Magħtab Environmental Complex, Malta

New Non-Hazardous Waste Landfill

Stability Risk Assessment

Report prepared for: Wasteserv, Malta

Date: 7 December 2021

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Quality management data

Name of Project	Magħtab Complex, Malta
CQA Reference No.	30500
Client details	WasteServ, Malta
Client Reference No.	
Type of document	Report
Title of this document	Stability Risk Assessment
Status / Version	Final, Version 3
Issue date and history	Draft Reviewed: December 2019 Version 1 Issued: 17 December 2019 Version 2 issued: 31 July 2020 Version 3 issued: 7 December 2020
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Reviewed by	Darren Bland
Authorised to be issued as a formal report from CQA International Ltd by	Peter Stevens Managing Director
Signature	Peter

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1. Introduction

1.1 Terms of reference

This assessment of stability issues relates to the construction and operation of a new landfill in the Magħtab Environmental Complex, Malta. The assessment includes the potential impact on nearby historic landfills (Magħtab and Ta'Żwejra) and the existing mechanical waste treatment plant.

The report has been prepared for Wasteserv Malta Ltd (WSM) by CQA International Ltd (CQA) and is based on documentary information and the results of data collection carried out on site between 21 and 22 November 2019.

A supplementary site visit was made during summer 2021, during which further data were acquired.

The potential for instability is considered for the different elements of the landfill and modes of failure, in the scenarios of excavation, liner construction and waste-filling operations.

1.2 Location and proposed development

The new landfill will be constructed in an excavation; a limestone quarry is being enlarged to provide greater capacity. Part of the site was previously intended to be used for hazardous waste disposal, although the designation has changed. This area is currently used as an RDF stockpile, and this material is currently being removed.

The dimensions of the available land present a major challenge to the development of an economically feasible landfill site. The narrow site width and relatively small footprint would provide very limited capacity with traditional landfill construction, CQA has addressed this problem with the design of an innovative "steep wall" system to maximise the void space available for disposal. This design is based on landfill steep wall systems that have previously been installed in rock excavations.

The side slopes of an engineered landfill are typically formed to a grade between 1:2.5 and 1:3. The appropriate slope angle is determined by the need for the mineral liner to remain stable, for the avoidance of shear stresses in the geosynthetic liner and for safety during installation. The width of the quarry is approximately 90m. If the base of the landfill is assumed to be 30m wide, the side slopes could extend to a depth of only 10 metres, if standard construction methods were used.

As the quarry is being excavated with near-vertical sides, much of the void would need to be backfilled to create conventional side slopes. This would provide a very small disposal volume, compared to the cost of construction, greatly reducing economic feasibility. This option would not provide the void space required to meet the national waste disposal targets.

CQA has designed a vertical lining system for the side slopes which will allow the full width of the quarry to be utilised, to a depth exceeding 40m. This increases the available void space by a factor of three.

1.3 Local context

There are three existing landfills, part of the Magħtab Environmental Complex, which partly surrounding the proposed new landfill.

The Magħtab landfill is the oldest and lies directly northeast of the new landfill. It was constructed as a dilute and disperse landfill in the 1970s. This landfill was constructed as a valley infill with subsequent land raise to an elevation of approximately 100m above sea level (ASL).

The Ta'Żwejra landfill was constructed in 2004 in compliance with the EU landfill directive. It is located on the south-eastern boundary of the new landfill and was also formed as a valley infill and subsequent land raise, to an elevation of approximately 80m ASL.

The Għallis landfill is currently being used for waste disposal operations. This was constructed in 2007 in compliance with the EU landfill directive. This landfill is located 100m northwest of the new landfill and is outside the zone of potential impact from instability in the new landfill.

The mechanical waste treatment plant is located directly to the north of the new landfill. It comprises an industrial unit and associated infrastructure. The plant is constructed on a limestone plateau with an elevation of approximately 40m. Also in this area are several concrete water tanks, which are buried below the ground surface. The tanks are located approximately 10m from the vertical face of the new landfill. The locations and dimensions of the tanks have been discerned from assessment reports of the firefighting system. The depth is typically 5m and the widths / lengths are typically 15-20m.

The area to the southwest is arable farmland. It is understood that future development to extend the excavation of the limestone quarry into this area is being investigated in order to provide long term waste management solutions on the island.

Roads are located on all sides of the new landfill, with elevations between 40m to 60m. The crest of the slopes in the new landfill will have similar elevations, following existing ground level.

2. Conceptual model of ground conditions

2.1 Data collection

CQA has previously visited the excavation works to obtain data for design purposes. Ground profile information was provided by WSM. A detailed existing topographic model of the site has been produced and this has been used to model the current ground slope conditions. The information also included the historic 1970s land contours to allow analysis of the land-raises formed by historic landfilling.

Geotechnical data were available from a ground investigation completed by Terracore Ltd. (Report No, TERR18/WAS/J3018). This involved five boreholes drilled to between 8 and 12 m below ground level from the upper perimeter of the new landfill. The field and laboratory results obtained from this investigation have been considered when determining geotechnical design parameters.

A previous ground investigation was completed by Terracore Ltd. (Report No. TERR17/WAS001/J2829) at Ghallies landfill site to the north of the new landfill. Ten boreholes were drilled to depths of between 6 and 33m using open-hole and coring techniques. The field and laboratory results obtained from this investigation have been reviewed when determining geotechnical design parameters.

A series of laboratory tests have been completed on the limestone bedrock; these have been compared with empirical values for massively bedded limestone.

Since these visits, the depth of excavation has increased, and an additional site visit was considered necessary for the assessment of stability issues. This was undertaken on 21 and 22 November 2019 and involved an engineering geological inspection and the measurement of parameters to calculate rock mass assessment indices.

A further site visit was made in summer 2021, in connection with other matters on the site. At this time, the base of the excavation was approaching the design level and WSM were interested to learn if the depth of the landfill could be increased. CQA examined the strata exposed in the excavation and arranged for additional geotechnical testing to be undertaken.

CQA understands that the routine geotechnical inspection of the rock faces during excavation works is being undertaken by others on behalf of WSM.

2.2 Geology and hydrogeology

The limestone that is being excavated from the quarry is the Lower Coralline formation of Oligocene age. This stratum forms the basal limestone unit on the Maltese islands. The rock is typically moderately weak to moderately strong and massively bedded (Pedley and Buxton, 1991). The formation is approximately 100m thick on the west of the island, with a shallow dip in an eastern direction. There is a small outcrop of Globigerina Limestone formation, of Miocene age, at the top of the excavation in the south-eastern part of the site.

Karstic features are present in the limestone bedrock of the Maltese islands. These are typically present in the upper beds of the Lower Coralline Limestone (Pedley and Buxton, 1991). A horizon of karstic features has been found on site, at a depth of 20m. These features comprise irregular, partly connected voids with some staining by red clay. Some localised instability was associated with these features, and the excavation was revised to leave a bench in this area.

The karst is not expected to be very active in the present day due to the low rainfall, and groundwater resources are scarce. The water table is understood to be close to sea level in the area around site. As the base of the landfill excavation will extend down to 7m above sea level at the lowest point, groundwater is expected to occur 5m-7m below this level.

Groundwater is not anticipated to be encountered during the construction of the new landfill, and has, so far, not been encountered.

2.3 Geological inspection

The geological inspection involved detailed visual assessment of the rockmass on each of the excavation faces, as accessible from the benches. The rockmass characteristics were evaluated, including estimated strength, RQD and discontinuity details.

The limestone bedrock is generally massive, with few discontinuities. The extraction method involves breaking the rock by hydraulic chisel, which complicates the measurement and interpretation of natural joints.

The bedding planes are widely spaced and dip 5-10 degrees towards the south or southwest. Evidence of slope instability observed during the site visit was very limited on all excavation faces.

Detailed rock mass assessments were made along 15 “scanlines” at various locations and orientations in the excavation. The following geological features were observed during the site visit with annotated photographs in Appendix 3.

On the southern rock face (adjacent to the Ta’Żwejra landfill) irregularly shaped karstic solution features were evident, up to 20m long and 5m in width, with variable height. The solution features were generally open, with some infilling of debris and soil. The features appeared to align with a joint plane, which was steeply dipping out of the southern face. The solution features were located above a rock horizon which appeared to have lower permeability.

Further small-scale solution features were evident on the western rock face, again occurring above a suspected low-permeability horizon. The solution features are a concern for slope stability because they represent potential zones of low strength, which may facilitate localised failure of the rockmass in the excavation face.

The lack of discontinuities meant that there are relatively few joint-controlled breaks on the excavated faces. The discontinuities on the western face suggest the potential for wedge failures. Small-scale block failures are possible on all faces.

Some normal faults were observed on the eastern rock face, below the Magħtab landfill. Faults were visible over the entire 20m face height and were steeply dipping to the north-west, having an east-west strike and weaker material on the hanging wall.

The excavation was already at formation level below the MBT Plant. One fault was observed in the rock face, though no instability was apparent. Small wedge failures were observed beneath the RDF store, with a maximum block size of 2m x 2m.

2.4 Rock Mass Assessment

A rock mass assessment was carried out using a series of scanlines along the excavation benches. The rockmass was characterised using both the South African “RMR” system and the Norwegian “Q” system. The results are summarised in Table 1.

Table 1 Rock Mass Assessment

Scanline	Slope Direction	RMR No.	RMR Rating	Q- No.	Q- Rating
1	N (046°)	69	Good	37.5	Good
2	NE (314°)	45	Fair	3.8	Poor
3	NW (318°)	45	Fair	11.3	Good
4	NW (308°)	74	Good	37.5	Good
5	SW (236°)	82	Very Good	37.5	Good
6	SW (228°)	82	Very Good	37.5	Good
7	W (288°)	69	Good	37.5	Good
8	SW (232°)	69	Good	37.5	Good
9	S (188°)	69	Good	37.5	Good
10	E (104°)	55	Fair	11.2	Good
11	SE (124°)	67	Good	37.5	Good
12	N (014°)	79	Good	37.5	Good
13	N (360°)	79	Good	37.5	Good
14	NE (040°)	77	Good	37.5	Good
15	E (068°)	77	Good	37.5	Good

The assessment indicates the rock mass is generally good quality. Where fair or poor-quality zones were recorded these relate to the solution features in the southern rock face.

A total of 190 discontinuity measurements were made over a distance of 180m, representing a low intensity of joints. Most of the discontinuity orientations have strikes to the NE or NW, although there is some scatter in the results. Wide spacing of discontinuities is beneficial for stability because the strength of the rock material is generally greater than the strength of discontinuities.

Bedding is sub-horizontal and will have little effect on stability. The bedding has controlled the location of the solution features, which have formed along a single level.

2.5 Geotechnical Testing

Terracore report TERR18/WAS/J3018 describes five boreholes (BH1-BH5): four were on the eastern side of the site and one was on the western side near the promontory. These boreholes extended to depths of 7-12m. Laboratory testing included bulk density, water content and UCS on ten samples. The average bulk density was 2060 Mg/m^3, and the average UCS was 12MPa.

Terracore report TERR17/WAS001/J2829 describes ten boreholes (BH3, BH5, BH7, BH9, BH11, and BHA-BHE). Boreholes A-E were inside the hazardous waste cell and were very shallow with no laboratory testing. The other boreholes were located around the site: three on the eastern side, one on the southern side and one on the western side near the promontory. These boreholes extended to depths of 19-33m. Laboratory testing included bulk density, water content and UCS on fourteen samples. The average bulk density was 2230 Mg/m^3. The results from BH7 and BH9 were higher, and the average without these boreholes was 2130 $Mg/^{m3}$. The average UCS was 24MPa. The results from BH7 and BH9 were higher, and the average without these boreholes was 12.4MPa.

The reports do not provide coordinates or collar elevations of any borehole. CQA has estimated approximate elevations from the location plans. This indicates that the test samples represent a narrow range of elevations, mostly between 30-40m above sea level, with 29.4m being the deepest sample. The anticipated basal elevations of the landfill excavation are 7m in the centre and 12m at the foot of the vertical faces.

The lower 20m of the excavation was not covered by any laboratory strength data. In summer 2021, in coordination with WSM, CQA organised a series of fourteen Schmidt Hammer tests in the excavation. These tests are designed for in-situ assessment of concrete, but the results can be converted to UCS using a suitable formula based on correlation studies. CQA selected the formula proposed by Katz, which was based on result from similar limestone rocks.

During this testing, CQA noticed that the rock mass properties changed with depth, becoming softer and more friable towards the base of the excavation. This observation was supported by the Schmidt Hammer results, which reduced with depth. All available UCS results are presented in Table 2. The variation of UCS with depth is illustrated in .

Table 2 UCS test results from the site

Location	Elevation, m	UCS, Mpa	Location	Elevation, m	UCS, Mpa
Terracore, 2018			CQA Schmidt Hammer tests		
BH1	38.6	10.1	SH1	40.0	28.2
BH1	37.1	14.4	SH2	35.0	12.6
BH2	39.0	11.4	SH3	31.0	14.4
BH2	35.4	12.4	SH4	12.0	5.6
BH3	43.6	6.8	SH5	12.0	6.5
BH3	43.2	7.8	SH6	20.0	6.0
BH4	50.2	15.1	SH7	28.0	9.6
BH4	48.4	13.1	SH8	22.0	14.4
BH5	43.2	16.2	SH9	30.0	13.5
BH5	40.8	12.4	SH10	37.0	15.4
Terracore, 2017			SH11	13.0	11.8
BH11	39.7	18.4	SH12	29.0	13.5
BH11	39.5	14.3	SH13	34.0	11.8
BH11	39.2	8.8			
BH11	37.9	8.3			
BH11	35.6	6.7			
BH11	35.4	7.6			
BH3	30.2	17.3			
BH3	29.4	9.6			
BH5	32.3	15.3			
BH5	31.6	17.4			
BH7	41.6	38.2			
BH7	39.6	58.7			
BH9	34.1	54.3			
BH9	33.6	62.3			

2.6 Design Parameters

Geotechnical design parameters were required to carry out numerical analyses of slope stability. Key parameters are bulk unit weight, strength and water pressure. The characteristics of discontinuities are also important.

The Bulk unit weight relates to the limestone bedrock and the effect of nearby loading, such as the waste in the old landfills and the access road. The values used in the analyses have been derived from the Terracore reports. The density was assumed to reduce with depth, together with the strength of the rock.

The required strength parameters for the analyses should represent the rock mass, and these are usually estimated from intact rock properties. Suitable intact properties include cohesion (both drained and undrained), friction angle and shear strength of the intact rock. Of these, only the latter (as UCS) has been determined. The other properties have been estimated by CQA on the basis of typical relationships and equivalent rock types.

Discontinuities play a key role as they may be planes of lower strength and preferential movement. These have been assessed by the rock mass assessments.

Groundwater can influence stability by increasing the unit weight of the strata and by reducing the drained shear strength.

Values have been assigned to the design parameters from published information, based on the site inspection and the available ground investigation data. The selected values for the design parameters are summarised in Table 3.

Table 3 Design parameters for analyses

Lithological Unit	Bulk Density (Mg/m^3)	UCS of intact rock (MPa)	Cohesion of rockmass (MPa)	Friction Angle of Rockmass (^{0})
Limestone bedrock – above 35m elevation	2070	13	0.9	40
Limestone bedrock – 20m-35m elevation	2050	9	0.4	40
Limestone bedrock – below 20m elevation	2030	6	0.2	40
Old waste in the adjacent landfills	1500	-	0.020	18

The values have been used to conservatively estimate the global slope stability of the side slopes of the new landfill to its proposed design depth. As groundwater was not encountered during the investigation and is not expected to be above the formation level of the new landfill, the slopes have been modelled as unsaturated.

