


Safety Factor of Section 09 – Northern Side – DRAINED

APPENDIX G: FEM FOR UNDRAINED ANALYSIS (Section 09 – Southern Side)

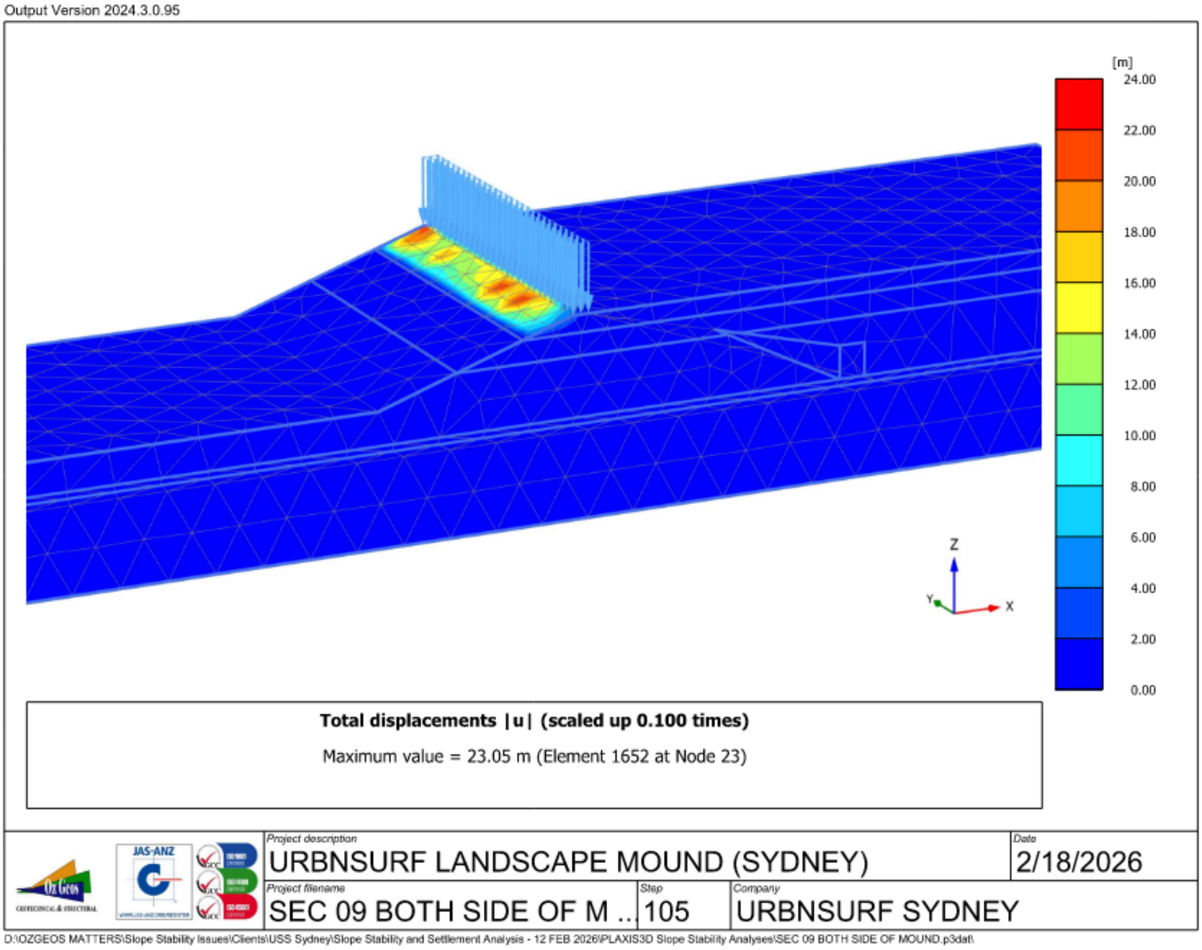


Slope Failure Plane – Section 09 – Southern Side – UNDRAINED

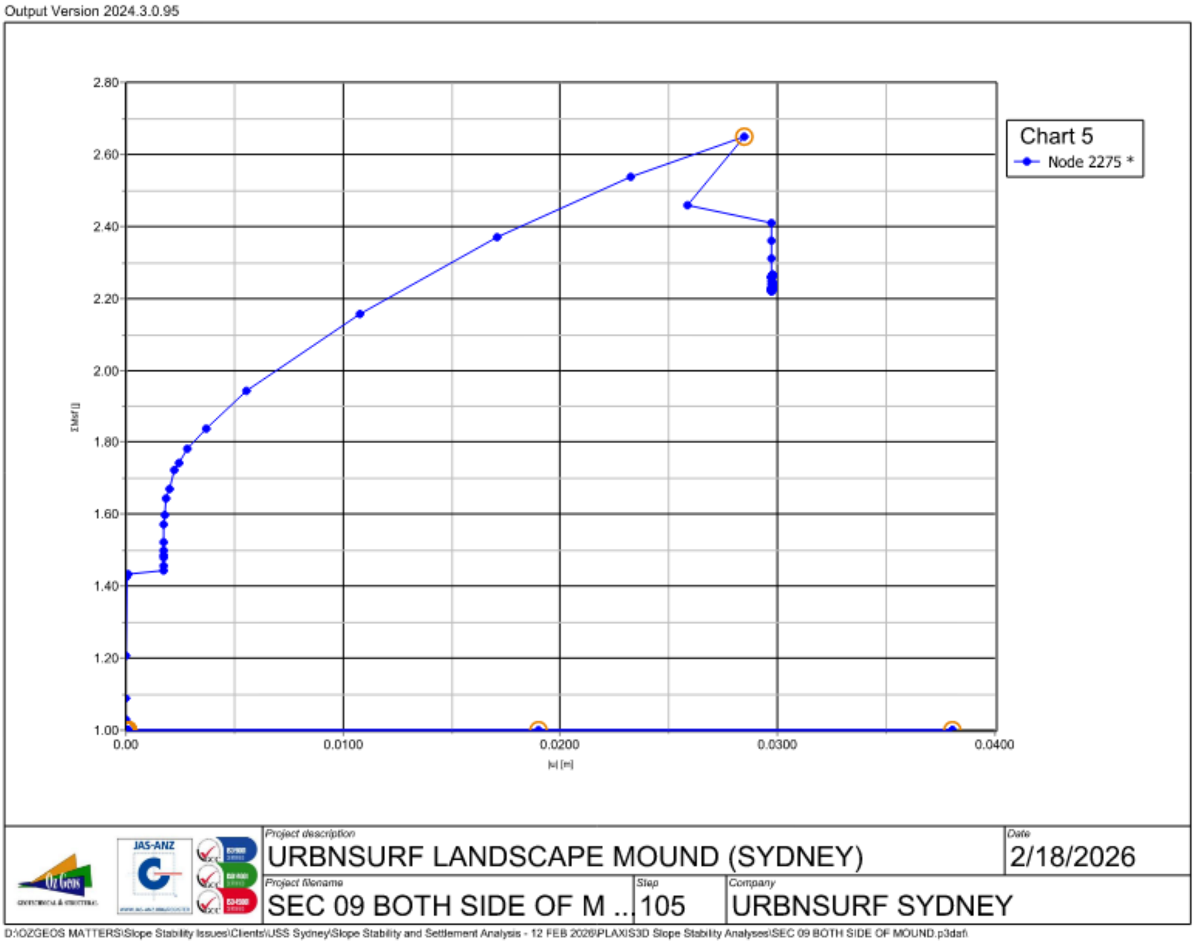


Safety Factor of Section 09 – Southern Side – UNDRAINED

APPENDIX H: FEM FOR DRAINED ANALYSIS (Section 09 – Southern Side)



Slope Failure Plane – Section 09 – Southern Side – DRAINED



Safety Factor of Section 09 – Southern Side – DRAINED

APPENDIX I: GABION WALL DESIGN CHECK

Oz Geos (AM & PU PTY LTD) U2 102-110 North View Dr, Sunshine West, VIC 3020	Project: URBNSURF SYDNEY LANDSCAPE MOUND				Job no.: GT1019-05N	
	Calcs for: GABION WALL DESIGN CHECK				Start page no./Revision: 1	
	Calcs by: PU	Calcs date: 2/18/2026	Checked by: PU	Checked date: 2/18/2026	Approved by: PU	Approved date: 2/18/2026

