ARUP

Kempsey Shire Council

Toose Road Landslide Remedial Works

Temporary Remedial Preliminary Concept Design Development

Reference: 293180-GE-RPT-02

1 | 23 June 2023

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1. Introduction

1.1 Project Background

A landslide occurred through a section of Toose Road, Kempsey Shire, approximately 5km west Bellbrook, NSW on 31 March 2022. Kempsey Shire Council (KSC) has previously engaged GHD and Regional Geotechnical Solutions (RGS) to provide support services, including advice on remediation of the landslide and alternative access arrangements.

KSC engaged Arup in December 2022 to prepare a geotechnical desktop review of the landslide, including Site walkover, review of available geotechnical information, aerial imagery and survey data and a peer review of previous remediation advice provided by GHD and RGS. The study also identified options for remediation of the landslide. The desktop review was completed in February 2023 (Arup, February 2023).

Key findings of the desktop study are summarised below:

- The section of Toose Road where the landslide occurred, appears to have been cut through an old landslide as evidenced by the colluvium exposed in the cutting and tension cracks in the road. There are multiple layers to the current slope activity. The slope is currently not stable and could move at any time.
- Previous work by others has not identified a viable remedial solution for stabilising the landside. It is unlikely that a practical, cost efficient and stable long-term remedial solution will be possible. Further slides in the future can be reasonably expected and a slide that permanently closes the road at some point in the future is likely.
- A temporary remedial solution, involving moving the road into the slope, controlling surface water and removing the active landslide material, should be possible. However, the upslope material will be marginally stable and further landslides are to be expected, especially during periods of significant rainfall. The risk to people would need to be managed by implementing practical controls, such as not using the road during rainfall, regularly inspecting the slope for signs of instability prior to crossing and the like. Access would need to be restricted to essential users only and they would need to be educated about the risks of continued use. Regular removal of landslide debris from the road is likely to be required.

Following completion of the desktop review and a meeting on Site with several Contractors, TfNSW and KSC, has requested Arup to further develop a temporary remedial earthworks solution Preliminary Concept design.

1.2 Purpose of This Report

The purpose of this report is to present the temporary remedial solution Preliminary Concept design, following completion of the desktop study.

1.3 Limitations

This report has been developed based on the information made available to Arup prior to 23 May 2023. Should further information that may change the content of this report be available, it should be reviewed and revised (if required) by a suitably qualified geotechnical engineer.

No geotechnical investigations have been completed as part of the scope of this report due to site access constraints and slope stability issues. The information is provided to inform the Preliminary Concept design for the temporary works solution and requires verification in future project stages and likely full-time geotechnical supervision during construction. The information is not for detailed design. No responsibility is provided to third parties using the information and interpretations provided within this report.

There are inherent uncertainties in geotechnical engineering. The ground is a product of continuing natural and artificial processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties to understand or predict the behaviour of the ground and groundwater on a particular site and under particular conditions.

2. Site Work Since Completion of the Desktop Study

2.1 2nd Site Visit 24 March 2023

Arup attended a second site visit on 24 March 2023 with representatives from KSC, TfNSW, Pan Civil, Rixam and Malcolm Civil Earthworks . Observations indicated that the site is still very active, with tension cracks/scarplets in the topsoil and vegetation mat, trees moving, local debris falls, especially through the bluff were observed. Amongst the Contractors who visited Site in March, there were indications that construction of the earthworks solution with precautions should be feasible.

2.2 Drone LiDAR and Aerial Imagery

A drone based survey of the landslide and surrounding area was carried out by Diospatial on 5 April 2023 (Diospatial, April 2023). The data captured included high resolution aerial imagery, drone-LiDAR and GNSS ground control survey.

This data was provided to Arup and has been compared to previous aerial photographs and LiDAR as discussed in the following sections.

2.2.1 April 2023 Aerial Photography

Figure 1 shows the aerial photography captured in April 2023 annotated with site features observed during the recent site visit and on the provided drone photography. Figure 2 shows a photograph captured by Diospatial in April 2023 compared to a photograph captured by Council in April 2022.

Comparison of the April 2022 to April 2023 photographs and observations during the site visit on 13 January 2023 and visit on 24 March 2023 indicates the overslip (yellow) to be active and currently moving. This is a development in understanding from the new information available since the desktop study.

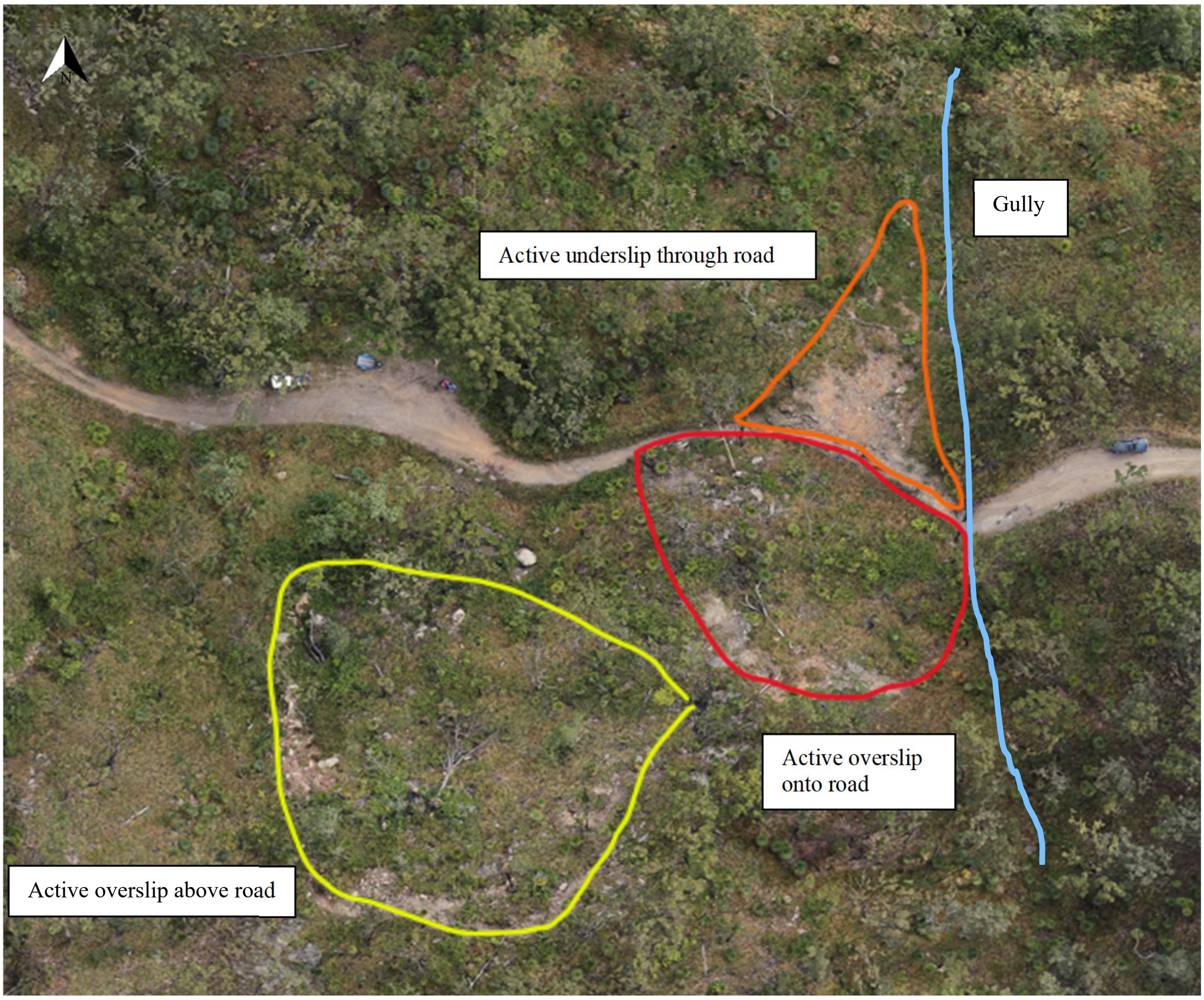

Figure 1: Aerial photographs from April 2023 annotated with observed features adjacent to the closed section of road

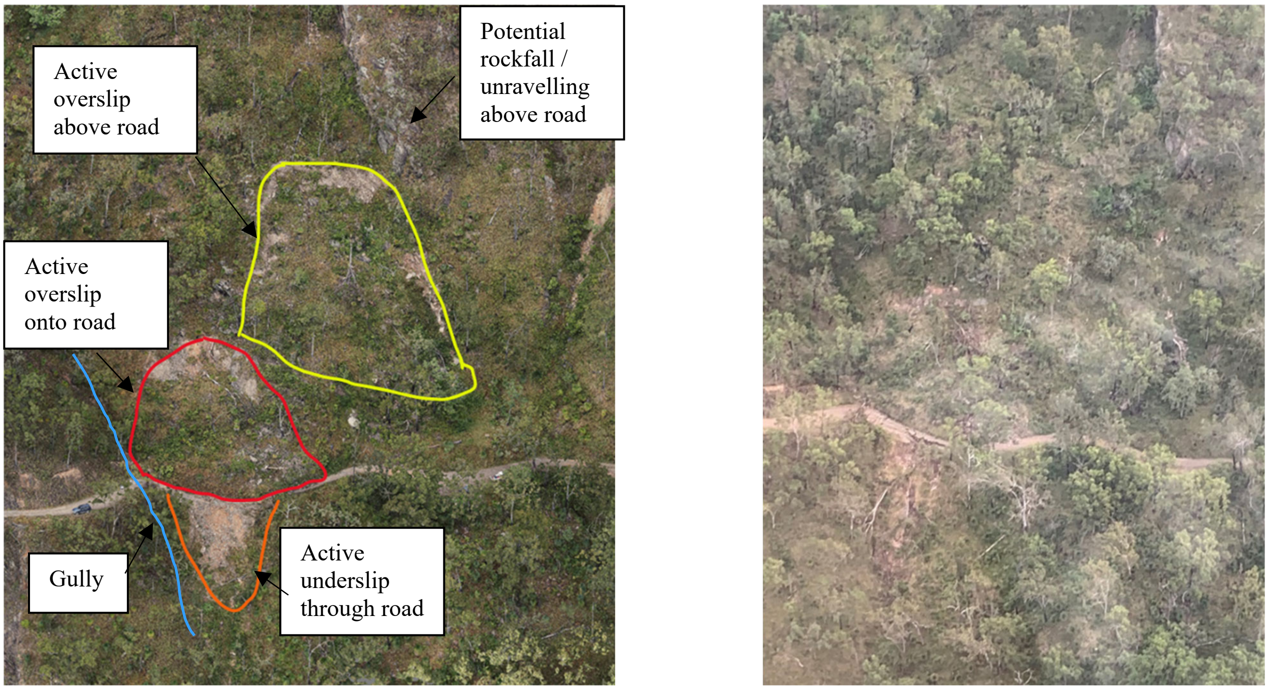

Figure 2: Annotated drone photographs from April 2023 compared to a similar photograph from April 2022

2.2.2 Comparison of 2023 and 2018 LiDAR

The LiDAR captured by Diospatial in April 2023 was compared to the LiDAR previously provided by Council which was understood to be captured in May 2018. An annotated plot of the difference in depth between the two LiDAR sets is provided in Figure 3 below. The slips shown in Figure 1 and Figure 2 are also sketched onto the plot in Figure 3. The scarp and toe of each slip can be seen.

Both sets of LiDAR have been plotted on the sections prepared for the Preliminary Concept design (refer Section 0).