The conditions of existing slopes can provide verification of parameters and recommendations. The excavation faces are currently prepared at an angle of approximately 85 degrees with a height of approximately 25m. No failures have occurred. Similar slopes are evident in similar strata in Malta, suggesting that the rock mass can support. near-vertical faces.

3. Proposed design of the new landfill

3.1 Overview

The side slopes of the new landfill will be close to vertical and will extend from elevations (relative to mean sea level) of between 40m to 60m down to a basal level of 12m at the edges and 7m in the centre of the landfill. The composite basal liner of the landfill will be formed from standard landfill construction techniques, with a mineral liner of compacted clay overlain by geosynthetic membranes and leachate collection system meeting EU landfill regulations.

The proposed vertical lining system will comprise a light-weight steel frame to support the composite liner of geomembrane and a bentonite-cement grout. The grout will be placed between the excavation face and the geomembrane, supported by a system of steel mesh and geotextile. This containment system will also meet EU landfill regulations.

The steel frame could be constructed fully at the outset or be extended in sections as the waste level rises need to be supported. The proposed design is for the frame to be fixed onto the excavation face using specially drilled rock anchors. If anchors were previous installed for stabilisation purposes, the frame may be fixed to these. The proposed solution for stability would is to extend the frame upwards in sections ("lifts").

3.2 Design summary

The landfill will comprise one disposal cell. Details can be found in the following documents: Specification and CQA Plan, the "Construction Management Plan" and the detailed drawings.

There is no natural geological barrier on site. The sub-grade for the containment system is the Attard member of the Lower Coralline Limestone formation. This formation is an aquifer, although it is not used for groundwater abstraction in the site area and the groundwater is expected to be brackish.

The design includes the installation of an artificially established mineral barrier, which will comply with the requirements of the Landfill Directive. On the base of the landfill there will be a "conventionally placed" engineered mineral liner, augmented by a geosynthetic clay liner, as described in Sections 4 & 7 of the Specification and CQA Plan. On the side slopes of the landfill, the mineral liner will be bentonite-enhanced concrete. Please refer to Section 5 of the Specification and CQA Plan.

The artificial sealing liner will be formed from 2mm HDPE geomembrane sheets, welded together to form a continuous barrier, as described in Section 8 of the Specification and CQA Plan.

The HDPE geomembrane on the base will be covered by a protection system comprising heavy-duty geotextile and a graded sand layer, as described in Section 9 of the Specification and CQA Plan. The protection system on the sides will be formed from either baled selected waste or filled bulk bags, to be decided in the operational plan.

3.3 Leachate and water management

The leachate collection system will comprise a layer of uniform-sized gravel with slotted pipes leading to the extraction sumps as described in Section 10 of the Specification and CQA Plan. The leachate extraction wells are shown on drawing 30374-WSM-SW-FD-14 of the Specification and CQA Plan.

The Magħtab Environmental Complex already has a leachate management system. The new landfill will be integrated with this system.

The design of the capping system includes a geocomposite drainage layer to provide surface water drainage.

A groundwater management system is not included in the design. Groundwater is not expected to be encountered in the excavation and construction works, or within 1m of the basal liner.

There is no leakage detection system in the design.

3.4 Subgrade

Apart from some sections around the upper perimeter of the landfill, the formation subgrade on the base and sides of the landfill will be freshly excavated into strata of the Lower Coralline Limestone. The bedrock is expected to have very low compressibility, probably less than 1mm from the planned depth of waste.

There are some karstic cavities at a depth of approximately 20m in the sides of the excavation, at the boundary between the upper Xlendi member and the lower Attard member of the Lower Coralline Limestone formation. The karstic processes are not expected to be very active at present, due to the dry climate. No cavities have been found in the lower levels of the excavation. It is considered unlikely that any would develop in the foreseeable future: due to the base being relatively close to th groundwater level; and the expected brackish-saline nature of the groundwater.

The potential for basal heave in the landfill is very low and so this is not a likely mechanism to affect the integrity of the sub-grade.

The presence of discontinuities (joints, faults etc) within the slope is a key aspect of the stability and is described in detail in this report. If the promontory is excavated by blasting, the final face in this area may be uneven and weakened and will probably need to be shaped before installation of the liner. Loose rocks may need to be scaled if this cannot be done during excavation.

The lining system on the sides of the landfill will be installed in phases as the height of the waste increases.

The small area of sloping sidewall, in the north-eastern corner, will be covered with an adapted lining system. This design will need to be confirmed during construction, in coordination with the development of the site road system, which will also be modified in this area.

Groundwater levels will be several metres below the base of the side slopes and will not greatly affect on stability.

3.5 Lining system

As the subgrade is rock, with a relatively low stress field, the magnitude of either basal heave due to excavation or settlement due to loading will be minimal and will not affect the stability of the landfill nor the integrity of the basal liner.

Cavities are not expected in the basal subgrade. The cavities in the upper part of the side slopes will be partially filled by bentonite-enhanced concrete. The slope will be supported by the ground-anchors and, within a short time, by the placement of waste. The construction methodology will reduce the risk of instability and any potential effects on the liner.

The basal liner will be a composite containment system involving both mineral liner and artificial sealing system. The slope angles on the base are very low and sliding on the interfaces in the containment system is not likely to occur.

The vertical side slopes will be lined with a composite containment system, comprising a bentonite-enhanced concrete mineral layer (500mm minimum thickness) and a geosynthetic sealing layer (2mm HDPE geomembrane).

The materials will be supported on a steel frame and interface properties will not affect the stability or integrity of the containment system.

The containment system will be constructed in lifts, typically 3m high, which will be supported by waste relatively quickly. The geosynthetic liner will be temporarily anchored at the top of each new lift.

Imposed shear stresses on the liner will be relatively low due to the vertical orientation. The protection materials will reduce transfer of any friction during settlement of the waste.

3.6 Operations

The new landfill is being constructed in an operational waste management complex with full management procedures in place. These procedures will be extended to cover operations in the new landfill.

The placement of waste into the cell is an operational issue, which although this will be influenced by the design. The proposed construction methodology is to install the side slope liner in 3m high lifts as the waste is placed.

As a result, temporary slopes in the waste should have a height of only a few metres and the risk of instability will be low.

The limited height of the waste slopes will mean that any recirculation of leachate will not affect the slopes.

The waste disposal methodology will be conventional placement and compaction in layers, with cover as required to control the risk of wind-blown materials.

We anticipate that the final waste profile will be raised, with pre-settlement side slopes no steeper than 1:3. This will ensure that the waste does not become unstable.

The height and grade of the landfill surface will reduce due to compression of the waste with time and so no instability issues are expected post-settlement.

Therefore, no impacts are expected on the collection of leachate or landfill gas.

The leachate collection system will include a drainage blanket, slotted pipes and vertical extraction shafts, which are constructed as the waste level rises.

3.7 Capping

The design of the capping system is a composite liner, with mineral and artificial sealing layers. The risk of instability has been greatly reduced by the design.

The design uses standard materials used in landfill cap construction for which the interface properties are well known. The design is formulated to avoid these properties being exceeded. The materials will not be subject to reduced integrity when used in landfills.

The capping design allows for settlements in the waste to ensure that the integrity is not compromised.

Gas management wells will prevent the build-up of pressures. These will be connected to the gas management system of the site.

The new landfill is being constructed on an operational waste management site for an experienced operator. Procedures will be in place to ensure that gas is managed and that damage to the cap is avoided.

4. Screening of instability risks

4.1 Lifecycle phases

The stability of the landfill excavation has been assessed in three phases, relating to the key stages in the lifecycle of the development:

Construction

Operations

Closure

The construction phase involves the current excavation works to produce the usable void. The removal of the RDF is also included in this phase. Thereafter, construction will involve the installation of the conventional landfill lining system on the base of the landfill and the framework for the steep wall lining system on the side walls. The access road will be built, and the leachate control system will be installed.

The excavation to formation level is a critical phase for stability. The side walls will reach maximum height and the risk of instability will be greatest at this time. A bench has been retained on the southern face due to a reported potential for instability on specific joints. Further stabilisation measures may not be required due to the planned short life span of the faces and the relatively low numbers of personnel who will work inside the landfill. Removal of the promontory may affect stability if the rockmass is disturbed by the excavation method. The possible effect of the underground water tanks on stability has also been assessed.

The operational phase will involve filling the landfill with waste. The vertical lining system will be completed in stages as the level of waste rises and the side walls are more easily accessible. Waste delivery vehicles will not drive close to the slopes, but the compactor and construction equipment / personnel would need to approach the vertical faces.

As the landfill becomes filled, the slope height will reduce as the waste provides a supporting pressure on the covered faces. The lining system will also protect the rockmass and may reduce the infiltration of water after rain. The stability risk will reduce during operations.

Slope instability will not be an issue during closure, as the faces will be fully supported.

4.2 Identification of key stability risks

The potential stability risks on this site have been screened to ensure that the assessment is focussed on the key issues. The screening has been described in this report and is summarised as follows:

Element	Summary of screening
Basal Sub-Grade	Bedrock with adequate shear strength Not compressible or subject to swelling Low risk of voids or groundwater No stability issues to be addressed
Side Slopes Sub-Grade	Bedrock with adequate intact shear strength Properties will be defined by the discontinuities Some cavities in the upper part of the slopes (particularly southern and western sides) Not compressible or subject to swelling Low risk of groundwater Stability issues need to be addressed due to the steep slopes Possible rockmass disturbance by excavation of promontory Possible effect of buried water tanks
Basal Liner	The stability of the basal subgrade will result in no impacts on the basal liner. No stability issues to be addressed
Side Slopes Liner	The side slope liner will be supported on a steel frame, fixed to the side slopes. The stability issues of the side slopes will also relate to the side liner system.
Waste	The waste will be placed in low lifts, which will not have stability impacts No part of the side slopes will be formed from older waste The design of the final profile includes grades and pre-settlement levels that will be stable with typical municipal waste Post-settlement levels are not expected to result in any increased slope grades or other issues. No stability issues to be addressed
Capping	The design of the final waste profile includes grades and pre- / post-settlement levels that ensure the slope angles are less than the capacity of the interface friction of the materials No stability issues to be addressed

The results of this screening are that the key stability risk is presented by the side slopes in the landfill excavation. The stability risk is described in this report.

4.3 Potential failure modes in the rockmass

It is important to predict the potential mode of failure in a rock mass in order to assess the potential for instability. The most likely modes are assessed to be as follows:

Wedge sliding	Possible when two persistent discontinuities which slope out from the face intersect to define a prism of rock. If the angle of the line of intersection is steeper than the friction angle, sliding may occur.
Toppling	Possible when persistent discontinuities occur which are near-vertical and parallel to the face. Other factors include the strength of intact rock on the base and sides.
Collapse	Possible if voids are present in the rock mass, which undermine overlying strata. The potential for failure is related to the size of voids, and strength of the rock mass to bridge any voids. Some solution features have become visible in the cut faces, which may produce voids.
Pseudo circular	Possible if there are many discontinuities at different orientations, which allow the rock mass to act like a soil. Although there are few discontinuities in this rock mass, this mechanism may be relevant due to the scale factor.

4.4 Stability of the lining system

In addition to the cut faces of the void, it is necessary to address the stability of the lining system. On gentle-grade side slopes this involves the assessment of frictional forces that will occur between the layers of the lining system: soils, geosynthetics and waste. These are compared to the out of balance forces in the slopes, which may cause movement.

This approach is not relevant to the steep wall lining system. In this case, the essential issues are:

Fixing to the rock	The steel frame of the lining system will need to be fixed to the rock face with sufficient strength to support the forces during the lifecycle. The proposed design is to attach the frame using rock anchors.
Weight of the liner	The steel frame will need to support the static load of the lining system, as this is constructed in phases.
Live loads	Exposed elements of the lining system will need to support live loads which may be imposed, such as from wind, seismic events and impacts by equipment.
Compaction of the waste	The settlement of waste may cause some vertical shear forces in the lining system, although these will be reduced by the steep angle of the faces.

4.5 Adjacent sites

The Magħtab landfill northeast of the new landfill is constructed as a land raise, infilling the existing valley.

The Ta'Żwejra landfill is directly southeast of the new landfill. This is an engineered landfill constructed in 2004 to EU guidelines. It is envisaged this was constructed with standard side slopes of 1:3 (vertical: horizontal).

The mass of waste in these landfills will not affect the stability of the new landfill excavation.

The MBT Plant of the Magħtab Environmental Complex is situated at natural ground level above the northern side wall of the new landfill. The water tanks for firefighting are also located in this area.

Instability in the excavation for the new landfill could affect these adjacent sites, and this is considered further in the risk assessment.

5. Stability assessment

5.1 Approach

The potential for wedge sliding and toppling has been assessed by graphical analysis of discontinuity measurements and geotechnical observations. The data have been analysed using the software package "Stereonet" by Richard Allmendiger.

The potential for collapse has been assessed by geotechnical assessment and rock mass analysis.

The potential for pseudo circular failure has been processed using slope stability software, SLOPE/W, part of the Geostudio suite. Limit equilibrium slip circle analysis has been performed (Janbu, 1956) to provide a global factor of safety. The sensitivity of variations in bulk density and peak angle of shearing resistance have been modelled to ensure a conservative approach, also allowing for a seismic load. The results are presented graphically in Annex A. Drawing 30374/WSM/SA/01, which shows the location of sections used to derive topographical parameters, is attached to this report.

Geomechanical analysis has been performed using FLAC3D® software. This finite-difference modelling package can analyse rockmass stress distributions and responses using a number of constitutive models, such as elastic, Mohr Coulomb and Hoek & Brown. The sections modelled were the western face near the promontory and the northern face adjacent to the water tanks. The eastern face adjacent to the existing landfills was also modelled. Graphical results are included in Annex B.

The lining system stability has been calculated by geotechnical methods, based on the rock mass assessment and design parameters.

The main issues relate to any instability which may be caused during excavation of the new landfill to formation level and if at base depth there will be instability in the surrounding landfills and MTP. Therefore, the side slope stability has been assessed along the southern, eastern and northern boundaries. A worst-case scenario of final target depth, unsupported cut face has been assessed along each orientation.

Three sections have been analysed to assess the consequence of slope failure in the new landfill on the Magħtab, Ta'Żwejra and MTP. The maximum slope design slope heights have been modelled with 85-degree slope angles.

5.2 Discontinuity-controlled mechanisms

The risk of instability resulting from discontinuity-controlled movement will primarily be present in the construction phase, when unsupported faces are at maximum extent. CQA has carried out screening analyses by stereonet methods. A kinematic stability analysis will be required to fully assess toppling and wedge failure. CQA understands that this is being completed by a geotechnical consultant.

CQA's observations of the potential for discontinuity -controlled movements are as follows. These are illustrated by the stereonets in Annex C.

East face	Relatively stable configuration of joints
	Moderate risk of wedges with small volume.
South face	Relatively stable configuration of joints
	Low risk of wedges with small volume (moderate in karstic zones)
West face	Stable configuration of joints, steeply dipping and perpendicular to face
	Low risk of wedges with small volume
	Possible joint orientation parallel to face, presents some risk of toppling, if release surfaces are present.
	Discontinuities and karstic features on the promontory could reduce stability during additional excavation works
North face	Relatively stable configuration of joints
	Joint orientation parallel to face, presents risk of toppling, release surfaces not so common.

