

Lot 15, 220 Withy Road, Awakaponga

Geotechnical report

Prepared for Mike Davies

Project 50721 - Rev.0 - 28/03/2023

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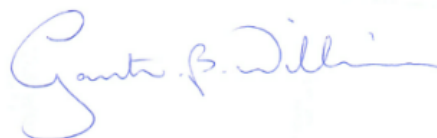
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The recommendations and opinions contained in this report are based on our visual reconnaissance of the site, information from geological maps and upon data from the field investigation as well as the results of in situ testing of soil. Inferences are made about the nature and continuity of subsoils away from and beyond the exploratory holes which cannot be guaranteed. The descriptions detailed on the exploratory hole logs are based on the field descriptions of the soils encountered.

This report includes Appendices. These appendices should be read in conjunction with the main part of the report and this report should not be considered complete without them.

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1.0 INTRODUCTION

1.1 Project Brief

Engineering Design Consultants Ltd (EDC) was commissioned by Mike Davies on June 2022 to provide a geotechnical assessment and report for the proposed new dwelling at Lot 15, 220 Withy Road, Awakaponga (Lot 15 DP 380141), referred to below as 'the site'.

1.2 Proposed Development/Works

It is proposed to construct a new dwelling on-site, the surveyor plan provided by Overington Surveyor forms Appendix A

1.3 Existing Engineers Reports

Manktelow Consulting Engineers (MCE) Limited has previously prepared a 'Geotechnical Investigation Report for Proposed Subdivision, 168-206 Withy Road, Whakatane' (Ref111.1 Dated 30 November 2004).

When MCE undertook the investigation, for Lot 15 *'no earthworks have been undertaken to establish this building platform. The house site is located on a north-south trending ridgeline... Evidence of groundwater seepage and past slope movements is evident down the eastern slopes.'*

MCE undertook one window sampler and four hand auger borelogs on the lot. The investigation indicates *'sand and silt combinations underlay the site down to the investigation depths.'* The investigation results is also further discussed in Section 5.

Slope stability analysis was also completed as part of the report. MCE recommended a 15m setback from the top of the eastern and 10m from the top of the western slope. MCE also recommends removing six to seven-metre depth of soil to establish a reasonable building platform. In addition, to reduce the possibility of soil creep and soil movement an in-ground palisade wall (barrier pile wall) was recommended between the house and the steeper slope.

1.4 Objective/Scope

The objective of this report is to advise on geotechnical constraints, provide geotechnical parameters for design of the proposed building and provide advice on appropriate foundation types.

In order to achieve the outlined objectives this geotechnical investigation comprised the following scope:

- A geotechnical desktop study to review geological mapping, geotechnical information resources and review of freely available historical aerial photographs.
- A site walkover and shallow intrusive investigation comprising hand auger boreholes with Scala Penetrometer and shear vane strength testing.

- Provision of an interpretive report summarising the above and providing geotechnical parameters for foundation design of the proposed development.
- A quantitative slope stability analysis based on hand auger findings

This report assesses the condition of the land only.

2.0 SITE DESCRIPTION

2.1 Site Location

The site is irregular in shape, situated approximately 100m from Withy Road and covering a total area of approximately 16,000m².

The Google Earth coordinates of the approximate centre of the site are: 37°56'46.4"S 176°44'11.5"E.

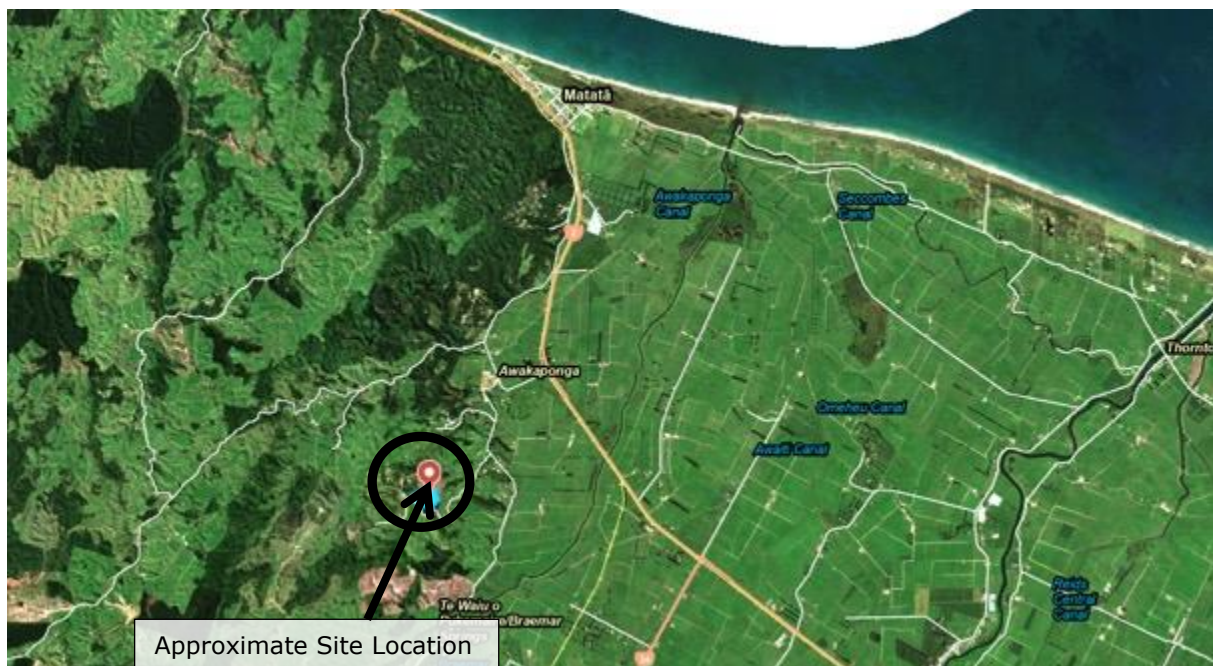


Figure 1: Site Location (Courtesy of BOPRC Maps)



Figure 2: Site Aerial Photo (Courtesy of BOPRC Maps)