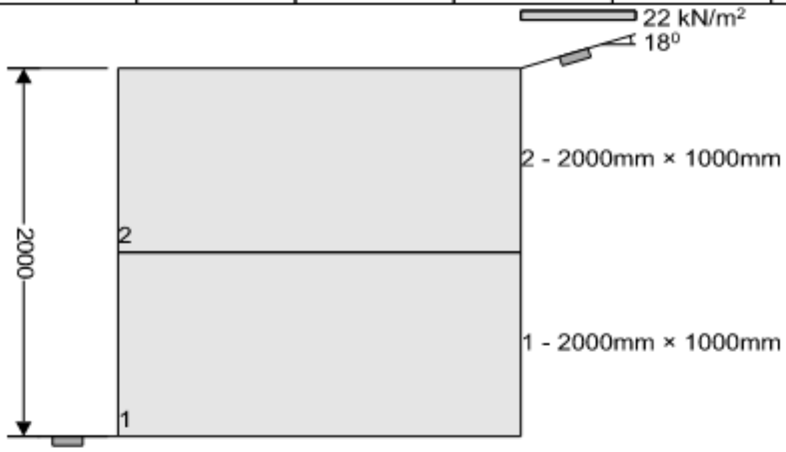
GABION RETAINING WALL ANALYSIS & DESIGN

Tedds calculation version 2.0.04

Design summary

Combination 1

Action	Resistance	Force	FoS	Allowable FoS	Status
Overturning, sliding and bearing at base level					
Overturning (kNm/m)	87.7	29.7	2.956	1.000	PASS
Sliding (kN/m)	46.1	36.6	1.260	1.000	PASS
Bearing (kN/m^2)	100.0	56.9	1.758	1.000	PASS
Eccentricity (mm)	Reaction acts within the middle third of base				PASS
Overturning and sliding between courses 1 and 2					
Overturning (kNm/m)	40.7	5.7	7.144	1.000	PASS
Sliding (kN/m)	26.8	13.1	2.047	1.000	PASS



Wall geometry

Width of gabion 1	w_1 = **2000** mm
Height of gabion 1	h_1 = **1000** mm
Width of gabion 2	w_2 = **2000** mm
Height of gabion 2	h_2 = **1000** mm
Step to front face between courses 1 and 2	s_2 = **0** mm
Wall inclination	ε = **0** deg

Gabion properties

Unit weight of fill	γ_d = **18.0** kN/m^3
Friction between gabions	$\delta_{bg.k}$ = **35.0** deg

Loading

Permanent surcharge	$p_{o,G}$ = **5** kN/m^2
Variable surcharge	$p_{o,Q}$ = **10** kN/m^2

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	Calcs for GABION WALL DESIGN CHECK				Start page no./Revision 2	
	Calcs by PU	Calcs date 2/18/2026	Checked by PU	Checked date 2/18/2026	Approved by PU	Approved date 2/18/2026

Soil properties

Slope of retained soil	β = **18.0** deg
Characteristic effective shearing resistance angle	$\phi'_{r.k}$ = **30.0** deg
Characteristic saturated density of retained soil	γ_{sr} = **21.0** kN/m^3
Coefficient for wall friction	$k_{membrane}$ = **0.75**
Wall friction angle	$\delta'_{r.k}$ = **22.5** deg
Characteristic wall friction angle	$\delta_{r.k} = \min(\delta'_{r.k}, \phi'_{r.k} \times k_{membrane})$ = **22.5** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **30.0** deg
Bearing capacity of founding soil	q = **100** kN/m^2

Wall geometry

Horizontal distance to centre of gravity gabion 1	$x_{g1} = w_1 / 2$ = **1000** mm
Vertical distance to centre of gravity gabion 1	$y_{g1} = h_1 / 2$ = **500** mm
Weight of gabion 1	$W_{g1} = \gamma_d \times w_1 \times h_1$ = **36.0** kN/m
Horizontal distance to centre of gravity gabion 2	$x_{g2} = w_2 / 2 + s_2$ = **1000** mm
Vertical distance to centre of gravity gabion 2	$y_{g2} = h_2 / 2 + h_1$ = **1500** mm
Weight of gabion 2	$W_{g2} = \gamma_d \times w_2 \times h_2$ = **36.0** kN/m
Weight of entire gabion	$W_g = W_{g1} + W_{g2}$ = **72.0** kN/m
Horiz distance to centre of gravity entire gabion	$x_g = ((W_{g1} \times x_{g1}) + (W_{g2} \times x_{g2})) / W_g$ = **1000** mm
Vert distance to centre of gravity entire gabion	$y_g = ((W_{g1} \times y_{g1}) + (W_{g2} \times y_{g2})) / W_g$ = **1000** mm
Correcting for wall inclination horiz dist	$X_g = x_g \times \cos(\varepsilon) + y_g \times \sin(\varepsilon)$ = **1000** mm
Vertical change in height due to wall inclination	$H_f = y_{g2} + h_2/2 - ((y_{g2} + h_2/2) \times \cos(\varepsilon) - (x_{g2} + w_2/2) \times \sin(\varepsilon))$ = **0** mm