Discontinuities are generally widely spaced. This will reduce the probability of sliding blocks being formed, thereby increasing stability. This will also facilitate the identification of locations with potential for block sliding failure.

5.3 Potential collapse

CQA observed karstic features in the mid height of the southern and western faces in the excavation. The karstic features seem to have formed above a specific bedding plane in the limestone. The features have formed by groundwater dissolving the limestone over a long period of time.

The karstic features could affect stability in two ways:

- The features could enlarge discontinuities, increasing the risk of sliding
- Larger features could collapse from the force of overlying strata

Neither of these stability issues were observed on the rock faces. This indicates that the rock strength is sufficient to support the hanging wall above the voids. However, the excavation works may alter the stress conditions around some solution features, and it is necessary to assess and, if necessary, mitigate these features.

The risk of instability as a result of collapse will primarily be present in the construction phase.

This applies particularly to the promontory, which will be excavated after the main face is completed. The discontinuities and solution features in this area could lead to instability as a result of equipment working above or below the face. This short-term construction risk will need to be addressed in method statements for the work.

Mitigation measures will be appropriate at this time. The risk may occur during the operation phase if mitigation is not carried, possibly if voids are not detected.

The rock bolting and netting would provide some support. However, the most suitable approach, if access is possible, would be to close the entrance of each void with masonry and fill the open space with strong mortar-grout. If this is not possible, a sprayed concrete lining could be a suitable alternative.

5.4 Pseudo-circular mechanism

The results obtained from the limit equilibrium analysis for the critical conditions are as follows:

Adjacent to Magħtab	Global factor of safety of 1.6 at the maximum slope height of 45m.
Slope adjacent to Ta'Żwejra Landfill	Global factor of safety of 2.1 at the maximum slope height of 49m.
Slope adjacent to MTP	Global factor of safety of 3.5 at the maximum slope height of 32m.

A typical factor of safety for short-term slope stability is between 1.3 – 1.5 (Trenter, 2001). The results indicate the cut faces of the landfill void will be stable at final formation depth to an acceptable level.

5.5 Stability issues during operations

As the new landfill is lined and begins to be filled, theoretically the slopes will become more stable as weight is being added to the toe (i.e. base) of the faces. The lining will also prevent precipitation entering the limestone, reducing dissolution risk.

Each rock face should be regularly assessed during excavation by geotechnical engineers. Suitable guidance would be the UK Health and Safety at Quarries Regulations 1999, Approved Code of Practise (HSE, 1999), in the absence of other systems.

Each rock face should be extensively inspected prior to liner installation to ensure the overhead rockface conditions are not prone to rockfall. This may require rope-access inspections should dissolution features be present. The area of the promontory may be especially prone to this risk due the late excavation and the possibility that scaling could not reach all loose rocks. This area may also be uneven and require final grading prior to installation of the lining system.

5.6 Potential impacts on adjacent structures

Five buried tanks are present near the northern face and the northern part of the eastern face and act as reservoirs for firefighting water. The tanks are constructed from reinforced concrete (base, sides and roof) with concrete columns to support the roof. The tanks are approximately 5m deep and are approximately 10m from the face of the excavation at the closest points.

The tanks represent areas of reduced loading on the excavation faces, due to the previous removal of soil and rock. The estimated 4m depth of water will replace less than 50% of the load. The changes in stress from changing water level will be small compared to the properties of the rock, and any stresses at depth and resulting elastic deformations will not affect the rock face.

Modelling results also show that deformations in the rock face due to the excavation will not affect the concrete tanks.

The roads around the landfill perimeter will not be affected by the small deformations of the ground surface in response to excavation. The deformations will be very small (mm scale) compared to the tolerances of the structures.

The adjacent landfills will not be impacted by any small deformations o the ground resulting from the excavation. If blasting is required to remove the promontory, the charge design and blasting sequence should be made to reduce the risk of flyrock.

5.7 Stability issues during closure

Stability issues are not anticipated during or after closure works. This would need to be reassessed if further landfill cells are excavated to connect to this cell.

6. Suggested ground control measures

6.1 Face stabilisation

The rock faces are expected to be mostly stable in the short to medium term after excavation. However, issues may occur in the longer term and so it will be important to construct the landfill as soon as possible after excavation is completed.

If local instability does occur, the most likely modes are:

- Toppling on the north face, and possibly west face
- Wedge sliding on the east face, and possibly on the south and west faces
- Collapse into contiguous solution features, along joint planes

If stabilisation measures prove to be necessary, the suitable methods would be:

- Rock bolts in a regular pattern on the north and west faces
- Rock bolts in locations determined by geotechnical survey on the south and east faces
- Supporting solution features by masonry and grout or shotcrete (as feasible)

The installation of any ground support will need to be coordinated with the excavation of the void and future expansion plans. For example, extension of the landfill in land to the west may be impeded if ground support and a lining system are installed on the western face. The decision concerning expansion will need to be made promptly to allow the lining system to be designed.

6.2 Rock bolts

Rock bolts act by imposing forces onto the rock face which prevent movement and would be a suitable method to support any sections of excavated faces which are deemed to be unstable. Rock bolts typically comprise textured steel bars with threaded external ends. These are installed slightly oversized, horizontal drill holes. The forces are produced by tightening nuts, which act against the section of the rock bolts that are anchored into the rock strata by expanding sections or epoxy grout. To be effective, rock bolts must be drilled sufficiently far into the strata that the anchored section is outside the zone that may be affected by failure. The design of the rock bolts will need to be verified by a series of trial pull-tests.

Rock bolts can be installed in a regular pattern over a rock face. This system is preferred when there are many discontinuities and the position of each cannot be predicted. This system is also suitable for failure modes such as toppling and pseudo-circular. Rock bolts can also be installed to support identified and delineated zones of potential instability, such as preventing the sliding of wedges between known joints.

It is recommended that if rock bolts are installed in a pattern, these shall be connected by a stiff steel mesh. This would have the immediate benefits of providing support to the whole face and catching possible loose rock.

6.3 Shotcrete, masonry and grout

If any solution features appear to be unstable, the risk could be reduced by closing the open faces of any voids with masonry. Suitable materials would be concrete blocks or engineering bricks, fixed in place with a strong mortar. After the mortar is hard, the voids behind the walls should be filled by pumping in a strong mortar grout. If this method is not feasible, due to access restrictions, a sprayed concrete solution may be suitable. For example. shotcrete is concrete that is sprayed onto a rock face to provide support prevent deterioration and reduce the risk from falling blocks. The tensile strength of the concrete can be increased by steel or fibre reinforcement.

We understand the WSM has retained engineers to monitor the excavation and they will be able to decide this issue on site, as required.

6.4 Steep wall lining system

The steep wall lining system will be supported on a steel frame. This will be constructed in lifts as the level of waste increases. The frame will be fixed to the face of the excavation using specially installed rock bolts. In addition to supporting the containment system, the rock bolts will also provide additional ground support.

6.5 Monitoring and inspection

It is recommended the excavations and rock faces are inspected on a regular basis by a competent person. The UK Health and Safety at Quarries Regulations 1999, Approved Code of Practise (HSE, 1999) provide helpful guidance for such inspections.

7. Preliminary Risk assessment

7.1 Risk receptors

Risks from slope instability will arise when the potential for instability creates hazards on receptors such as:

Persons	Construction workers and operators (long exposure)
	Delivery drivers, engineers (short exposure)
Infrastructure	Perimeter road around the landfill
	The lining system (from slope instability)
Equipment	Construction and operational plant
	Delivery vehicles
Natural environment	Encroachment onto land, loss of containment

7.2 Geotechnical risk register

A geotechnical risk register has been defined in Table 5, based on the scoring system in Table 4. This identifies the hazards and potential risks from ground movement on the proposed works. This risk register indicates that a number of scenarios are unsatisfactory in an unsupported excavation, but that satisfactory stability can be achieved by suitable mitigation measures.

The mitigation measures, using the techniques summarised in this report, will need to be designed. When the design details are known, the risk assessment will need to be repeated with the actual parameter values to provide the working risk assessment.

Table 4 Risk scoring and ranking system

Probability (P)		Impact / Consequence (I)		Risk Rating (P x I = R)		Response
Very Likely	5	Very High	5	Intolerable	17 to 25	Unacceptable
Probable	4	High	4	Intolerable	13 to 16	Unacceptable
Likely	3	Medium	3	Substantial	9 to 12	Early attention
Unlikely	2	Low	2	Tolerable	5 to 8	Regular attention
Negligible	1	Very Low	1	Trivial	1 to 4	Monitor

Table 5 Geotechnical risk register

Potential Risk	Cause	Initial			Mitigation Measures	Hazard rating after mitigating measures			Description of residual risk
		Likelihood	Impact	Risk		Likelihood	Impact	Risk	
Large-scale collapse exposing historic landfills Magħtab & Ta'Żwejra	Rock face failure causing loss of containment in surrounding landfills	3	5	15	Pattern rock bolting as required , monitoring of rock faces during excavation and operations, confirmation of stability risk assessment	1	5	5	Variations in rock mass behind the rock face
Large-scale collapse causing loss of foundation of MTP or water tanks	Rock face failure causing damage to existing infrastructure	3	5	15	Pattern rock bolting as required , monitoring of rock faces during excavation and operations, confirmation of stability risk assessment	1	5	5	Variations in rock mass behind the rock face
Toppling failure in rock face – rockfall	Discontinuity orientations and solution features	3	4	12	Pattern rock bolting as required, monitoring of rock faces during excavation and operations, confirmation of stability risk assessment	2	2	4	Variations in rock mass behind the rock face
Wedge failure in rock face – rockfall	Discontinuity orientations, , release surfaces in rock mass	3	4	12	Isolated rock bolting as required, monitoring of rock faces during excavation and operations, confirmation of stability risk assessment	3	2	6	Variations in rock mass behind the rock face. Possibility that some blocks were not identified
Collapse of rock face due to solution features	Solution feature size and location, effect of excavations	3	4	12	Masonry walls and grouting as required, monitoring of rock faces during excavation and operations, confirmation of stability risk assessment	2	2	4	Variations in rock mass behind the rock face
Failure of side-lining system	Pull-out of lining frame. Pull-out of rock anchors. Leakage of bentonite-cement grout	3	4	12	Full pull-out analysis and on-site testing during installation. Limited installation height to ensure mineral liner is not overfilled	1	4	4	Variations in rock mass behind the rock face

8. Conclusions and recommendations

As with any deep, vertical excavation, the extension of the former quarry to enable construction of the new landfill involves the risk of instability of the rock faces. Such instability would potentially impact people working in and around the landfill site; and may also affect nearby facilities, including closed landfill sites, the waste treatment plant and the water reservoir tanks.

The most likely modes of instability are block sliding and potential collapse of solution features. However, ground conditions are generally favourable and the mitigation measures will be responses to identified problems, rather than general stability measures. Any mitigation measures should be designed and installed by a specialist company.

The probability of block sliding is low as a result of the wide spacing of discontinuities and the generally favourable orientations. None of the small solution features were observed to have collapsed, and this risk is also low.

If necessary, the mitigation measures block sliding will comprise rock bolting and stiff steel mesh to provide support and protection from small falling rocks. Mitigation measures, if required for solution features, will include masonry to close any openings and the placement of stiff grout behind the walls, or sprayed concrete where this is not feasible.

The mechanical excavation methods produce relatively little disturbance of the rockmass behind the face, allowing a smooth profile to be formed. This excavation method reduces the risk of instability.

The composite lining system will be placed against this smooth face.

If blasting is used, such as to remove the promontory on the western side, there is higher risk of disturbance of the rockmass in this area. This may increase instability risk locally and also require cleaning of the face prior to liner installation.

Regular inspections of the excavation by a geotechnical engineer are recommended, leading to recommendations for any mitigation measures that are considered necessary. Additional geotechnical data should be collected during the site visits.

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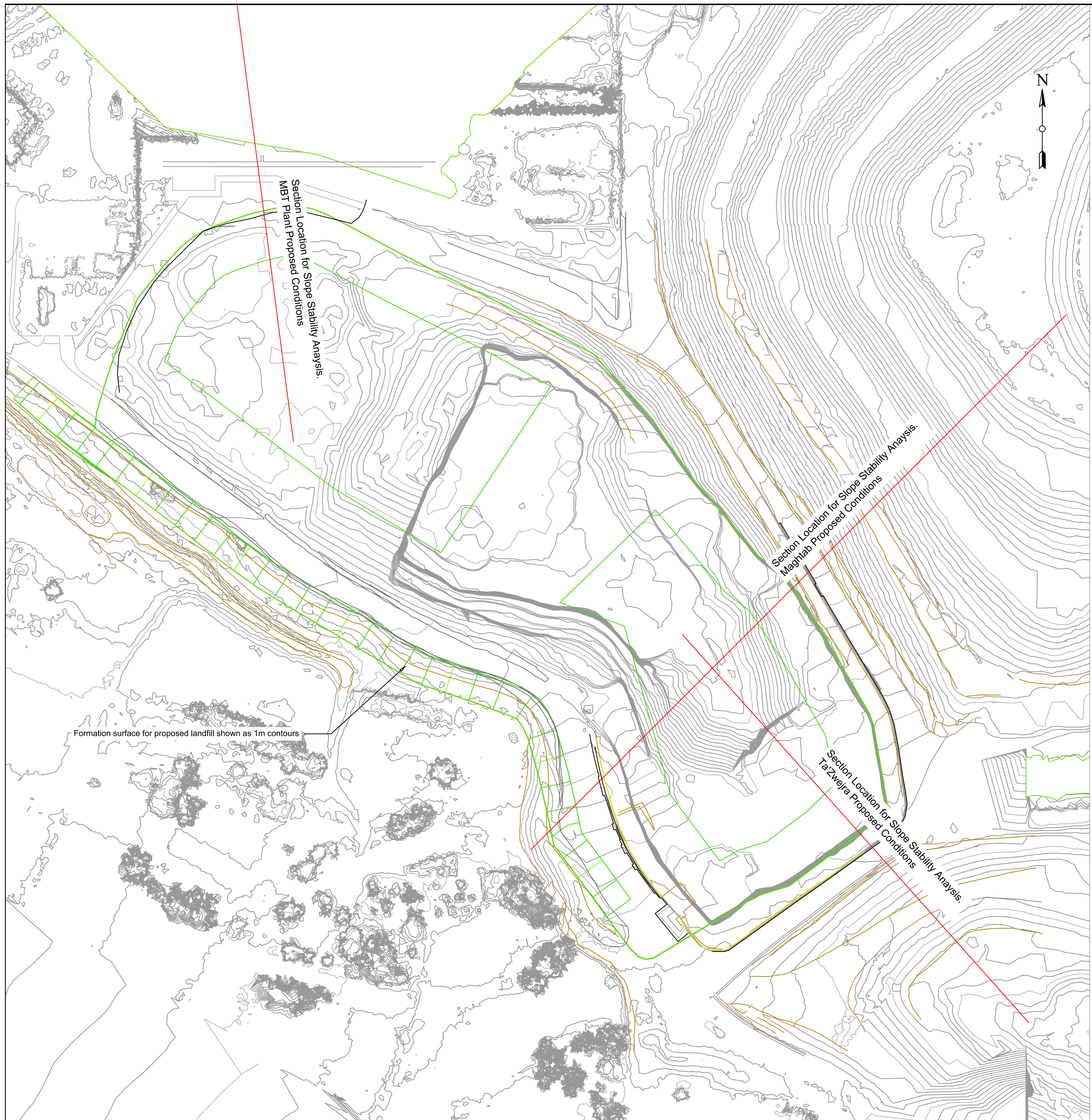
Annex A Slope stability Modelling Output

Please refer to attached drawing 30374/SWM/SA/01 for sections used in calculations.