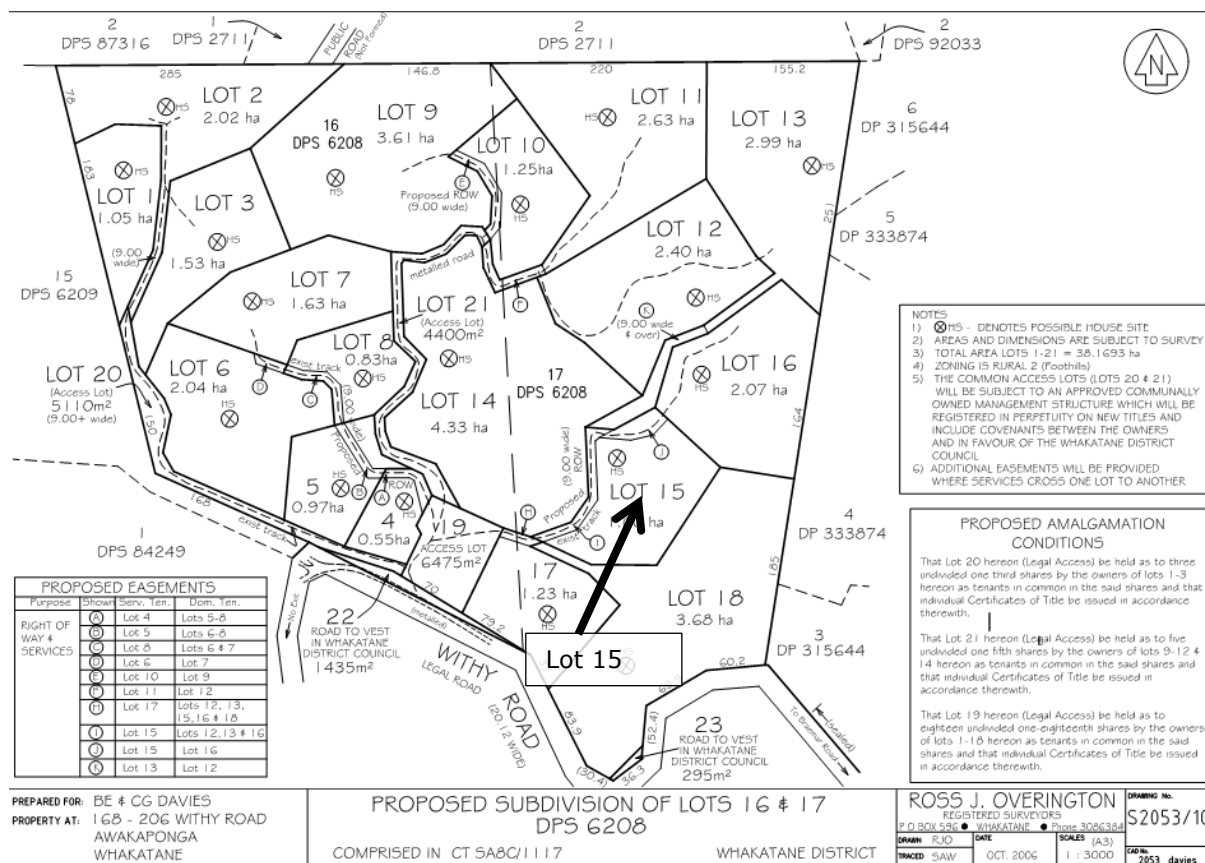


Figure 3: Subdivision Plan

2.2 Site Walkover

The proposed building platform is situated approximately 200m north of Withy Road and can be accessed through a steep driveway to the south or from the right of way from the west. The land on the proposed building platform is generally flat, with the subsoil exposed on the surface with a very thin layer of topsoil.

The building platform is also surrounded by slopes on all sides, but the slopes can be differentiated into three main sections, referred to below as the eastern, northern and western slopes. These all dip away from the building platform.

The northern slope is approximately 50m high, with a slope of approximately 35°. At the time of EDC's visit, we noted an access track leading to Lot 14 to the north that was cut off in the middle. This was likely part of the earthworks for the subdivision.

Based on the report produced by MCE in 2004, we understand that the eastern slope had failed previously. During EDC's visit, the eastern slope was overgrown with vegetation and thus was not possible to assess the slope conditions. Based on the BOPRC contours, the slope is approximately 36m high at 30°.

The western slope is approximately 15-20m high, with the angle varying between 25-30°. The slope leads down to Lot 20 to the west. Approximately 5m west of the building platform is a cut at approximately 60-70° for the driveway leading up to the site. On the cut surface, shallow slips were noted and the pumiceous subsoil was exposed.

Figures 4 - 8 show the site, taken during EDC's site walkover:



Figure 4: View on the proposed building platform looking southwest



Figure 5: View of the shallow slip on the western slope



Figure 6: View on the proposed building platform and eastern slope



Figure 7: View on the northern slope



Figure 8: View of the eastern slope

3.0 PUBLISHED GEOLOGY & HYDROGEOLOGY

EDC has reviewed the following resources as part of the geological and hydrogeological desk-based study:

- New Zealand Geotechnical Database (NZGD)
- GNS 1:250,000 online QMap
- EDC's Geological and Geotechnical Database
- GNS NZ Active Fault Map

3.1 Geological Mapping



Figure 9: GNS Geology QMap

According to the GNS Geological Unit QMap, the site is underlain by Pleistocene (~303,000-year-old) Matahina Formation comprising 'Predominantly pink to purplish-grey columnar jointed, welded, locally eutaxitic rhyolite ignimbrite; minor fall deposits' (Q9z).

3.2 Faults

Information contained on the Geological Mapping & GNS Active Fault database in relations to the local faults is included below:

Name	Active / Inactive	Location	Fault Sense	Recurrence Interval (years)	Recurrence Class
Matata Fault (Zone)	Active	Immediate Vicinity*	Normal	$\leq 2,000$	I
Otakiri Fault (#1752)	Active	3km E	Normal	$\leq 2,000$	I
Omeheu Fault (#2112)	Active	4.4km E	Normal	$\leq 2,000$	I

Table 1: Fault Details

NOTE: According to the GNS Active Fault Map, the site is within the Matata Fault Zone, however, there does not appear to be any active fault trace within 100m of the site. The closest Matata Active Fault trace is approximately 800m NW of the site.

The topography of the local area suggests the presence of a number of other faults (not shown on the maps mentioned above). These might not be 'active' (and hence are not on the active faults database), but have affected the topography in the past.

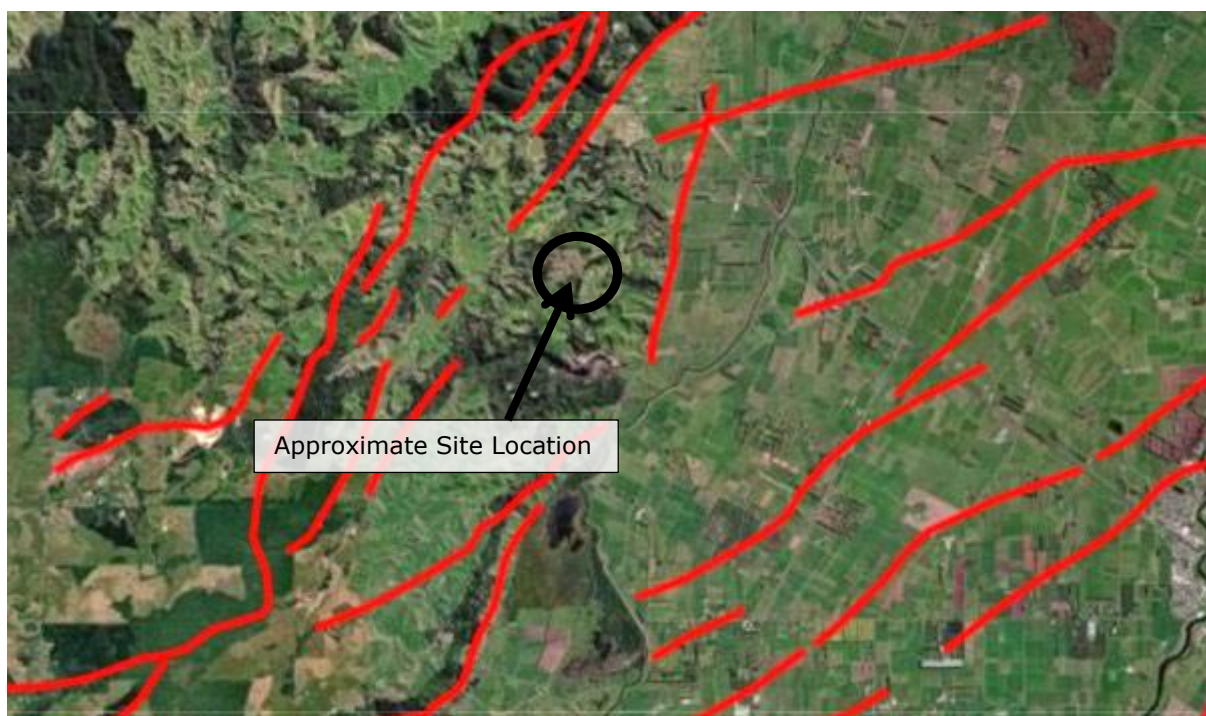


Figure 10: GNS Active Fault Map

4.0 GEO-HAZARDS DESK-BASED INFORMATION

4.1 Site History

4.1.1 Historical Aerial Photographs

Historical aerial photographs available on RetroLens have been reviewed by EDC for the years 1943 through to 2011. This review indicated that between 1943 to 1961, the sites surface appears to be grass-covered and was likely used as grazing ground. From 1974, the site and its surrounding started to transition to forestry use.

In the 1974 and 2003 photos, there appear to be potential signs of slope failure southeast of the building platform.