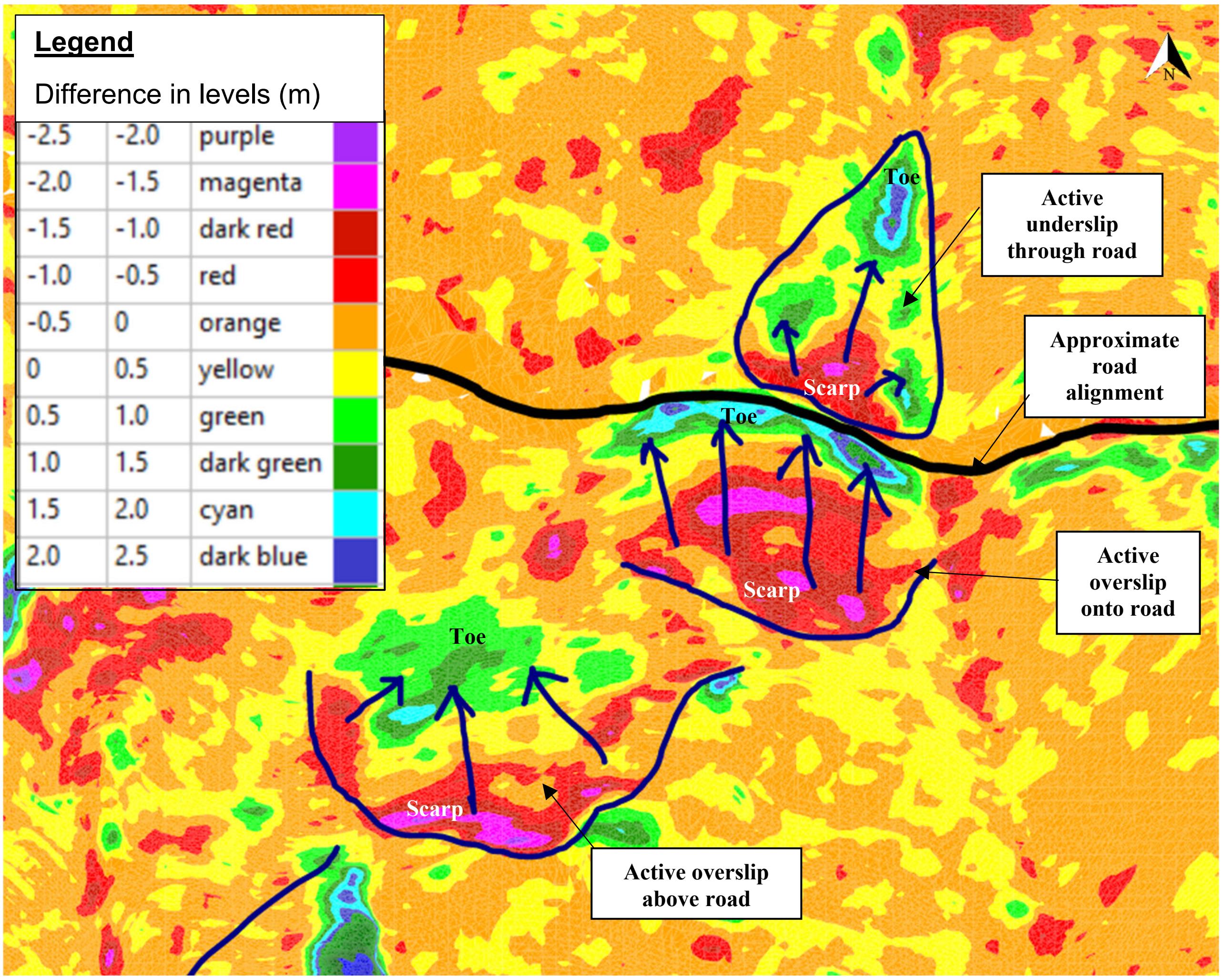

Figure 3: Comparison of 2023 and 2018 LiDAR

It is noted that the 2018 LiDAR was previously understood to be in GDA94 datum, however on comparison to the Diospatial LiDAR appeared to be to the same datum i.e. GDA2020 datum. This was discussed with Council on 1 May 2023 and it was noted that there are known instances where similar data is labelled as GDA94 but is in GDA2020. It was suggested that if it looks like the 2018 data is in GDA2020 it probably is, so we have assumed that both data sets are in GDA2020.

3. Temporary Remedial Preliminary Concept Design Development

The temporary remedial Preliminary Concept design has been further developed to incorporate the findings of the new information discussed in Section 2.

The temporary remedial Preliminary Concept involves realigning the road into the slope, controlling surface water, and removing the active landslide material.

Preliminary Concept earthworks drawings have been developed and are attached in Appendix A. Note the comment about datums in Section 2.2.2.

Geotechnical analysis of the of the temporary remedial earthworks Preliminary Concept is presented in Appendix B.

4. Conclusion and Recommendations

The rainfall around the time of the initial failure, site observations and stability analysis all indicate that rainfall is a significant factor driving instability.

The slope stability analysis indicates the following key points:

- further shallow failures during periods of significant rainfall are to be expected. These will be manifested as further movements of any remaining vegetation mat on steeper sections and movement of the surface materials in any unvegetated zones.
- deeper failures are unlikely especially during dry periods.
- The temporary Preliminary Concept design section analysed has an adequate FoS for a dry slope when assessed against GTD2018/001.

The temporary remedial solution could be implemented with appropriate controls to reduce risk to workers during construction and residents once completed. The key control would be to restrict access to the area during periods of wet weather. Some examples of controls are:

- Detailed design – keep slopes as flat as possible, improve drainage and control surface water to direct water away from the slip area and to prevent concentrated flows.
- The existing streams will need to have a shallow concrete apron/channel developed to direct flows in an appropriate fashion.
- During construction – *Additional precautions and controls will be needed during wet spells with potential restrictions on work*, implement exclusion zones in unstable areas, full time supervision of the work by a geotechnical engineer to direct work and assess temporary stability during construction, daily inspections of all work areas for signs of instability etc.
- Completed works – *Additional precautions and controls will be needed during wet spells with potential restrictions on work*, regularly inspecting the slope for signs of instability prior to crossing and the like, restricting access to essential users only and educating users about the risks.
- Practical monitoring such as installing ‘trip lines’ or similar should be considered in detailed design.

Regular removal of landslide debris would be required, and it is possible that another slip which permanently closes the road may occur.

5. References

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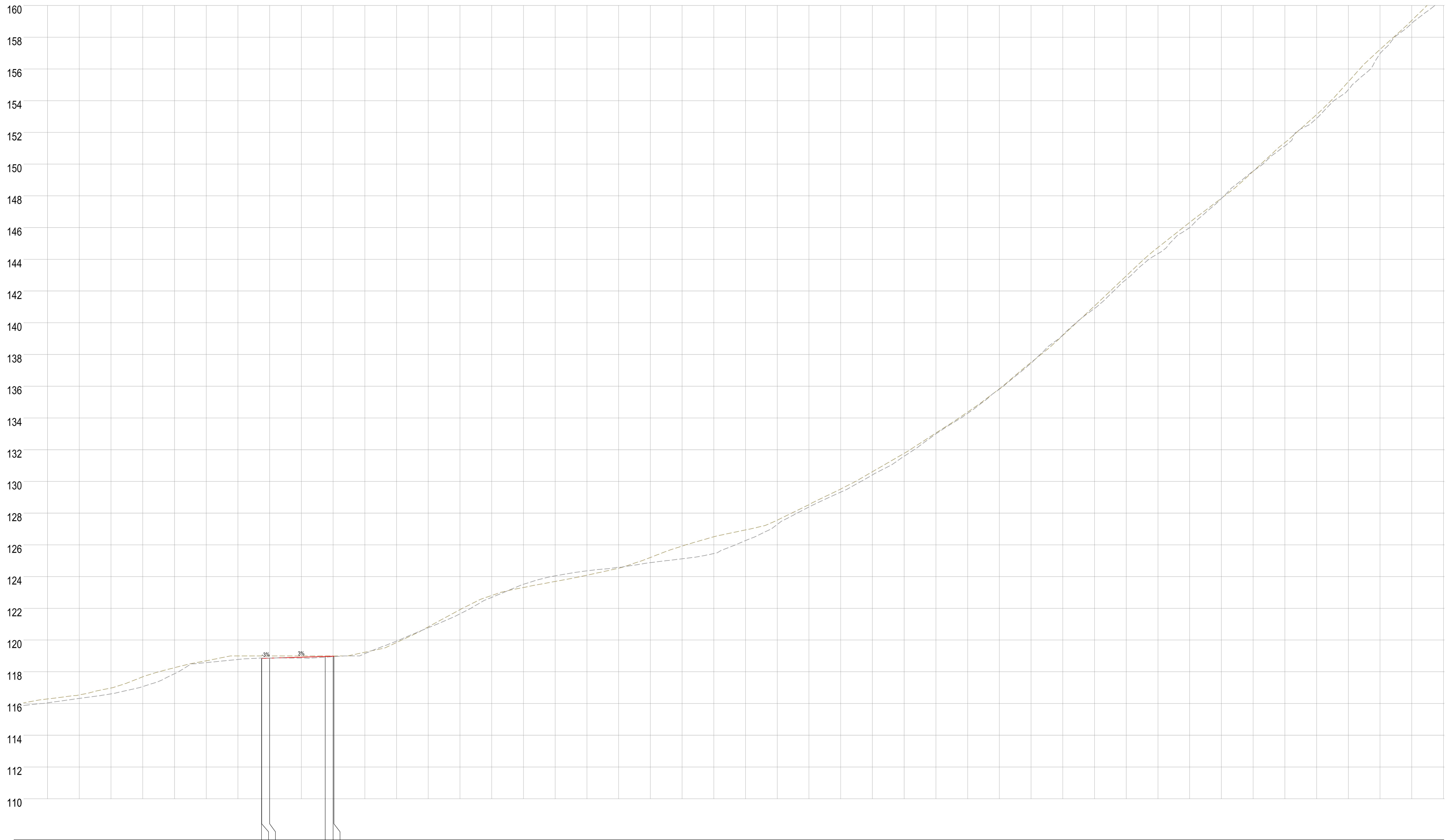
Appendix A

Preliminary Concept Earthworks Drawings

TOOSE ROAD SLIP REALIGNMENT PLAN

PRELIMINARY CONCEPT

12d Model
Scale 1:200
Wed May 3 14:55:10 2023



	IA00C	EH00	ES00	LE0I	EH0I	IA0IF
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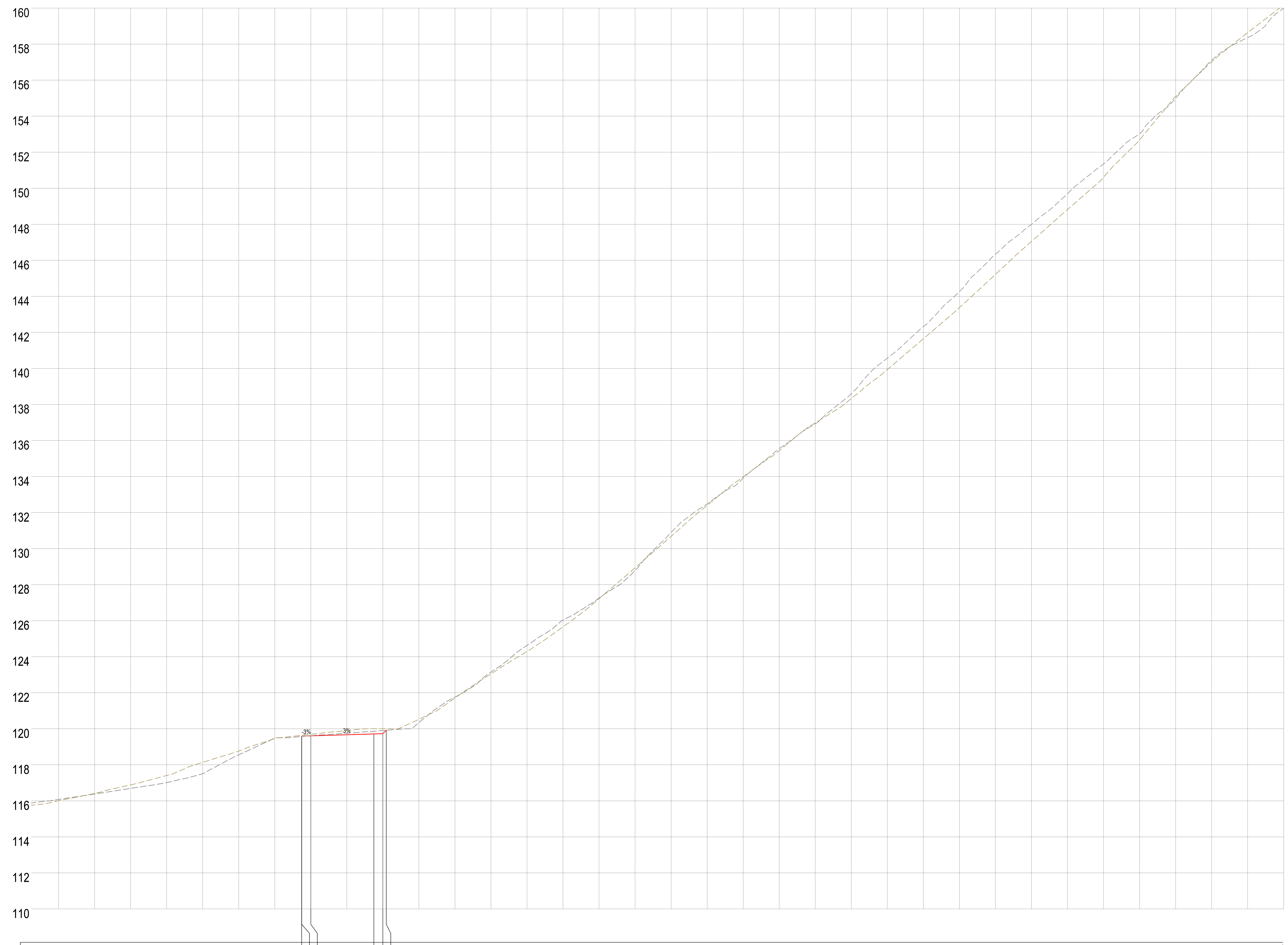
DESIGN LEVELS	118.864	118.845	118.860	118.965	118.980	118.955
EXISTING LEVELS	118.864	118.864	118.868	118.919	118.952	118.955
DESIGN OFFSET	-0.509	-0.500	0.000	3.500	4.000	4.051