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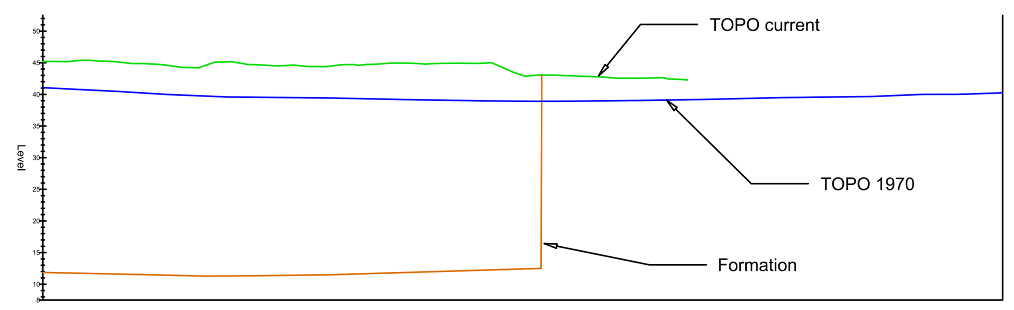

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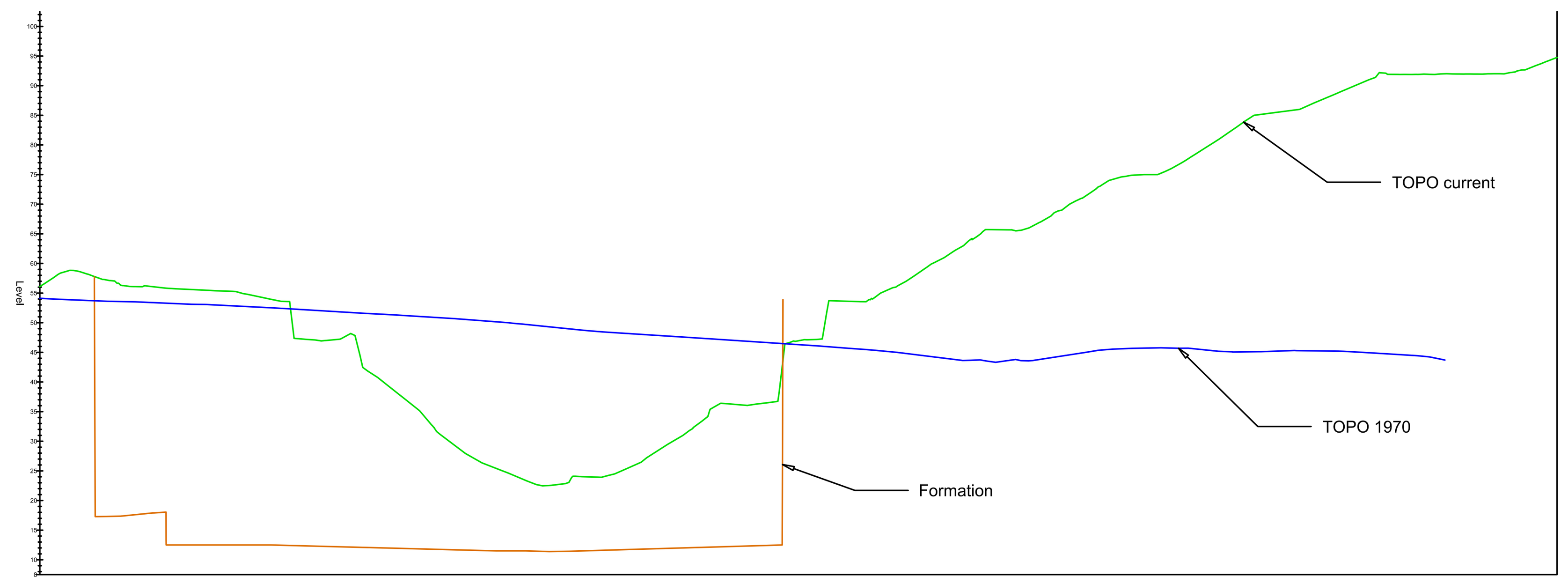
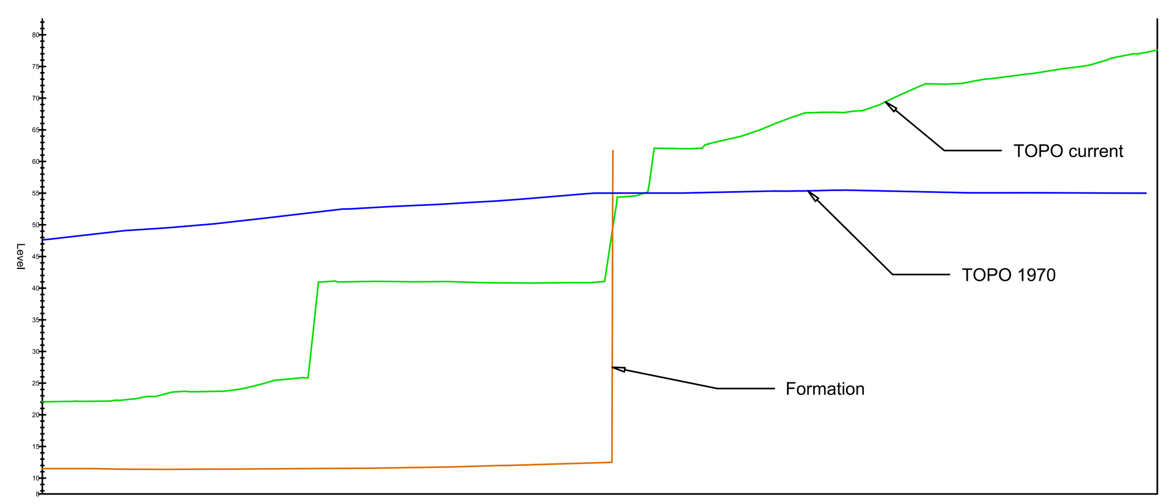
Maghtab Proposed Conditions



Section for Slope Stability Anaysis.
Ta'Zwejra Proposed Conditions



NOTES

Topographic survey utilised June 2019

WASTESERV
CREATING RESOURCES FROM WASTE

PROJECT TITLE
Construction of a Steep Wall
Non-Hazardous Waste Cell

PROJECT NUMBER
30374

CLIENT
Wasteserv Malta Ltd.

LOCATION
Maghtab Environmental Complex

DRAWING TITLE
Section Locations & Sections for
Stability Analysis

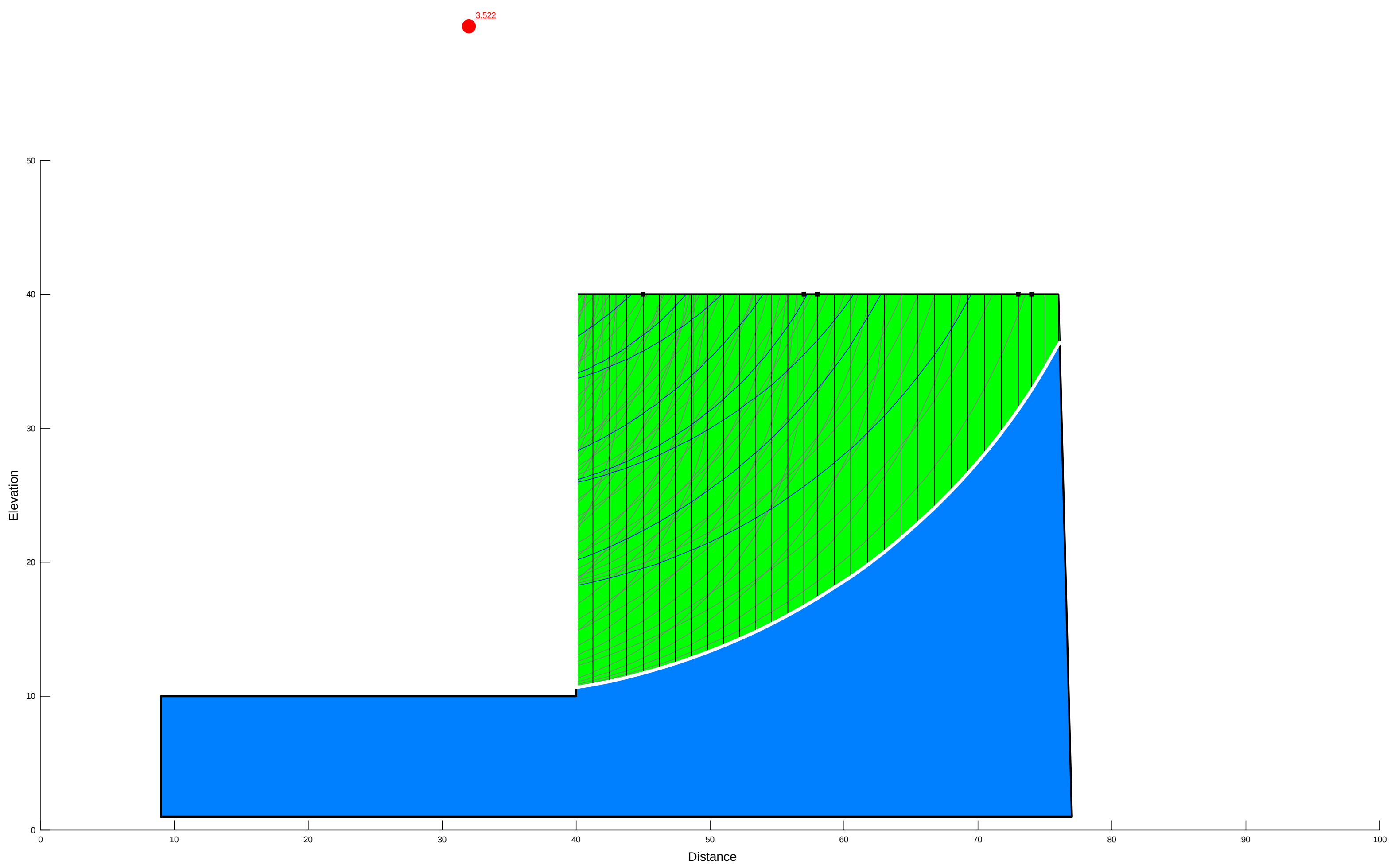
REVISIONS

Rev	Date	Chkd	Description
0	16/12/2019	DB	First issue

DESIGNED	RWS	DRAWN	RWS
SCALES @ A1	1:1000	DATE	16/12/2019

CQA
CQA International limited

DRAWING NUMBER	REV
30374/WSM/SA/01	00



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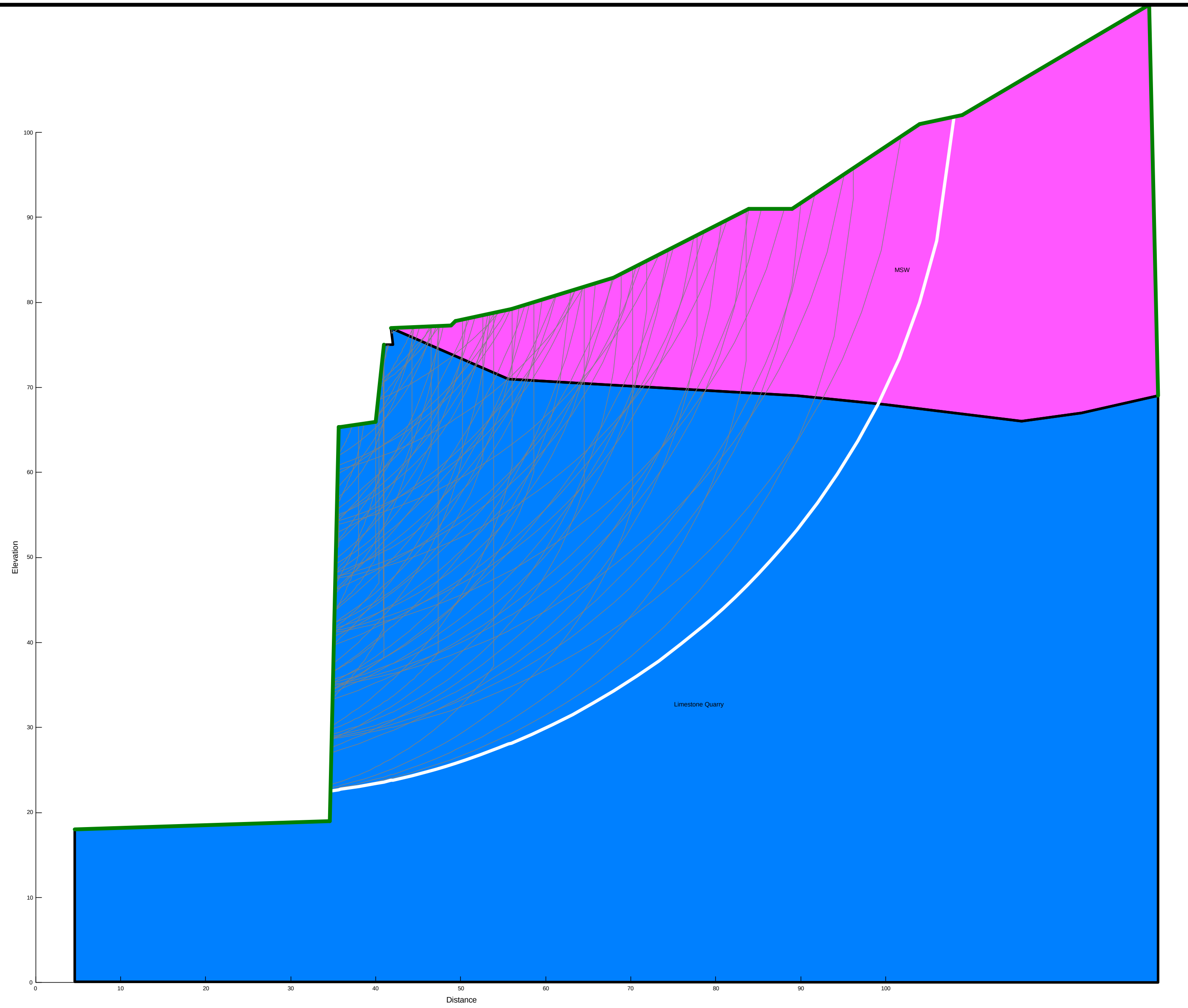