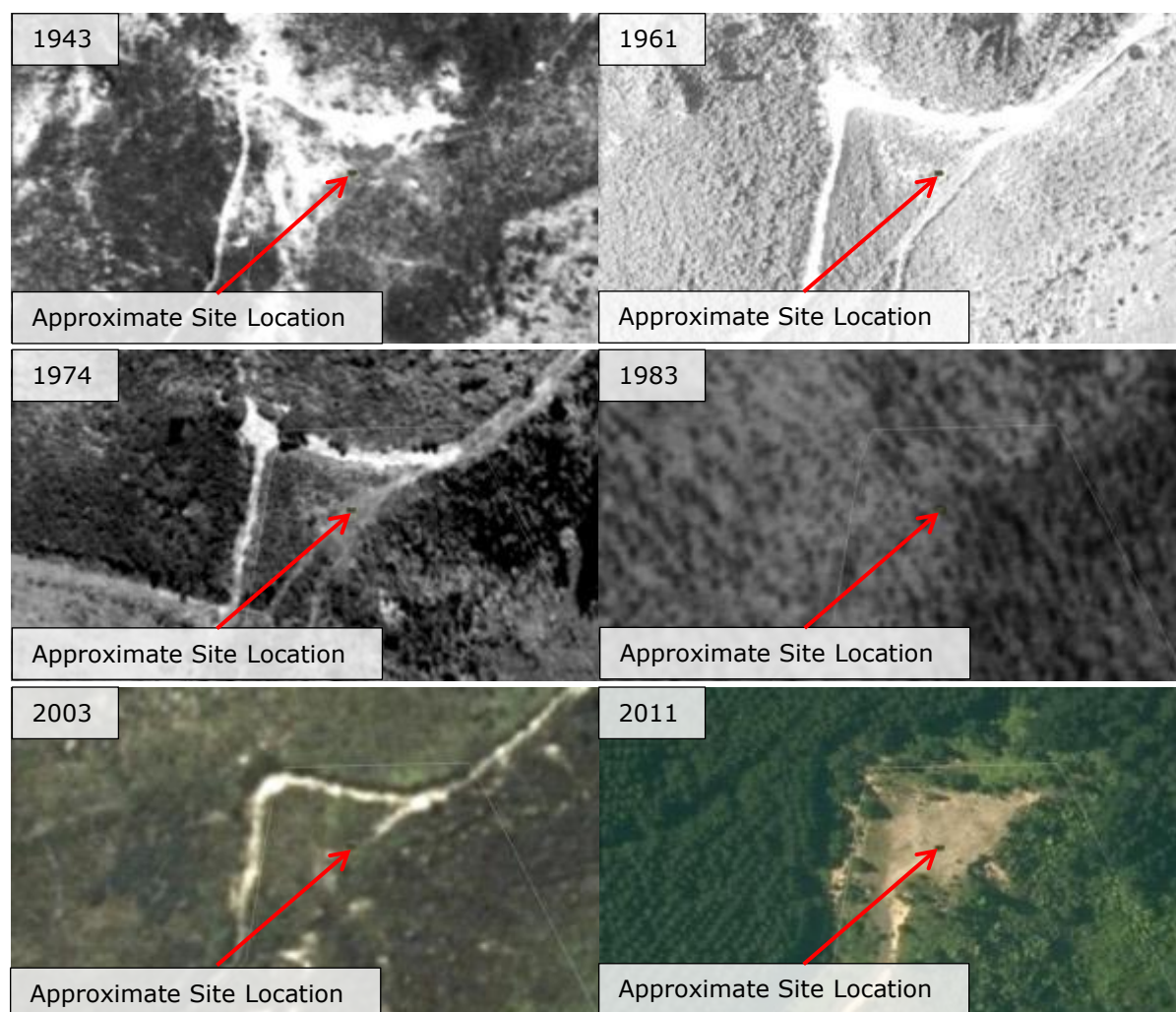


Figure 11: Historical Aerial Photographs

4.1.2 Hazardous Activities and Industries Register

The Bay of Plenty Reginal Council Land Use Register (LUR) holds information regarding sites that have been or currently are used for activities which have the potential to cause contamination.

The LUR does not currently have any information about a Hazardous Activities or Industries List site on or within 100m of the site.

4.2 Liquefaction Susceptibility

The BoPRC Bay Explorer GIS includes the 'Liquefaction Vulnerability Assessment for Bay of Plenty Regional' (Tonkin & Taylor Ltd., 2021). The map indicates the site is in a 'Undetermined' liquefaction vulnerability zone.

4.3 Flooding

As the proposed building platform is well-elevated compared to the surrounding land, it is not likely to be at risk of flooding. This is not a guarantee that the site will not flood, and it may still be at risk from local surface ponding.

Minimum Floor Levels are set by Councils with consideration to the New Zealand Building Code, the Resource Management Act, the applicable District Plan and the latest information available. Please contact Whakatāne District Council for the Minimum Floor Level for this property.

5.0 INTRUSIVE INVESTIGATION

5.1 Rationale

The objective of this report is to describe ground conditions and advise on foundation options for the proposed dwelling. To achieve the outlined objectives, shallow hand testing involving hand auger boreholes (HA's) with Scala Penetrometer and Shear Vane strength testing was considered sufficient to determine ground conditions for the proposed works, in accordance with the MBIE guidance.

We undertook:

- 4 hand augered exploratory holes (HA01, HA02, HA04 and HA05)
- 2 Scala Penetrometer Tests

In addition, we understand that MCE undertook four hand auger and shear vane tests to 4m depth and one window sampler borehole to 10m. The results from the previous investigation have been integrated to the following section (BH01).

5.2 Intrusive Investigation Summary

5.2.1 Shallow Hand Testing

EDC visited the site on 13/07/22 to undertake a shallow geotechnical investigation in order to determine the subsoil conditions.

Soils have been described generally in accordance with NZGS: Field Description of Soil and Rock. The results of these investigations are shown on the HA Log sheet attached in Appendix B.

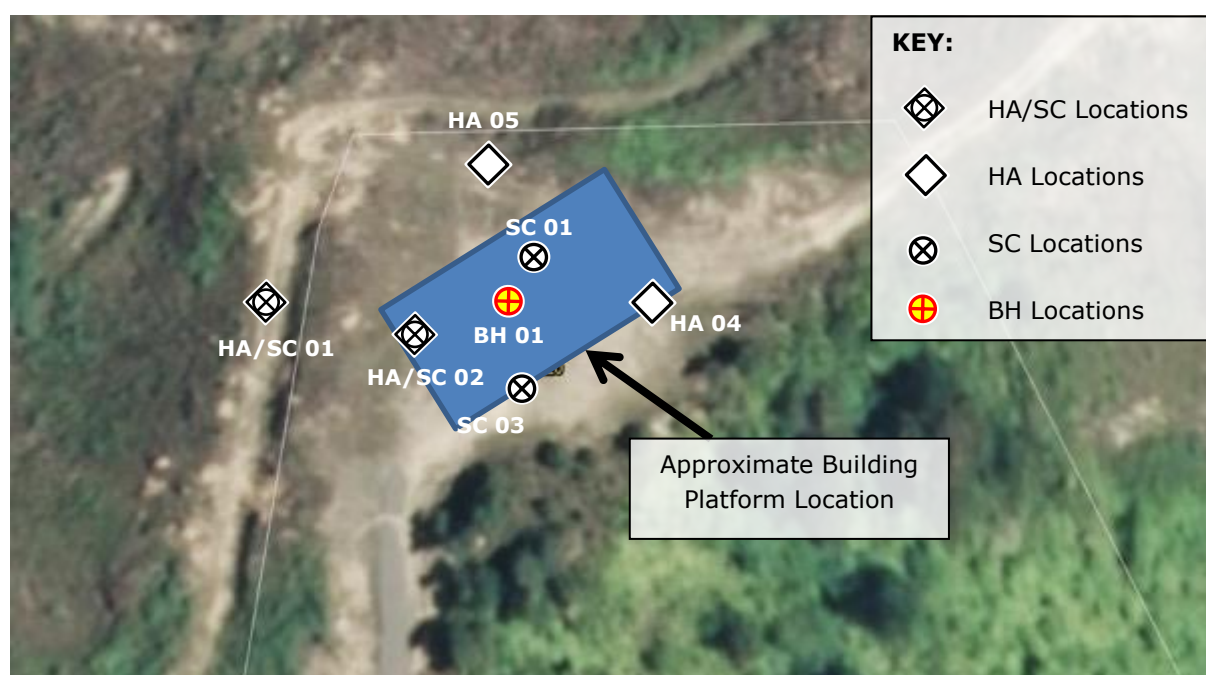


Figure 12: Intrusive Investigation Approximate Locations

The following table shows the depths reached by EDC's hand testing:

Test Ref.	Depth Reached – Reason for Termination
HA01	4.9m – Poor Sample Return
HA02	1.8m – Unable to Penetrate
SC03	1.9m – Target Depth Achieved
HA04	2.2m – Target Depth Achieved
HA05	2.7m – Poor Sample Return
SC06	1.9m – Target Depth Achieved
BH01 (MCE)	10.0m – Target Depth Achieved

Table 2: Shallow Hand Testing Summary

5.3 Ground Conditions Encountered

5.3.1 Topsoil

A very thin layer (~100mm) of organics was encountered in all the exploratory holes comprising loose brown organic silty sand.

5.3.2 Fill Material

No fill materials were encountered in any of the exploratory holes.

5.3.3 Subsoils

The hand auger exploratory holes indicate that the subsoil on the building platform consists of closely interbedded pumiceous silt and sandy silt to 1.2m, underlain by medium dense to dense pumiceous sand and gravelly sand to the termination depth of 2.2m.

The borehole from MCE indicates that the subsoil on the building platform consists of sand to 1.6m, underlain by silt to 4.0m. The silt is underlain by sand and gravelly sand to 9.0m, underlain by silt to the termination depth of 10.0m. While the results from the borehole appear to differ from the hand auger results, we understand that the borehole was conducted prior to earthworks. At the time of EDC's visit, the building platform had been flattened and it is evident that some excavation had taken place previously. In addition, MCE recommended an excavation of between six to seven meters on the building platform.

Based on the overlay of the investigation results, it is likely that the building platform was excavated approximately 3m after MCE completed the investigation.

This material is likely to be fall deposits (tephra) and is consistent with the anticipated soil type based on the geological information described above.

5.3.4 Rock

No rock was encountered in the exploratory holes

5.3.5 Ground Water

While a permanent groundwater table was not encountered in the exploratory holes, saturated soil was encountered in some. It must be noted that the days prior to the investigation was a period of relative heavy rain.

The following table shows the groundwater comment for each holes.

Exploratory Hole	Groundwater Comment
HA01	The soil was wet from 1.2m, and saturated from 1.3m
HA 02	The soil was wet from 0.8m and saturated from 1.0m
HA04	The soil was wet from 0.9m
HA05	The soil was saturated from 2.4m

Table 3: Groundwater Depths Encountered

5.3.6 Summary of Ground Conditions

This investigation indicates the following approximate soil section on the proposed building platform, the columns shaded in grey were the inferred overlay from MCE's borehole results.

General Depth Range (m)	Generalised Ground Description	Density/ Consistency	Geological Unit
Surface to 1.2	Yellowish brown pumiceous sandy silt and silt	Very stiff	Tephra
1.2 to 2.2	Light yellow pumiceous sand	Medium Dense to Dense	Tephra
2.2 to 4.0	Pumiceous sand as above	-	Tephra
4.0 to 7.0	'Cream' silt	-	Tephra

Table 4: Ground Condition Summary

6.0 GEOTECHNICAL ASSESSMENT OF INTRUSIVE DATA

6.1 Static Ultimate Bearing Capacity Assessment

The peak shear strengths, as measured using the Shear Vane, have been used to establish ultimate bearing capacities (UBC) using the multiplication factor of 5.14 outlined by Verification Method B1/VM4 Foundations (Department of Building and Housing, 2008). The Scala Penetrometer results have been assessed using a correlation between Scala blow count and allowable bearing capacity by Stockwell (Stockwell, 1977).

Using this correlation, the investigation results indicate that a geotechnical ultimate bearing capacity (UBC's) of 300kPa can generally be achieved from the surface of the building platform.

A copy of the Scala Penetrometer Ultimate Bearing Capacity Graph forms Appendix C.

6.2 Seismic Site Subsoil Classification

EDC considers that a seismic site subsoil classification 'Class C – Shallow Soil Site', as defined in NZS:1170.5 is appropriate.

6.3 Qualitative Liquefaction Assessment

For liquefaction to occur it is generally considered that there needs to be three preconditions, coupled with ground motions of at least 0.1g:

- Young (Holocene: less than 10,000 years old) sediments;
- The soils include fine-grained and non-cohesive (silts and sands);
- The soils are saturated (below the water table).

The sand and silt encountered in our intrusive investigation is considered to have been deposited during the Pleistocene Epoch. While the groundwater table was not encountered in the exploratory holes, layers of saturated soil was encountered at various depths.

The BoPRC 'Liquefaction Vulnerability Assessment for Bay of Plenty Regional' indicates the site is within an area of 'Undetermined' liquefaction vulnerability.

In accordance with the 'Planning and engineering guidance for potentially liquefaction-prone land' (MBIE/MfE, 2017), we have undertaken a risk-based liquefaction risk assessment. Please refer to Appendix "D" for the assessment matrix and further information.

Based on the desk-based information, shallow site investigation and EDC knowledge of the area we make the following assessment of the site using Table 3.7 of the MBIE/MfE Guidance:

- The site falls within the 'Rural Residential Setting' category;

- Ground damage is expected to be 'none to minor' under SLS conditions (25-year return period earthquake);
- Ground damage is likely to be 'none to minor' under ULS conditions (500-year return period earthquake), however, the hand auger noted that certain layers of soils were saturated during hand auger drilling, and might possibly be susceptible to liquefaction;
- It is acknowledged in the MBIE Guidance that there are acceptable levels of residual uncertainty in this assessment.

In this case the liquefaction vulnerability category is likely to be 'Very Low' - "There is a probability of more than 99 percent, that liquefaction-induced ground damage will be: None to Minor for 500-year shaking".

6.4 Slope Stability Analysis

6.4.1 Qualitative Assessment

A visual site assessment of the slopes adjacent to the proposed building platform was undertaken during the site walkover. It is understood that the eastern slope exhibited signs of failure during MCE's investigation in 2004. During EDC's site visit, we noted shallow failure on the cut surface of the western slope.

6.4.2 Model Inputs and Assumptions

In the absence of a topographical survey, EDC's site measurement and the contours available on the BOPRC GIS have been used to construct a cross-section through the site. The cross-section location is shown on Figure 12 and was selected at the inferred worst-case location for instability risk; based on gradient and slope height.

While no groundwater table was encountered in EDC's hand auger boreholes, the soil was saturated at various depths.

Soil parameters have been obtained from the table below;

Soil Description	Unit weight γ (kN/m ³)	Friction Angle ϕ (°)	Cohesion c (kPa)
Near Surface Pumiceous Sand	17	38	1
Near Surface Silt	18	32	10
Dense Pumiceous Sand	18	40	1
Stiff Silt	18	32	12

Table 5: Soil Parameters

The following assumptions have been made in the design:

The house design and foundation loads have not been confirmed, as such it is assumed that the foundation imposes a vertical loading of 10kPa on the proposed building platform.

Due to the high risk of shallow topsoil failure on steep slopes, and the low impact on the proposed development, the topsoil has been excluded from the analysis.