CHAINAGE 0

PRELIMINARY CONCEPT

EXISTING LIDAR SURVEY 2018
EXISTING LIDAR SURVEY 2023

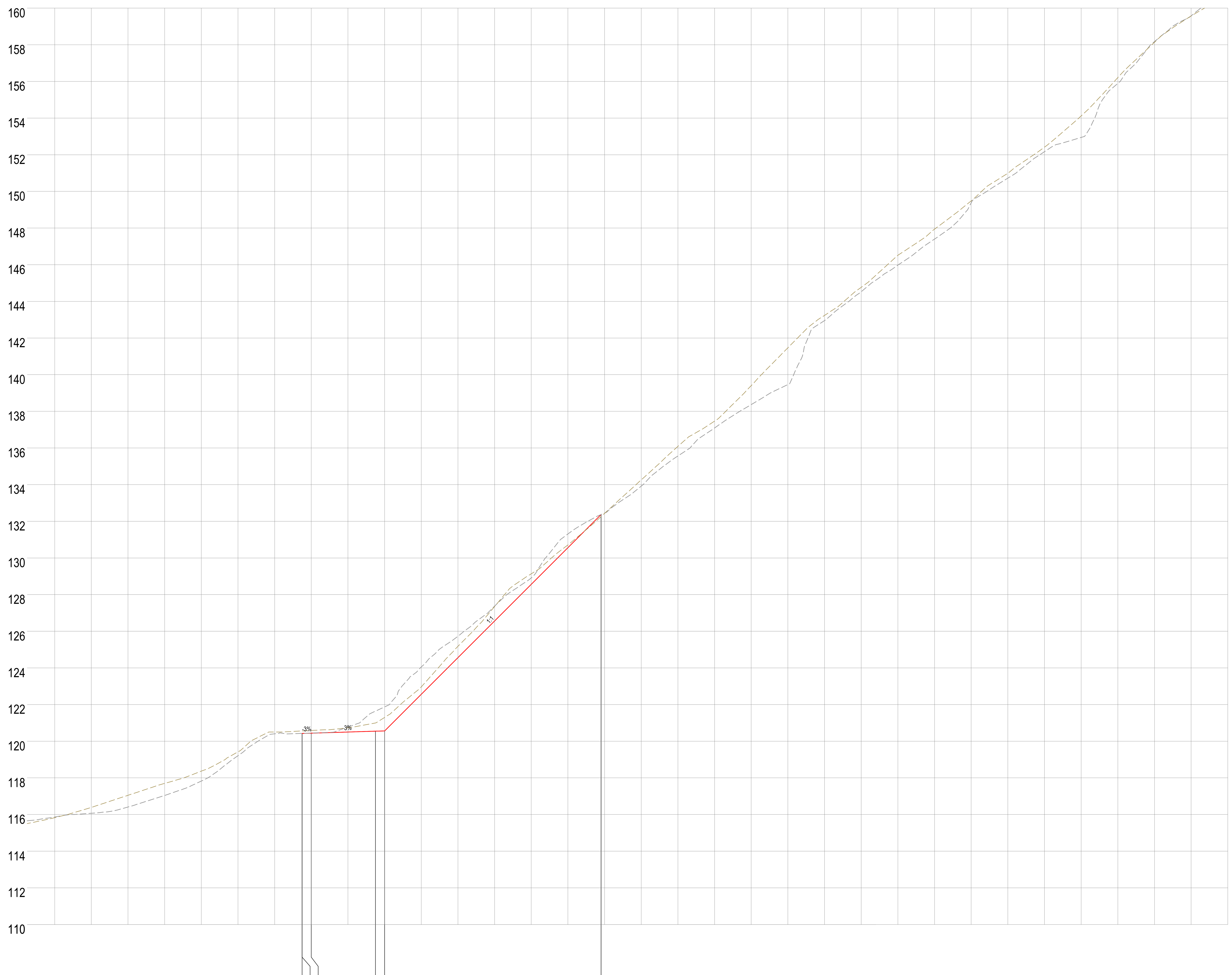


CUT STRINGS	IA00F	EH00	ES00	LE0I	EH0I	IA0IC
LEVEL DIFFERENCE	0.000	0.021	-0.009	-0.153	-0.177	0.000
DESIGN LEVELS	119.569	119.591	119.606	119.711	119.726	119.917
EXISTING LEVELS	119.569	119.570	119.615	119.865	119.903	119.917
DESIGN OFFSET	-0.511	-0.500	0.000	3.500	4.000	4.191

CHAINAGE 10

PRELIMINARY
CONCEPT

EXISTING LIDAR SURVEY 2018
EXISTING LIDAR SURVEY 2023

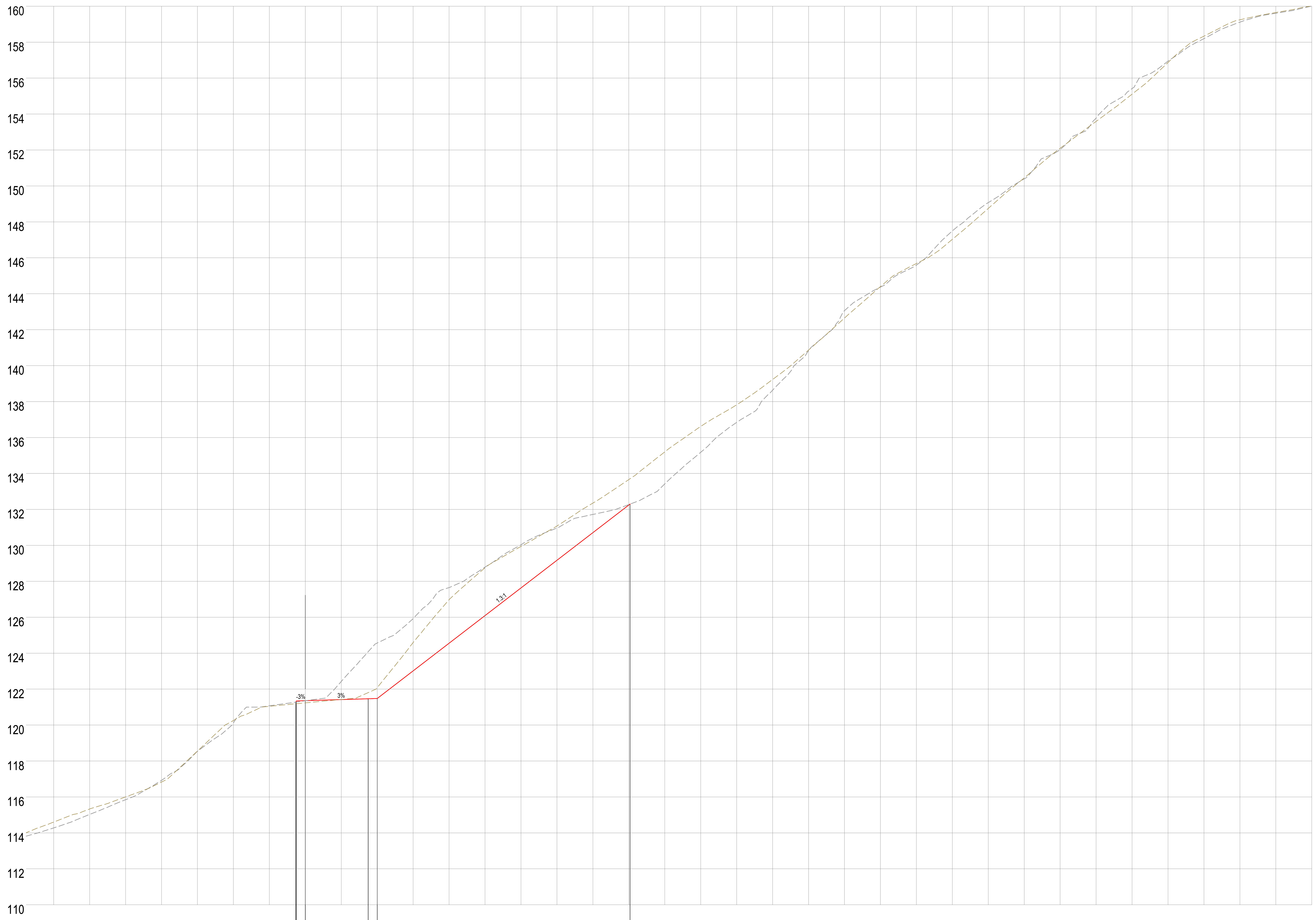


CUT STRINGS	IA00C	EH00	ES00	LE0I	EH0I	IA0IC
LEVEL DIFFERENCE	0.000	-0.003	-0.006	-1.094	-1.313	0.000
DESIGN LEVELS	120.429	120.426	120.441	120.546	120.561	132.369
EXISTING LEVELS	120.429	120.429	120.447	121.640	121.874	132.369
DESIGN OFFSET	-0.502	-0.500	0.000	3.500	4.000	15.808

CHAINAGE 20

PRELIMINARY CONCEPT

EXISTING LIDAR SURVEY 2018
EXISTING LIDAR SURVEY 2023

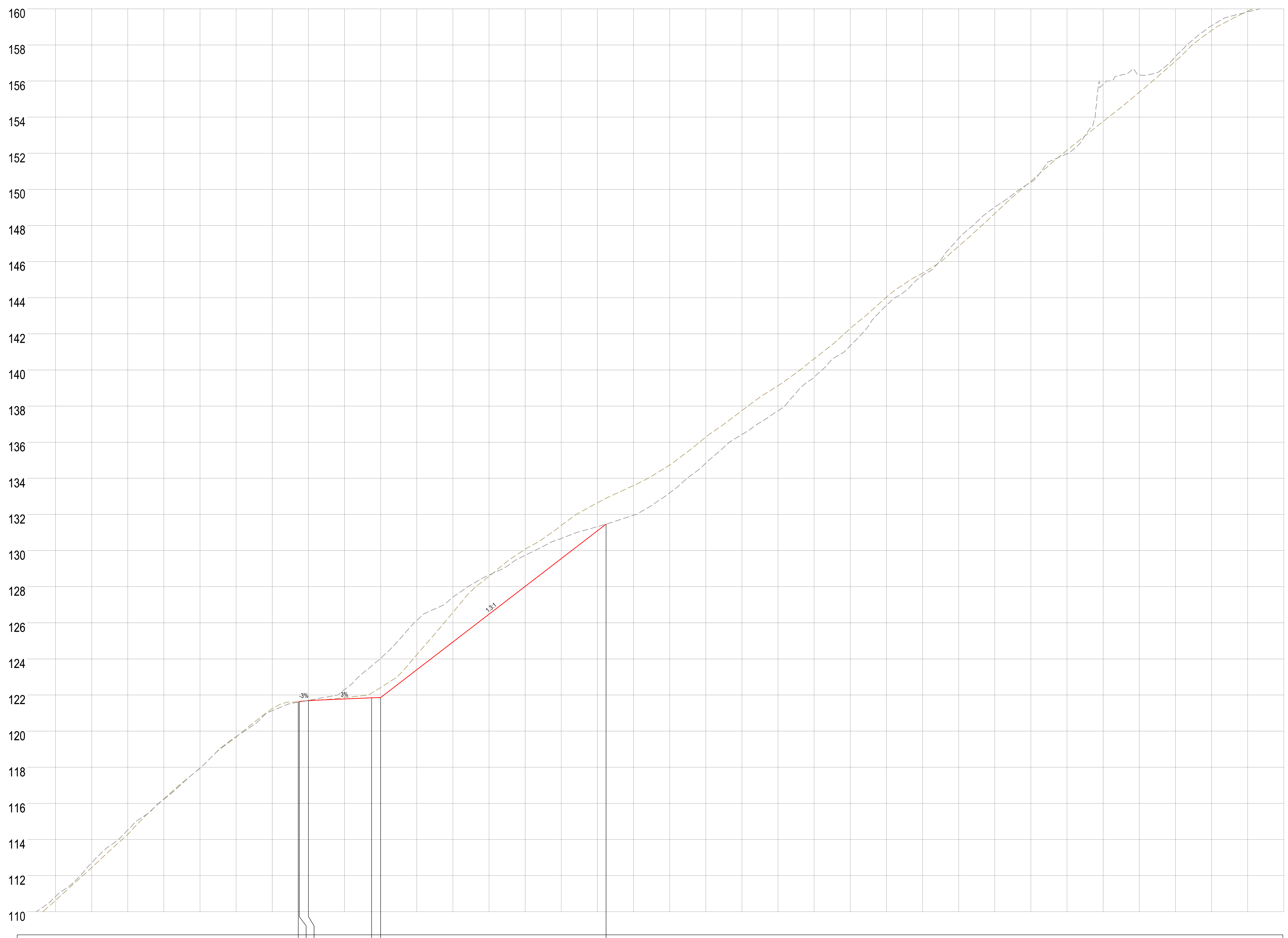


CUT STRINGS	IA00F	EH00	ES00	LE0I	EH0I	IA0IC
LEVEL DIFFERENCE	0.000	0.061	0.006	-2.624	-3.086	0.000
DESIGN LEVELS	121.276	121.342	121.357	121.462	121.477	132.296
EXISTING LEVELS	121.276	121.281	121.351	124.086	124.563	132.296
DESIGN OFFSET	-0.533	-0.500	0.000	3.500	4.000	18.065