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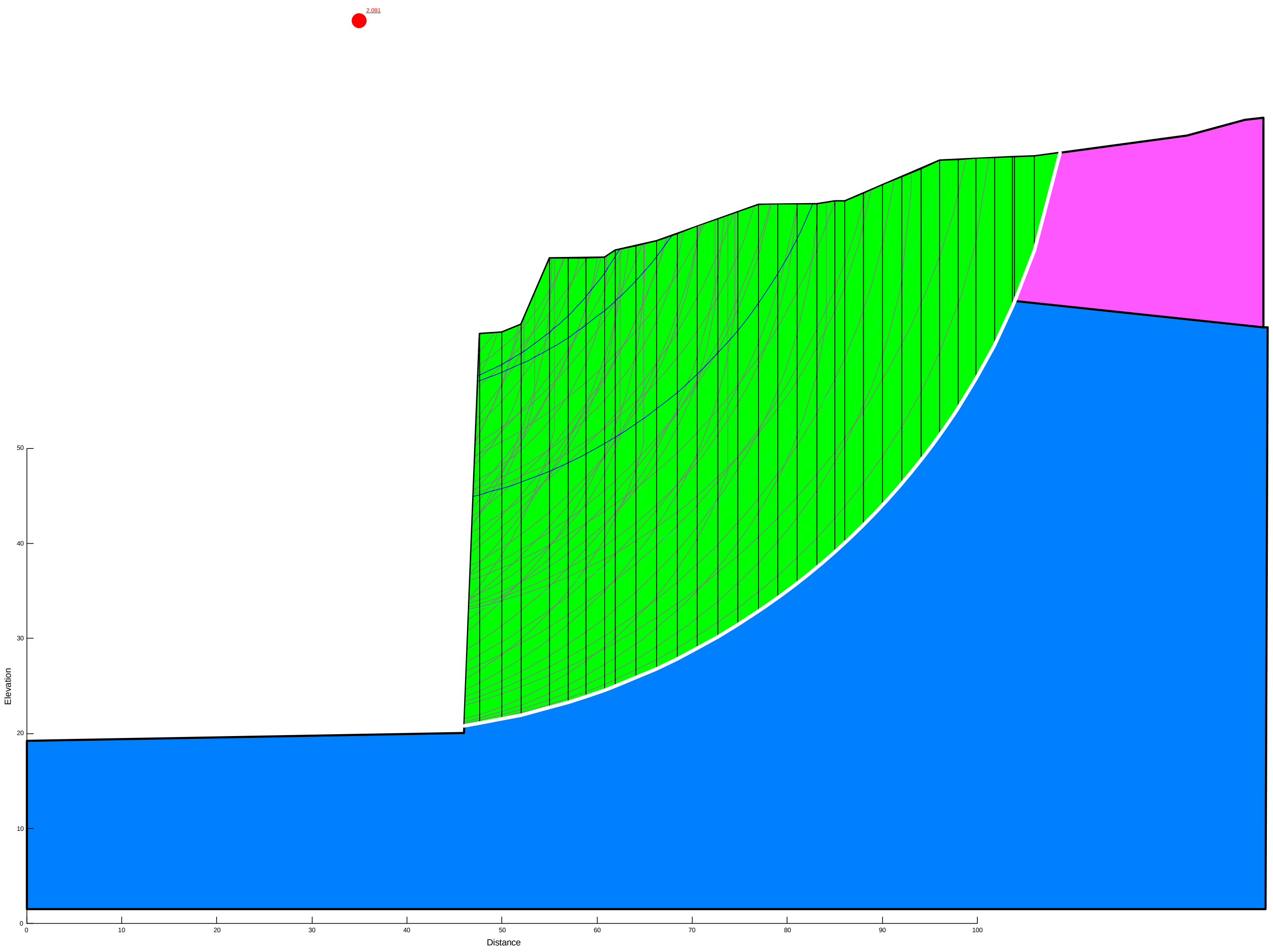


Slope Stability

Zwerja Proposed condition.gsz

17/12/2019 1:750

Annex B Geomechanical Modelling Output

Deepest part of excavation, with access ramp

Block model of deepest excavation, with access ramp, after removal of promontory. Blue = lower unit, Green = middle unit, Red = upper unit (refer to section on design parameters for values).

Maximum stress in rockmass, showing locally high values at the toe of the higher parts of the slope

Ratio of rockmass strength to the maximum stress – highest stress at tow of slope but not sufficient to cause failure

State of finite difference elements at end of simulation – light blue colour state indicates lack of instability

Displacements due to deepest excavation near access road and promontory; movements very small, insufficient to cause issues

Northern part of excavation adjacent to a buried water tank

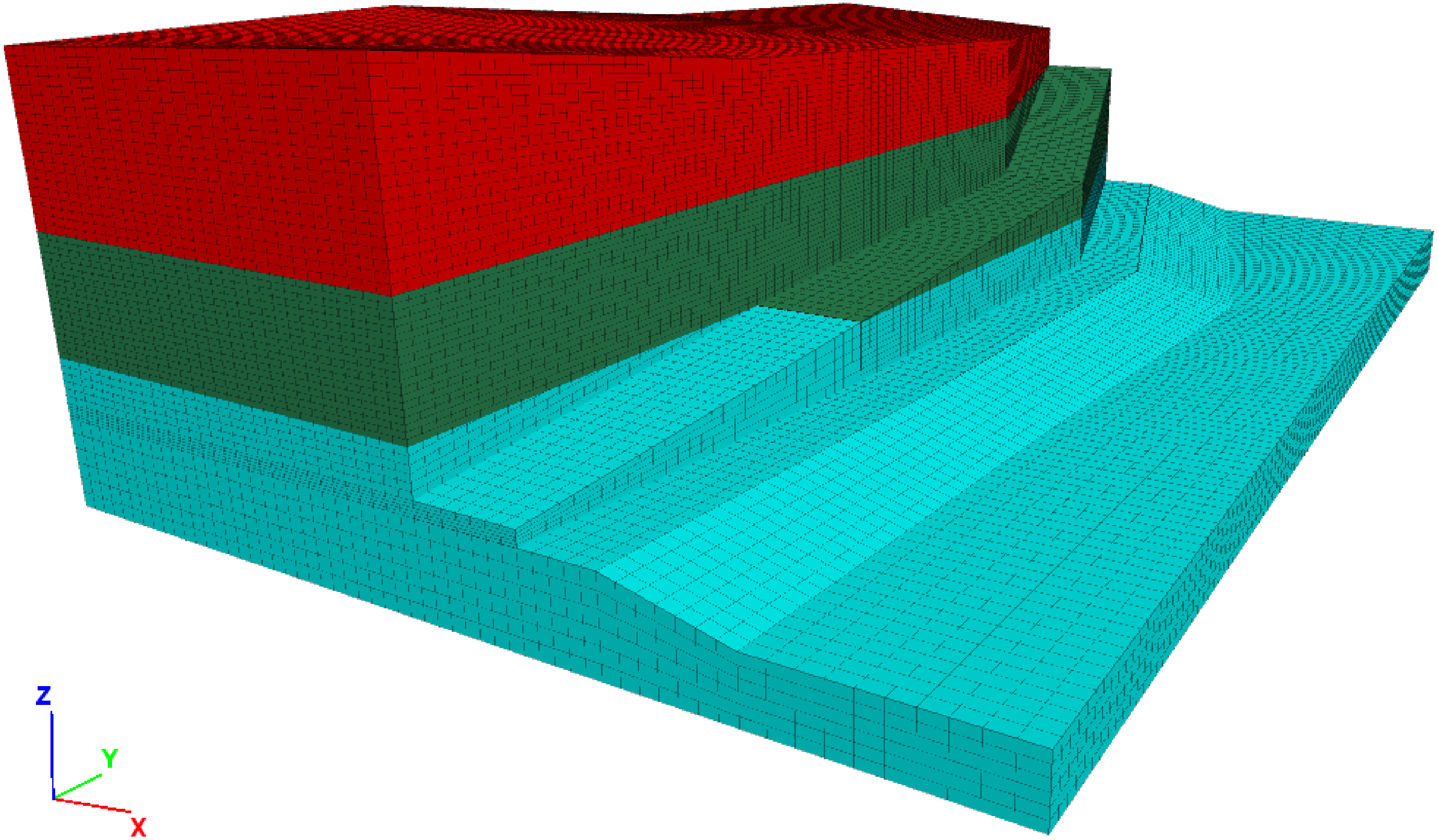
Block model of northern part of excavation adjacent to a buried water tank. Blue = lower unit, Green = middle unit, Red = upper unit (refer to section on design parameters for values).

Maximum stress in rockmass, showing reduction under tank and locally high values at the toe of the slope

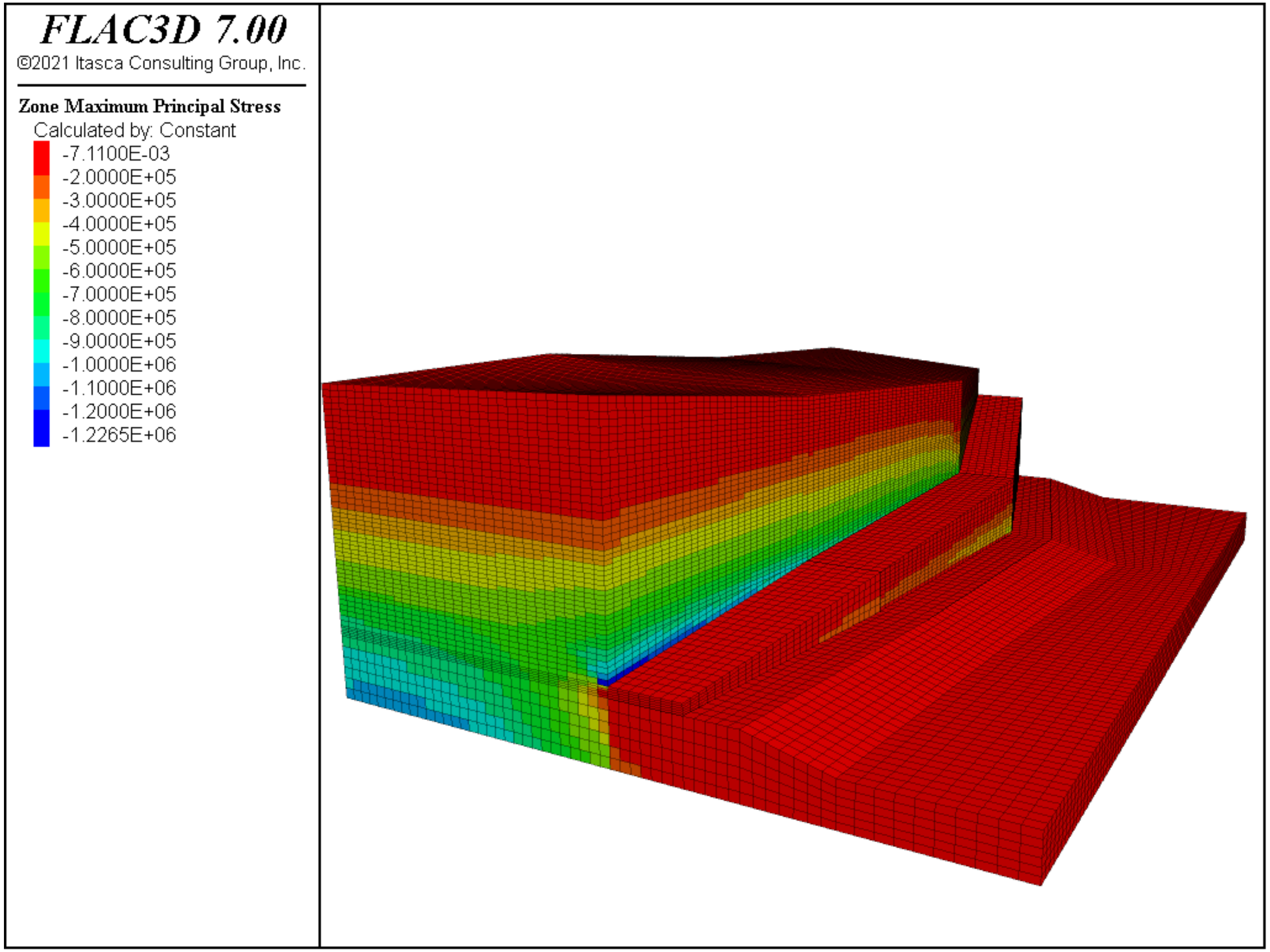
Ratio of rockmass strength to the maximum stress – highest stress at tow of slope but not sufficient to cause failure

State of finite difference elements at end of simulation – blue indicates lack of instability. Neither tank nor slope are affected.

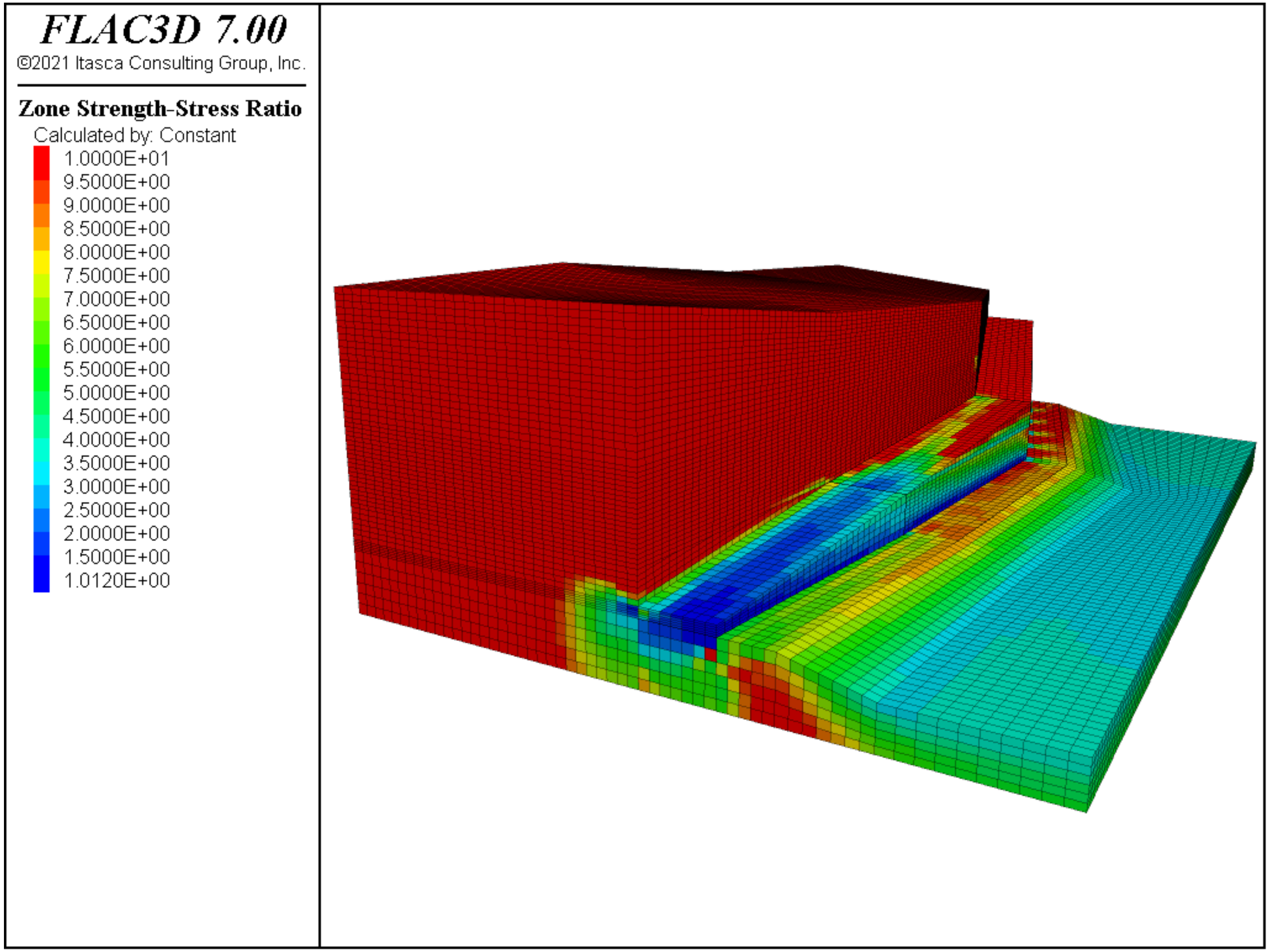
Displacements due to excavation near a reservoir tank; movements very small, insufficient to cause issues.