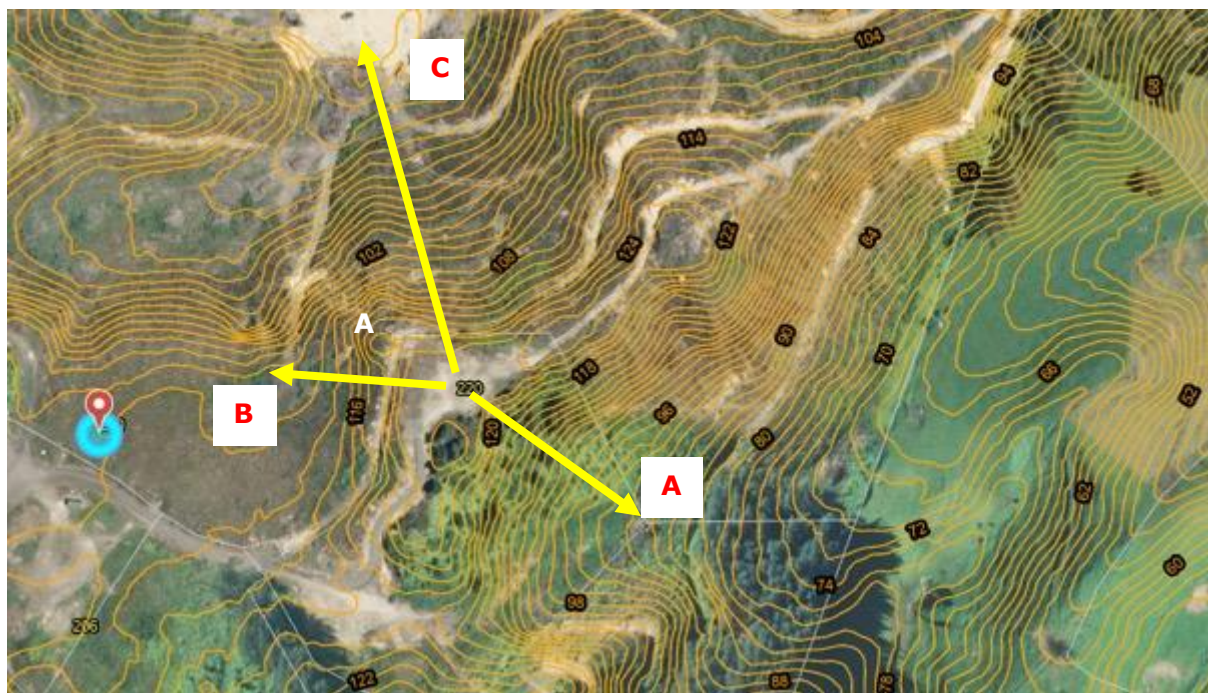


Figure 13: Slope Stability Analysis Cross-Section

6.4.3 Analysis Scenarios

Static/normal, transient/raised groundwater and Ultimate Limit State (ULS) seismic scenarios have been analysed for the slope section above. The ULS scenario analysis is undertaken for the purpose of assessing future ground performance under 'design' earthquake loadings.

Peak Ground Accelerations (PGA) for the purpose of pseudo-static analysis have been quantified using Table A1 from the MBIE 'Earthquake Geotechnical Engineering Practice'

It should be noted that the following pseudo static analysis has used horizontal PGA's that are likely to be conservative since the calculation method assumes the PGA as a constant earthquake force acting only in a direction that promotes slope instability. As such reduction coefficients can be used, which reduce the horizontal PGA model input. However, the correct derivation of a pseudo static coefficient is highly complex and not considered warranted for this assessment.

A Factor of Safety (FOS) ≥ 1.5 for normal, ≥ 1.3 for transient and ≥ 1.0 for seismic slope stability analysis is required for a slope to be considered suitable for development. Where failure planes with lower FOS's exist, the soils between the failure plane and surface are potentially at risk of failure.

The following table provides a generalised interpretation of the Factor of Safety (FoS) against instability:

Factor of Safety	Likelihood of Instability	Comments
1.5 and greater	Very Unlikely	A stable slope. In most cases, such a slope will survive moderately severe ground shaking during an earthquake and will accommodate construction on its surface without affecting the state of stability.
1.25 to 1.5	Unlikely	No signs of instability. Surface building loads reduce factors of safety. At the lower end of this category, slopes are sensitive to groundwater rise and earthquake loads. About 50% of slopes in this category are expected to survive moderately severe earthquake loading, the remainder would yield, deform or fail under earthquake loading.
1.1 to 1.25	Likely	Signs of general stability with some indications of instability by tension and creep. Normal small variations in soil strength characteristics have a big effect on the state of stability. This category is sensitive to changing groundwater conditions, external loading and earthquake loading as they all reduce safety and promote instability. Moderately severe shaking is expected to produce an unstable condition. Imposed building loads reduce the state of stability.
1.0 to 1.1	Very Likely	Generally failure or near failure by landslipping. Small changes in ground and water conditions could precipitate ground rupture. Slopes have no tolerance to earthquake loads. Moderately severe shaking would produce instability. Signs of creep movements are likely to be evident.
Less than 1.0	Almost certain	Slope is theoretically unstable and likely to fail. Slopes have no tolerance to earthquake loadings. Signs of creep movements are likely to be evident.

Table 6: Slope Stability Factor of Safety Interpretation

6.4.4 Analysis Results

Using the *Slide2* (version 9.023) software produced by RocScience, EDC has analysed both circular and non-circular failures using the Spencer and GLE/Morgenstern-Price methods. The results of the slope stability analysis is attached in Appendix E.

Our overall analysis indicates that in the absence of a retaining structure, it is not possible to establish a stable building platform that achieves the minimum required factor of safety. Therefore scenarios with retaining structures have also been taken into consideration for transient and seismic analysis. Based on our analysis, the retaining structure required to achieve the minimum factor of safety would have a shear capacity of 600kN per meter, at 12m depth.

The following Table shows the minimum calculated Factor of Safety results for the analysed scenarios.

Slope	Analysis Condition	Target FoS	Minimum Calculated FoS	Comments
AA'	Normal	1.5	1.6	The slope remains stable
	Transient	1.3	0.5	Potential deep failure extends to the building platform (FOS-1.2)
	Seismic	1.0	0.7	Potential deep failure extends to the building platform
	Transient w/ support	1.3	0.5/1.3	Potential shallow failure to the edge of the support (FOS-0.5), the ground above the barrier pile wall is stable (FOS-1.3)
	Seismic w/ support	1.0	0.7/1.0	Potential shallow failure to the edge of the support (FOS-0.7), the ground above the barrier pile wall is stable (FOS-1.0)
B-B'	Normal	1.5	1.5	The slope remains stable
	Transient	1.3	0.5/1.3	Shallow failure along the cut surfaces (FOS-0.5), potential failure surfaces extending to the edge of the building platform (FOS-1.3)
	Seismic	1.0	0.7/1.0	Shallow failure along the cut surfaces (FOS-0.7), potential failure surfaces to the edge of the building platform (FOS-1.0)
	Transient w/ support	1.3	0.5/1.3	Shallow failure along the cut surfaces (FOS-0.7), the ground above the barrier pile wall is stable (FOS-1.3)
	Seismic w/ support	1.0	0.7/1.1	Shallow failure along the cut surfaces (FOS-0.7), the slope is stable from above the crest (FOS-1.1)
C-C'	Normal	1.5	1.8	The slope remains stable
	Transient	1.3	0.6/1.3	Shallow failure along the steeper section of the slope (FOS-0.6), the soil at the building platform is stable (FOS-1.3)
	Seismic	1.0	0.9	Potential deep failure extends through the whole building platform (FOS-0.9)
	Transient w/ support	1.3	0.6/1.3	Shallow failure along the steeper section of the slope (FOS-0.6), the soil at the building platform is stable (FOS-1.3)
	Seismic w/ support	1.0	0.7/1.0	Shallow failure along the steeper section of the slope (FOS-0.7), the ground within the support is stable (FOS-1.0)

Table 7: Slope Stability Model Results

6.4.5 Building Limitation

A Building Limitation Line (BLL) is defined as the edge of a zone which is the closest that a building should be located towards a slope or significant feature. Building above (i.e. on the side away from the slope) the BLL will have no specific design limitations due to stability issues.

