Vale of Glamorgan Council

Penarth Headland Link

Stage 1 Maritime and Geotechnical Review

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1 Executive summary

For the maritime assessment of the proposed structure (earth causeway with rock revetment) the alignment considered assumes approximately 20m offset from the cliff.

The design has been assessed to determine rock sizes required for the revetment and viable crest levels (based on overtopping criteria) compared to an initial outline proposal prepared by the Penarth Headland Group.

The initial proposed level of +8m OD is considered too low to provide safety to pedestrians from wave overtopping. Based on the existing data, an adequate level is estimated to be indicatively +9m OD. The exact level is to be decided after further studies considering risks and options and sensitivity to climate change.

By comparison this level is below the +9.7m OD 'as built' level at Cardiff Bay Barrage outer harbour breakwater, but higher than the +8.0m OD on the Parliamentary plans for the (unbuilt) Penarth Headland Link.

As another benchmark, the predicted 1:100 year flood level allowing for sea level rise but with no allowance for wave action is +8.5m OD.

The wave data available for this stage 1 study isn't sufficient to permit an assessment of the frequency that the causeway would have to be closed off as unsafe.

The rock armour on the causeway outer face appears reasonable for preliminary design, except that a toe of armour will be required.

Some issues that will require consideration in the next stage:

- Assess gaps and inconsistencies in the wave and water level data which is available for this stage 1 assessment.
- Operational restrictions: prevent access during storms.
- Potential for a parapet wall along the seaward edge of path to reduce wave overtopping.
- Allowance for sensitivity to increased storminess and sea level rise scenarios.
- Need for culverts through the causeway to prevent 'leaky dam'.
- Area between causeway and cliff likely to become silty, collecting rubbish and possibly becoming smelly.
- Need to consider escape options if overtopping or security concerns.
- Environmental and landscape impacts.

2 Introduction

A proposal to build a combined pedestrian and cycle path link between the western end of the Cardiff Bay Barrage and Penarth is being considered.

It is proposed that the link takes the form of a causeway, constructed with rock, running along the beach, between the cliff and intertidal zone.

The aim is for the path to be at a sufficient elevation to remain open in all tide and most storm conditions. The causeway is also to have a stand-off from the cliff that is sufficient for no stabilisation works to be required to the cliff face and for users of the causeway to be at negligible risk from rockfalls.

3 Objectives

The objective of this "Stage 1" report is to provide:

- Initial assessment of concept proposals, focussing on key maritime engineering issues
- Engineering strategy to take the project forward
- Review minimum horizontal distance between rock face and the nearest edge of the carriageway to protect users of the carriageway

Key maritime engineering issues (at stage 1)

- Seastate conditions are uncertain. This has major impact on:
    - Height of causeway (and therefore cost);
    - Size of rock armour (and therefore cost)

Key geotechnical engineering issues (at stage 1)

- Location at which, without a rock protection fence along the edge, the path can be used by walkers and cyclist with negligible danger of harm from future rock falls.

4 Scope

4.1 Maritime scope

The purpose of the Stage 1 maritime study is to provide a preliminary assessment of the volume of civil works required to form the Penarth Headland Link, based exclusively on maritime criteria (wave loads and wave overtopping).

The assessment is made for a typical section of causeway protected by a rock revetment (see Figure 1). This has been presented as an initial concept by the Penarth Headland Group. Although no other type of structure has been analysed, alternative configurations of the section that could help reduce the overtopping are suggested at the end of this study.

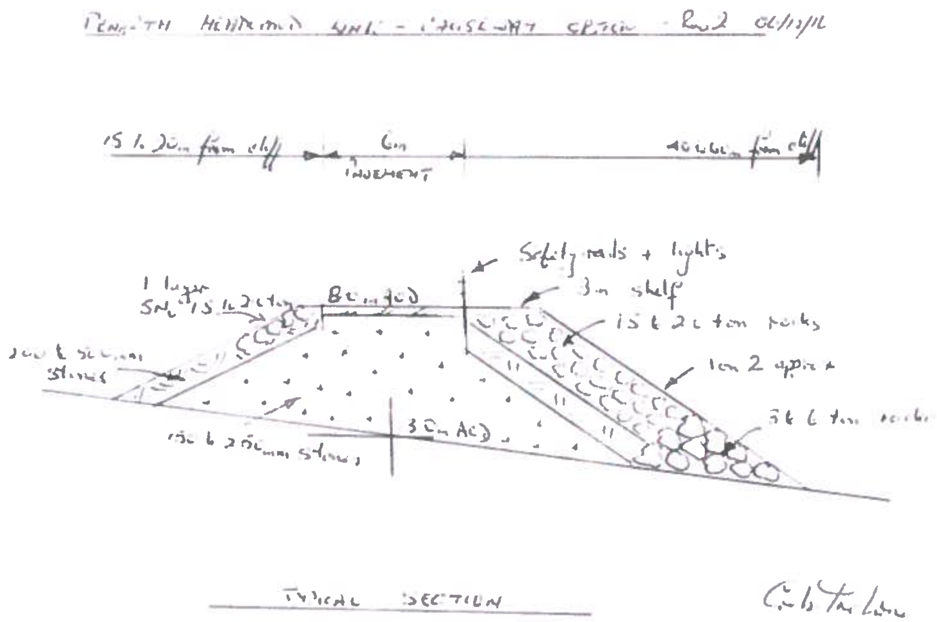

Figure 1: Sketch of typical section

The proposed tasks to undertake for this stage are:

- Desk study of what information on seastate and bathymetry is available from work already carried out for the previous planning application and by Penarth Headland Link, and identify any gaps in the available information.
- Review work already carried out for the previous planning application and by Penarth Headland Link.
- Carry out some initial assessment of waves at the site based on available wave data from Penarth Headland Link.
- Initial assessment of high tides and high waves.
- Assess allowance for sea level rise.

- Review approximate causeway levels and rock size based on the initial assessments.
- Co-ordinate with geotechnical engineers on geotechnical design and stability.

4.2 Geotechnical scope

The purpose of the Stage 1 geotechnical study is to review the location at which, without a rock protection fence along the edge, the path can be used by walkers and cyclist with negligible danger of harm from future rock falls.

The proposed geotechnical related tasks to undertake for this stage are:

- Site walkover to identify key areas of risk and gather data to inform the assessment
- Initial characterisation of cliff in terms of rock type and form
- Undertake rock fall simulations for selected locations to assess risks (using RocFall software)
- Initial review of proposed walkway location in relation to risks from rockfall
- Initial consideration of possible measures to mitigate risks

5 Maritime preliminary analysis

5.1 Site description

Penarth Headland Link coastal line is located between Cardiff Barrage harbour and the Penarth Esplanade. The approximate length of the frontage of study is 800m.

The northern end is coincident with the South breakwater of the barrage outer harbour, whereas the site ends on the South at one of the existing outfall structures, after crossing two concrete groynes.

The topography shows a sharp drop in levels at the cliff along the whole site, with an average height of 45m. The beach then slopes down to the foreshore with an approximate gradient of 1 in 10. The beach is formed of sand and rocks of different sizes with a dry area above mean high water springs between 2m and 7m. Over the first 200m after the breakwater, on the North end of the site, there are four existing concrete groynes.

Figure 2: Site aerial view

Figure 3: Existing groynes at the northern end of the site

5.2 Topography and bathymetry

The Vale of Glamorgan Council provided the topographic survey used for the previous scheme, that was produced in 2001.This includes levels of the foreshore and it has been used to obtain three representative sections of the area of study.

These sections will be checked and updated with the more recent Lidar survey results at a later stage.

Admiralty Charts 1182 and 1176 have been also used to confirm the approximate foreshore gradient in the area.

5.3 MetOcean conditions at Penarth

This section provides baseline information which is used for the assessment of:

- Hydraulic stability of the structure, i.e. the required rock size to withstand the incident waves

- Hydraulic performance of the structure, i.e. the wave overtopping rate

5.3.1 Water Levels

Tidal water levels

Tidal water levels at Cardiff are derived from the Admiralty Tide Tables (ATT), 2018, Vol 1B. These are presented in Table 1 in both chart and Ordnance Datum.

Table 1 –Tidal water levels, (ATT, Cardiff, 2018)

Cardiff	Tidal Level (mOD)	Tidal Level (mCD)
Highest Astronomical Tide	7.3	13.6
Mean High Water Springs	6.0	12.3
Mean High Water	4.4	10.7
Mean High Water Neaps	2.8	9.1
Mean Sea Level	0.3	6.6
Mean Low Water Neaps	-2.3	4.0
Mean Low Water Springs	-5.1	1.2
Lowest Astronomical Tide	-6.4	-0.1

Extreme water levels

The Environment Agency Coastal Flood Boundary 2011 (CFB) provides results of extreme water levels which include astronomical tide and surge components.

The extreme levels reported in CFB in proximity of Penarth at Chainage 412 and Chainage 414 (see location in Figure 4) are presented in

Table 2 for different return periods. Although the differences between the two datasets are considered small, Chainage 412 gives slightly higher tide levels and is used in the current assessment.

Figure 4: CFBD Chainages 412 and 414 in proximity of Penarth

Table 2 – Extreme water levels for different return periods relative to OD (Coastal Flood Boundary Chainages 412 and 414)

Chainage	412	Chainage	414
Return Period	Extreme water level (m)	Return Period	Extreme water level (m)
1:1	7.2	1:1	7.1
1:2	7.3	1:2	7.2
1:5	7.4	1:5	7.3
1:10	7.5	1:10	7.4
1:20	7.6	1:20	7.5
1:25	7.6	1:25	7.6
1:50	7.8	1:50	7.7
1:75	7.8	1:75	7.8
1:100	7.9	1:100	7.8
1:150	8.0	1:150	7.9
1:200	8.0	1:200	8.0
1:250	8.1	1:250	8.0
1:300	8.1	1:300	8.1
1:500	8.2	1:500	8.2
1:1000	8.4	1:1000	8.4

Sea level rise

Sea level rise projections were retrieved from UKCP09 grid cell 23681. The cell location is shown in Figure 5.

Figure 5: UKCP09 grid cell 23681 for relative sea level rise

UKCP09 provides sea level rise projections up to 2100. Assuming a present day of 2018 and an end of design life of the works of 2120, the sea level rise projections have been extrapolated up to 2120. Figure 6 shows the central estimates of low, medium and high emissions, as well as the extrapolation to 2120 for high emissions only. Between 2018 and 2120 the sea level rise for high emissions scenario is approximately 0.6m.

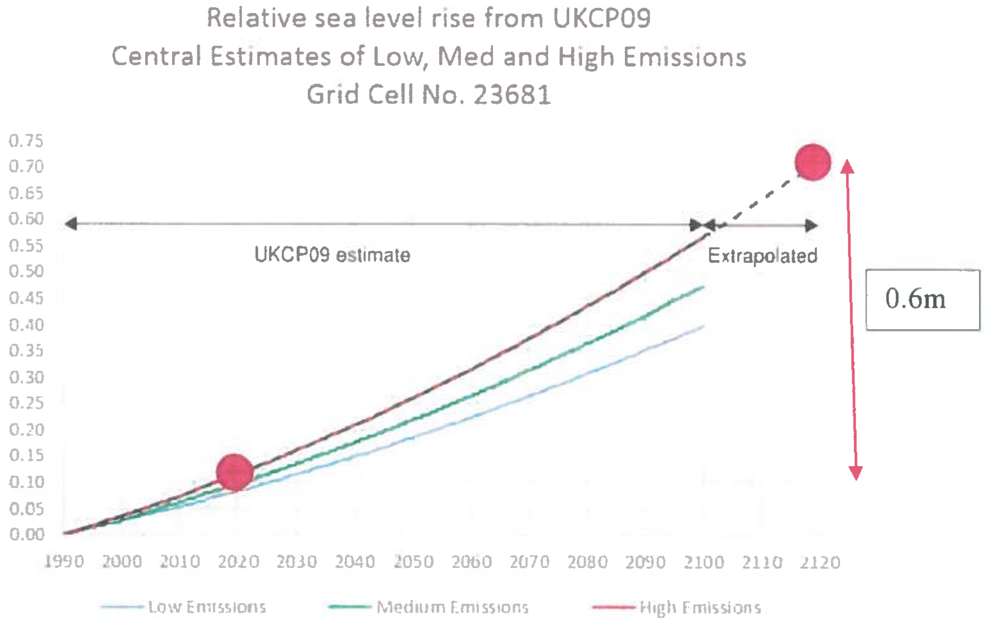

Figure 6: Relative sea level rise from UKCP09

Note, however that the DefRA guidelines Flood risk assessments: climate change allowances 2017 state that the more onerous "high ++ allowance" should be considered. This would give approximately 1.0m increase in sea level to 2120.