Design dimensions

Effective angle of rear plane of wall	α = 90 deg + ε = **90.0** deg
Effective face angle	θ = 90deg - ε = **90.0** deg
Effective height of wall	$H = (y_{g2} + h_2 / 2) + (w_1 \times \sin(\varepsilon)) - H_f$ = **2000** mm
Height of wall from toe to front edge of top gabion	$H_{incl} = ((y_{g2} + h_2 / 2) \times \cos(\varepsilon) - (x_{g2} - (w_2 / 2)) \times \sin(\varepsilon))$ = **2000**mm
Active pressure using Coulomb theory	$K_a = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times (1 + \sqrt{}(\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))))^2)$ = **0.395**
Active thrust due to soil	$P_{a,soil} = 0.5 \times K_a \times \gamma_{sr} \times H^2$ = **16.6** kN/m

Pressure at base

Horizontal forces

Retained soil	$F_{soil\_h,q} = P_{a,soil} \times \cos(90 - \alpha + \delta_{r.k})$ = **15.3** kN/m
Height of soil thrust resolved vertically	$d_{h,soil} = H / 3 - w_1 \times \sin(\varepsilon)$ = **667** mm
Surcharge	$F_{surch\_h,q} = (p_{o,G} + p_{o,Q}) \times K_a \times H \times \cos(90 - \alpha + \delta_{r.k})$ = **11.0** kN/m
Height of surcharge thrust resolved vertically	$d_{h,surch} = H / 2 - w_1 \times \sin(\varepsilon)$ = **1000** mm

Vertical forces

Gabion weight	$F_{gabion\_v,q} = W_g$ = **72.0** kN/m
Retained soil	$F_{soil\_v,q} = P_{a,soil} \times \sin(90 - \alpha + \delta_{r.k})$ = **6.4** kN/m
Horizontal dist to where soil thrust acts	$b_{v,soil} = w_1 \times \cos(\varepsilon) - (H / 3) / \tan(\alpha)$ = **2000** mm
Surcharge	$F_{surch\_v,q} = (p_{o,G} + p_{o,Q}) \times K_a \times H \times \sin(90 - \alpha + \delta_{r.k})$ = **4.5** kN/m
Horizontal dist to where surcharge thrust acts	$b_{v,surch} = w_1 \times \cos(\varepsilon) - (H / 2) / \tan(\alpha)$ = **2000** mm
Total horizontal unfactored force	$T_q = F_{soil\_h,q} + F_{surch\_h,q}$ = **26.3** kN/m
Total vertical unfactored force	$N_q = F_{gabion\_v,q} + F_{soil\_v,q} + F_{surch\_v,q}$ = **82.9** kN/m
Force normal to base	$N_s = N_q \times \cos(\varepsilon) + T_q \times \sin(\varepsilon)$ = **82.9** kN/m

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	Calcs by PU	Calcs date 2/18/2026	Checked by PU	Checked date 2/18/2026	Approved by PU	Approved date 2/18/2026

Total unfactored overturning force	$M_{O,q} = F_{soil\_h,q} \times d_{h,soil} + F_{surch\_h,q} \times d_{h,surch} = \mathbf{21.2}$ kNm/m
Total unfactored restoring force	$M_{R,q} = F_{gabion\_v,q} \times X_g + F_{soil\_v,q} \times b_{v,soil} + F_{surch\_v,q} \times b_{v,surch} = \mathbf{93.8}$ kNm/m
Eccentricity	$e = w_1 / 2 - (M_{R,q} - M_{O,q}) / N_s = \mathbf{124}$ mm
	Reaction acts within middle third of base
Pressure at toe	$\sigma_{toe} = N_s / w_1 \times (1 + (6 \times e / w_1)) = \mathbf{56.9}$ kN/m^2
Pressure at heel	$\sigma_{heel} = N_s / w_1 \times (1 - (6 \times e / w_1)) = \mathbf{26.0}$ kN/m^2
Factor of safety	$FoS_Q = q / \max(\sigma_{toe}, \sigma_{heel}) = \mathbf{1.758}$
Allowable factor of safety	$FoS_{Q\_allow} = \mathbf{1.000}$