CHAINAGE 30

PRELIMINARY CONCEPT

EXISTING LIDAR SURVEY 2018
EXISTING LIDAR SURVEY 2023

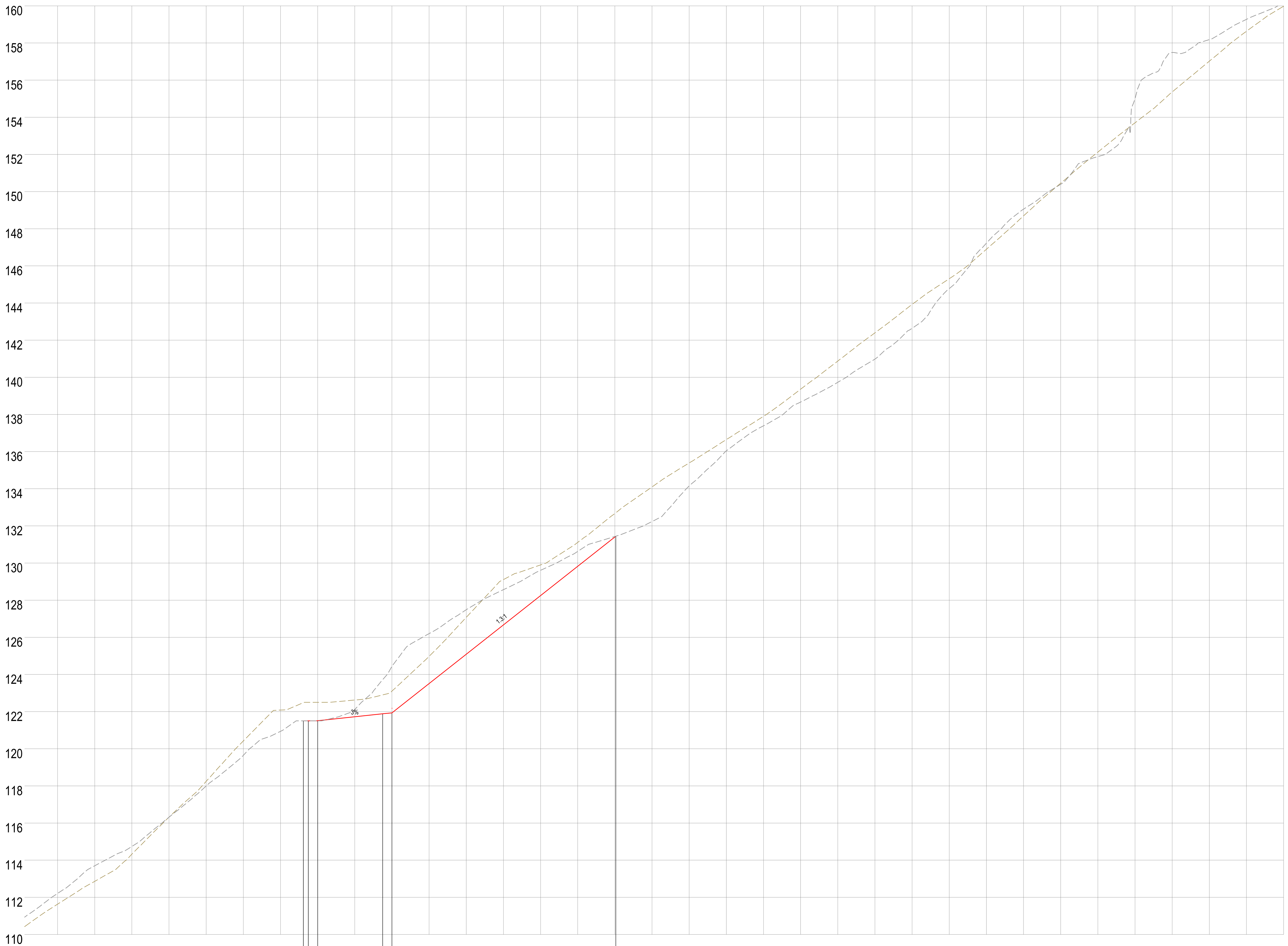


CUT STRINGS	IA00F	EH00	ES00	LE0I	EH0I	IA0IC
LEVEL DIFFERENCE	0.000	0.111	0.035	-1.763	-2.166	0.000
DESIGN LEVELS	121.603	121.725	121.740	121.845	121.860	131.461
EXISTING LEVELS	121.603	121.614	121.705	123.608	124.026	131.461
DESIGN OFFSET	-0.561	-0.500	0.000	3.500	4.000	16.482

CHAINAGE 40

PRELIMINARY CONCEPT

EXISTING LIDAR SURVEY 2018
EXISTING LIDAR SURVEY 2023

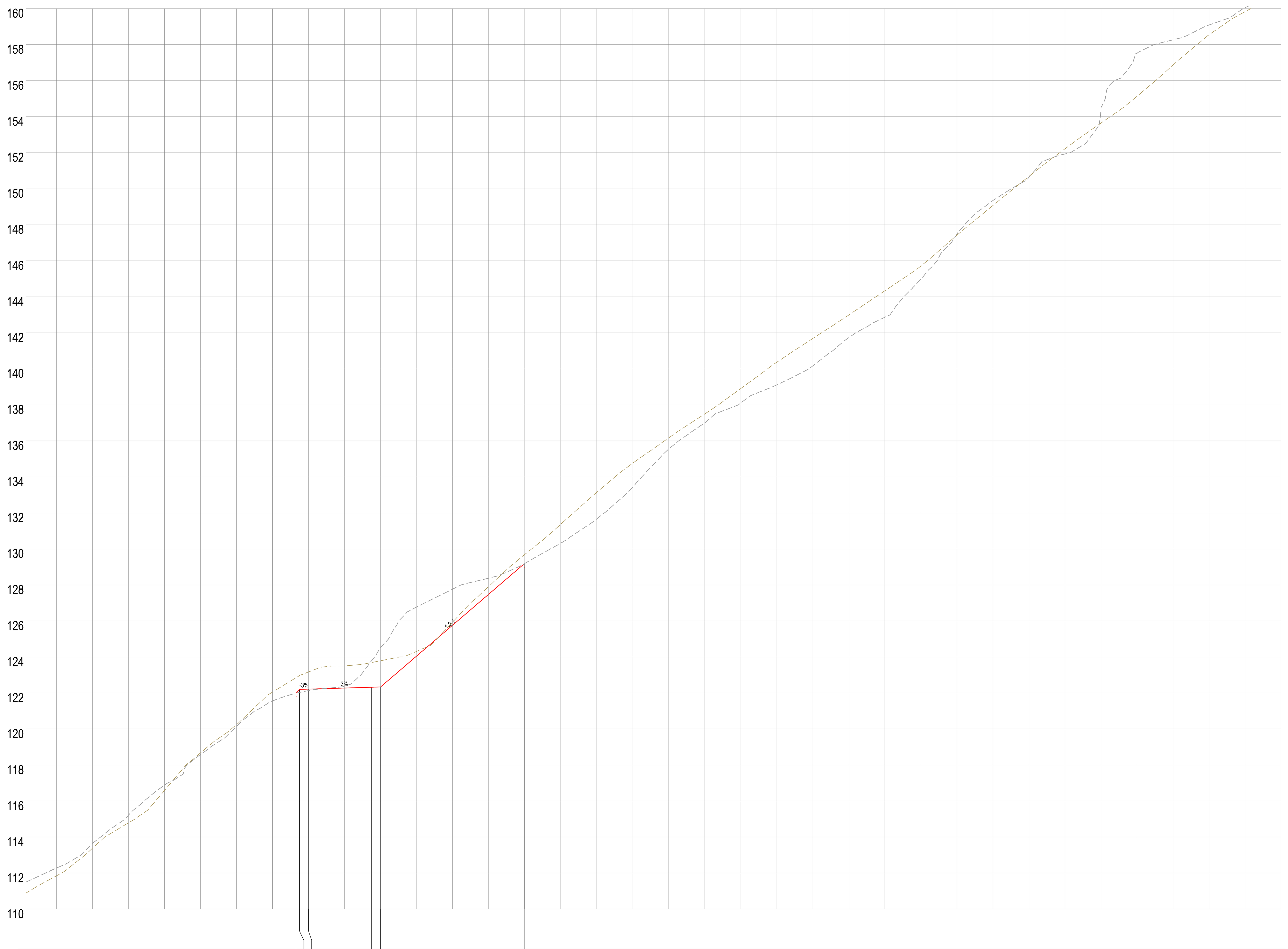


CUT STRINGS	IA00F	EH00	ES00	LE0I	EH0I	IA0IC
LEVEL DIFFERENCE	0.000	0.244	0.259	-1.854	-2.557	0.000
DESIGN LEVELS	121.500	121.744	121.759	121.864	121.879	131.446
EXISTING LEVELS	121.500	121.500	121.500	123.717	124.436	131.446
DESIGN OFFSET	-0.772	-0.500	0.000	3.500	4.000	16.048

CHAINAGE 50

PRELIMINARY CONCEPT

EXISTING LIDAR SURVEY 2018
EXISTING LIDAR SURVEY 2023

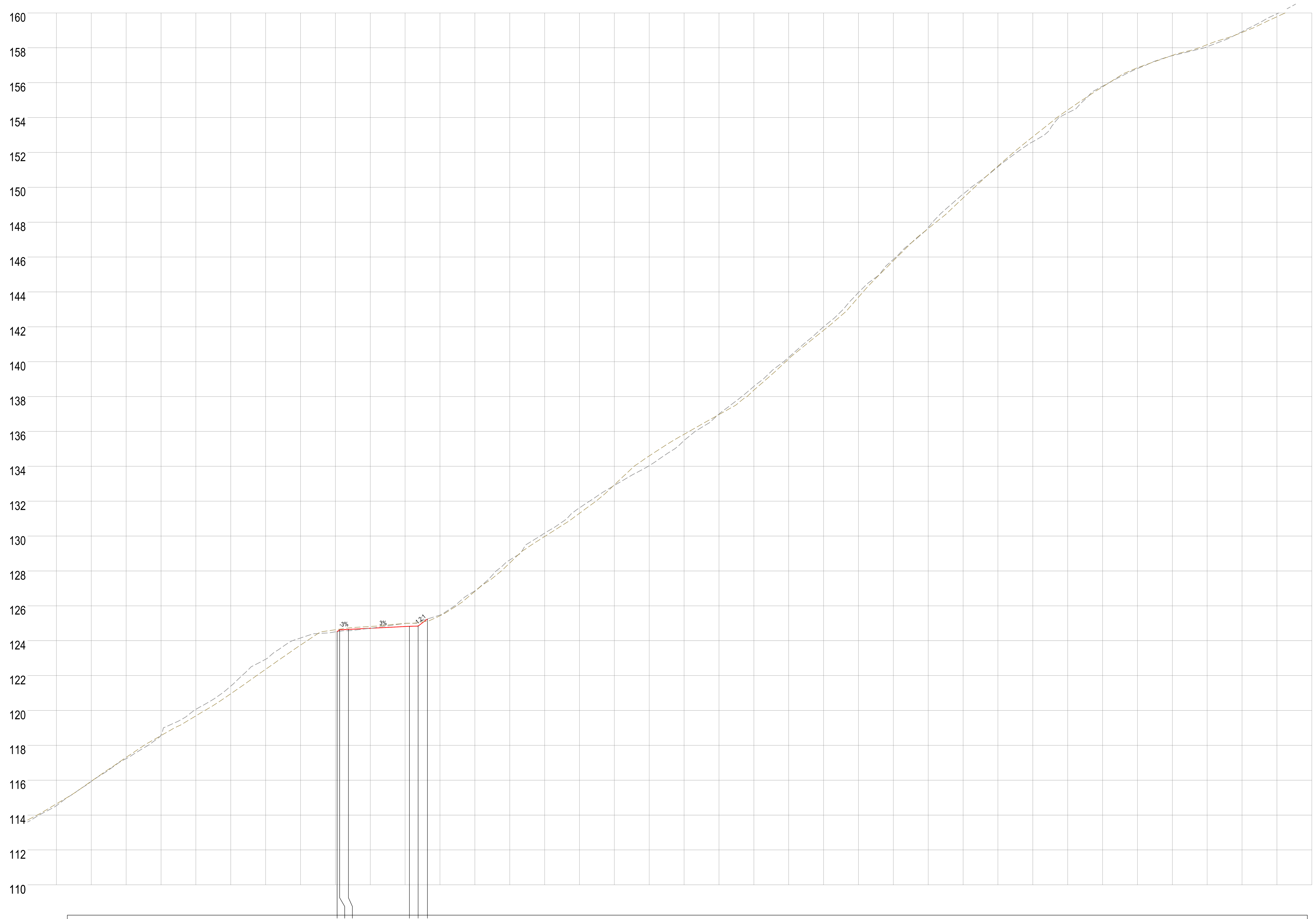


CUT STRINGS	IA00F	EH00	ES00	LE0I	EH0I	IA0IC
LEVEL DIFFERENCE	0.000	0.358	0.277	-1.280	-1.996	0.000
DESIGN LEVELS	121.996	122.394	122.409	122.514	122.529	129.175
EXISTING LEVELS	121.996	122.036	122.132	123.793	124.525	129.175
DESIGN OFFSET	-0.699	-0.500	0.000	3.500	4.000	11.975