Table 3 presents the extreme water levels including the predicted sea level rise. They derive from Chainage 412 data.

Table 3 – Extreme water levels including sea level rise predicted in 2120 relative to OD

Chainage	412
Return Period	**Predicted extreme water level in 2120 (m)**
1:1	7.8
1:2	7.9
1:5	8.0
1:10	8.1
1:20	8.2
1:25	8.2
1:50	8.4
1:75	8.4
1:100	8.5

1:150	8.6
1:200	8.6
1:250	8.7
1:300	8.7
1:500	8.8
1:1000	9.0

5.3.2 Waves

At this stage of the project, the assessment of waves is based on available information and preliminary empirical calculations.

Wave information for Penarth Headland Link

Information about waves in the area of interest has been found in the following reports/publications. In particular, waves used for the Cardiff Barrage studies are considered representative to Penarth.

Table 4 - Information on waves as extracted by sources provided by Penarth Headland Group

Ref	Source of information	Information extracted
1	Swansea Bay Shoreline Management Plan Stage 1 Vol. 3/SMP/Nov 99	**Wave Data:** The data has a number of sources and dates from 1980 to present. Data in the Penarth area uses the barrage study and indicates significant wave height and period for return periods ranging from one year to 1000 years. The one in one hundred year event stated values of Hs = 2.3m and Tz = 6.4. Refer to specific Barrage studies for wave transform data and assessment of relevance to Penarth study frontage. **Tide Data:** Data from Proudman Oceanographic Ltd for Cardiff indicates Max recorded tide height of 7.53m AOD (1936) with a one in one hundred year event of 7.48m AOD.(This value is slightly different to data cited in Severn Estuary Study - data sets use different locations - Check gradient Lavernock/Penarth/Cardiff) Sea level rise trends are shown for Lavernock at 2.2mm/year **Sediment Transport:** Strategic data indicating ebb tide dominated fine sediment drift with wave induced drift generally up-channel with exception at Penarth Head where a drift divide is known to be present. Further investigation of barrage data required. **Wind Data:** Annual wind rose at Rhoose provided. Examination in conjunction with fetch lengths highlights importance of taking account of north east to south eastern sector

<table>
<tr><th>Ref</th><th>Source of information</th><th>Information extracted</th></tr>
<tr><td>2</td><td>Cardiff Bay Barrage Design Study, Report EX 2175, September 1991, HR Wallingford</td><td>
TABLE 18 : Recommended design water levels and wave conditions: by direction sector and overall
<table>
<tr><th>Return period (years)</th><th>Direction sector (°N)</th><th colspan="13">Significant wave height (m) for given high water level (mOD)</th></tr>
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</table>
The table entries refer to conditions at high water, and indicate water levels and wave heights at Cardiff expected, on average, to be equalled or exceeded once per return period. For sectors 30-190°N peak wave period may be estimated from the relationship $T_p = 13.5 \sqrt{H_s/g}$ where g is the acceleration due to gravity. For 190-230°N sector the relationship is $T_p = 30.9 \sqrt{H_s/g}$.
</td></tr>
<tr><td>3</td><td>Burt N., 2002. Cardiff Bay Barrage: Overview of Hydraulic Studies, Proc. Civil Engineers - Water and Maritime Engineering, ICE, Volume 154 Issue 2, June 2002, pp. 93-102, ISSN 1472-4561 | E-ISSN 1753-7800</td><td>
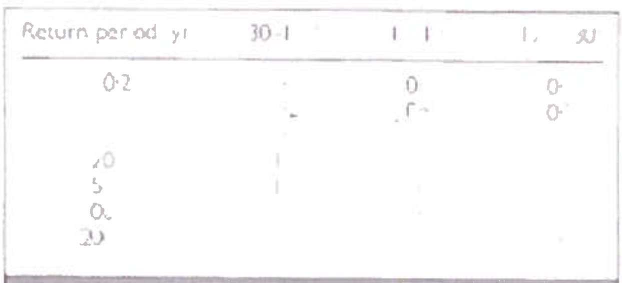
Table 1 Extreme wave heights, metres
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Wave data from nearby sites

There is no measured wave data for Penarth directly available for use at this stage.

Records from buoys located nearby at Weston Bay (14km away) and Minehead (30km away) provide additional information about the wave climate in the Bristol Channel, see Figure 7. These buoys are part of the Channel Coastal Observatory (CCO) monitoring programme and have been deployed in 2009 and 2007 respectively in Weston Bay and Minehead. However, their exposure conditions to waves is considered to be significantly different from the expected local conditions at Penarth and therefore not considered further.

Figure 7: Wave buoys locations (http://www.channelcoast.org/data_management/online_data_catalogue/metadata/search/index2.php)

Estimation of wind generated waves

For a simplified estimation of waves based on empirical methods, information about winds is required.

To understand the wind climate in the area of Penarth, wind data was retrieved from the DHI MetOcean portal which accesses directly the Climate Forecast System Reanalysis (CFSR) model produced by the National Centre for Environmental Prediction (NCEP). 37 years of hourly wind speed and direction are summarized in the wind rose presented in Figure 8.

There is a predominant westerly wind (wind coming from the western sector) and the highest wind speeds are also registered from this sector, between West and Southwest.

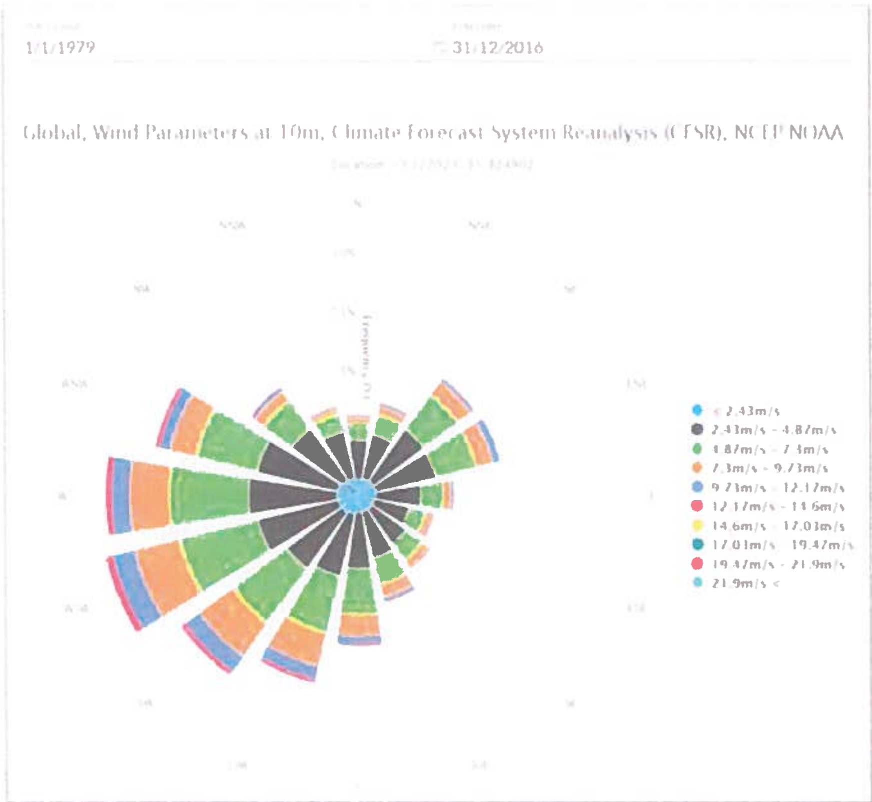


Figure 8: Wind rose at Penarth as extracted from the CFSR model from 37 years of hourly records.

The British Standards 6399-2:1997 provides extreme hourly wind speeds for 50 year return period which can be converted to other durations and return periods

For Penarth a basic wind speed of 22m/s was adopted in all scenarios (see Figure 9). The reduction factor for wind direction was not adopted at this stage. A correction factor of 1.08 was applied to transform the 50 year return period wind speed into a 200 year return period, in accordance with BS 6399-2:1997 Annex D.

Wind generated waves in deep waters have been estimated based on a simplified wave prediction method following the guidance provided in Coastal Engineering

Manual (CEM, 2011), Part II-2. The method assumes waves are generated and grow due to wind blowing with a constant direction over a certain distance (fetch) for a certain amount of time.

The assessment was carried out based on 30deg sectors, between 60deg and 270deg offshore at a distance from the structure. For each sector the maximum fetch was estimated and the results are presented in Table 5.

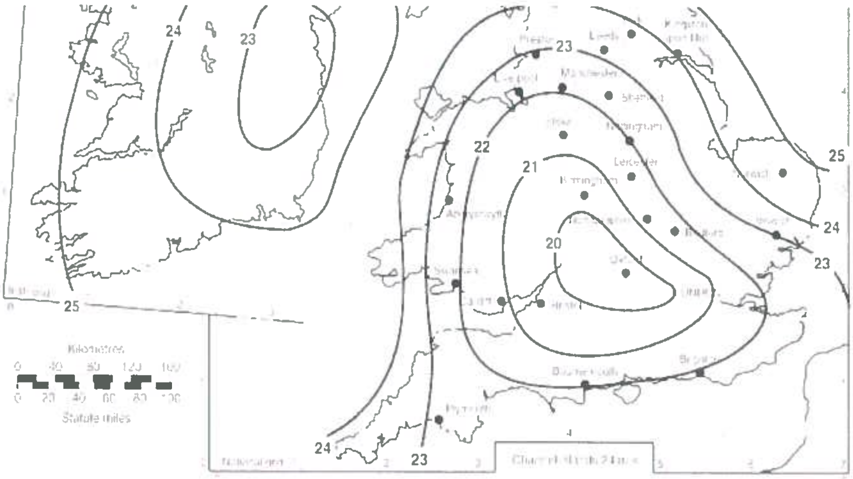


Figure 6 — Basic wind speed V_b (in m/s)

Figure 9: Basic wind speed in the UK. Source: BS 6399-2-1997

The results show that westerly waves propagating from the Atlantic Ocean to Lavernock Point (direction 270deg) along a 300km long fetch can reach a height of 5.9m. Such a wave height is not applicable directly to the site since the site is partially sheltered by Lavernock Point. Offshore waves from South-western sectors can reach a height of 2.5m; however, the site is partially sheltered from waves from this direction. More locally offshore wind generated waves along a shorter fetch of approximately 25-30km from the North-Eastern to Southern sectors generate lower offshore wave heights.

Table 5 – Analytical prediction of wind generated waves (CEM, 2011)

Ref.	Guidance	Analytical prediction							
4	Coastal Engineering Manual (CEM, 2011), Part II-2		Direction (wind coming from, in deg relative to North						
			60	90	120	150	180	210	240
		Hs (m) 1:200 years	1.5	1.3	1.1	1.3	1.6	2.0*	2.5*
		* offshore waves at a distance from structure; will be subject to diffraction							

Proposed design waves

Based on references 2, 3 (see Table 4) we have selected the design waves in deep waters:

Hs=2.0m, Tp=6.1s for a 1:200 years return period.

A higher wave height H=2.3m for a 1:100yrs return period is stated in ref. 1 (see Table 4, however without any justification and this is not considered to support an increase of the selected wave height.

Wave heights proposed in the available references are deep water waves, which in reality correspond to the conditions at the Cardiff Barrage and other locations, and not at Penarth Head frontage. Wave heights can increase as the waves reach shallow waters.

At this stage of the project for the estimation of rock sizing and wave overtopping, it has been assumed the selected design wave has a normal direction to the armour structure. At a next stage, waves from different directions approaching the structure after refraction and diffraction should be assessed using a numerical model.

5.4 Rock sizing

The hydraulic stability is assessed in terms of the required size of rocks to withstand the incident waves. The stability formula by Hudson (1953, 1959) gives the relationship between the median weight of armourstone W_{50} (N) and wave height at the toe of the structure, H (m) and the various relevant structural parameters. The Hudson formula is presented below. It assumes a 0-5% damage based on the volume of units displaced from the rock armour zone around the water level (up to a depth equal to one wave height):

$$M_{50} = \frac{\rho_s H^3}{K_D \left(\frac{\rho_s}{\rho_w} - 1\right)^3 cota}$$

M_{50} medium mass of rocks, $M_{50} = \rho_s {D_{n50}}^3$

ρ_s mass density of rocks

ρ_w mass density of water

a slope angle

K_D is a dimensionless stability coefficient, calculated for different kinds of armour by laboratory experiment. It depends on the rock shape and placement as well as the wave breaking. Most common values are 2.0 for breaking waves and 4.0 for non-breaking waves.

For an armour layer slope (V:H) 1:2 and rock density 2.65t/m^3, it is the wave height that mainly controls the hydraulic stability.