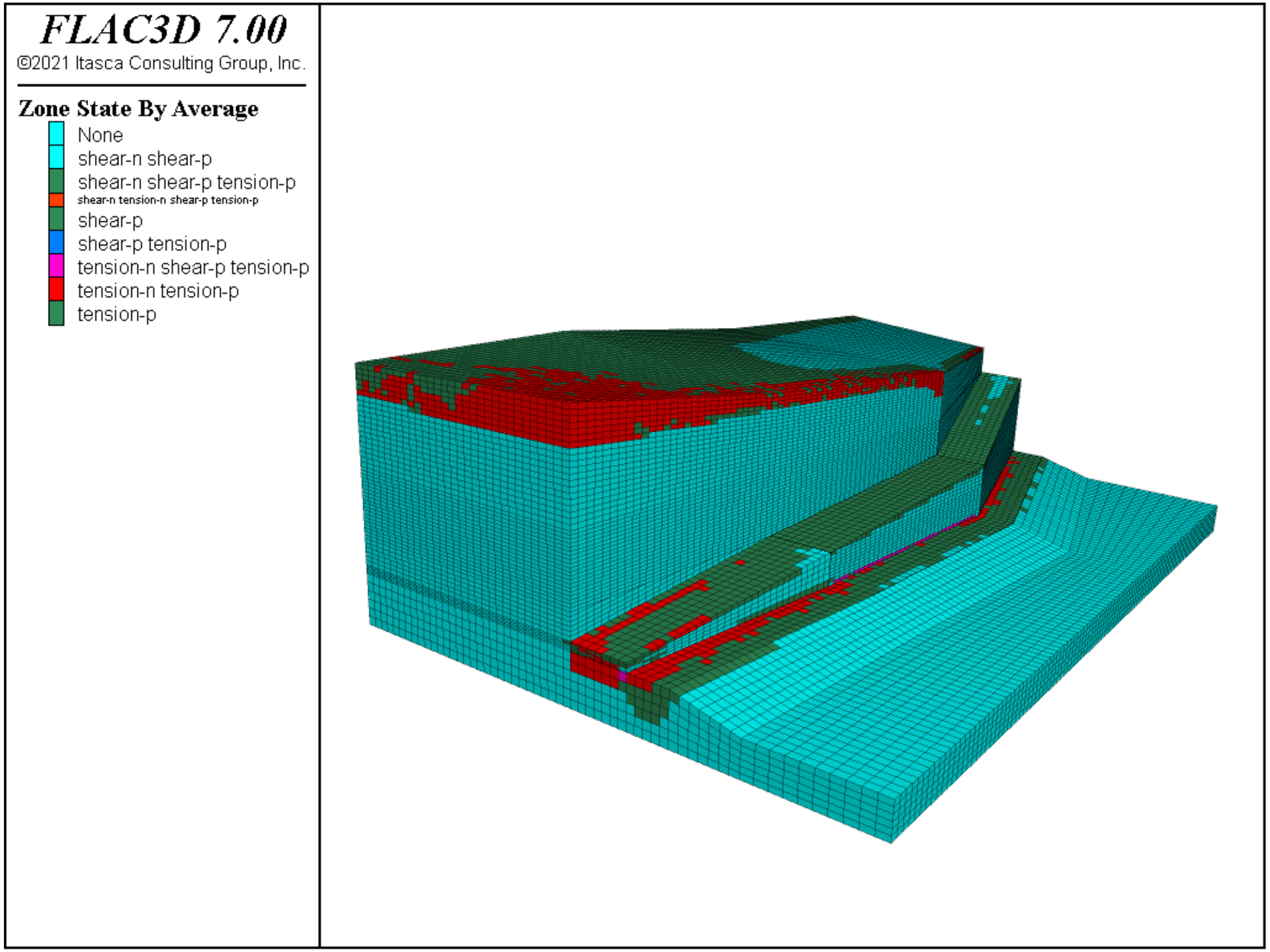
Block model of deepest excavation, with access ramp, after removal of promontory. Blue = lower unit, Green = middle unit, Red = upper unit (refer to section on design parameters for values).



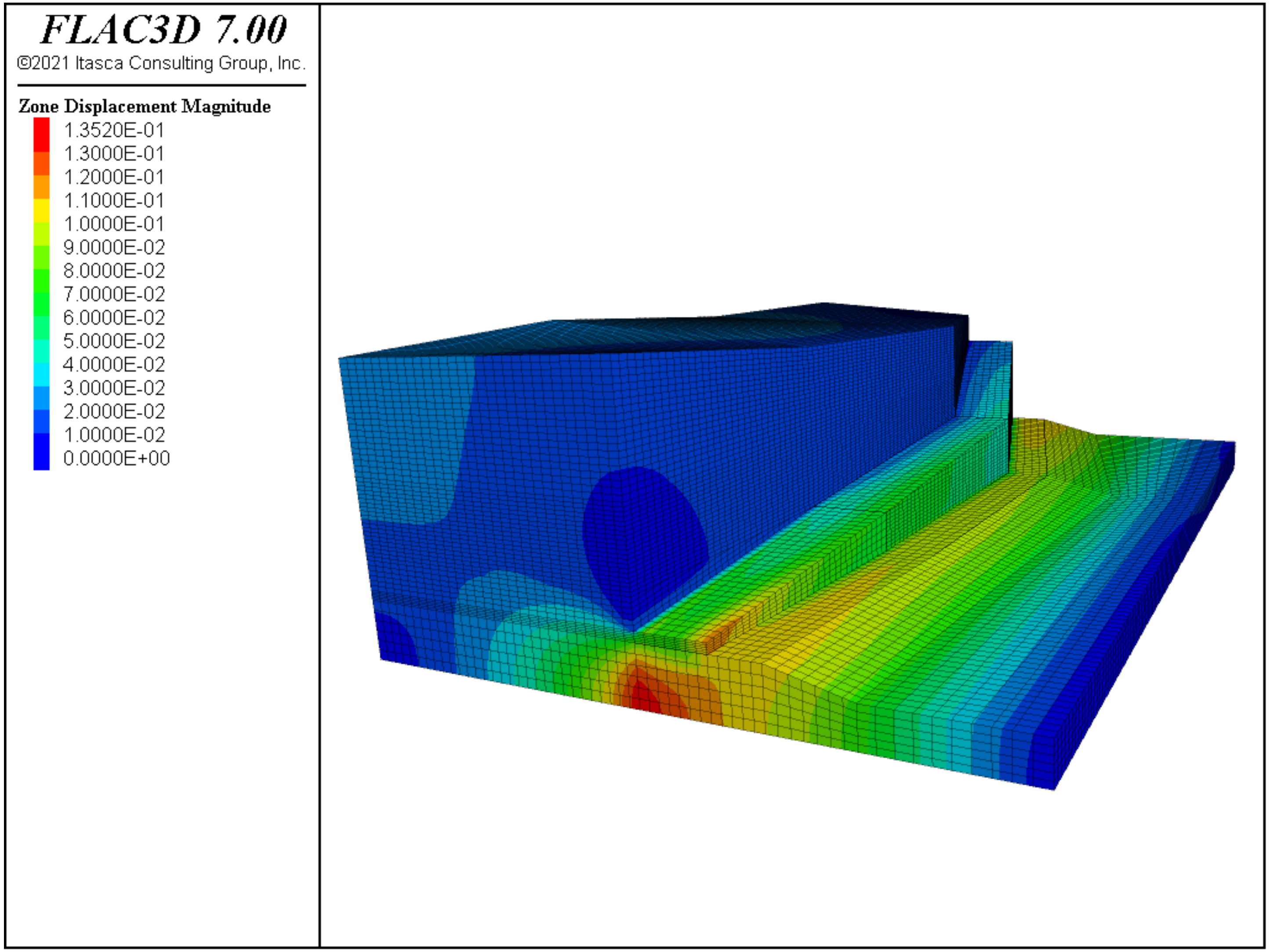
Maximum stress in rockmass, showing locally high values at the toe of the higher parts of the slope



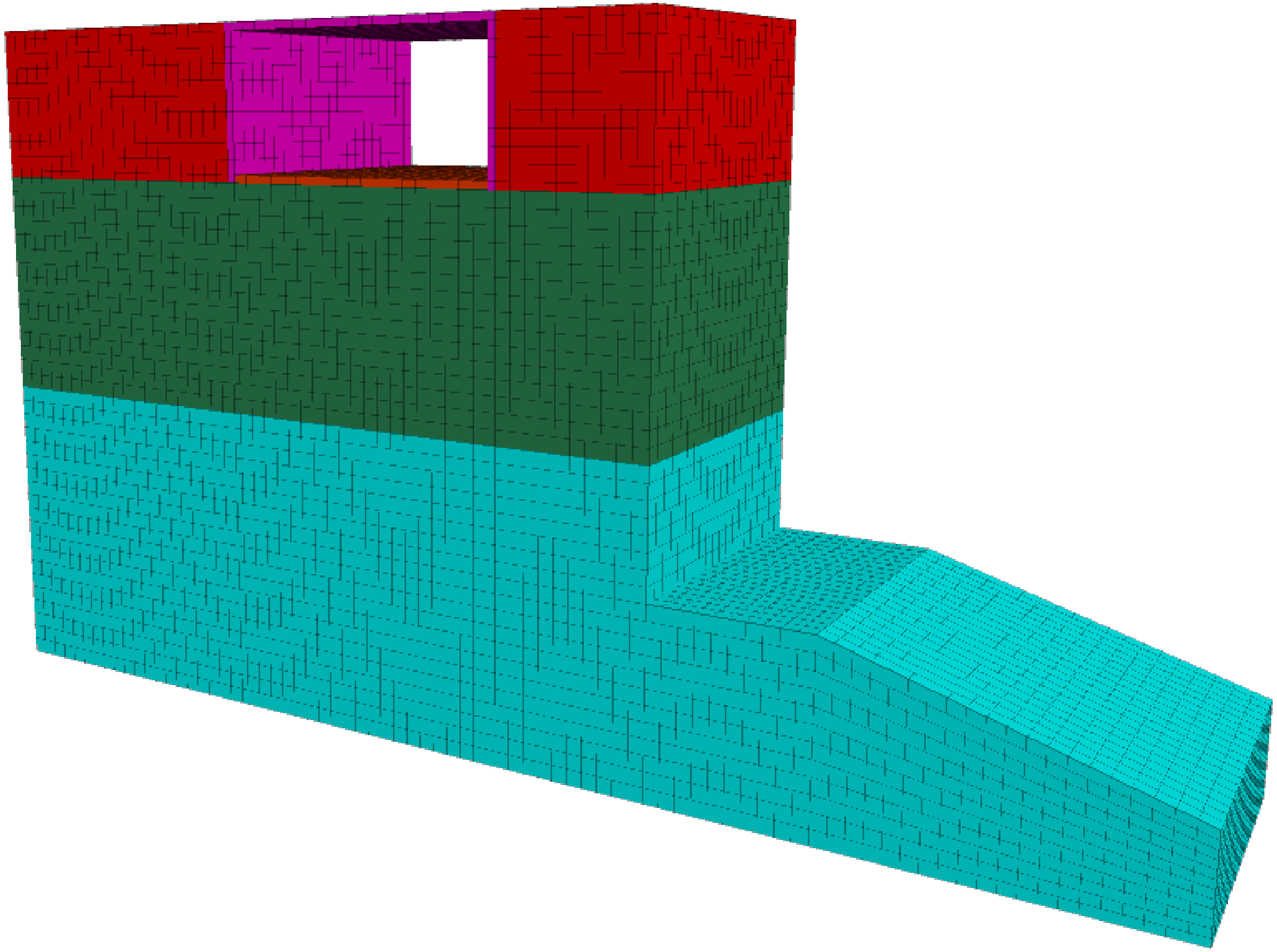
Ratio of rockmass strength to the maximum stress – highest stress at tow of slope but not sufficient to cause failure.



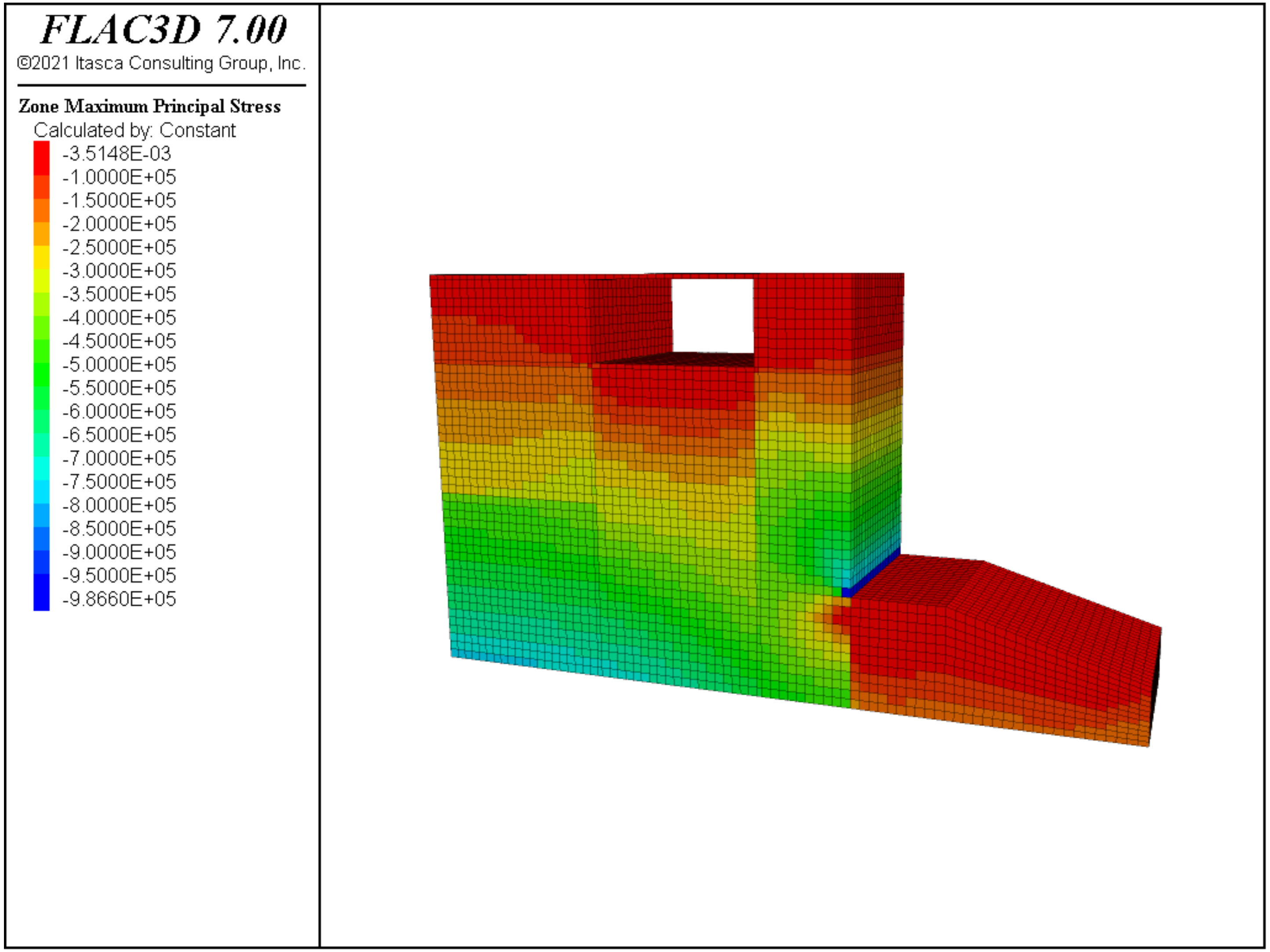
State of finite difference elements at end of simulation – light blue colour state indicates lack of instability.