A Building Restriction Line (BRL) is defined as a line beyond which no development is allowed.

Building between the BLL and the BRL would require further geotechnical investigation and analysis and/or require the construction of a retaining structure.

Based on the outcome of the analysis, without additional work to stabilise the slopes around the building platform, the required factor of safety cannot be achieved. We therefore recommend in-ground retaining wall be constructed 5m from each of the slope crests. The in-ground barrier pile wall or equivalent should be designed to have a minimum shear capacity of 600kN per meter length, and 12m in depth; the barrier pile wall should be designed by a chartered structural engineer.

Following the construction of the barrier pile wall, we consider a BLL of 1m as measured from the edge of the retaining structure is considered acceptable. Additionally, it is permissible for the barrier pile wall to be constructed further than 5m away from the slope crest if a smaller building platform is preferred, however, a BLL of 1m from the retaining structure still remains.

We also recommend a BRL at the crest of each slope, if the proposed building platform is to extend beyond BLL, additional investigation deep testing such as CPT deep borehole or similar should be conducted at the crest of each slope.

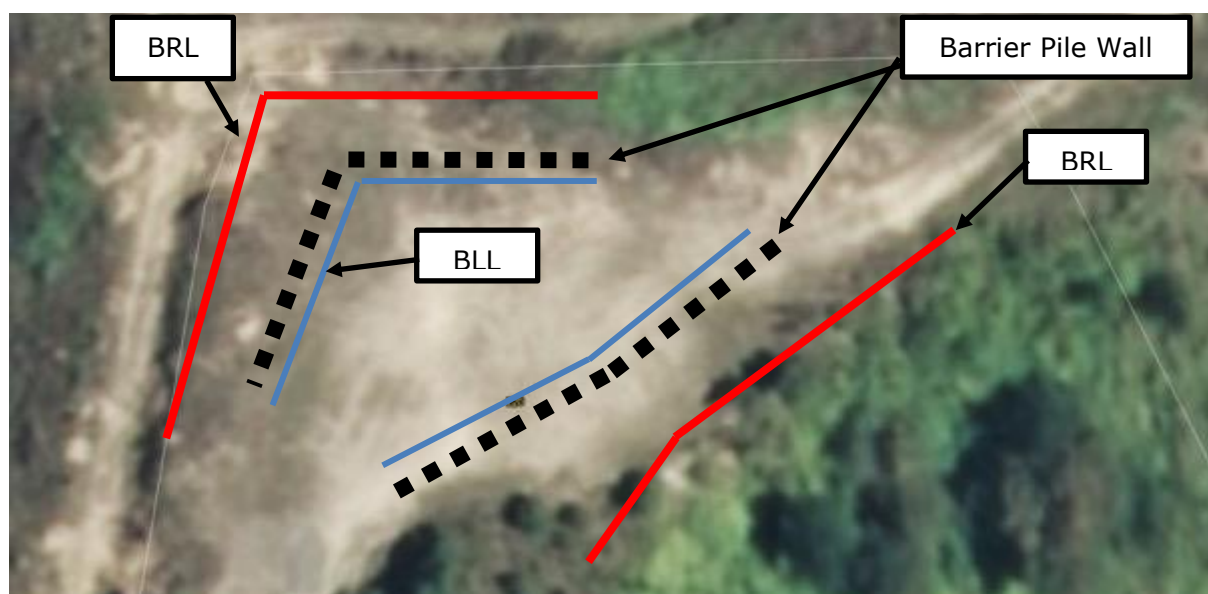


Figure 14: Approximate Building Setbacks and Barrier Pile Wall Locations

6.5 NZS 3604 "Good Ground" Assessment

In accordance with NZS 3604: 2011 "Good Ground" is defined as "Any soil or rock capable of permanently withstanding an ultimate bearing capacity of 300kPa (i.e. an allowable bearing pressure of 100kPa using a factor of safety 3.0)". It excludes expansive soils, topsoil or organic rich soils, uncompacted loose gravel and any ground likely to experience ground movements of 25mm or more.

The soils at the site do not the NZS 3604 definition of 'Good Ground' due to the potential for nearby slope instability.

7.0 RECOMMENDATIONS

7.1 Engineering Requirements

It is the structural engineer or designer's responsibility to ensure that the recommendations of this report are correctly understood and applied. We are happy to discuss the project with the structural engineer or designer and recommend that we review the final design documentation prior to construction.

7.2 Site Restrictions and Additional Works

Due to the potential for slope instability, similar to the recommendation from MCE's report, we consider that an in-ground barrier pile wall or similar ground stabilised feature will be required to create a viable building platform. Please refer to Section 6.4.5 for the in-depth requirement of building setback and retaining structure requirements. The building setback should be marked out in the site plan prior to consent and must be confirmed with a suitably qualified geotechnical engineer.

7.3 Potential Foundation Types

Based on the ground assessment, the following foundation types are considered appropriate to the ground conditions following the construction of a specific engineer design ground stabilisation structure.

7.3.1 Concrete Floor

A waffle slab foundation in general accordance with the MBIE Canterbury Residential Guidance is considered suitable to site conditions. The topsoil (anticipated to be 0.05m deep) will require removal. After the completion of the earthworks, a geotechnical UBC of 300kPa can be used for design.

If fill is required, the fill beneath the slab should consist of an appropriately compacted, well-graded gravel (AP40 or AP65, pumiceous fill or engineer-approved alternative). The above foundation should be produced by a Chartered Structural Engineer.

7.3.2 Timber Floor

A specific engineer design timber floor suspended on pile foundations with either a piled perimeter or reinforced concrete perimeter footing, in general accordance with NZS 3604 (with B1/AS1 modifications) are considered suitable to site conditions. Alternatively, screw pile foundation is also considered acceptable.

The above foundation design should be produced by a Chartered Structural Engineer in accordance with the MBIE guidance.

7.4 General

If gravel hardfill is required, it should be compacted in accordance with NZS 4431:2022 Code of Practice for Earthfill for Residential Development. Validation testing of the compacted gravel should be undertaken and signed off by a suitably experienced Geotechnical Engineer.

According to the New Zealand Building Code, Ultimate Bearing Capacities should be multiplied by a reduction factor based on the design scenario. The MBIE Earthquake geotechnical engineering practice Module 4 'Earthquake resistant foundation design', suggests 0.45 – 0.60 (EDC recommends 0.6) for all Ultimate Limit State load combinations and the NZBC recommends 0.40 – 0.55 (EDC recommends 0.50) for all other load combinations.

Any topsoil/fill, very soft, organic or otherwise unsuitable materials encountered are not considered a suitable bearing stratum for new foundations and will require removal beneath the building platform.

Advice from a geotechnical engineer should be sought if ground conditions different to those encountered in the intrusive investigation are observed during foundation construction works.

7.5 Services

It is anticipated that stormwater will be collected from roofs and used for domestic purposes. Any overflow from stormwater tanks must be collected and with any stormwater from impermeable surfaces or groundwater collected from behind retaining walls (if any are required in the design), must be disposed of in a council approved manner. It is critical that stormwater is not discharged directly on the building platform or on the adjacent slope to prevent destabilising the slope.

The new dwelling will require an on-site wastewater treatment system. We recommend that a secondary treatment system is used. We consider that a suitable system can be developed in accordance with AS/NZS 1547:2012 and any local/regional Council requirements.

The client should consult their consent documents to confirm if a BoPRC schedule 5 assessment will be required. We recommend that the wastewater design be undertaken in accordance with any Schedule 5 assessment report and/or local governing body regulations.

7.6 Ground Inspections

It is recommended that a Chartered Professional Engineer with appropriate geotechnical experience be engaged to supervise any future bulk earthworks, foundation excavations. This is in accordance with normal council practice at the Building Consent stage. It should also be noted that under the Building Act (2004), there are specific requirements for supervision by appropriately qualified personnel.