We have selected the following 1:200 year return period design wave conditions for application of the stability formula:

Hs = 2.0m – in deep water

Tp = 6.1s

During high water levels, when water depth in front of the structure is larger than approximately 3.0m, rocks of M_{50}=1.3t are stable against waves. A rock grading of 1-3t can withstand these waves. However, in shallower waters, for water levels lower than 3.0m, the armour will be in the breaking zone, where the waves increase in height. Heavier rocks of M_{50}=4t are required to withstand the breaking waves. A rock grading of 3-6t is selected for the armour at levels lower than 3.0mOD.

It is noted that a more detailed assessment of incident waves may result in different wave breaking conditions at the structure and therefore, different size of rocks.

5.5 Preliminary assessment of wave overtopping

Eurotop (2016) provides the tolerable overtopping discharges and overtopping wave volumes in a table as below (see Table 6).

Table 6 - Limits for overtopping for people and vehicles as extracted from Eurotop (2016)

Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
People at structures with possible violent overtopping, mostly vertical structures	No access for any predicted overtopping	No access for any predicted overtopping
People at seawall / dike crest. Clear view of the sea		
H_{m0} = 3 m	0.3	600
H_{m0} = 2 m	1	600
H_{m0} = 1 m	10-20	600
H_{m0} < 0.5 m	No limit	No limit
Cars on seawall / dike crest, or railway close behind crest		
H_{m0} = 3 m	<5	2000
H_{m0} = 2 m	10-20	2000
H_{m0} = 1 m	<75	2000
Highways and roads, fast traffic	Close before debris in spray becomes dangerous	Close before debris in spray becomes dangerous

Wave overtopping was calculated for a simple rock armour layer with a 1:2 slope at Mean High Water (MWH) level with and without sea level rise projections in future included. For a toe of the structure set at +0.0 mOD, the water depth in front of the structure is 4.4m in 2018 and 5.0m in 2120. The toe level of +0.0mOD is based on Arup drawing SK100 reproduced below in Figure 10.

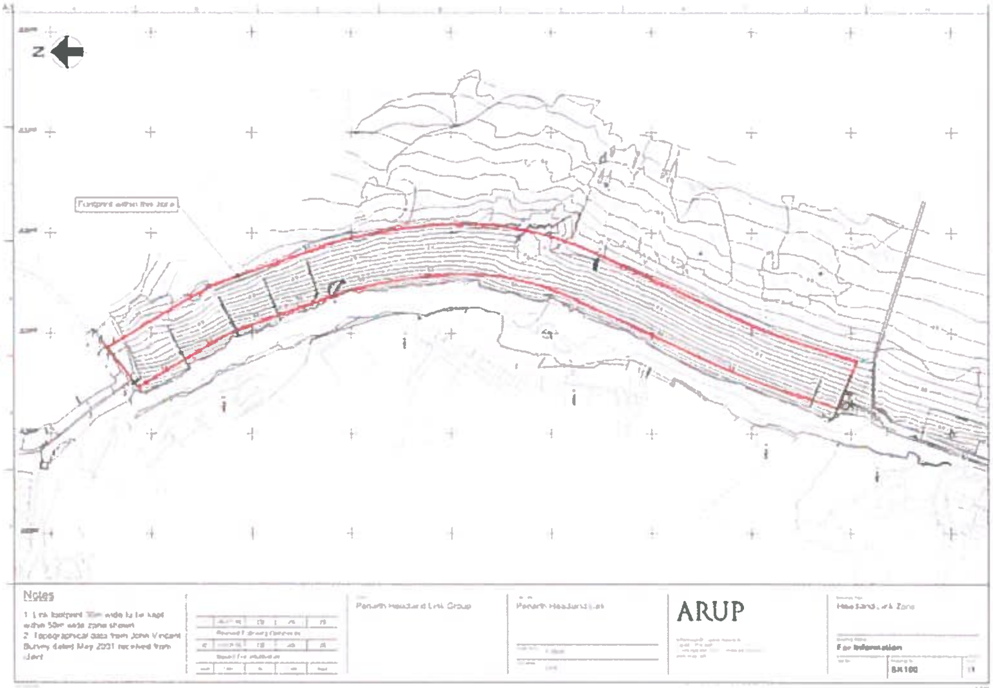


Figure 10: SK100 and the link footprint boundaries currently being considered

Given that the future predicted HAT level is +8.0mOD and extreme water levels for different return periods exceed the 8.0m (see Table 3), the wave overtopping calculations have been carried out for crest levels set higher than +8.0mOD until the tolerable wave overtopping value is met based on the criteria of Table 6. Results are presented in

Table 7.

The wave overtopping criteria for the 'safe' use of the structure by pedestrians is satisfied for a crest set +8.0mOD for the tested wave and water-level conditions in 2018. However, considering the future sea level rise in 2120, a crest level set at a higher level than +8.5mOD is estimated 'safe'.

Table 7 – Wave overtopping rate at MHW levels in years 2018 and 2120 for different structure crest levels

MHW	Wave Height –water depth limited wave	Crest level	Wave overtopping rate	Safe?
MHW 2018 4.4 mOD	2.0 m	+8.0 mOD +8.5 mOD +9.0 mOD	0.9 l/s/m 0.3 l/s/m 0.1 l/s/m	Yes/Marginal Yes Yes
MHW 2120 5.0 mOD	2.0m	+8.0 mOD +8.5 mOD +9.0 mOD	4.0 l/s/m 1.2 l/s/m 0.3 l/s/m	No No/Marginal Yes

NB these figures allow for sea level rise but not for the effects of increased storminess. Wave overtopping for different conditions will be estimated based on a more detailed assessment of incident waves and following the agreed functional requirements.

5.6 Typical cross section

A conventional rock armour structure with the following characteristics is designed considering estimations based on the metocean inputs of previous sections. It is presented in Figure 11 .

Armour layer – sea side

The slope of the armour layer is (V:H) 1:2. The armour's stability requires different size of rocks to withstand the incident waves at different levels. A rock grading 1-3t is selected at levels higher than 3.0m. A rock grading of 3-6t is required at levels lower than 3.0m.

Such gradings correspond to median nominal rock diameter Dn_{50}=0.9m and Dn_{50}=1.22m, respectively based on a rock density 2.65t/m^3. Assuming a relatively tight placement of the smaller rocks with a layer coefficient Kt=0.9 and a layer coefficient Kt=1.0 for the larger rocks, the thickness of the armour layer can be uniform 2.4m. It can consist of 3 layers of 1-3t rocks (with Dn_{50}=0.9m) above the +3.0mOD and 2 layers of 3-6t rocks (with Dn_{50}=1.22m) below +3.0mOD. This armour layer configuration is similar to the one presented in Figure 1. Figure 11 presents a rock armour which consists of 3 layers of 1-3t and a rock berm consisting of 3-6t below the +3.0mOD, as an alternative configuration.

The horizontal crest of the armour layer shall consist of a minimum 3 stones of 1-3t which totals 2.7m.

Underlayer – sea side

The underlayer shall satisfy the filter criteria between the armour layer above and the core material below and a detailed calculation can be carried out at the next stage. At this stage a rock grading of 0.3-1t is selected (based on a rock size W/10; W is the weight of the armour rock). Based on a rock density 2.65t/m^3, the nominal rock diameter is D_{n50}=0.63m. Assuming layer coefficient Kt=1.0 the thickness of the underlayer is 1.26m.

Armour Layer – rear side

A double layer consisting of 1-3t rocks may form the rear side armour. The rear side armour may have a steep slope 1:1.5. Its thickness is 1.8m assuming a layer coefficient Kt=1.0. At the next stage more detailed calculations can be carried out to explore opportunities to adopt a smaller size of rocks.

Core

The core material can be rock with a wide grading, e.g. 0.75kg to 30kg (based on a rock size W/200-W/4000; W is the weight of the armour rock). The choice of grading for the core shall satisfy the filter criteria for both the rock layers of 1-3t and 0.3-1t which are immediately adjacent on the rear side and sea side respectively.

Footpath

A 6m wide footpath will be formed at the crest of the structure. The footpath should be of durable flexible material which allows for some settlement of the causeway

Crest Level

The typical cross section shows indicatively a crest level set at +9.0mOD. This crest level results in a relatively low wave overtopping rate. But it is lower than the crest level of the rock armour of the Cardiff Barrage closure breakwater, which is located at the northern end of the current scheme. It is noted however that wave conditions and required safety of pedestrians might be different for these two structures.

The crest level of the structure of interest will be defined based on a more detailed wave assessment and the functional requirements in terms of accessibility and wave overtopping.

<u>Toe</u>

The toe of the armour layer shall be designed in more detail considering both wave and seabed ground conditions. Generally, in shallow waters conditions where waves may break on the structure, the toe can be made by extending the main armour layer i.e. to form a rock berm. This toe configuration is shown in Figure 11. Its crest is set at +3.0mOD. It is made of a 3-6t rocks. The design at the next stage shall consider the combination of wave breaking conditions in a steep foreshore e.g. 1:10. In general, a more expensive solution is to construct the toe in a dredged trench, which however makes it possible to lower the toe level and use a smaller stone size. If rock is found on a seabed a small trench of a depth equal to 70% of the rocks diameter (Dn_{50}) may provide toe support.

At the next stage the cross section configuration will be assessed in more detail.

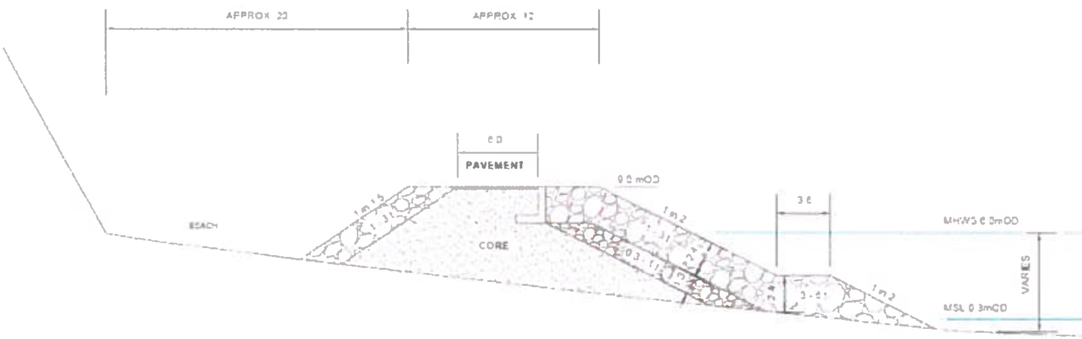


Figure 11: Simple conventional typical cross section

5.7 Functional requirements, uncertainties and potential alternatives

The two main constraints considered in relation to the position of the causeway from the cliff are the rock fall risk (20m) and the environmental risk zone on the foreshore side (approximately coincident with the 0mOD contour line).

No functional requirements have been defined by the Client at this stage. The proposal for Penarth Link in this report is based on the typical section provided to make it work for the two main wave-driven issues: revetment rock sizes and overtopping.

The first conclusion of this assessment is that a crest level at +8mOD should not be considered for this scheme given that this coincides with the 1:200 years extreme water level (see

Table 2) as well as the future predicted HAT and extreme water levels for different return periods (see Table 3)

For the tested conditions, a crest level of +8.50mOD (see

Table 7) may satisfy the wave overtopping criteria. However, wave overtopping predictions are sensitive to the incident waves. For example, if the sensitivity to increased storminess is considered (including a future wave height increase of 10%) as is recommended by Environment Agency (2016), a crest higher than +9.5mOD would be required to provide protection against wave overtopping for the pedestrians. A sensitivity analysis for the wave period and wave steepness impact is also recommended at a next stage.

Other exposed structures in the area, such as Cardiff Barrage outer harbour breakwaters, have their crest levels at around 10mOD. We understand that lower levels of the causeway would be more acceptable, with lower capital expenditure.

There are two possible options:

i) Restrict the structure's access by its users when storms are forecast: keeping the basic section provided (see Figure 1). For this case, people should be warned of potential hazards from wave overtopping and, according to Eurotop (2016), a more focused 'duty of care' shall apply to staff.

The structure may be particularly dangerous in storms where people can be washed off. In some instances an operating authority may be able to exclude access, but at others the public may still be able to access under severe wave conditions, even when such wave overtopping could be dangerous for people.

If this is the selected option a weather forecast system linked to wave overtopping discharge and maximum overtopping volumes calculations for predicted wave conditions and water levels can be developed. Eurotop (2016) gives an example of an operation plan to warn or exclude visitors at the breakwater at Oostende, Belgium which is used by pedestrians and cyclists for recreation; as soon as a weather condition is predicted to exceed the maximum overtopping volume the breakwater is closed at low water preceding the expected high tide overtopping. A similar warning operation has been used for approximately 20 years at the Samphire Hoe reclamation, near Dover UK where a warning system was calibrated against observations of hazardous conditions at the reclamation.

ii) Modifications to the basic section: Limit the wave overtopping to an acceptable 'safe' limit to all extreme present and future conditions by revising the structure design.