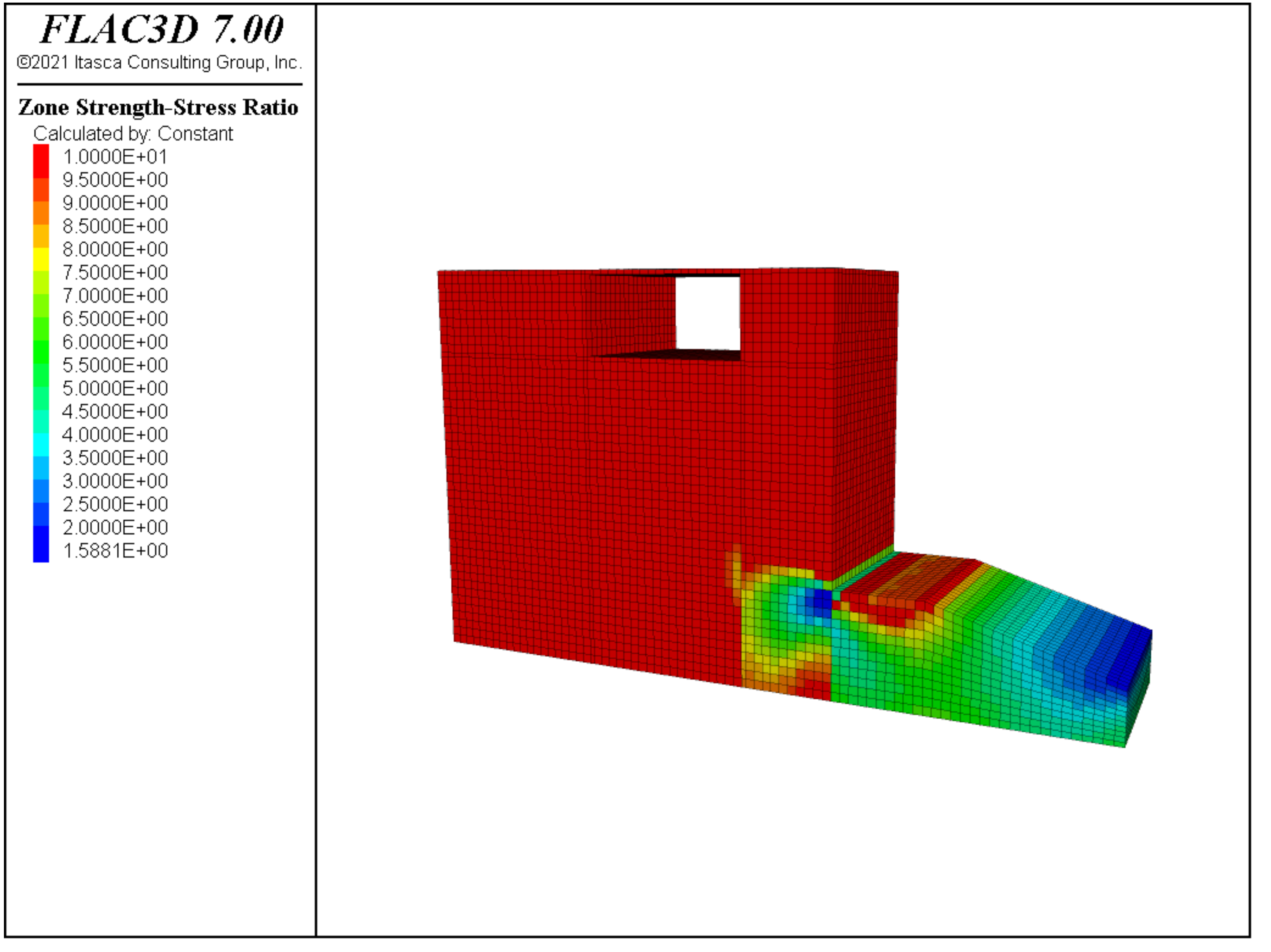
Displacements due to deepest excavation near access road and promontory; movements very small, insufficient to cause issues.



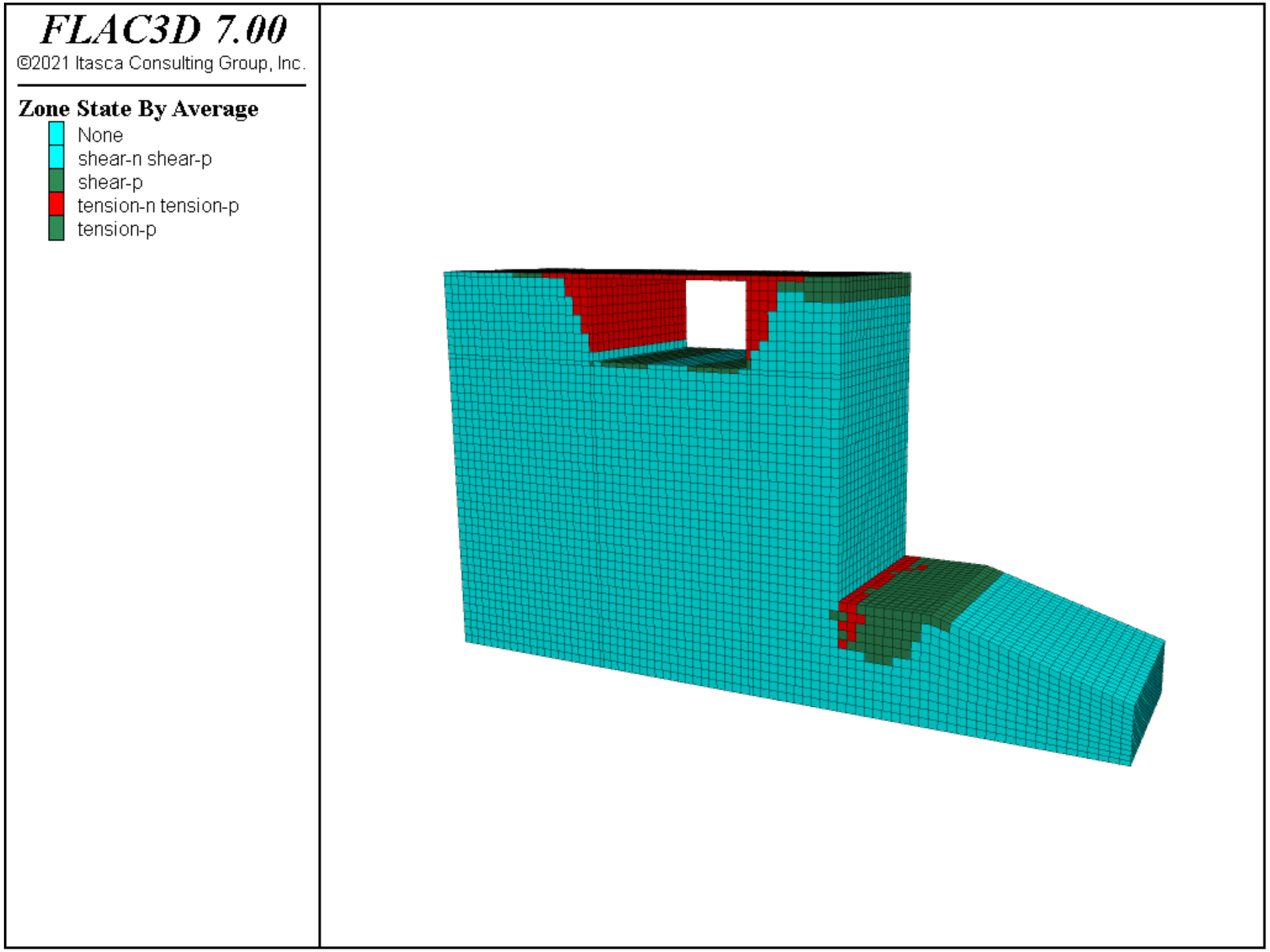
Block model of northern part of excavation adjacent to a buried water tank. Blue = lower unit, Green = middle unit, Red = upper unit (refer to section on design parameters for values).



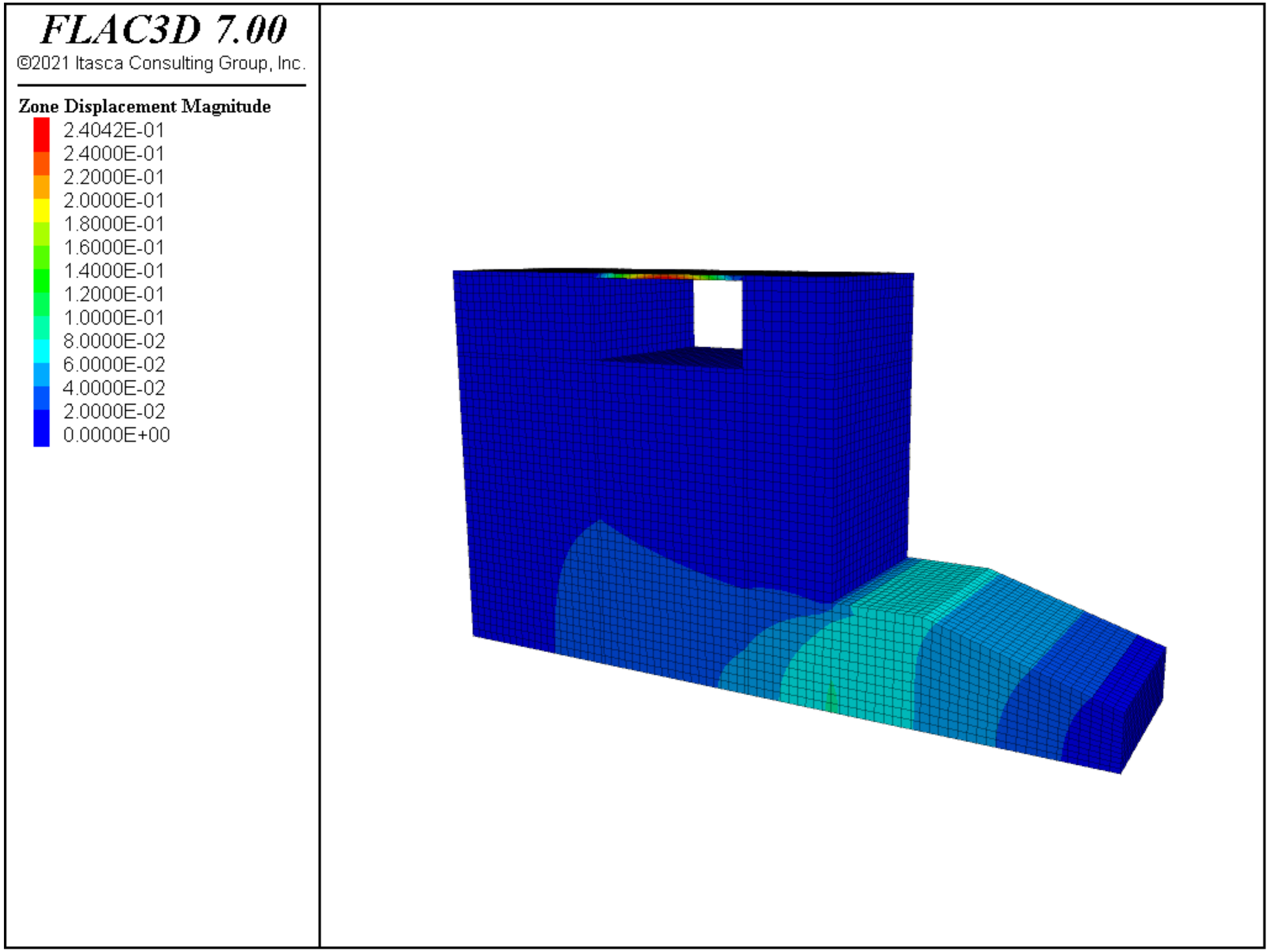
Maximum stress in rockmass, showing reduction under tank and locally high values at the toe of the slope



Ratio of rockmass strength to the maximum stress – highest stress at tow of slope but not sufficient to cause failure.



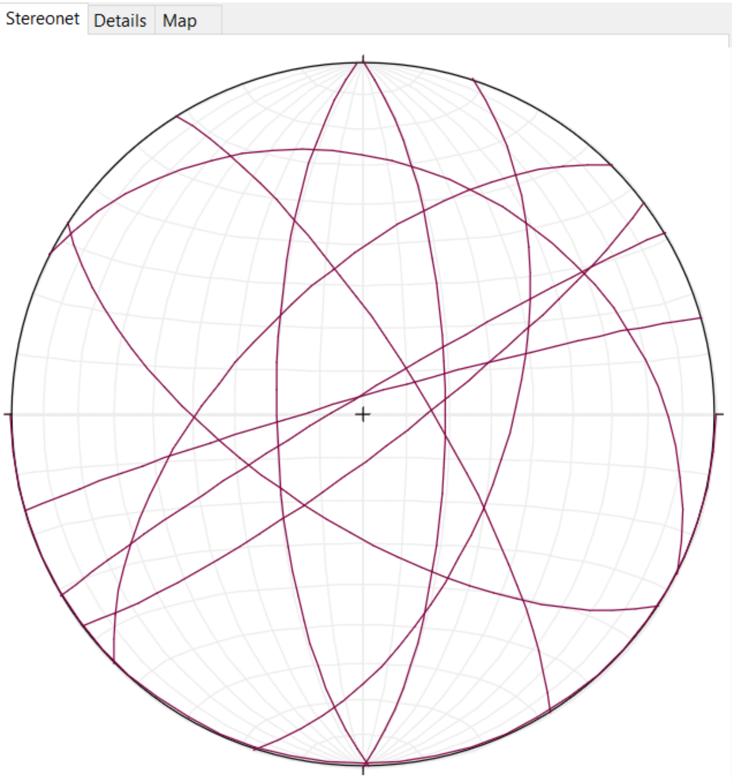
State of finite difference elements at end of simulation – blue indicates lack of instability. Neither tank nor slope are affected.



Displacements due to excavation near a reservoir tank; movements very small, insufficient to cause issues.