PASS - Design FoS for allowable bearing pressure exceeds min allowable pressure to base

Design approach 1

Partial factors on actions - Combination 1

Permanent unfavourable action	$\gamma_G = \mathbf{1.35}$
Permanent favourable action	$\gamma_{G,f} = \mathbf{1.00}$
Variable unfavourable action	$\gamma_Q = \mathbf{1.50}$
Variable favourable action	$\gamma_{Q,f} = \mathbf{0.00}$

Partial factors for soil parameters - Combination 1

Angle of shearing resistance	$\gamma_{\phi'} = \mathbf{1.00}$
Weight density	$\gamma_\gamma = \mathbf{1.00}$

Design soil properties

Design effective shearing resistance angle	$\phi'_{r.d} = \mathrm{Atan}(\tan(\phi'_{r.k}) / \gamma_{\phi'}) = \mathbf{30.0}$ deg
Design saturated density of retained soil	$\gamma_{s.d} = \gamma_{sr} / \gamma_\gamma = \mathbf{21.0}$ kN/m^3
Design wall friction angle	$\delta_{r.d} = \min(\mathrm{atan}(\tan(\delta_{r.k}) / \gamma_{\phi'}), \phi'_{r.d} \times k_{membrane}) = \mathbf{22.5}$ deg
Design base friction angle	$\delta_{bb.d} = \mathrm{Atan}(\tan(\delta_{bb.k}) / \gamma_{\phi'}) = \mathbf{30.0}$ deg
Design friction between gabions	$\delta_{bg.d} = \mathrm{Atan}(\tan(\delta_{bg.k}) / \gamma_{\phi'}) = \mathbf{35.0}$ deg
Active pressure using Coulomb theory	$K_a = \sin(\alpha + \phi'_{r.d})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.d}) \times (1 + \sqrt{}(\sin(\phi'_{r.d} + \delta_{r.d}) \times \sin(\phi'_{r.d} - \beta) / (\sin(\alpha - \delta_{r.d}) \times \sin(\alpha + \beta))))^2) = \mathbf{0.395}$
Active thrust due to soil	$P_{a,soil} = 0.5 \times K_a \times \gamma_{s.d} \times H^2 = \mathbf{16.6}$ kN/m

Horizontal forces

Retained soil	$F_{soil\_h} = \gamma_G \times P_{a,soil} \times \cos(90 - \alpha + \delta_{r.d}) = \mathbf{20.7}$ kN/m
Surcharge	$F_{surch\_h} = (p_{o,G} \times \gamma_G + p_{o,Q} \times \gamma_Q) \times K_a \times H \times \cos(90 - \alpha + \delta_{r.d}) = \mathbf{15.9}$ kN/m

Vertical forces

Gabion weight	$F_{gabion\_v,f} = \gamma_{G,f} \times W_g = \mathbf{72.0}$ kN/m
Retained soil	$F_{soil\_v,f} = \gamma_{G,f} \times P_{a,soil} \times \sin(90 - \alpha + \delta_{r.d}) = \mathbf{6.4}$ kN/m
Surcharge	$F_{surch\_v,f} = (p_{o,G} \times \gamma_{G,f} + p_{o,Q} \times \gamma_{Q,f}) \times K_a \times H \times \sin(90 - \alpha + \delta_{r.d}) = \mathbf{1.5}$ kN/m

Overturning stability - take moments about the toe

Overturning moment	$M_O = F_{soil\_h} \times d_{h,soil} + F_{surch\_h} \times d_{h,surch} = \mathbf{29.7}$ kNm/m
Restoring moment	$M_R = F_{gabion\_v,f} \times X_g + F_{soil\_v,f} \times b_{v,soil} + F_{surch\_v,f} \times b_{v,surch} = \mathbf{87.7}$ kNm/m
Factor of safety	$FoS_M = M_R / M_O = \mathbf{2.956}$
Allowable factor of safety	$FoS_{M\_allow} = \mathbf{1.000}$