Alternative cross section configurations and/or additional design elements on the sea-side rock armour may provide reduction of wave overtopping for crest levels of +9.0mOD. Indicative solutions which are commonly adopted, but would require further analysis to determine their viability (technical, economic and environmental), involve, :

- A crest wall to provide additional protection against wave overtopping. It would also protect people from washing off during severe wave overtopping events.

- Increasing the crest width to reduce the overtopping volumes that reach the footpath in a similar way to the existing road north of the barrage sluices. At this location, the existing crest width is generally greater than 13m.

Other options are not considered cost-effective, particularly for the wave conditions we have considered herein:

- Rock armour with a wide berm which would absorb part of the incident wave energy and reduce wave overtopping. A flatter slope than 1:2 e.g. 1:4 may reduce the wave overtopping rate and/or form more horizontal overtopping flows, which might be less dangerous than the overtopping from steeper flow-jets.
- A detached rock armour at a distance and parallel to the main structure; it can reduce the incident wave energy at a distance from the main structure, so that the main structure will be hit by the much lower transmitted waves.

The structure itself being at a distance from the cliffs might create conditions of standing waters. The transversal drainage of the causeway will be studied and an adequate system will be proposed at a later stage if necessary.

At this stage, all the estimations are based on the available information and high level analytical calculations. Further analysis recommended at the next stage is discussed in Section 7.

Other related issues should include:

- Strategy and design of culverts
- An assessment of the likely development of the beach area between the causeway and the cliff. Consider risk of build-up of silt, rubbish and possible smells
- Assessment of safety and security issues
- Environmental and landscape impacts

6 Geotechnical Study

6.1 Site description

The area of interest stretches for some 800m around Penarth Head from the outer harbour at the Cardiff Bay Barrage in the north to a masonry and concrete walkway of Penarth Esplanade in the south.

The upper sections of the beach are made up of sand, gravels, cobbles and boulders largely derived from the adjacent cliff face. The elevation of the beach at the base of the cliff is generally at between 7 and 8mOD.

The height of the cliff face above beach level increases from around 30-40m in the north (40-50mOD), up to 50-60m around Penarth Head (55-65mOD) and back down to 30-40m in the south (40-50mOD).

Four groynes are present at the northern end of the beach and two groynes are present at the southern end of the beach, see Figure 12.

The area above the cliff is generally developed with houses, blocks of flats, a school and public open space.

No streams or other water features are shown on the published Ordnance Survey mapping but minor depressions are present in the ground above the cliff which are likely to form drainage channels for surface water runoff, towards the cliff.

6.2 Geology and hydrogeology

The geology and hydrogeology of the site has been reviewed based on published information and the information to support the previous planning application for the headland link.

Based on review of the 1:10,000 scale geological map Sheet ST17SE and rock exposures seen during the site walkover, the beach deposits present along the alignment of the proposed link are underlain by the Mercia Mudstones, except around the Penarth Fault where the Blue Anchor Formation outcrops.

A sequence of strata is exposed in the cliff with the Red Mudstones and Blue Anchor Formation of the Triassic Mercia Mudstone Group at the base, overlain by the Triassic Penarth Group and in turn overlain by the Jurassic strata of the Blue Lias.

The geological map shows two northwest to southeast trending faults to the south of Penarth Head. The northern fault is known as the Penarth Fault. The area between the faults is down thrown exposing the Blue Anchor Formation on the beach.

The bedrock is generally near horizontally bedded along the majority of the cliff, with localised disturbance to the south of the Penarth Fault.

The geological sequence exposed in the cliffs is summarised in Table 8.

Table 8 - Summary of geological stratigraphy:

Unit	Formation	Strata description
Blue Lias	Porthkerry Formation (Po)	Limestones and subordinate mudstones
	Lavernock Shales (Lvn)	Dark grey mudstone with nodules and nodule beds of limestone
	St Mary's Well Bay Formation (StM)	Bluish grey mudstones with limestones
Penarth Group (PnG)	Lilstock Formation (Langport Member)	Limestones overlain by grey mudstones with thin sandstones, siltstones and limestones
	Lilstock Formation (Cotham Member)	Grey green calcareous mudstones with thin siltstones, sandstones and limestones
	Westbury Formation	Dark grey shales with thin limestone and sandstones
Mercia Mudstone Group	Blue Anchor Formation (BAn)	Grey green mudstones with subordinate thin dolomites and limestones plus beds of gypsum nodules
	Red Mudstones (MM)	Red brown mudstones and silty mudstones with subordinate grey beds. Gypsum nodules and beds of nodules

6.3 Site walkover

Site walkovers were undertaken on 21 February 2018 and 10 April 2018. The walkovers were undertaken to review the form of the cliff, identify signs of instability and any features that may influence the instability.

The walkovers followed the base of the cliff from the south breakwater of the barrage outer harbour to the masonry and concrete walkway of Penarth Esplanade.

The cliff has been divided into four zones based on the nature of the cliff.

6.3.1 Outer harbour breakwater to Headland (Zone 1)

A footpath is present between the cliff and south breakwater. In this area the cliff is faced with masonry. The southern end of the masonry wall has collapsed in places exposing the bedrock. The collapsed area of wall is cordoned off with heras fencing.

The site visit on the 10 April 2018 was undertaken after heavy rain. A fast seepage of water was observed from the upper part of the cliff immediately to the south of the breakwater. The location of the seepage corresponds with a depression in the upper part of the cliff that appears to be funnelling surface water runoff.

The profile of the cliff face undulates with stronger bands of gypsum (in the Mercia Mudstone Group strata) and limestone (in the Penarth Group and St Marys Well Formation) forming overhangs where the weaker mudstones have been preferentially weathered. The bands of gypsum are generally around 250mm thick and form overhangs of up to around 1m. The individual limestone beds are up to around 250mm thick with the thicker bands locally forming overhangs of around 500mm; see Photographs P1-P3. The limestone beds contain sub vertical joints that together with the bedding planes form tabular blocks.

The base of the cliff is generally obscured by a "soil" slope built up of debris that has fallen from the cliff. The debris comprises gravel sized fragments of mudstone and limestone with occasional cobbles and boulders of limestone and gypsum.

The beach is covered by sands and gravels and larger blocks that have fallen from the cliff, see Photographs P1-P3. Blocks of limestone and gypsum up to $1.25x1.25x0.25m^3$ were observed on the beach.

6.3.2 Headland to Penarth Fault (Zone 2)

This section of cliff is similar to the previous section with exposures containing bands of gypsum and limestone. To the south of the fourth groyne the general size of blocks on the beach increases, coinciding with the increased height of the cliff and the increased exposure of blocky limestone in the upper cliff; see photographs P5 and P6.

During the 10 April 2018 site visit a number of rock falls were observed originating from a gully close to the crest of the cliff face, see Photograph P5. The rockfalls consisted of soil and blocks of limestone up to around 200x200x200mm. Blocks were seen to roll and bounce down the cliff face, with blocks resting up to around 20m from the base of the cliff. The source of the rock blocks was a shallow gully that was acting as a drainage channel.

A large area of debris is present covering the base of the cliff between a rib of rock that runs down the headland and the Penarth Fault. The debris is made up of material from the upper part of the face and includes detached sections of cliff, see Photograph P7. The debris extends a length of around 40m along the cliff and approximately 10-15m from the base of the cliff.

Beds of limestone with minimal mudstone is present towards the top of the cliff. In some areas these beds form blocky overhangs, see Photograph P8.

6.3.3 Penarth Fault Zone (Zone 3)

At the location of the Penarth Fault, the cliff face recesses in slightly, and the upper slope is cut back forming a rough ledge at around 30m (38mOD), see Photograph P9. Vegetation comprising grass, gorse bushes and other small trees / bushes are present on the ledges and in gullies.

Interbedded limestones and mudstones are exposed in the cliff face, with the bedding disturbed by faulting and folding. As with other areas the limestone beds contain sub vertical joints that together with the bedding planes form tabular blocks.

During the site walkover seepages down the cliff face were observed emanating from topographical depressions in the crest and cliff face.

Minimal debris was observed at the base of the cliff in this area. The beach is covered by limestone blocks generally in the order of 150x150x250mm.

6.3.4 Penarth Fault Zone to Penarth (Zone 4)

Compared to the areas to the north, a greater thickness (approximately 15m) of Red Mudstone is exposed at the base of the cliff in this section. The Red Mudstone is overlain by the Blue Anchor Formation. Bands of gypsum are present which form overhangs of up to around 1m, see Photographs P11-P15. Blocky limestone of the Penarth Group and St Mary's Well Formation are present towards the top of the cliff face in the north, with the thickness decreasing to the south. The bands of jointed limestones in the Blue Anchor Formation, Penarth Group and St Mary's Well Formation form small overhangs.

The frequency of larger limestone blocks on the beach increases from north to south, relative to the outcrop of the St Mary's Well Formation at the top of the cliff. Blocks on the beach are generally up to around 200x300x300mm with occasional larger blocks.

The remnants of a structure constructed from concrete panels is present at the crest of the cliff, see Photograph P10. The structure overhangs the crest of the cliff.

In places within the Red Mudstones, 4-5m high buttresses have formed following collapse of the cliff face. In places sub vertical joints are present that could result in toppling failure of the buttresses, see photograph P11.

Minimal debris was observed at the base of the cliff in this area with the exception of the southern end close to the end of the cliff. At this location a large amount of debris is present that has resulted from a slip from the upper part of the cliff, see Figures 12-14. The debris extends a length of around 40m along the cliff and approximately 5-10m from the base of the cliff. This slip occurred in 2014 and was captured on video: https://www.walesonline.co.uk/news/wales-news/penarth-landslide-watch-150-tonnes-6990560

6.3.5 Summary

Based on the observations made during the site walkover, the nature of instability along the cliff relates to the rock types present in the cliff face. Failure of large individual blocks occurs where overhanging beds of limestone and gypsum have formed. Smaller blocks are also released on sub vertical joints from bands of limestone throughout the strata exposed in the cliff. Instability is particularly contracted around topographical depressions at the crest of the cliff that form drainage channels.

The concentration of limestone blocks on the beach increases around Penarth Head, where the thickness of the St Mary's Well Formation exposed in the cliff thickens. Locally overhanging masses of blocky limestone are present at the top of the cliff, see figures 15 and 17 below. Based on the blocky nature of the limestone in this area, it is expected that should the rock mass associated with the overhangs fail, the material would break up into individual blocks behave as rocks falling rather than a debris slip.

Gypsum exposed in the cliff at either end of the area also contributes to large fallen blocks found on the beach. The Gypsum bands are generally encountered within the bottom 10m of the cliff.

The release of loose materials from the upper parts of the cliff appears to be associated with surface water flows above the cliff and erosion.

6.4 Cliff Stability Assessment

6.4.1 Mass Slippages

There is evidence of the build-up of debris along the cliff, associated with the weathering of weak mudstone bands and the release of limestone blocks. The build-up of debris generally extends a few meters from the base of the cliff.

Locally there is evidence of larger slips (to the south of Penarth Head and as the southern end of the cliff, see Photographs P7 and P15). In the area to the south of Penarth Head a large segment of the cliff has collapsed and at the southern end of the cliff the majority of material slipped from the upper section of the cliff.

Based on observations made during the sitewalk over the maximum extent of debris at the base of the cliff is some 10-15m from the base of the cliff.

A large ledge is present on the cliff to the south of the Penarth Fault. There is a possibility that this was formed as a result of a large slip.

It is difficult to assess the impact of mass slippages, but an evidence based approach and be used by gathering data of past events. Further review of historical information maps and aerial photographs to understand past events should be undertaken to assess the potential mechanisms, size and extent of slips.

6.5 Individual block rock fall

As outlined in Section 5.2 and 5.3, the desk study and site walkover has identified potential risks posed by individual rock blocks falling from the slope. In order to assess the risks posed, Rocfall software has been used to model the trajectory of falling rock blocks. The results and findings are presented in Appendix A and summarised below.

The Rocfall software models the trajectory of rock blocks (seeders) falling down the slope profile. Parameters used in the model have been derived from site information and published literature. The simulation has been run using the lumped mass approach due it having a larger amount of published input parameters. This method has the following assumptions:

- each rock is modelled as a particle;
- the rocks are not considered to have any size (or shape), only mass (used to calculate the kinetic energy for graphs and results);
- air frictional resistance is not considered;
- the slope is modelled as one continuous group of straight line segments

Profile

Rockfall along three sections has been considered representing typical sections of cliff face. The profile locations are shown on Figure 12. Figures 15, 16 and 17 illustrate the three profiles used.

The cliff profiles have been taken from point cloud survey undertaken by the Vale of Glamorgan Council.