CHAINAGE 60

PRELIMINARY CONCEPT

EXISTING LIDAR SURVEY 2018
EXISTING LIDAR SURVEY 2023

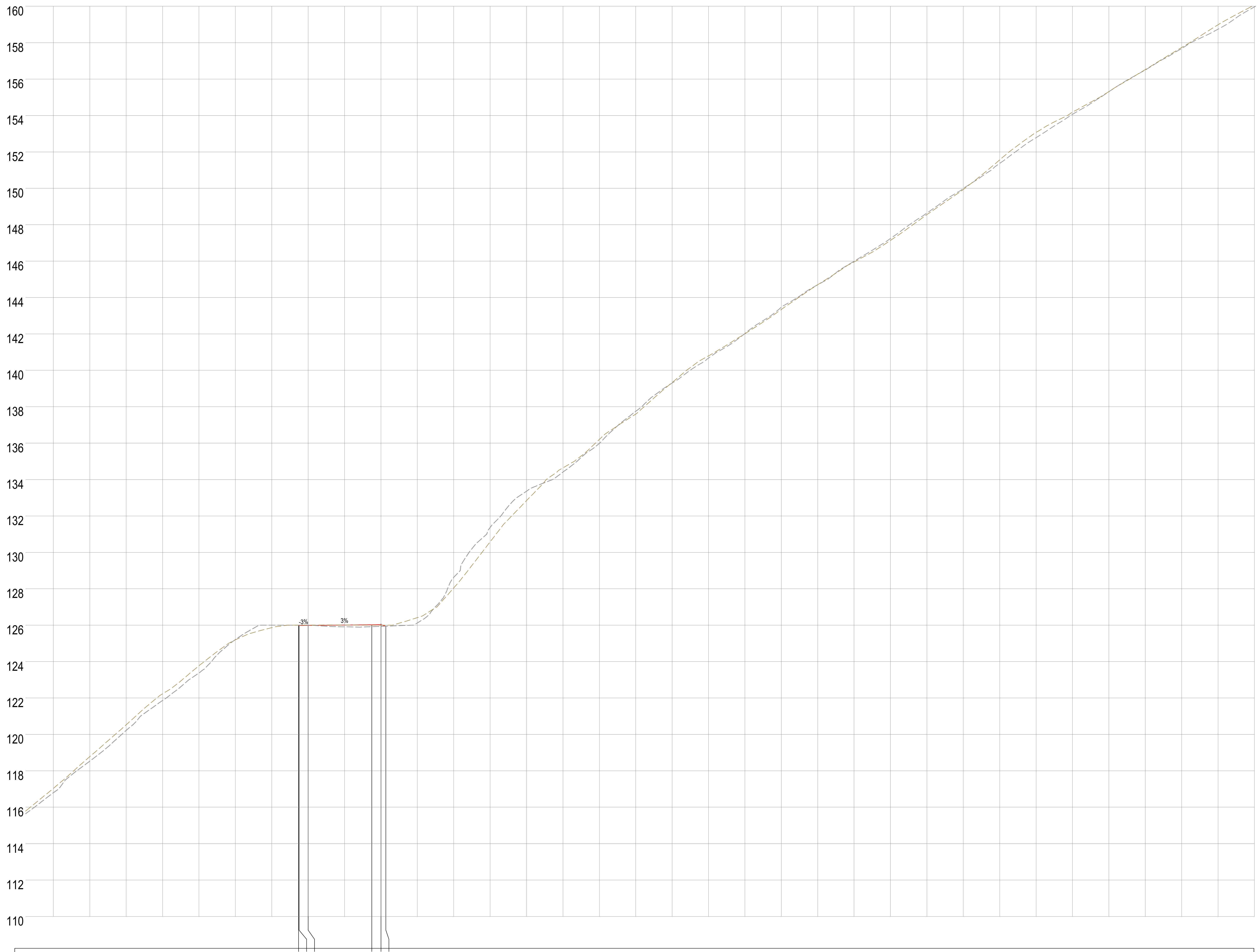


CUT STRINGS	IA00F	EH00	ES00	LE0I	EH0I	IA0IC
LEVEL DIFFERENCE	0.000	0.222	0.064	-0.327	-0.370	0.000
DESIGN LEVELS	123.910	124.205	124.220	124.325	124.340	124.785
EXISTING LEVELS	123.910	123.983	124.156	124.651	124.710	124.785
DESIGN OFFSET	-0.648	-0.500	0.000	3.500	4.000	4.534

CHAINAGE 70

PRELIMINARY CONCEPT

EXISTING LIDAR SURVEY 2018
EXISTING LIDAR SURVEY 2023



CUT STRINGS	IA00C	EH00	ES00	LE0I	EH0I	IA0IF
LEVEL DIFFERENCE	0.000	-0.053	-0.038	0.149	0.146	0.000
DESIGN LEVELS	126.000	125.947	125.962	126.067	126.082	125.946
EXISTING LEVELS	126.000	126.000	126.000	125.917	125.936	125.946
DESIGN OFFSET	-0.527	-0.500	0.000	3.500	4.000	4.272

CHAINAGE 80

PRELIMINARY CONCEPT

EXISTING LIDAR SURVEY 2018
EXISTING LIDAR SURVEY 2023

Appendix B

Geotechnical Analysis

B.1 Preliminary Geotechnical Profile

The geotechnical model for the Site can be summarised as follows:

- The geotechnical profile at the slip site is expected comprise colluvium / talus overlying metasiltstone bedrock at depth.
  - The colluvium is expected to be a mixture of sand, silt, gravel, and larger size rock pieces derived from the metasiltstone siltstone bedrock.
  - The metasiltstone bedrock in the vicinity of the slip is expected to be around 25m deep at the 'active overslip' based on extrapolating the slope of the exposed bedrock between RL 200 to 250m through the slip plane (refer to Figure 4).

    This is supported by the geophysical investigation undertaken by GHD (GHD, 2022) which concluded a range of 17m to 34m to bedrock at the slip site.

For the slope stability analysis, the slope has been assumed to comprise deep colluvium / talus within the expected failure surface planes.

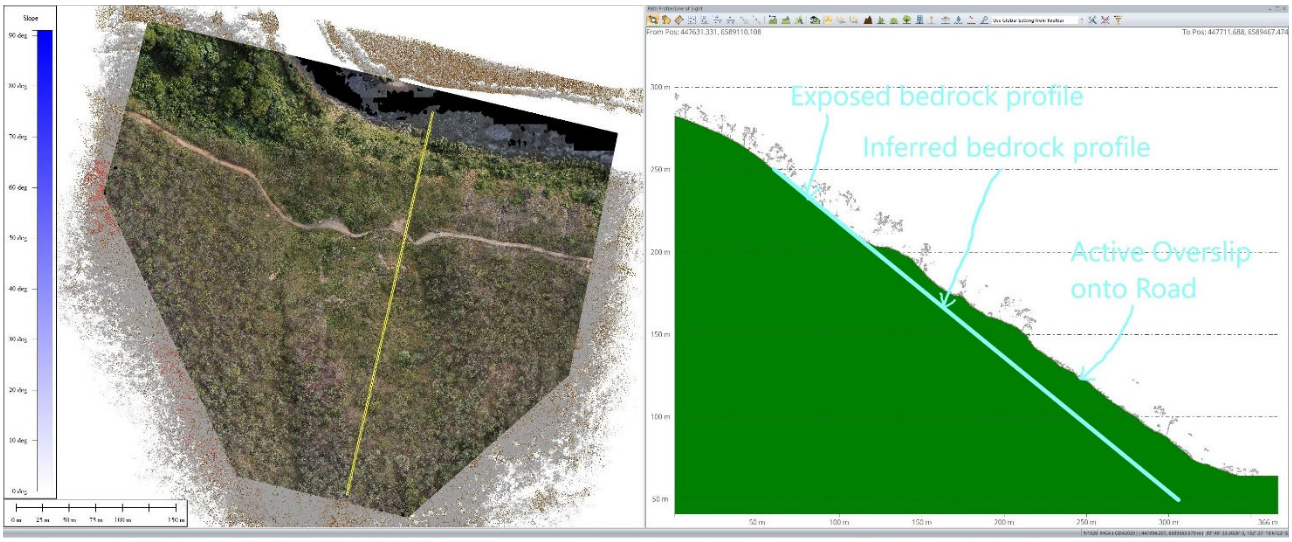


Figure 4: Section prepared by Diospatial with Inferred Bedrock Profile Annotated

From site observations and comparison of LiDAR the following has been inferred about the slips currently impacting the road:

- The 'active slip onto the road' (shown in red in Figure 2 and Figure 3) is relatively shallow translational failure, approximately 1 to 2m deep, extending about 50m along the slope.
- There is a shallow (estimated about 0.5m thick) layer of vegetation / root mat and topsoil. Underneath this is probably a finer mat of colluvium / talus material and transition zone of up to about 1m thick. These two layers are what enabled the moisture to be held leading to the active overslip failure onto the road.
- The 'active underslip through the road' (shown in orange in Figure 2 and Figure 3) appears to be through fill comprising reworked colluvium assumed to be from cut to fill for construction of the road. It is up to about 2m deep and likely contributed by erosion from flows down the gully.

A section (annotated version of Chainage 50 from the draft of the earthworks Preliminary Concept drawings) showing the inferred geotechnical profile is presented in Figure 5 below.

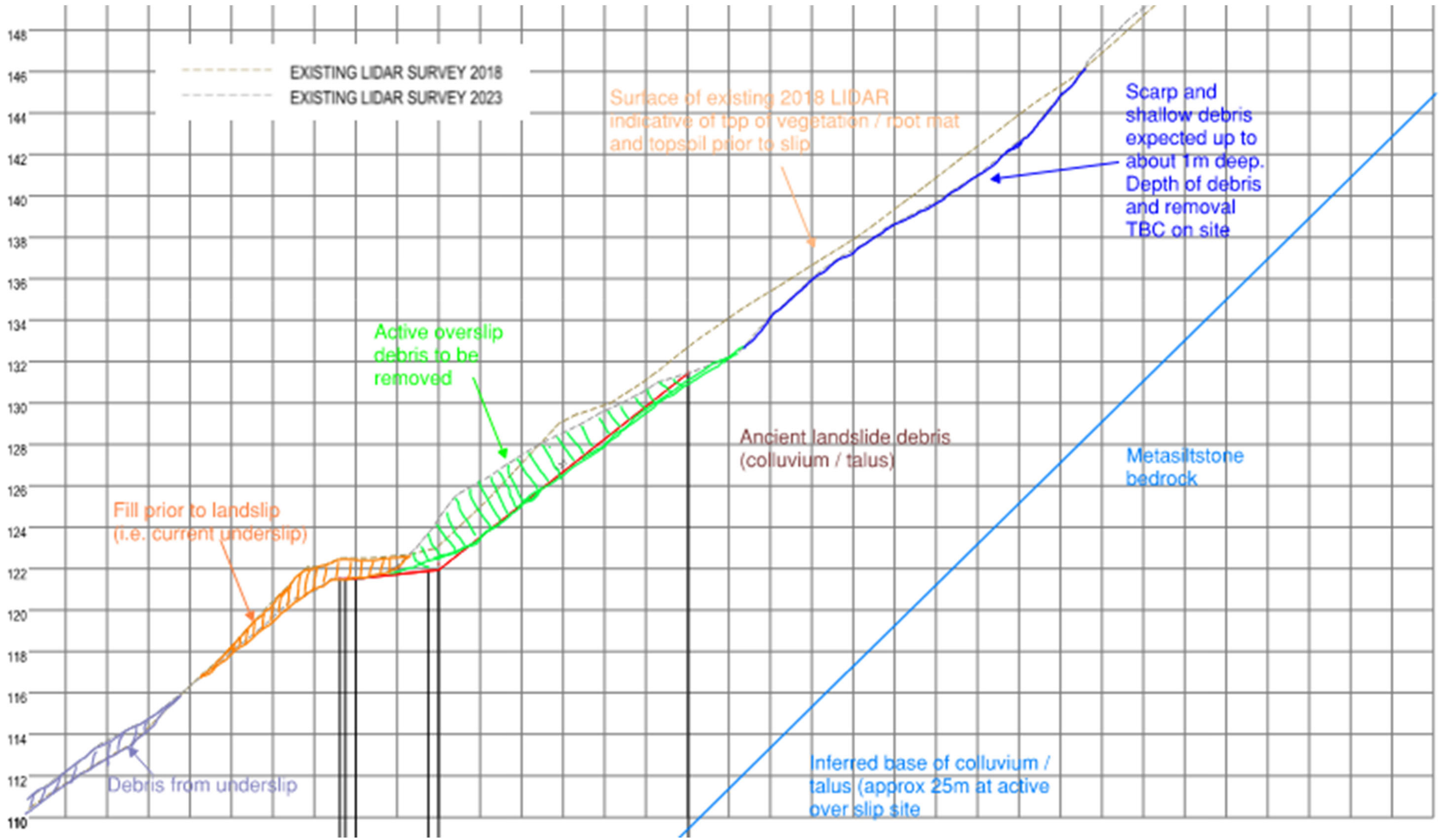


Figure 5: Inferred Geotechnical Profile

B.2 Selection of Geotechnical Parameters

Access for geotechnical investigations at the active slip site is limited due to potential stability issues. Additionally, characterisation of shear strength parameters for colluvium and talus, where with the inclusion of larger than gravel size rock by conventional investigation methods is difficult due to relative size of testing apparatus to rock fragments.