Annex C Stereonet summaries

All discontinuities



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☐	Main planes	Lines	TP
☑	planes from poles to Main plan...	Planes	AD

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4	033.0	25.0
5	089.0	20.0
6	323.0	09.0
7	288.0	34.0
8	270.0	19.0
9	238.0	14.0
10	207.0	60.0
11	000.0	89.0

----------10/12/2019 at 09:44----------
Analysis of 196 planes from file 'All data.txt'

---------- Poles from Planes | 10/12/2019 at 09:45 ----------
calculated from 196 planes from Data set: 'All data.txt'

----- Kamb Contouring | 10/12/2019 at 09:45-----
Data set name = poles to All data.txt
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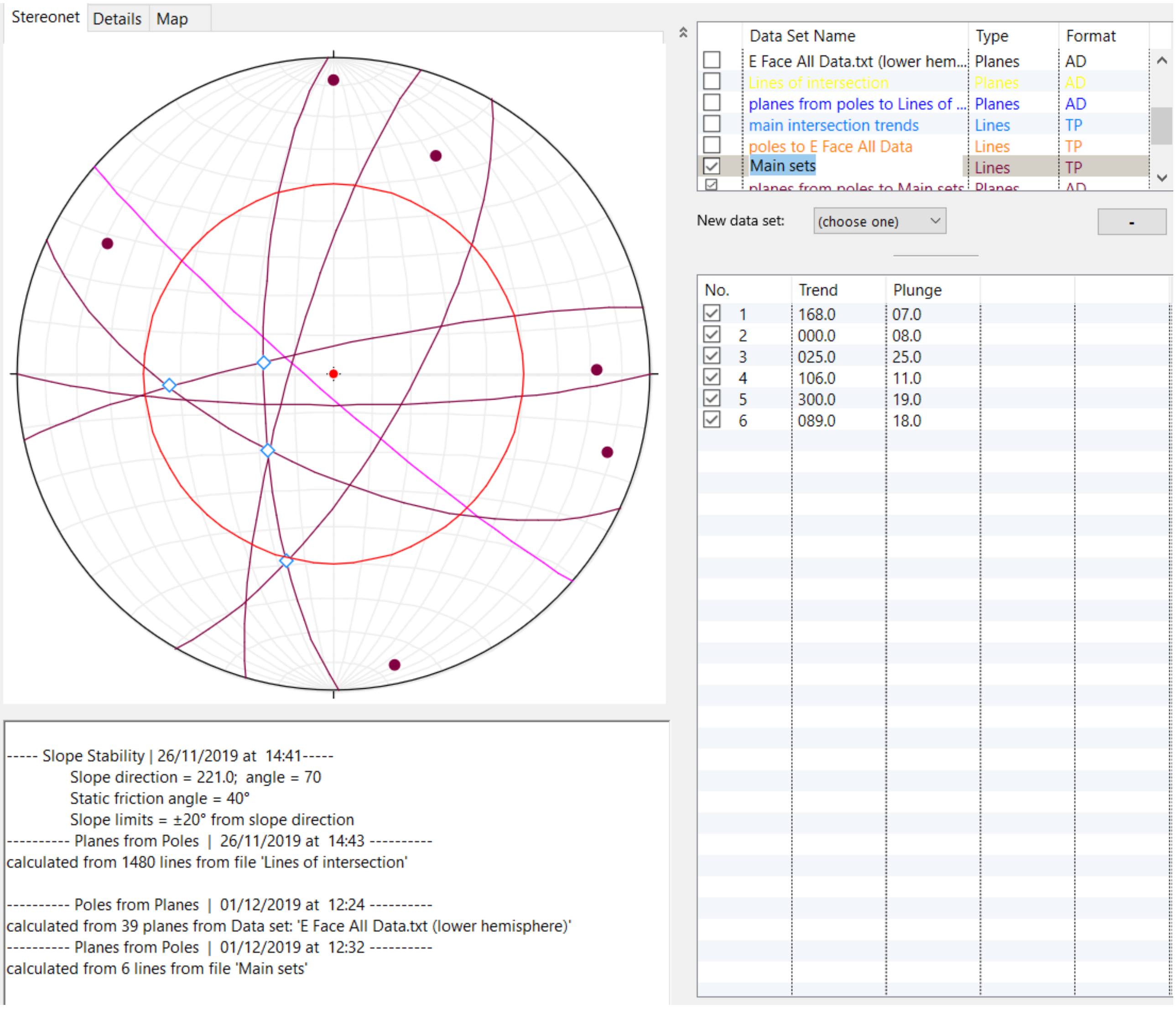
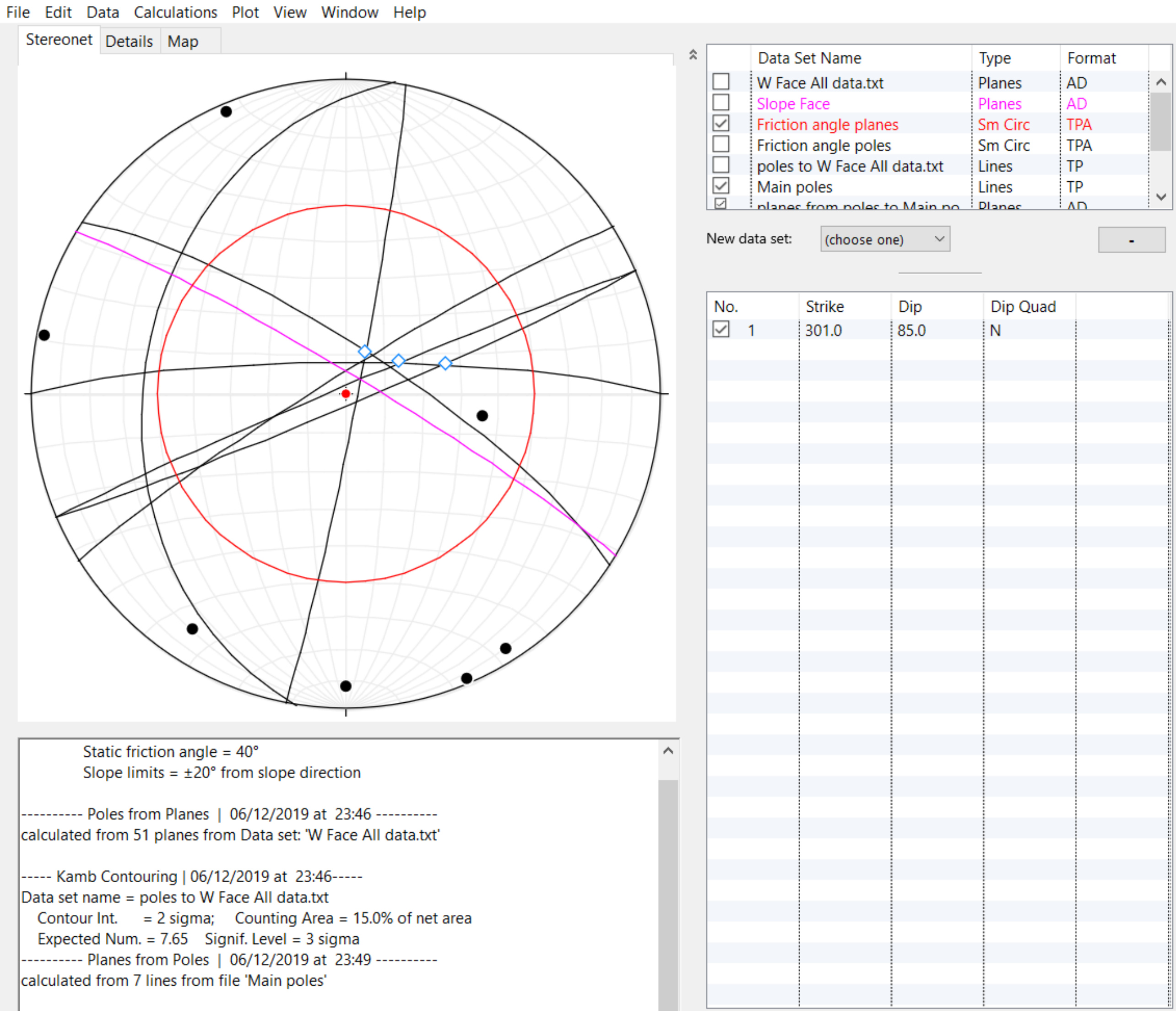
calculated from 11 lines from file 'Main planes'

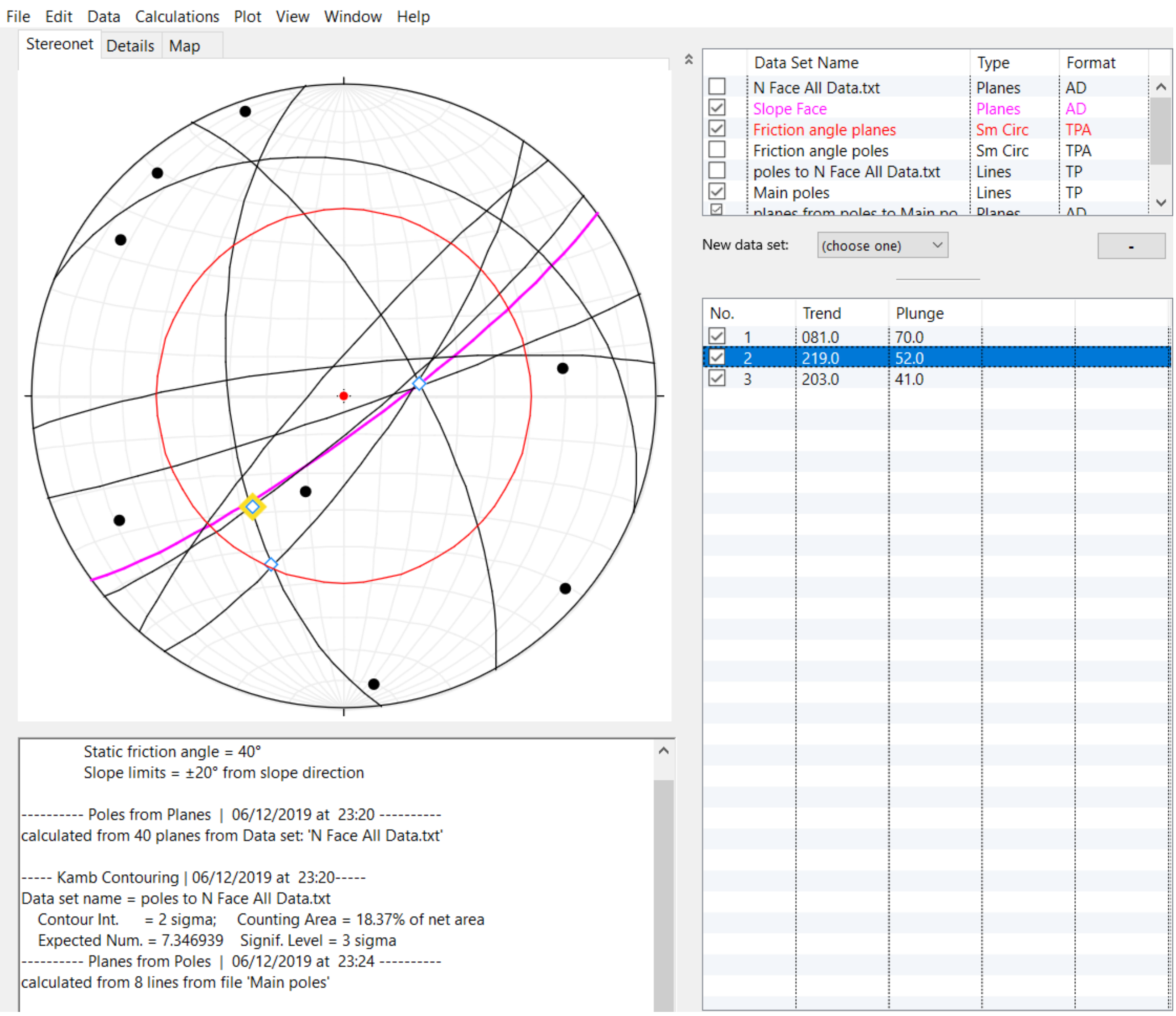
Eastern face



Western face



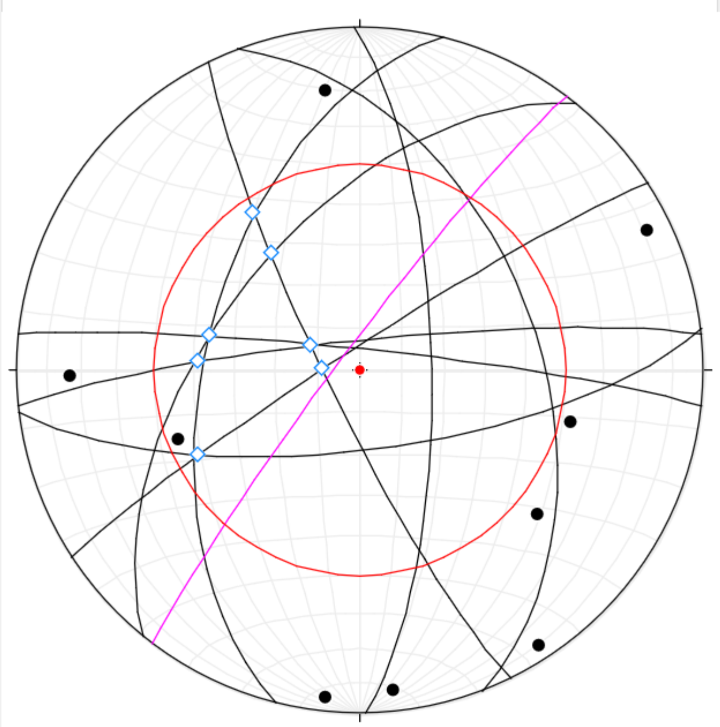
Northern face



Southern face

File Edit Data Calculations Plot View Window Help

Stereonet Details Map



	Data Set Name	Type	Format
☐	S Face All data.txt	Planes	AD
☐	Slope Face	Planes	AD
☑	Friction angle planes	Sm Circ	TPA
☐	Friction angle poles	Sm Circ	TPA
☐	poles to S Face All data.txt	Lines	TP
☑	Main poles	Lines	TP
☑	planes from poles to Main po	Planes	AD

New data set: (choose one)

No.	Strike	Dip	Dip Quad
☑ 1	217.0	85.0	W

----- Slope Stability | 26/11/2019 at 14:57-----
Slope direction = 307.0; angle = 85
Static friction angle = 40°
Slope limits = ±20° from slope direction

---------- Poles from Planes | 06/12/2019 at 23:32 ----------
calculated from 59 planes from Data set: 'S Face All data.txt'

----- 1% Area Contouring | 06/12/2019 at 23:38-----
Data set name = poles to S Face All data.txt
Contour Int. = 2%; Counting Area = 1% of net area
---------- Planes from Poles | 06/12/2019 at 23:40 ----------
calculated from 9 lines from file 'Main poles'

Annex D Site Photographs

Tile 1

New Landfill rock face beneath MTP, base at formation level.

Discoloured joint or small fault evident in corner of new landfill excavation. Water tank is located under the hardstanding with containers. No instability observed. Anchors / shotcrete to be installed as face extends back towards MTP

Tile 2

New Landfill rock face beneath Magħtab historic landfill.

Subvertical master joints/ faults extend through face. Some movement locally causing near-face rockfall. Water tanks located behind access ramp. Road to be widened. Stabilisation through rock anchors / shotcrete to be competed.

Tile 3

New Landfill rock face beneath Ta'Żwejra historic landfill.

Solution features – voids with brown staining - occur in the wall of the middle bench above a low-permeability horizon. Bench to be left in place to provide support to the southern face.