The exact elevation and alignment of the causeway is yet to be confirmed. For this Stage 1 assessment, simulations have been modelled with the top of the embankment at 8mOD and at 10mOD, and with the inner crest line at 20m and 10m from the toe of the cliff.

Seeder

During the site walkover, the limestone and gypsum blocks on the beach and in the cliff were observed up to 1.25m x 1.25m x 0.25m. The limestone blocks were generally angular cubic or tabular shapes (which is typical of the limestone in the area). The density of the seeder is based on a limestone density of 2500kg/m^3, as provided by Rocscience Rock Density Table. In addition, a few limestone blocks, up to approximately 0.2m x 0.2m x 0.2m were observed falling from the top of the cliff.

Each of the simulations have been run dropping 5000 rocks that have masses up to 1000kg, which have been released from the slope (dropped with no elevation above the slope) and have a starting velocity of 0.2m/s and a rotational velocity of 1°/sec.

Ground material properties

After reviewing tables provided by Rocfall and after some sensitivity tests the following parameters were considered most appropriate for the slope, the beach and the embankment.

Table 9: Slope parameters used in the rockfall simulations for each profile

Parameter	Mean	Distribution	Standard Deviation	Relative max/min
Cliff				
Coefficient of normal restitution (Rn)	0.4	Normal	0.05	±0.15
Coefficient of tangential restitution (Rt)	0.8	Normal	0.04	±0.12
Slope Roughness	0°	Uniform		±60°
Friction angle	Calculated from Rt			
Beach (boulders, cobbles and sand)				
Coefficient of normal restitution (Rn)	0.32	Normal	0.04	±0.12
Coefficient of tangential restitution (Rt)	0.8	Normal	0.04	±0.12
Slope Roughness	0°	Uniform		±60°
Friction angle	Calculated from Rt			
Embankment				
Coefficient of normal restitution (Rn)	0.35	Normal	0.04	±0.12
Coefficient of tangential restitution (Rt)	0.85	Normal	0.04	±0.12
Slope roughness	0°	Uniform		±60°
Friction angle	Calculated from Rt			

Values for the normal restitution and the tangential restitution have been obtained from the table at:
https://www.rocscience.com/help/rocfall/webhelp/baggage/rn_rt_table.htm

Results

The rock fall simulations have been run for a number of different profiles as described below. Results for each of the simulations are included in Appendix A.

1) **For causeway at 10mOD ad inner edge 20m from cliff toe:**

For Sections 1 and 2, none of the falling rocks reached the path, although a number come close to the crest of the embankment.

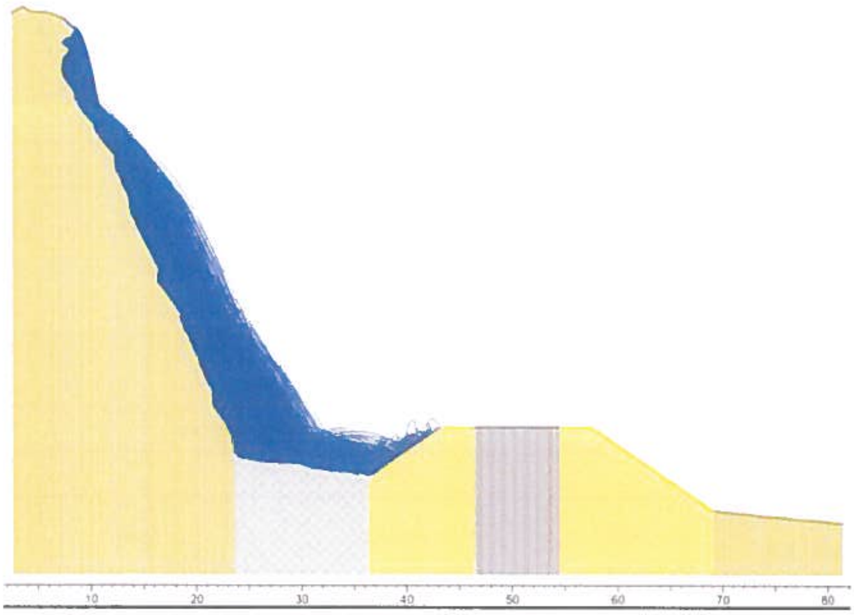


Figure 15: Section 1 with the crest of the embankment at 10mOD and 20m from the toe of the cliff

For Section 3, one rock reached the pathway and five reached beyond the crest of the embankment, out of the 5000 dropped.

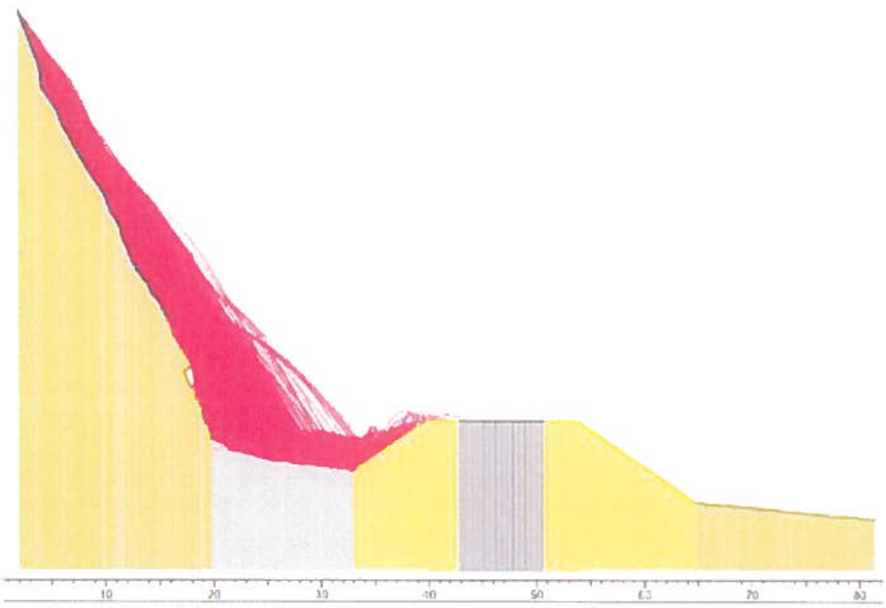


Figure 16: Section 3 with the crest of the embankment at 10mOD and 20m from the toe of the cliff

2) For causeway at 10mOD and inner edge 10m from cliff toe:

When the embankment is moved 10m closer to the slope, a significant number of rocks bounce over the pathway.

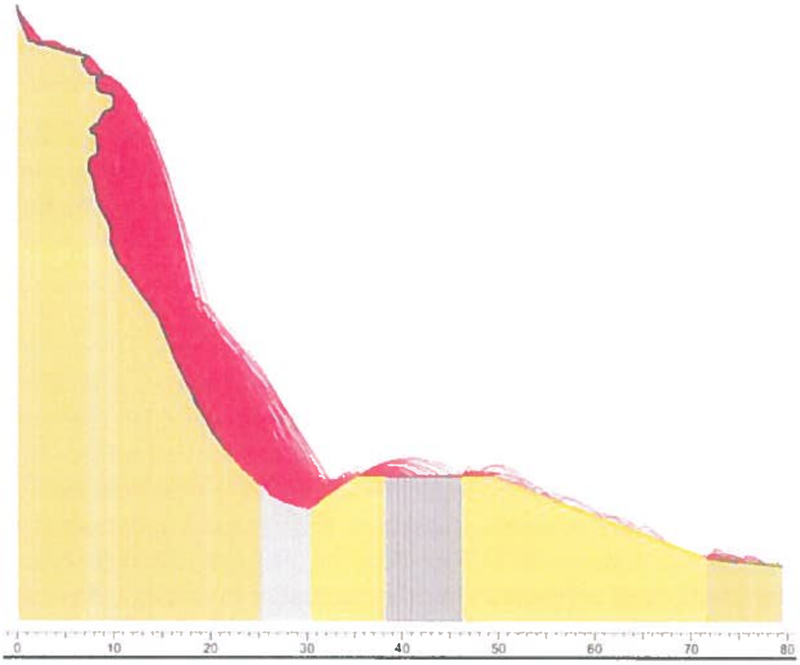


Figure 17: Section 2 with the crest of the embankment at 10mOD and 10m from the toe of the cliff

3) For causeway at 8mOD and inner edge 20m from cliff toe:

When the embankment is lowered to 8mOD, a number of rocks bounce over the pathway.

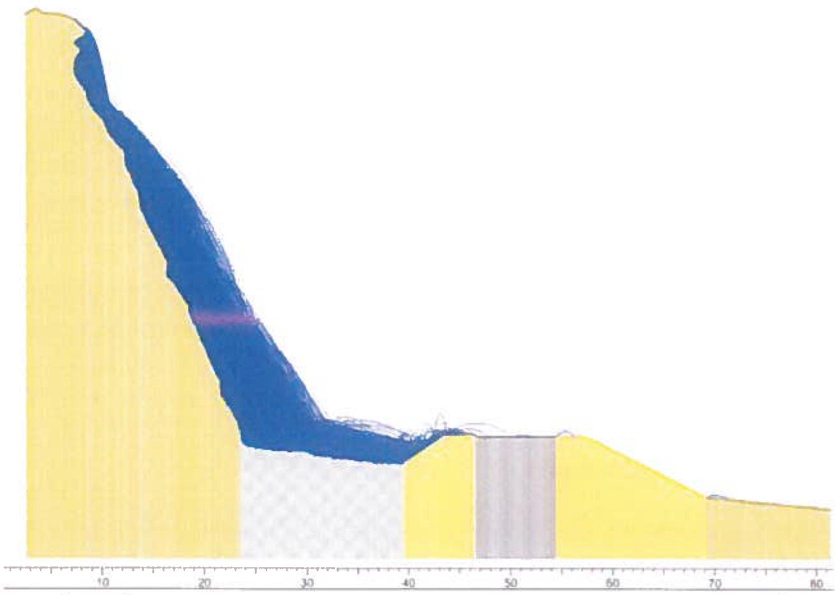


Figure 18: Section 1 with the crest of the embankment at 8mOD and 20m from the toe of the cliff

Conclusions

Based on the analysis undertaken on the three selected sections, when the top of the causeway is at an elevation of 10mOD and is 20m from the toe of the cliff, there is a very low risk that individual rockfall will reach the path.

Based on the trajectories of the falling blocks that were seen during the site walkover, the results of this assessment are considered reasonable.

Lowering the causeway or moving it closer to the cliff increases the risk that individual rockfall may reach the path. During the next stage it is recommended that a more detailed rock fall assessment is undertaken to fully inform the proposed vertical and horizontal alignment of the causeway.

6.6 Geotechnical Considerations

Based on observations made on site and interrogation of survey data, the debris slides that have occurred recently along the cliff extend some 10-15m out onto the beach. It is considered likely that some blocks will have rolled further down the beach in front of the main mass of debris. Based on these observations it is considered likely that future debris slides have the potential to extend as far at the proposed causeway, with a crest at 20m from the cliff face. Further review of historical instability and consideration of the potential to damage the causeway structure will need to be undertaken during the next stage of works.

Limestone and gypsum blocks present on the beach provide a good indication of material that currently falls from the cliff. Based on the rock fall assessments at the three selected sections, a path on a causeway with a crest at 20m from the cliff and at an elevation of 10mOD has a very low risk of being impacted by rockfall. Given that at this stage the rock fall analysis shows there is a residual risk that rock blocks could reach the path, consideration could be given to providing additional protection in locations where the public may spend more time such as areas of seating.

Consideration should be given to inspecting the top of the cliff and removing any potentially unstable structures that may fall from the cliff, for example the remnants of a concrete structure shown in Photograph P10. It is possible that this and other such structures may pose an increased risk over and above that of rock blocks natural falling from the cliff.

If at the connections to land at either end of the causeway, the crest elevation of the causeway is higher than that of the land, transition ramps will be needed. Given accessibility requirements the transitions will need to be at shallow gradients which may result in extensive lengths of ramp. It may be possible to accommodate the change in height on the causeway, potentially providing wave protection with a wall structure.

The causeway will reduce the amount of erosion of the base of the cliff, reducing the potential for undermining and collapse. However, erosion of the face and associated rockfall will continue.

As is evident from the piles of material locally present at the base of the cliff, slips occur, predominantly from the upper parts of the cliff face. These piles of material are gradually eroded away by the sea. Construction of the bund will prevent erosion of the cliff base and therefore over time material will build up between the cliff and causeway. The space between the causeway and cliff will act as a catch ditch for rock fall and therefore future maintenance may be required to manage the debris collecting in the area.

There will be a change in geometry of the cliff over time, and changes at the toe of the cliff as debris collects from the slope and rockfall events that will continue. This change over time will need to be considered in further assessments to be undertaken in Stage 2.

7 Further work

Based on the Stage 1 review presented above, the further maritime and geotechnical works required to progress the development of the Penarth Headland Link can be divided into two stages. Stage 2 being feasibility and scheme design and stage 3 being preparation of documentation for planning application and D&B tender.