All inspections will require a minimum of 48 hours' notice and it should be noted that unless we are given the chance to undertake all appropriate inspections for the items specified in the Building Consent, a Producer Statement (PS4) will not be issued.

7.7 Health and Safety

We recommend that 'Safety in Design' principles are included during the design of the proposed structure/development. It should be noted that all parties involved in

construction, including the client have responsibilities under the current 'Health and Safety at Work Act 2015'. This includes ensuring that the chosen contractor is both competent and suitably qualified/experienced to undertake the commissioned works.

Specifically, with respect to earthworks and foundations for this site, consideration needs to be made with respect to the presence of:

- Steep and narrow access;
- Site stability during construction.

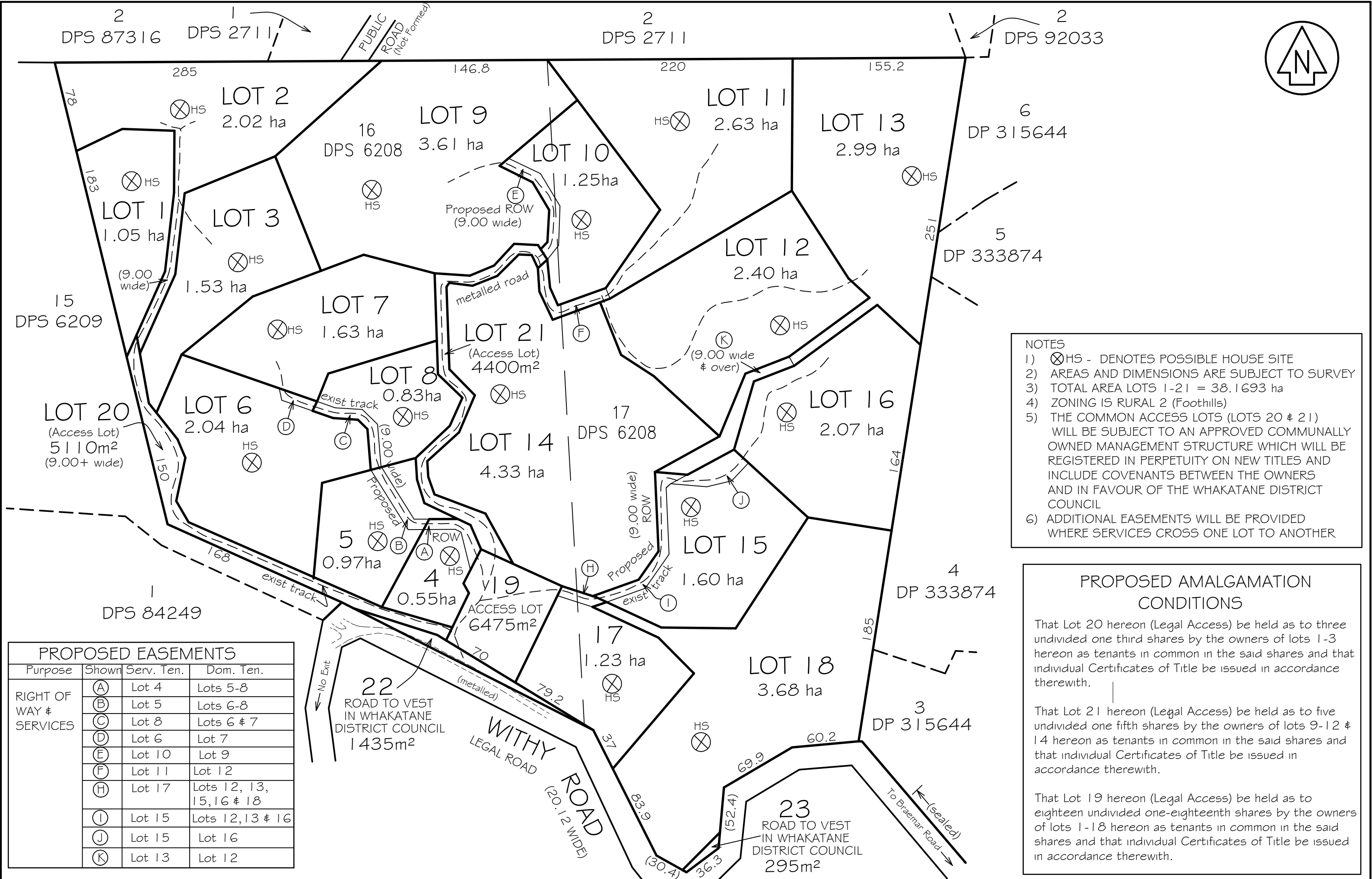
This list is not exhaustive and there might also be other specific considerations not listed above. EDC would be happy to provide geotechnical/geoenvironmental advice in relation to these issues at any workshops or design meetings for this project. In addition, our Structural and Fire Safety Design Engineers can provide advice as appropriate.

8.0 REFERENCES

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APPENDIX A

SURVEY PLAN



- NOTES
- 1) ⊗ HS - DENOTES POSSIBLE HOUSE SITE
 - 2) AREAS AND DIMENSIONS ARE SUBJECT TO SURVEY
 - 3) TOTAL AREA LOTS 1-21 = 38.1693 ha
 - 4) ZONING IS RURAL 2 (Foothills)
 - 5) THE COMMON ACCESS LOTS (LOTS 20 & 21) WILL BE SUBJECT TO AN APPROVED COMMUNALLY OWNED MANAGEMENT STRUCTURE WHICH WILL BE REGISTERED IN PERPETUITY ON NEW TITLES AND INCLUDE COVENANTS BETWEEN THE OWNERS AND IN FAVOUR OF THE WHAKATANE DISTRICT COUNCIL
 - 6) ADDITIONAL EASEMENTS WILL BE PROVIDED WHERE SERVICES CROSS ONE LOT TO ANOTHER

PROPOSED AMALGAMATION CONDITIONS

That Lot 20 hereon (Legal Access) be held as to three undivided one third shares by the owners of lots 1-3 hereon as tenants in common in the said shares and that individual Certificates of Title be issued in accordance therewith.

That Lot 21 hereon (Legal Access) be held as to five undivided one fifth shares by the owners of lots 9-12 & 14 hereon as tenants in common in the said shares and that individual Certificates of Title be issued in accordance therewith.

That Lot 19 hereon (Legal Access) be held as to eighteen undivided one-eighteenth shares by the owners of lots 1-18 hereon as tenants in common in the said shares and that individual Certificates of Title be issued in accordance therewith.

PROPOSED EASEMENTS			
Purpose	Shown	Serv. Ten.	Dom. Ten.
RIGHT OF WAY & SERVICES	(A)	Lot 4	Lots 5-8
	(B)	Lot 5	Lots 6-8
	(C)	Lot 8	Lots 6 & 7
	(D)	Lot 6	Lot 7
	(E)	Lot 10	Lot 9
	(F)	Lot 11	Lot 12
	(H)	Lot 17	Lots 12, 13, 15, 16 & 18
	(I)	Lot 15	Lots 12, 13 & 16
	(J)	Lot 15	Lot 16
	(K)	Lot 13	Lot 12

PREPARED FOR: BE & CG DAVIES
PROPERTY AT: 168 - 206 WITHY ROAD
AWAKAPONGA
WHAKATANE

PROPOSED SUBDIVISION OF LOTS 16 & 17
DPS 6208
COMPRISED IN CT SA8C/1117
WHAKATANE DISTRICT

ROSS J. OVERINGTON
REGISTERED SURVEYORS
P O BOX 596 ● WHAKATANE ● Phone 3086384
DRAWN RJO DATE OCT. 2006
TRACED SAW SCALES (A3) 1 : 3000
DRAWING No. S2053/1C
CAD No. 2053 daves