As such, geotechnical parameters have been estimated by a back analysis with consideration of the survey, in conjunction with site observations and literature review. A back analysis of the active overslip onto the road (shown in red in Figure 2 and Figure 3) was carried out to assess geotechnical parameters for this slip. Key considerations for the back analysis are below:

- The slip was first observed on 31 March 2022 after a period of rainfall. Review of the daily rainfall data for the Bellbrook (Bureau of Meteorology, May 2023) indicates that daily rainfall the day prior i.e. on 30 March 2022 was 70mm and the highest daily rainfall experienced in 2022 for that site. The slip is inferred to have been triggered by this rainfall event.
- The active upper slip appears to be a shallow translational failure, approximately 1-2m deep.
- There is a shallow vegetation / root mat present which is hypothesised to have become saturated during the rainfall event (see explanation below)
- The slope angle prior to the slip was typically 35 to 40 degrees.
- The road cutting is understood to have been constructed in the 1950's and has not experienced any other landslides, in particular any deep-seated landslides or shallow slides resulting in road closure.
- The colluvium / talus observed at the site is expected to be a mixture of sand, silt, gravel, and larger size rock pieces derived from the metasiltstone siltstone bedrock. It has a significant proportion of rock. It is expected to behave in a predominantly non cohesive manner, it is expected to be relatively highly permeable. Due to the large variability and rock sizes in the material it is unlikely that realistic shear strength parameters can be obtained from conventional testing, and instead suggested to undertake a back analysis and literature review.

It has been inferred that during the rainfall event the vegetation, root mat and interface with the underlying colluvium / talus to the depth of the approximate slide plane has become saturated. Due to the observed slip angle, expected depth and relatively high expected permeability of the underlying colluvium /talus, it is unlikely that the saturation has extended far below the vegetation and root mat . Therefore, the zone of saturation is expected to be limited to the upper 1.5m. An infinite slope analysis assuming a factor of safety of 1 during steady state seepage parallel to the slope with a slope angle of 35°, saturated unit weight (γ_{sat}), and no cohesion was used to back calculate the friction angle for the talus.

For a factor of safety of 1, the back calculated phi value is approximately 52 degrees.

Due to the observed rocky nature of the colluvium / talus it is expected that it would exhibit similar shear strength properties to rockfill. It is generally agreed in industry/through literature that the friction angle of rockfills reduces with confining pressure.

The Review of Shearing Strength of Rockfill (Leps, 1970) assembled a significant number of large scale triaxial shear test data for rockfills of various type, and provided a figure showing estimated φ° reducing with normal pressure.

For the stability analysis we have assumed a $\varphi = 52°$ at an effective normal stress of approximately 10kPa based on the back analysis, and φ that reduces with normal effective stress as shown annotated on Figure 6 (reproduced in kPa in (Barton, 1981)).

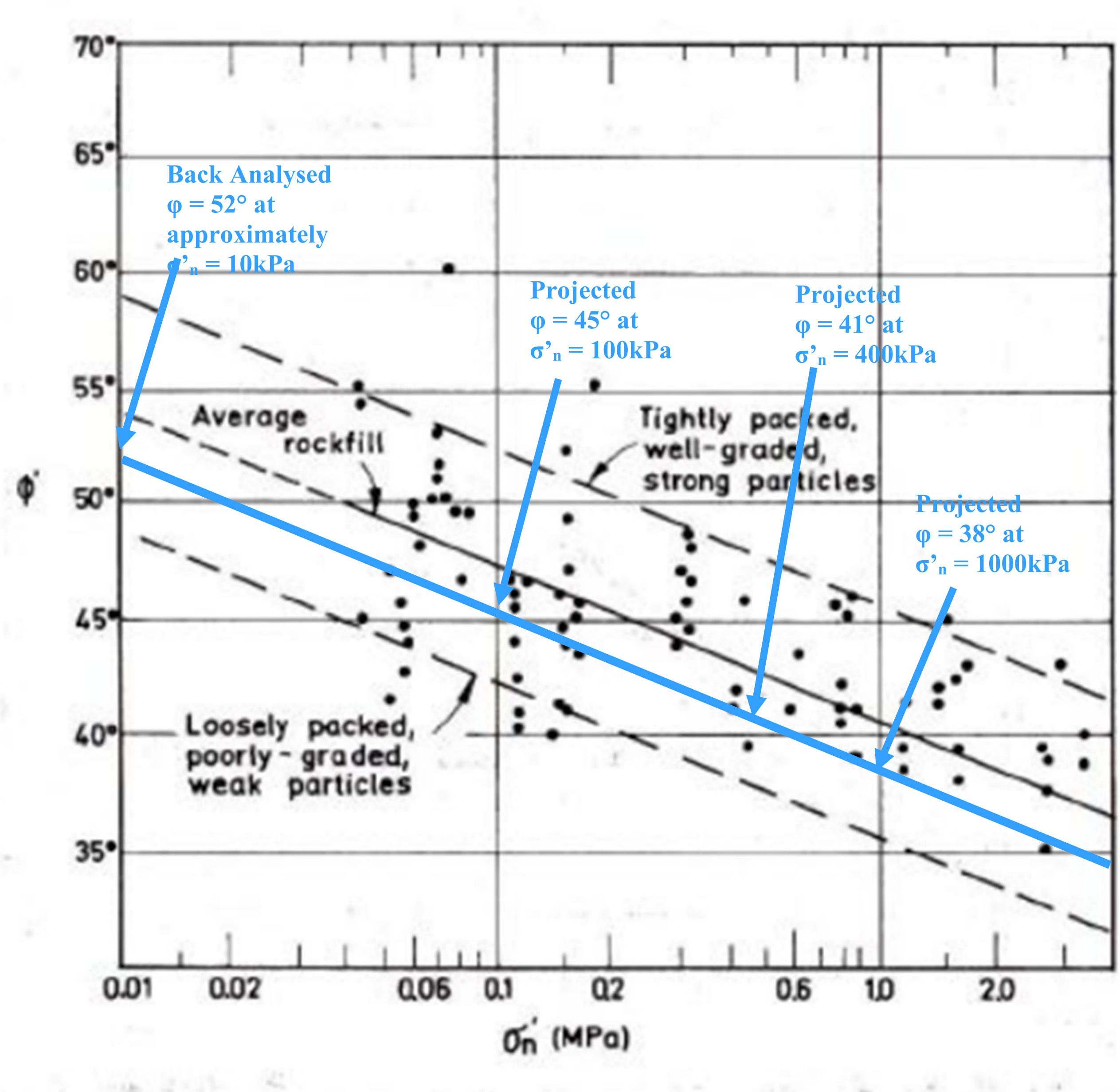


Figure 6: Annotated Triaxial Tests Results for Rockfill from LEPs 1970 (reproduced from (Barton, 1981)) blue line represents Site specific back analysis for Toose Road

Comparison to other strength models for similar materials has been carried out. One example is against the model presented in Strength, Stress-Strain and Bulk Modulus Parameters for Finite Element Analysis of Stresses and Movements in Soil Masses (Duncan, 1980). Duncan 1980 reported stress stain and strength parameters for seven results for 'GM' (i.e. gravel-silt and gravel-sand-silt) soil samples under drained conditions.

The paper provides shear strength parameters as φ at $\sigma'_3 = 1$ atmosphere i.e. at $\sigma'_3 = 100$ kPa with a reduction of φ per 10 fold increase in σ'_3. A comparison is provided in Table 1 below.

Table 1: Comparison of Selected φ values against φ values for 'GM' samples results in Duncan 1980

	φ (°) at σ'$_3$ = 100kPa	φ (°) at σ'$_3$ = 1000kPa
Min	44	35
Max	53	46
Average	50	42
Adopted[1]	44.5	37.5
Notes: approximated by converting φ at σ'$_n$ values from Figure 6 to φ at σ'$_3$ values.		

Additionally the adopted model was compared against the strength model presented in Charles and Watts (Watts, 1980) which was based on large scale triaxial testing of heavily compacted rockfills. Charles and Watts noted that 'the weak, soft slate (B2) might have been rejected as unsuitable for rockfill in many civil engineering works'. The approximate shear stress values converted from selected φ values on Figure 7 have been overlain for comparison.

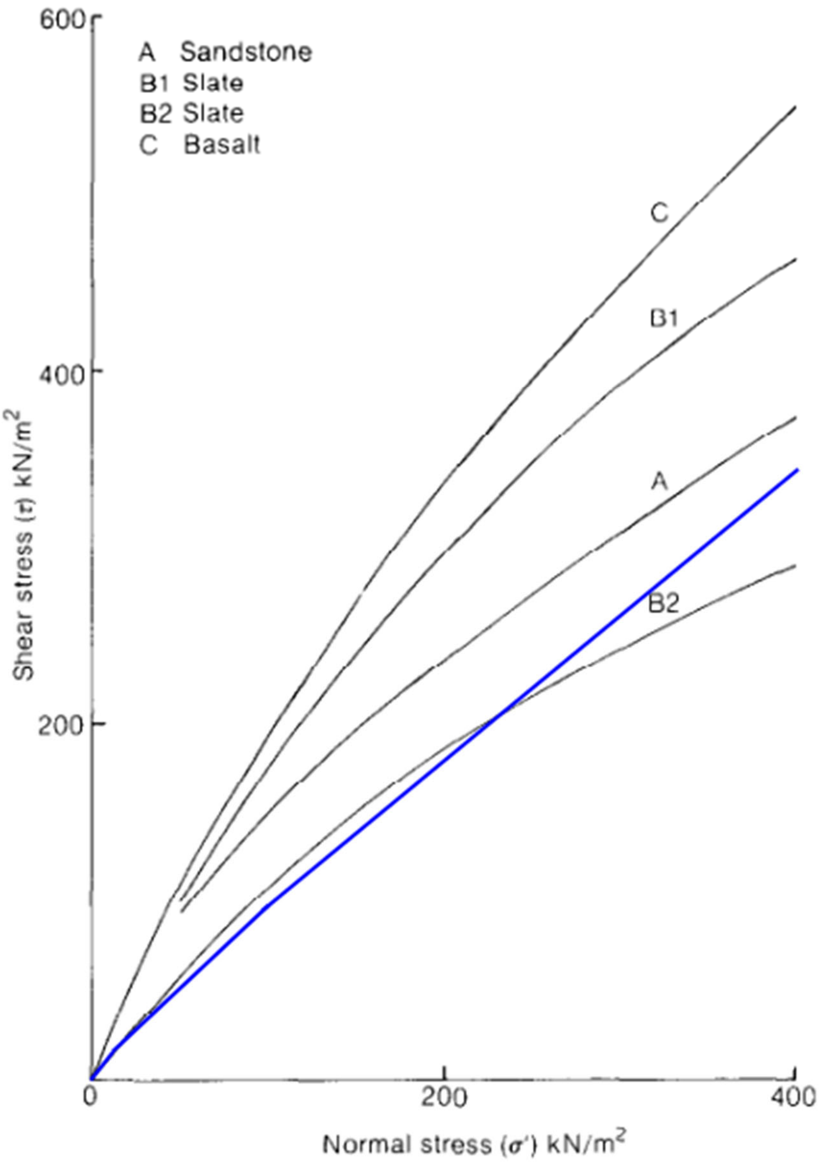


Figure 7: Annotated excerpt from Charles and Watts (1980) [7] showing adopted strength overlain for comparison.