The identified works are described below:

7.1 Stage 2

7.1.1 Objective

Feasibility and scheme design.

7.1.2 Scope

Scope as defined at end of Stage 1 to include:

- Review of Stage 1 outcomes and agree way forward; with client and key stakeholders
- Site visit by maritime engineers
- Acquisition of offshore wave data; Joint Probability Assessment of wave heights and water levels; numerical modelling to determine seastate conditions for detailed designs
- Assessment of impacts of the proposed scheme on coastal processes
- Consideration of tie-ins between mainland and causeway and the proposed area of reclaimed land to the north of the proposed causeway
- Detailed desk study (including geology, survey data and aerial photography) to review type and extent of historical instability
- Further assessment of rock fall hazards based on updated causeway alignment
- Review implications of changes I slope geometry due to degradation of the cliff and build-up of material at the base
- Outline geotechnical design of the causeway earthworks
- Integration of design and construction strategy with the constraints and opportunities identified by others (e.g. environmental)
- Pre-application consultations with maritime consenting authorities
- Input into EIA screening and scoping

7.1.3 Deliverables

- Feasibility report
- Scheme design drawings

7.2 Stage 3

7.2.1 Objective

- Documentation for planning application and D&B tender

7.2.2 Scope

- Support to EIA
- Support to stakeholder consultations

7.2.3 Deliverables

- Maritime and geotechnical engineering input into:
    - Planning applications
    - Employers Requirements & Reference Design for D&B construction

References

[1] Coastal Engineering Manual, US Army Corps of Engineers, 2011

[2] Environment Agency, 2016. Adapting to Climate Change: Advice for Flood and Coastal Erosion Risk Management Authorities

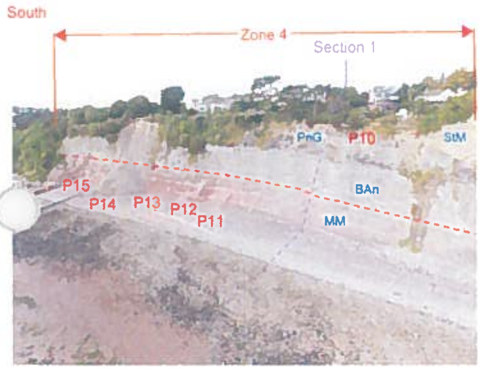
[3] Overtopping Manual, www.overtopping-manual.com (EurOtop, 2016).

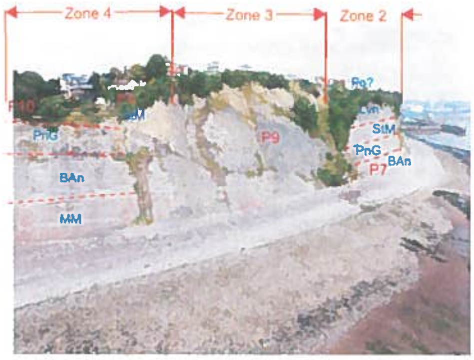
[4] The Rock Manual: the use of rock in hydraulic engineering (2nd edition), C683, CIRIA. London (CIRIA, CUR, CETMEF, 2007)

[5] Penarth Headland Link Planning Application, 2003/01045/REG3

[6] 2014 landslide: https://www.walesonline.co.uk/news/wales-news/penarth-landslide-watch-150-tonnes-6990560

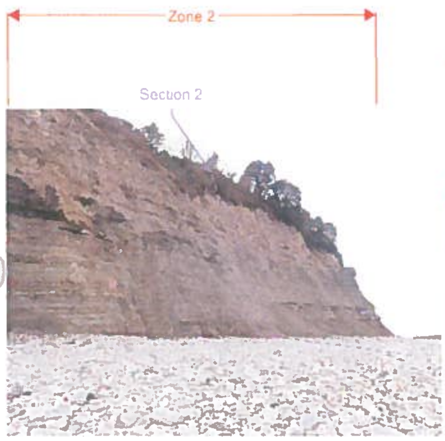
Figures

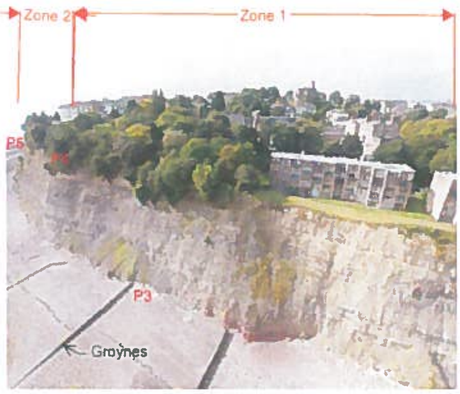


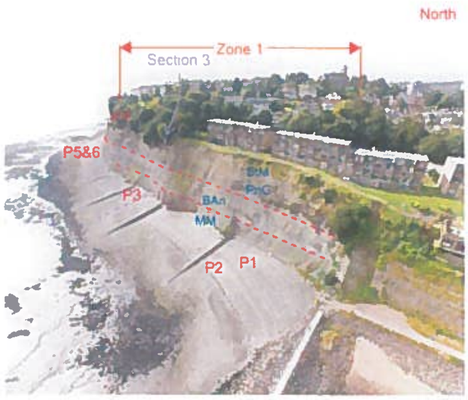


Legend:
P9 = Photograph location
Section 2 = Line of rock fall assessment cross sections

Geological Strata
MM = Mercia Mudstone
BAn = Blue Anchor Formation
PnG = Penarth Group
StM = St Mary's Well Bay Formation
Lvn = Lavernock Shales
Po = Porthkerry Formation







South

P6 - Northern part of Zone 2

rockfall during site visit

P5 - overhangs

P4 - Gypsum bands forming overhangs

P3 - Gypsum and Limestone blocks on the beach

P2 - fallen blocks and debris

North

P1 - Debris at base of cliff

South


P15 - Debris slide


P14 - Debris from the 2014 Slip


P13 - slip and MMG

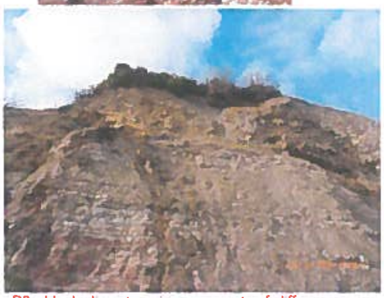

P12 - MMG


P11 - Buttresses of mudstone with vertical joints


P10 - concrete structure at crest

North


P9 - Faulted area

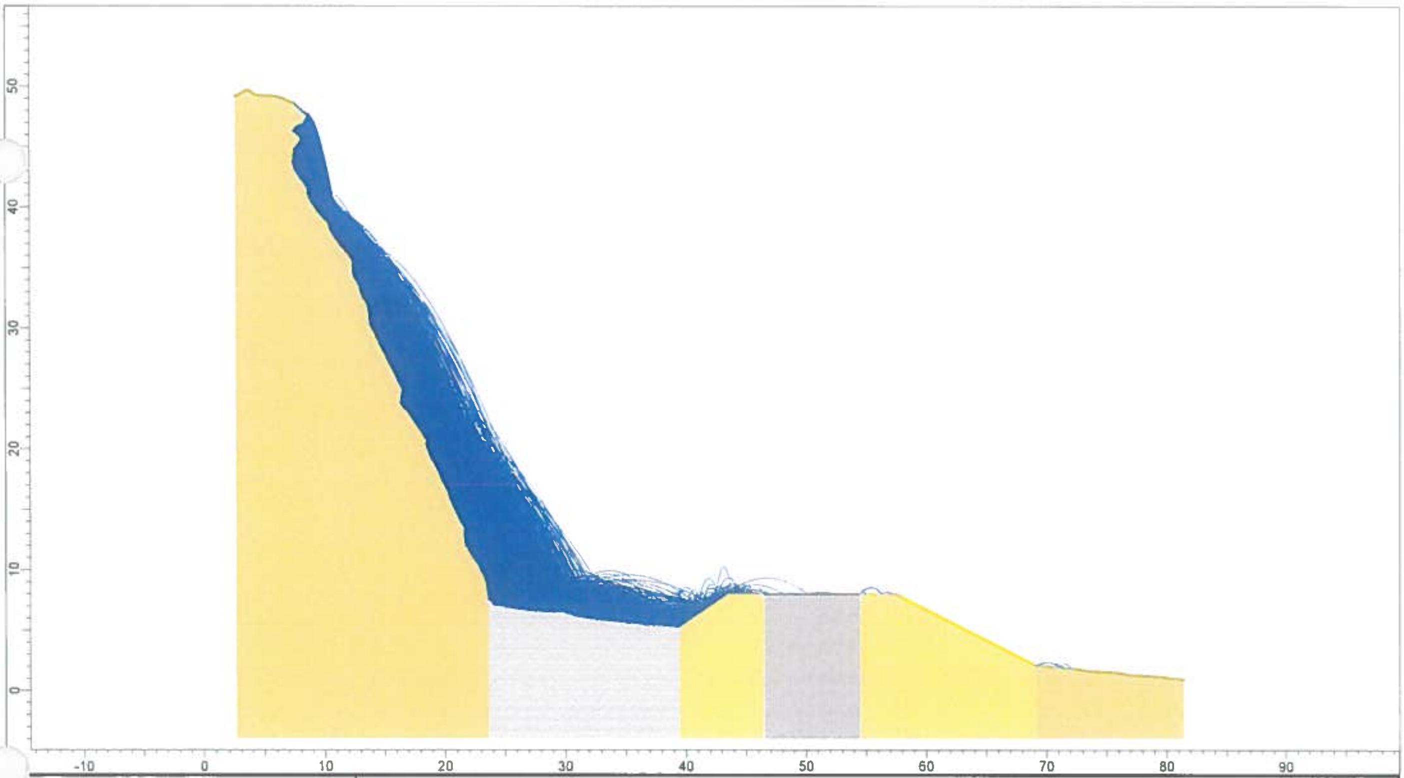

P8 - blocky limestone in upper parts of cliff


P7 - Debris slide and slumped blocks

Figure 14 Site Photographs - Southern

Appendix A

Rock Fall Assessment



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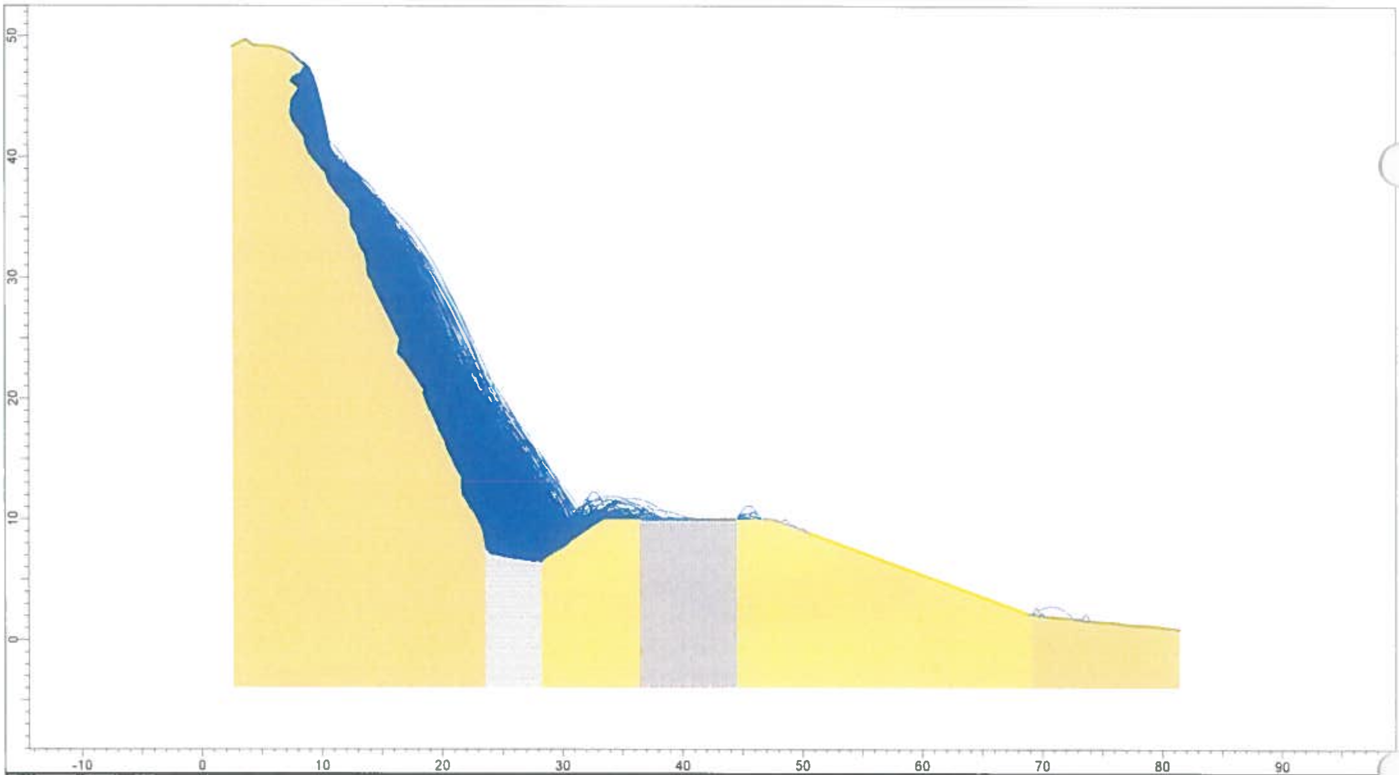
rocscience