PASS - Design FOS for overturning exceeds min allowable FOS for overturning

Sliding stability - ignore any passive pressure in front of the structure

Total horizontal force	$T = F_{soil\_h} + F_{surch\_h} = \mathbf{36.6}$ kN/m
Total vertical force	$N = F_{gabion\_v,f} + F_{soil\_v,f} + F_{surch\_v,f} = \mathbf{79.9}$ kN/m

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	Calcs by PU	Calcs date 2/18/2026	Checked by PU	Approved by PU	Approved date 2/18/2026

Sliding force	$F_f = T \times \cos(\varepsilon) - N \times \sin(\varepsilon) =$ **36.6** kN/m
Sliding resistance	$F_R = (T \times \sin(\varepsilon) + N \times \cos(\varepsilon)) \times \tan(\delta_{bb.d}) =$ **46.1** kN/m
Factor of safety	$FoS_S = F_R / F_f =$ **1.260**
Allowable factor of safety	$FoS_{S\_allow} =$ **1.000**

PASS - Design FOS for sliding exceeds min allowable FOS for sliding

Check overturning and sliding between courses 1 and 2

Wall geometry

Horizontal distance to centre of gravity gabion 2	$x_{g2} = w_2 / 2 =$ **1000** mm
Vertical distance to centre of gravity gabion 2	$y_{g2} = h_2 / 2 =$ **500** mm
Weight of gabion 2	$W_{g2} = \gamma_d \times w_2 \times h_2 =$ **36.0** kN/m
Weight of entire gabion	$W_g = W_{g2} =$ **36.0** kN/m
Horiz distance to centre of gravity entire gabion	$x_g = ((W_{g2} \times x_{g2})) / W_g =$ **1000** mm
Vert distance to centre of gravity entire gabion	$y_g = ((W_{g2} \times y_{g2})) / W_g =$ **500** mm
Correcting for wall inclination horiz dist	$X_g = x_g \times \cos(\varepsilon) + y_g \times \sin(\varepsilon) =$ **1000** mm
Vertical change in height due to wall inclination	$H_f = y_{g2} + h_2/2 - ((y_{g2} + h_2/2) \times \cos(\varepsilon) - (x_{g2} + w_2/2) \times \sin(\varepsilon)) =$ **0** mm

Design dimensions

Effective angle of rear plane of wall	$\alpha = 90$ deg $+ \varepsilon =$ **90.0** deg
Effective face angle	$\theta = 90$deg $- \varepsilon =$ **90.0** deg
Effective height of wall	$H = (y_{g2} + h_2 / 2) + (w_2 \times \sin(\varepsilon)) - H_f =$ **1000** mm
Height of wall from toe to front edge of top gabion	$H_{incl} = ((y_{g2} + h_2 / 2) \times \cos(\varepsilon) - (x_{g2} - (w_2 / 2)) \times \sin(\varepsilon)) =$ **1000**mm
Active pressure using Coulomb theory	$K_a = \sin(\alpha + \phi'_{r.d})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.d}) \times (1 + \sqrt{}(\sin(\phi'_{r.d} + \delta_{r.d}) \times \sin(\phi'_{r.d} - \beta) / (\sin(\alpha - \delta_{r.d}) \times \sin(\alpha + \beta))))^2) =$ **0.395**
Active thrust due to soil	$P_{a,soil} = 0.5 \times K_a \times \gamma_{s.d} \times H^2 =$ **4.1** kN/m

Horizontal forces

Retained soil	$F_{soil\_h} = \gamma_G \times P_{a,soil} \times \cos(90 - \alpha + \delta_{r.d}) =$ **5.2** kN/m
Surcharge	$F_{surch\_h} = (p_{o,G} \times \gamma_G + p_{o,Q} \times \gamma_Q) \times K_a \times H \times \cos(90 - \alpha + \delta_{r.d}) =$ **7.9** kN/m

Vertical forces

Gabion weight	$F_{gabion\_v.f} = \gamma_{G.f} \times W_g =$ **36.0** kN/m
Retained soil	$F_{soil\_v.f} = \gamma_{G.f} \times P_{a,soil} \times \sin(90 - \alpha + \delta_{r.d}) =$ **1.6** kN/m
Surcharge	$F_{surch\_v.f} = (p_{o.G} \times \gamma_{G.f} + p_{o.Q} \times \gamma_{Q.f}) \times K_a \times H \times \sin(90 - \alpha + \delta_{r.d}) =$ **0.8** kN/m