B.3 Geotechnical Design Standard

KSC has requested that an assessment of the stability of the slope following construction be carried out in accordance with GTD2018/001.

The required minimum factors of safety (FoS) for remediation design (Table 6.2 of GTD2018/001) assuming that the consequence class is C4 or C5, is 1.25 for long term and 1.2 for short term analysis.

B.4 Infinite slope analysis

Prior to the slip, the slope angle across the active overslip site was about 35° to 40°, with slopes of up to about 45° nearby to the slip The slope angle is expected to be around 40°, potentially up to about 45° in areas following temporary remediation works.

Infinite slope analysis both with and without steady state seepage (i.e. wet and dry) are presented in Figures 8 and 9 respectively. The analysis has been carried out for slope angles of 35°, 40° and 45° with a range of φ angles from 45° to 52° being the expected range of φ within about 5m depth. The results are discussed below:

The 'wet' slope arrangements analysed all have a low FoS, typically below 1. Indicating that further movements are to be expected, especially during periods of high rainfall. Given the performance of the slopes over varying slope angles and climactic conditions it is considered unlikely that operative friction angles below 50° are possible for near surface material within the expected depth of the saturated slip plane. This is supported by reference to published literature.

- The 'dry' slope arrangements analysed showed:
  - The FoS for phi = 52° had a factor of safety above 1.25 for all slopes angles analysed.
  - The factor of safety reduces with increased slope angle or reduced phi value. The sensitivity analysis indicates the FoS is less than 1.25 for a slope angle of 40° and phi of less than 47°, and a slope angle of 45° and phi of less than 52°.

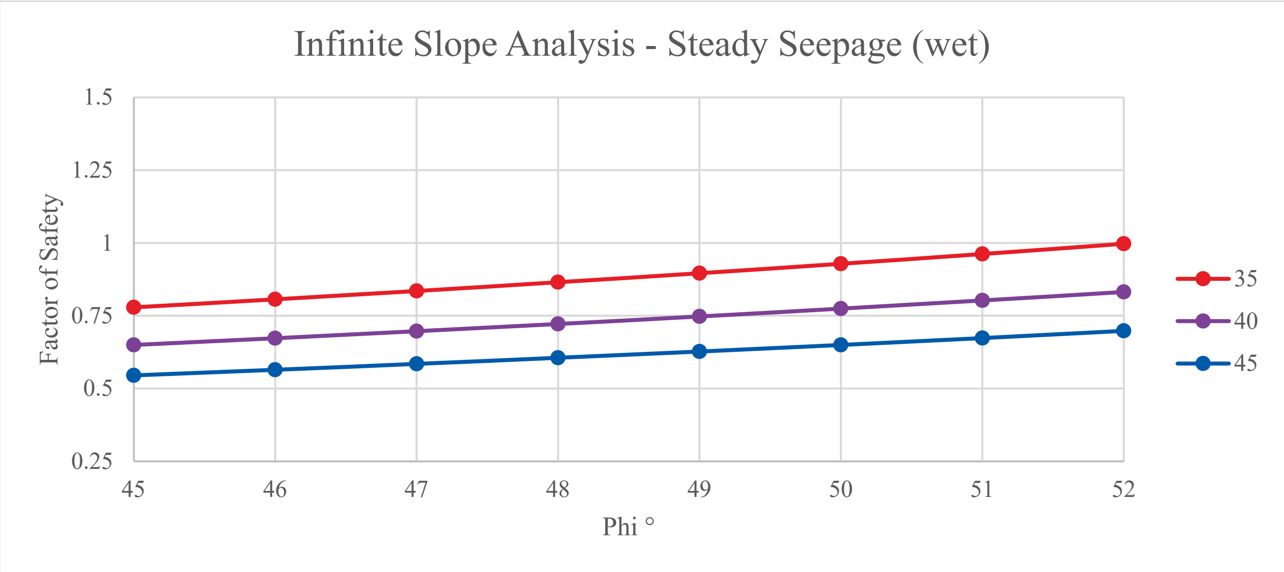


Figure 8: Results of Infinite Slope Analysis – Steady State Seepage (wet)

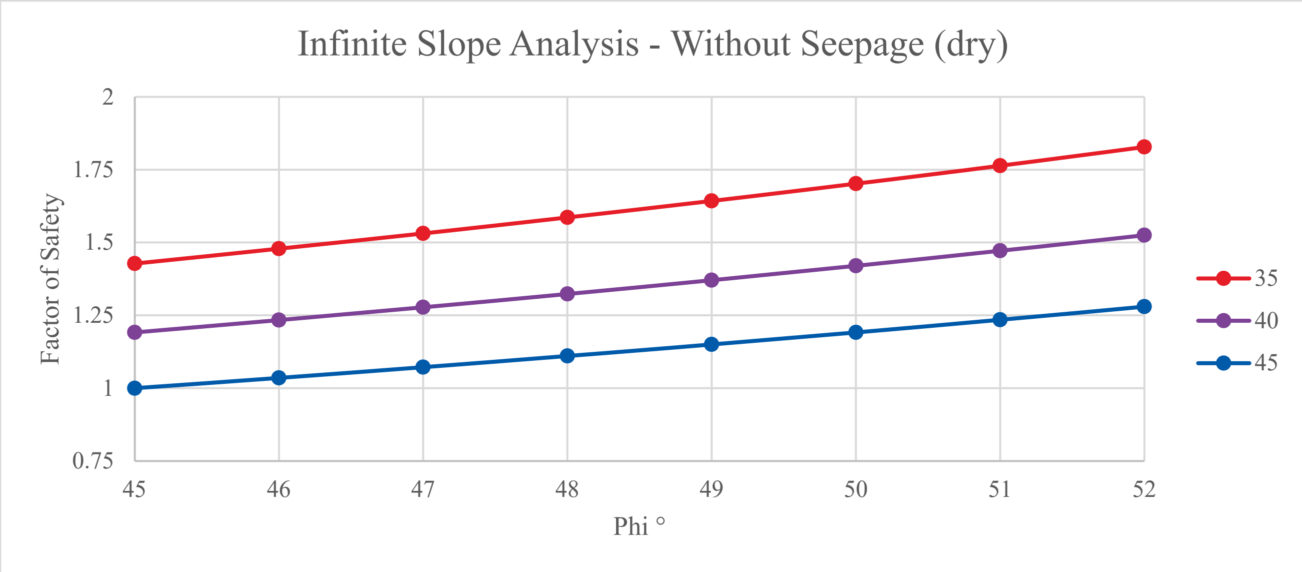


Figure 9: Results of Infinite Slope Analysis – Without Seepage (dry)

B.5 Stability Analysis of Completed Works

Slope stability analysis of CH40 (with reference to the draft earthworks Preliminary Concept design provided in Appendix A) has been carried out using limit equilibrium slope analysis software GeoStudio Slope/W version 23.1.0.520. Morgenstern-Price analysis has been adopted. The results are provided in Key inputs and assumptions are outlined below:

- The talus / colluvium was modelled as Mohr-Coulomb material, with saturated unit weight = 22 kN/m^3, unsaturated unit weight = 20kN/m^3, and phi values varying according to the approximate effective stress with values 52° at the surface, reducing to 45° at approximately 5m deep / effective stress of 100 kPa and 38° at approximately 50m deep / effective stress of 1000 kPa.
- 'Grid and radius' slip surfaces were used.
- The modelling was carried out for a 'dry' slope, where no groundwater water was included in the analysis. It was also carried out for a 'wet' slope, with a groundwater layer extending from the surface to 1.5m depth to represent the expected partial saturation as discussed in section 5.1. This groundwater condition has been implemented using the piezometric surface function, applied to a 1.5m thick soil layer from the surface down only.
- For the 'wet' slope the analysis indicated a FoS < 1 for shallow failures, which indicates that shallow failures are to be expected under these conditions. To analyse the minimum factor of safety for deeper failures where a significant proportion of the slip is below the seepage surface a minimum slip surface depth of 3m has been applied. For all other analyses the minimum slip surface depth has been set to 1m. The above not withstanding, the ancient slide is still moving and likely to develop with ongoing creep, erosion and loss of toe support through further flooding.

A summary of the results is provided in Table 2 below. The spatial model and results for each analysis are presented in Figure 10 to Figure 13 below.

Table 2: Results of Limit Equilibrium Stability Analysis

Analysis Scenario	FoS
Wet shallow	0.37
Wet deep (i.e. minimum 3m slip surface depth)	1.24
Dry	1.32