ROCFALL 6.005



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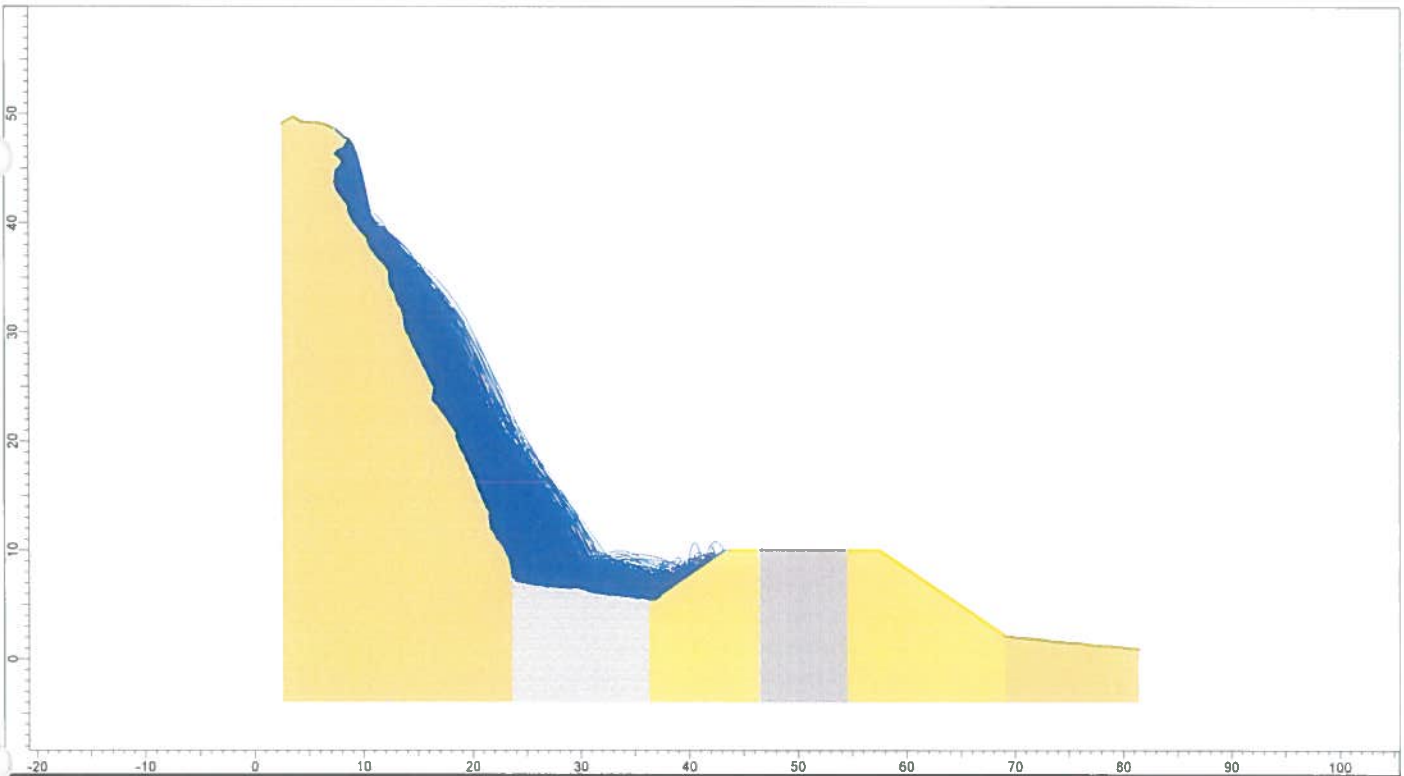
Analysis Description	Section 1 - 10mOD 10m from cliff		
Drawn By	TBW EB	Company	Arup
Date	28/03/2018, 16:08:39	File Name	Section 1 10mOD 20m.fal6

rocscience

ROCFALL 6.005



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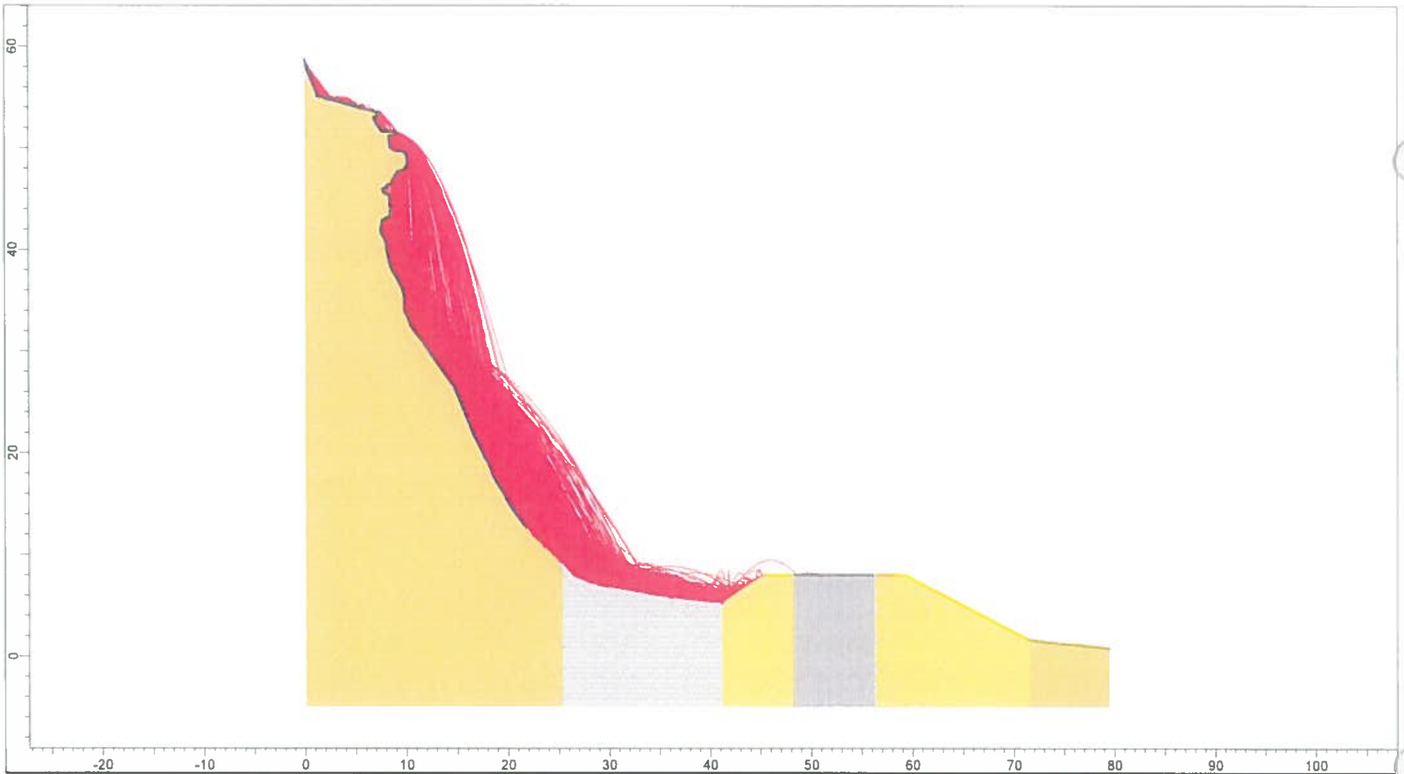

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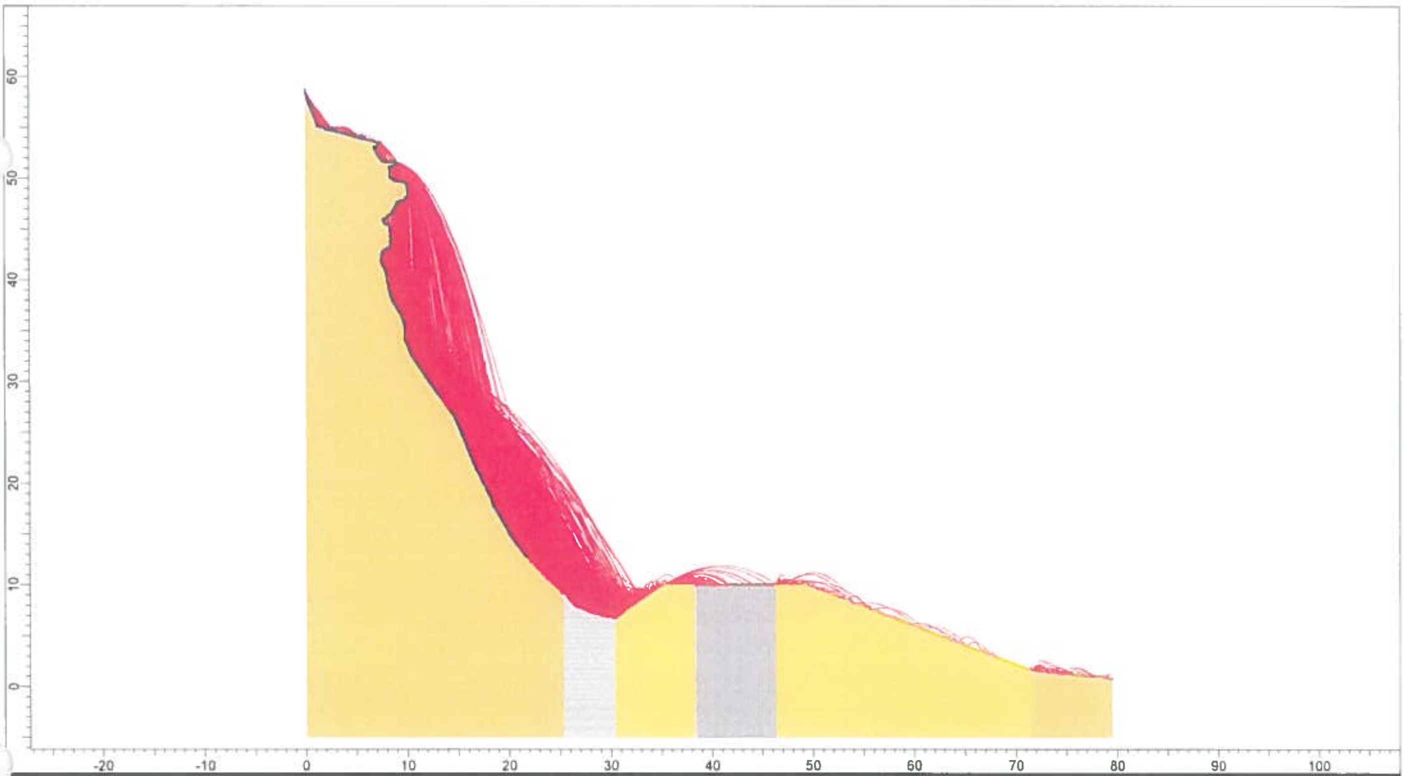