Overturning stability - take moments about the toe

Overturning moment	$M_o = F_{soil\_h} \times d_{h.soil} + F_{surch\_h} \times d_{h,surch} =$ **5.7** kNm/m
Restoring moment	$M_R = F_{gabion\_v.f} \times X_g + F_{soil\_v.f} \times b_{v,soil} + F_{surch\_v.f} \times b_{v.surch} =$ **40.7** kNm/m
Factor of safety	$FoS_M = M_R / M_o =$ **7.144**
Allowable factor of safety	$FoS_{M\_allow} =$ **1.000**

PASS - Design FOS for overturning exceeds min allowable FOS for overturning

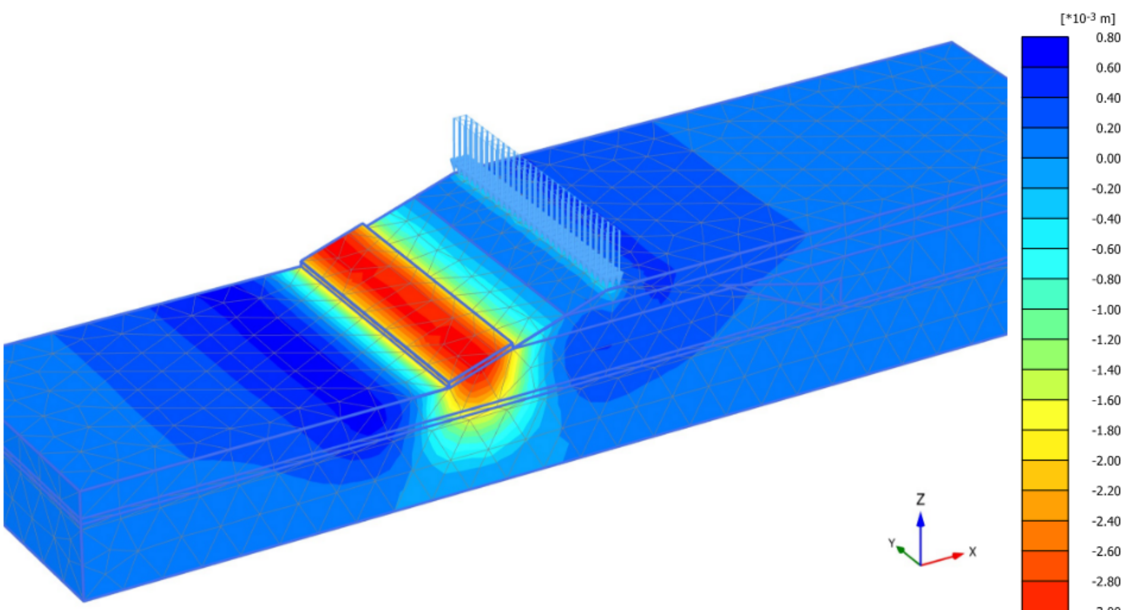
Sliding stability - ignore any passive pressure in front of the structure

Total horizontal force	$T = F_{soil\_h} + F_{surch\_h} =$ **13.1** kN/m
Total vertical force	$N = F_{gabion\_v.f} + F_{soil\_v.f} + F_{surch\_v.f} =$ **38.3** kN/m
Sliding force	$F_f = T \times \cos(\varepsilon) - N \times \sin(\varepsilon) =$ **13.1** kN/m
Sliding resistance	$F_R = (T \times \sin(\varepsilon) + N \times \cos(\varepsilon)) \times \tan(\delta_{bg.d}) =$ **26.8** kN/m
Factor of safety	$FoS_S = F_R / F_f =$ **2.047**
Allowable factor of safety	$FoS_{S\_allow} =$ **1.000**

PASS - Design FOS for sliding exceeds min allowable FOS for sliding

APPENDIX J: SANDSTONE BLOCK SETTLEMENT ANALYSIS

Output Version 2024.3.0.95



Total displacements u_z (scaled up 500 times)

Maximum value = $0.6814*10^{-3}$ m (Element 5589 at Node 111)

Minimum value = $-3.186*10^{-3}$ m (Element 3585 at Node 4311)

Project description: URBNSURF LANDSCAPE MOUND (SYDNEY)			Date: 2/18/2026
Project filename: SOIL BEARING CAPACIT ...	Step: 5	Company: URBNSURF SYDNEY	

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Settlement of Sandstone Block