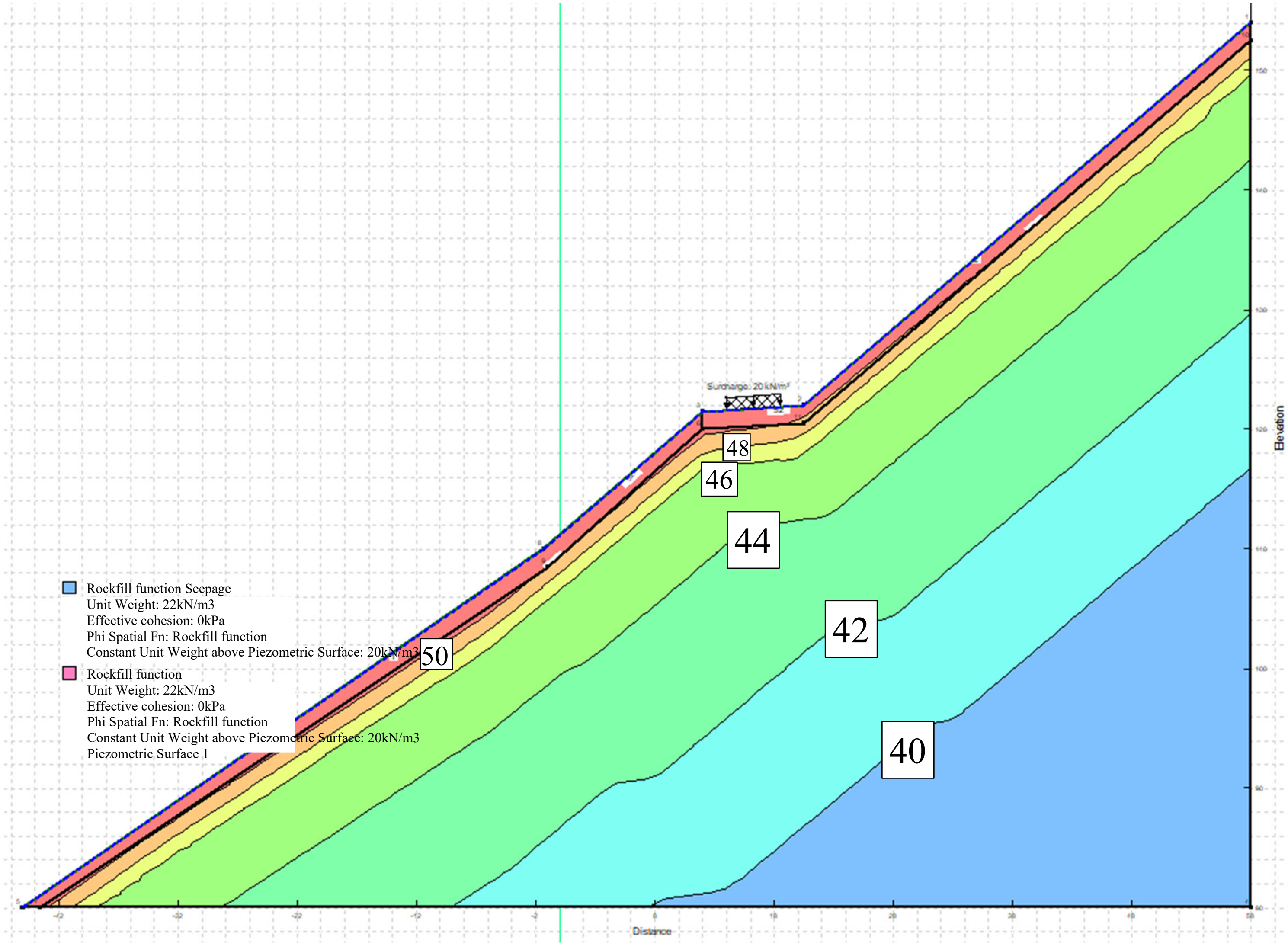


Figure 10: Adopted Spatial φ Function

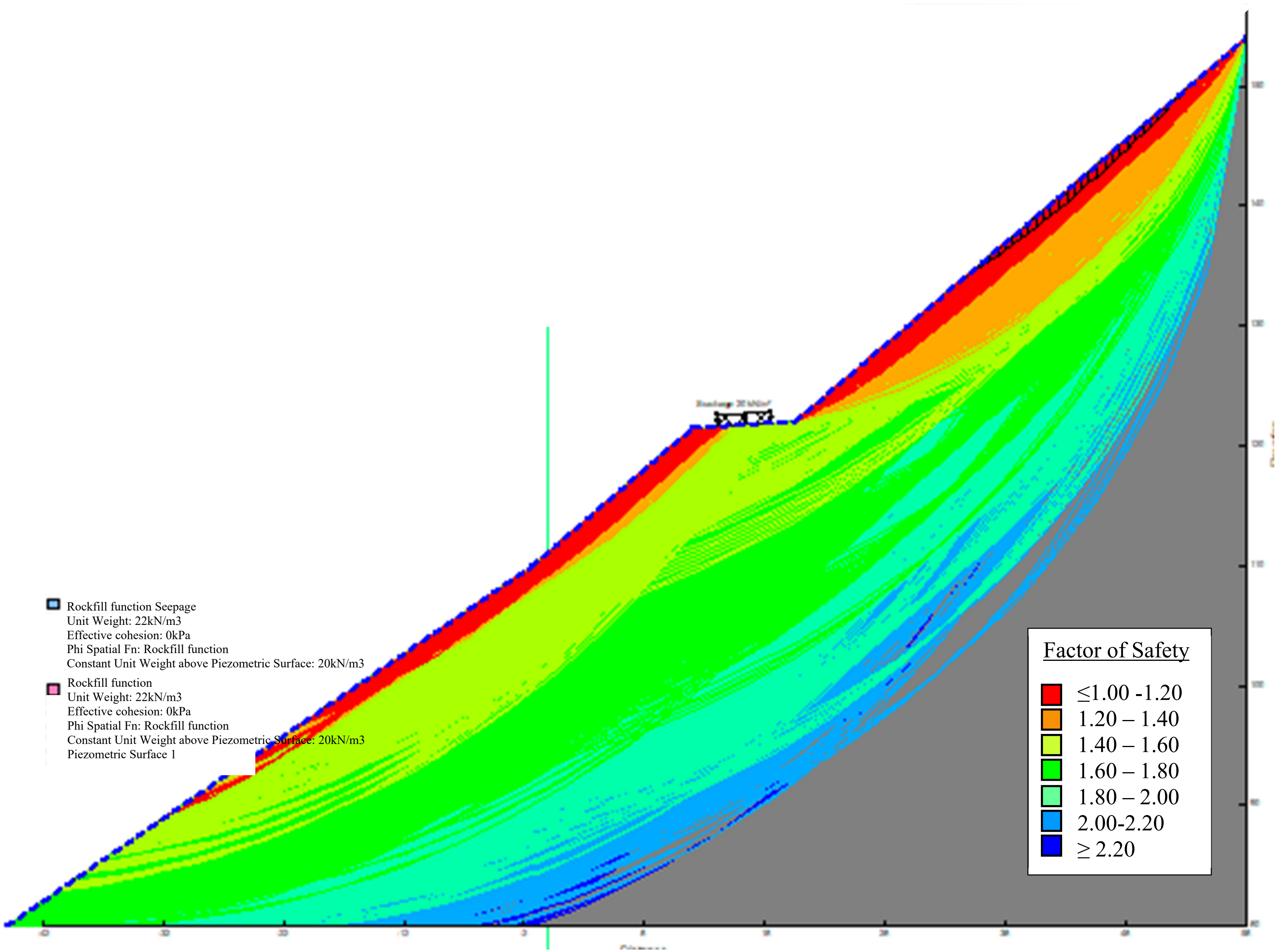


Figure 11: Slope/W Result for 'Wet Shallow' Slope Analysis

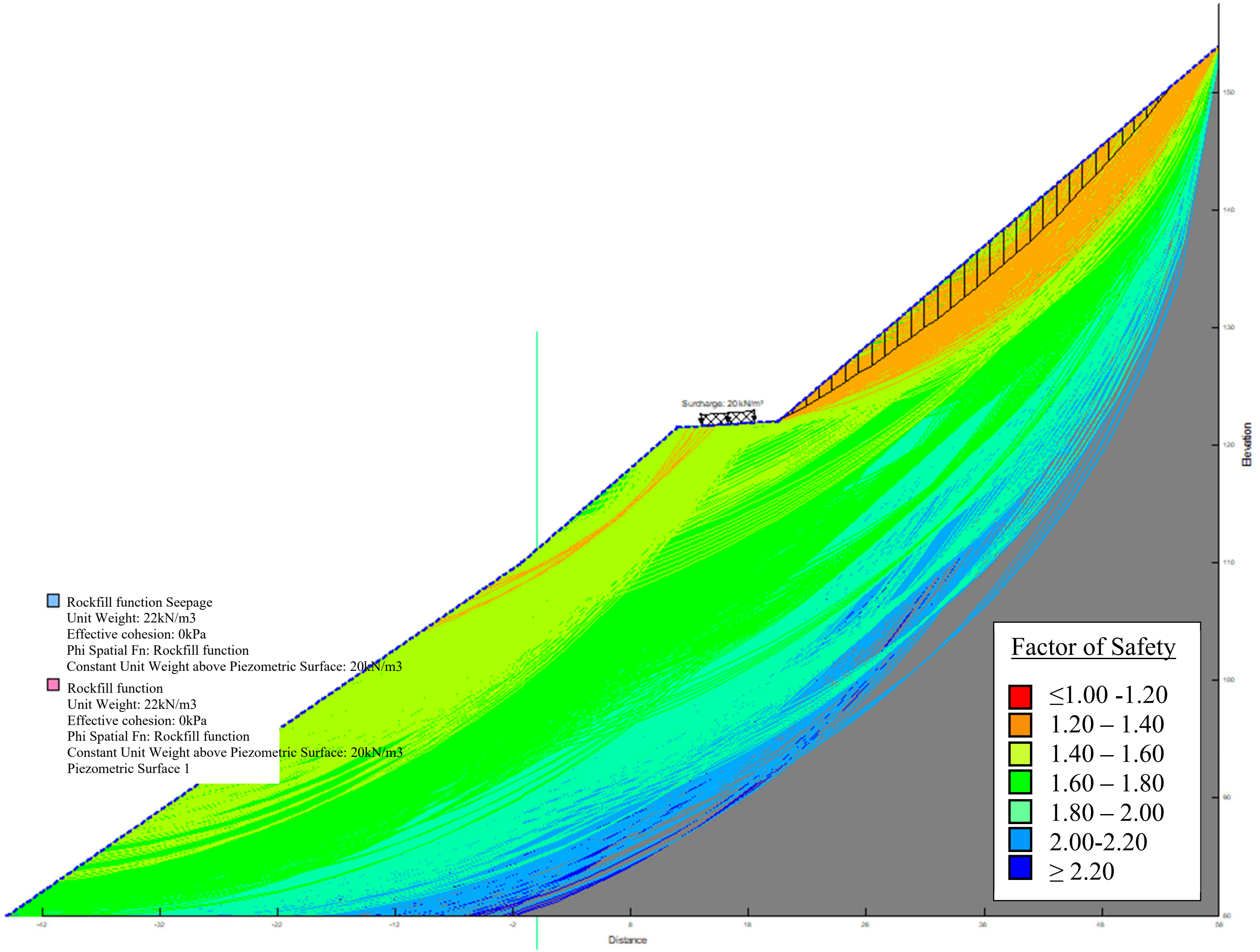


Figure 12: Slope/W Result for 'Wet Deep' Slope Analysis

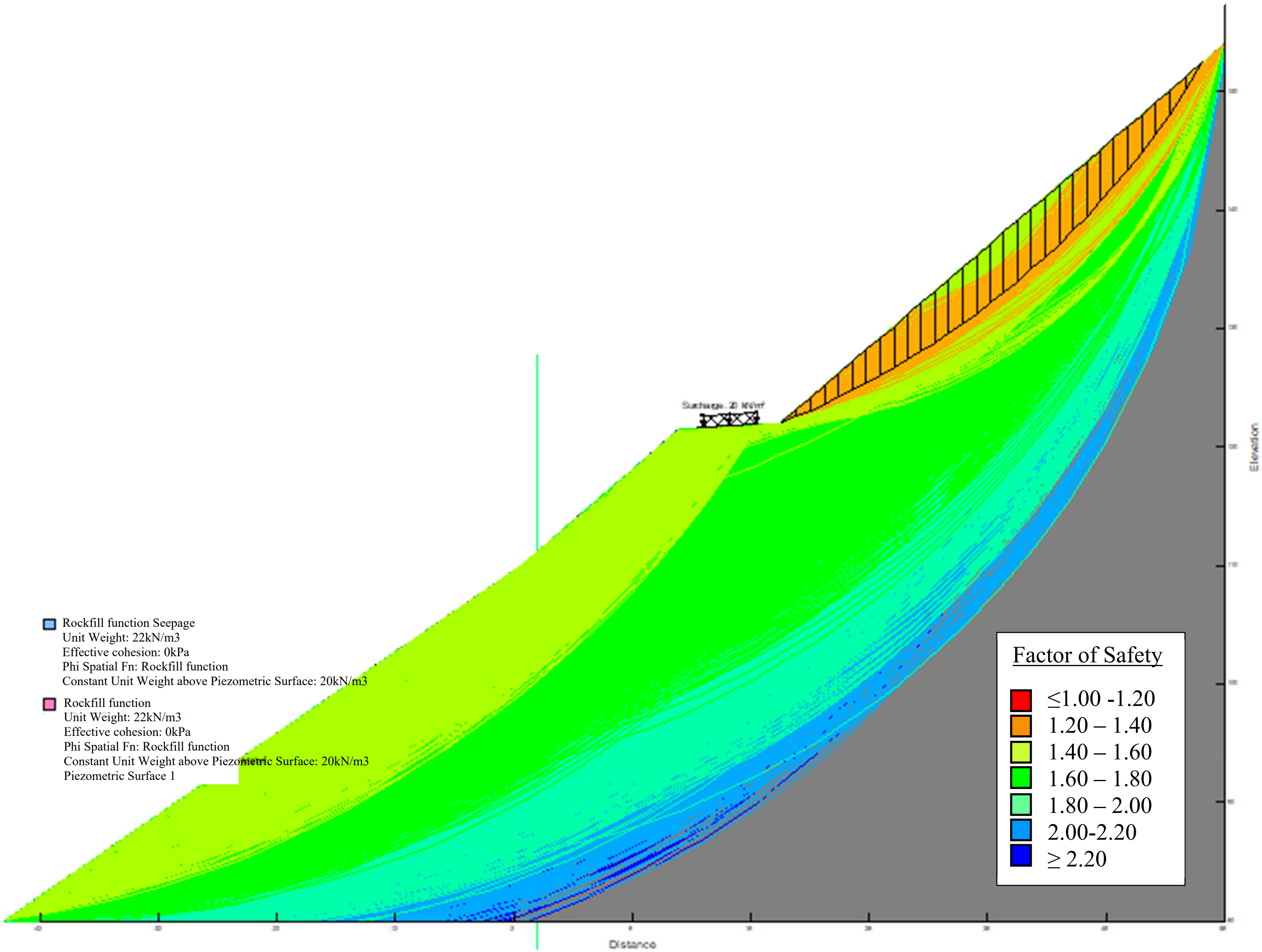


Figure 13: Results of Slope/W Analysis for 'Dry' Slope Analysis

The results indicate the following:

- The ‘wet shallow’ slope is marginally stable. Further shallow movements are to be expected both up and down slope, especially during periods of significant rainfall. This will reflect any surface saturation moving any loose rocks down the surface. Similar movements can be observed currently on site.
- The ‘wet deep’ slope has an adequate a FoS, i.e. above 1.2.
- The ‘dry’ slope has an adequate FoS, i.e. above 1.25.

B.6 Stability During Construction

Accessing the slip from the west appears safer and less costly than cutting in an elevated track from the east as was originally proposed. It is expected that a long reach excavator, could initially be positioned mid-slope in a relatively stable area and progressively remove the landslide debris in a top down manner working from west to east.

It is expected that a construction methodology with an adequate factor of safety during construction could be developed. This has been notionally modelled by the 20kPa surcharge in the assessment undertaken. This assessment should be compared with the proposed methodology once available for applicability.

During construction care would be required to maintain the stability of plant and safety of workers on the slope.

Full time geotechnical supervision would be required to direct the work and assess temporary stability during construction. Appropriate controls would need to be implemented eg additional precautions and controls will be needed during wet spells with potential restrictions on work, regularly inspecting the slope for signs of instability prior to crossing and the like, restricting access to essential users only and educating users about the risks. Other practical measures such as minimising new slope angles and keeping similar to those already exhibited, checking the stability of any above trees and boulders and removing those that pose a threat should also be implemented to aid in maintaining safety and stability.