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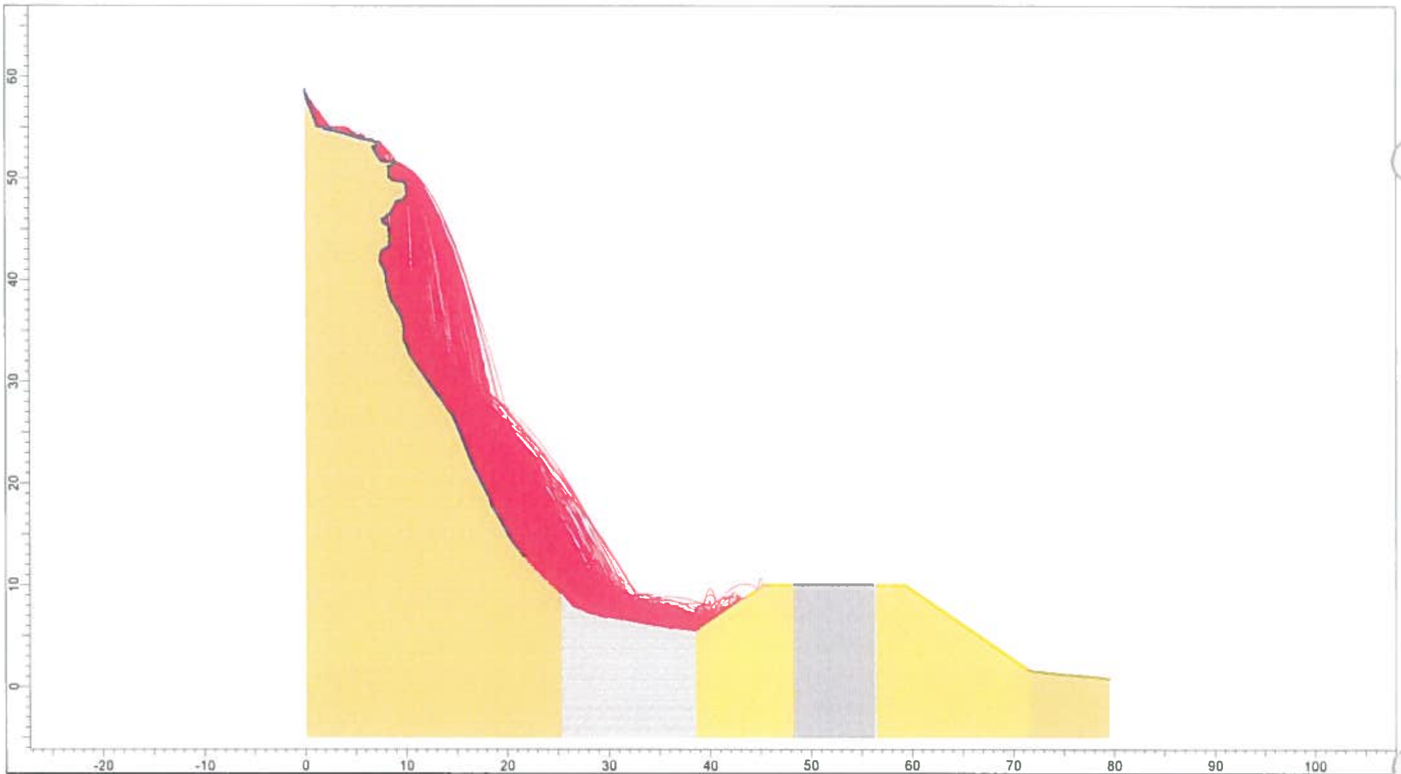

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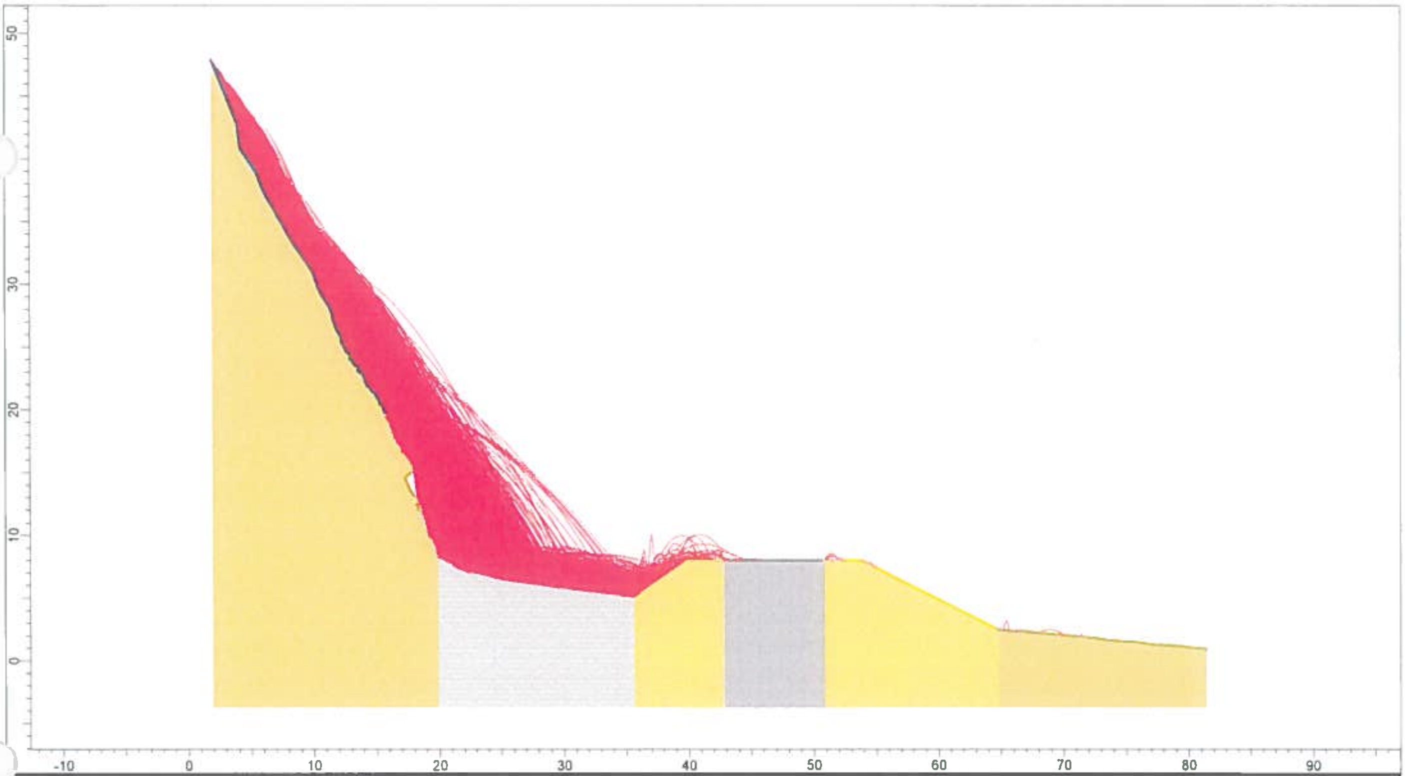

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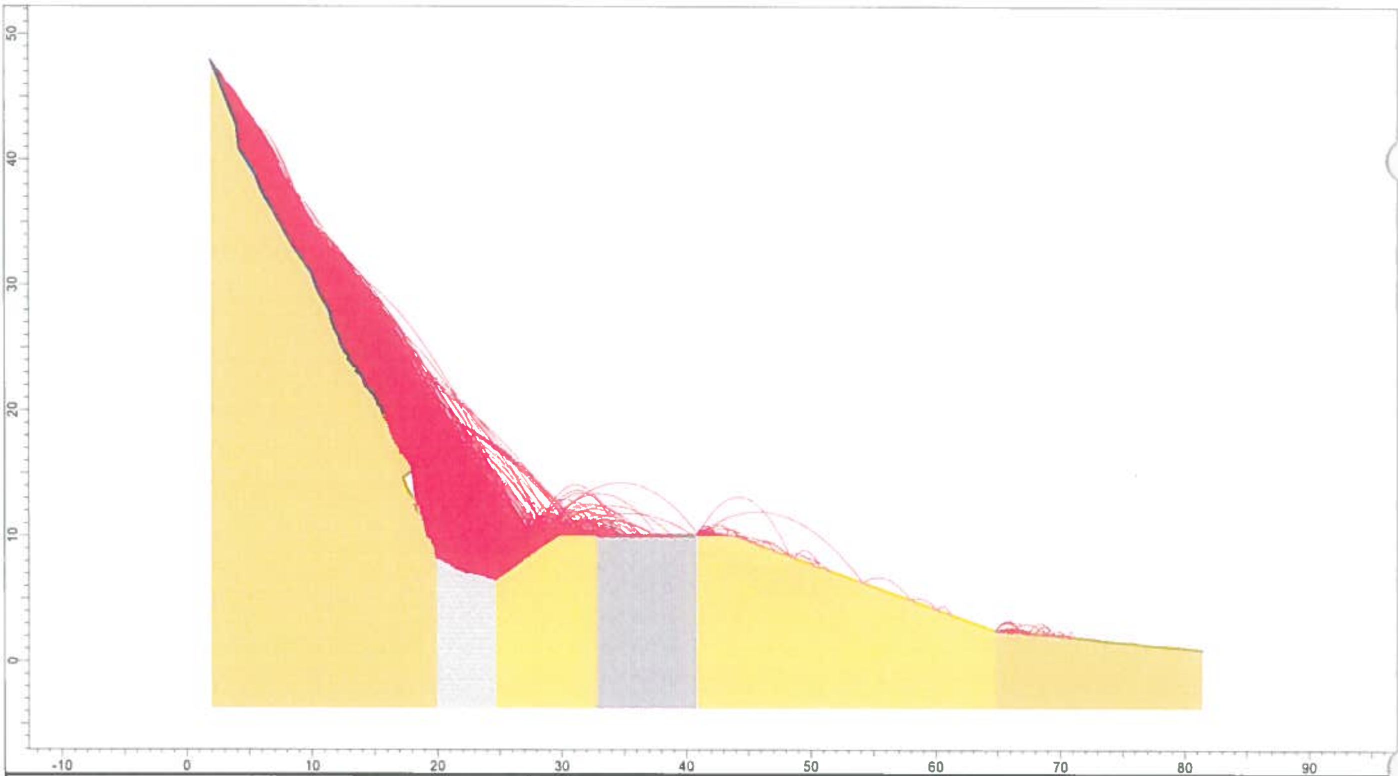

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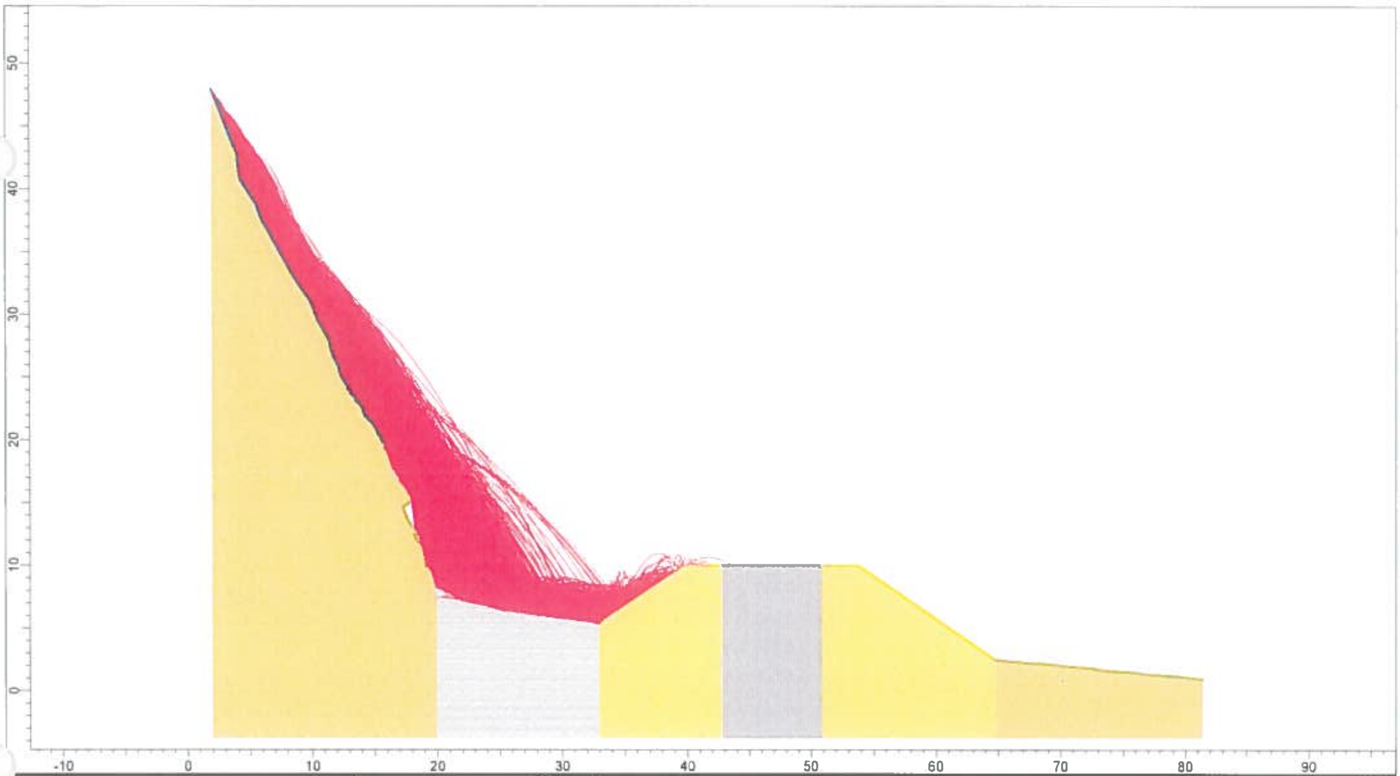

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