APPENDIX B

EDC'S HAND AUGER AND MCE'S BOREHOLE LOGS

PROJECT: New Dwelling

CLIENT: Mike Davies

ADDRESS: Lot 15, 220 Withy Road, Awakaponga

LOGGED

WW

PROCESSED

WW

CHECKED

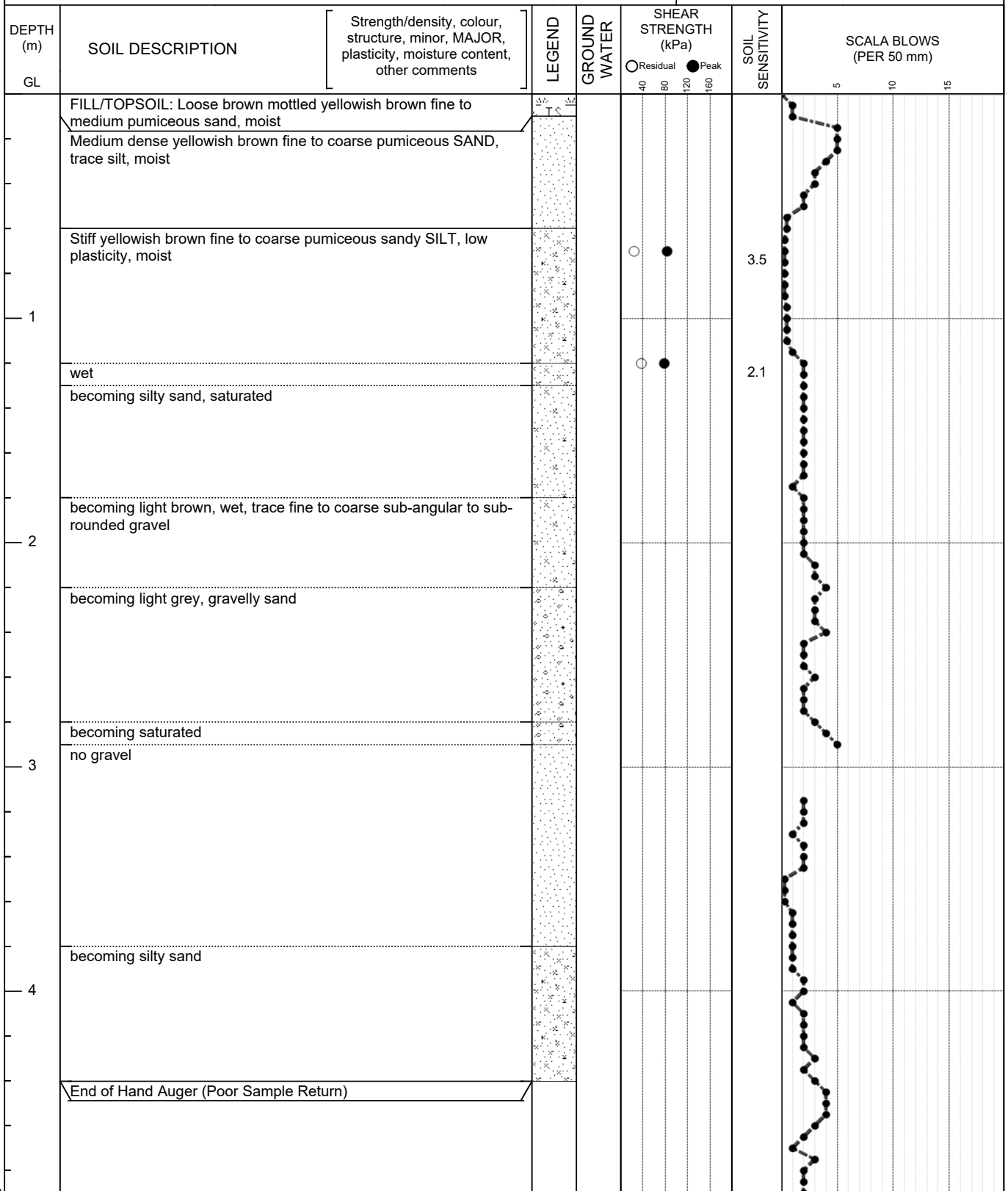
GW

BH LOCATION: COORDS:

RL GROUND:

SHEAR VANE ID#:

1955



NOTES:

While groundwater was not encountered during hand auger drilling, the soil was wet from 1.2m and saturated from 1.3m.

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FIRE ENGINEERS

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SUITE 7,202 THE STRAND
PO Box 855,WHAKATANE 3158

PH (07) 922 0075

1st FLOOR, UNIT 1, 100 BUSH ROAD,
ALBANY, AUCKLAND

PH (09) 451 9044

PROJECT: New Dwelling

CLIENT: Mike Davies

ADDRESS: Lot 15, 220 Withy Road, Awakaponga

LOGGED

WW

PROCESSED

WW

CHECKED

GW

BH LOCATION: COORDS:

RL GROUND:

SHEAR VANE ID#:

1955

DEPTH (m)	SOIL DESCRIPTION	Strength/density, colour, structure, minor, MAJOR, plasticity, moisture content, other comments	LEGEND	GROUND WATER	SHEAR STRENGTH (kPa) ○ Residual ● Peak	SOIL SENSITIVITY	SCALA BLOWS (PER 50 mm)
GL					40 80 120 160		5 10 15
	TOPSOIL: Loose dark brown organic fine to medium silty sand, wet						
	Medium dense pale brown fine to coarse pumiceous sand, trace silt, some fine to coarse sub-angular pumiceous gravel, moist						
	Medium dense brown fine to medium pumiceous SAND, moist						
	becoming light yellow, sand becoming fine to coarse						
1	Very stiff yellowish brown sandy SILT, low plasticity, wet. Sand is fine to coarse, pumiceous				Unable To Penetrate		
	Dense light yellow fine to coarse pumiceous SAND, trace silt, trace fine to coarse angular pumiceous gravel, saturated						
	becoming gravelly sand						
	End of Hand Auger (Unable to Penetrate)						
2							

EOH @ 2.90 m

NOTES:

While groundwater was not encountered during hand auger drilling, the soil was wet from 0.8m and saturated from 1.0m.

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CLIENT: Mike Davies

ADDRESS: Lot 15, 220 Withy Road, Awakaponga

LOGGED

WW

PROCESSED

WW

CHECKED

GW

BH LOCATION: COORDS:

RL GROUND:

SHEAR VANE ID#:

1955

DEPTH (m)	SOIL DESCRIPTION	Strength/density, colour, structure, minor, MAJOR, plasticity, moisture content, other comments	LEGEND	GROUND WATER	SHEAR STRENGTH (kPa) ○ Residual ● Peak	SOIL SENSITIVITY	SCALA BLOWS (PER 50 mm)
GL					40 80 120 160		5 10 15
	TOPSOIL: Loose dark brown organic fine sand, some silt, wet						
	Medium dense yellowish brown silty fine to coarse pumiceous SAND, moist						
	Very stiff yellowish brown sandy SILT, low plasticity, moist. Sand is fine, pumiceous						
	no sand						
1	becoming wet				Unable To Penetrate	2.4	
	Medium dense light yellow mottled white fine to coarse pumiceous SAND, trace fine to coarse pumiceous gravel, trace silt, moist						
2							

End of Hand Auger (Target Depth Achieved)

EOH @ 2.20 m

NOTES:

While groundwater was not encountered during hand auger drilling, the soil was wet from 0.9m.

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PH (09) 451 9044

PROJECT: New Dwelling

CLIENT: Mike Davies

ADDRESS: Lot 15, 220 Withy Road, Awakaponga

LOGGED

WW

PROCESSED

WW

CHECKED

GW

BH LOCATION: COORDS:

RL GROUND:

SHEAR VANE ID#:

1955

DEPTH (m)	SOIL DESCRIPTION	Strength/density, colour, structure, minor, MAJOR, plasticity, moisture content, other comments	LEGEND	GROUND WATER	SHEAR STRENGTH (kPa) ○ Residual ● Peak	SOIL SENSITIVITY	SCALA BLOWS (PER 50 mm)
GL					40 80 120 160		5 10 15
	TOPSOIL: Loose dark brown organic fine to medium silty sand, wet						
	Dense yellowish brown fine to coarse pumiceous SAND, trace silt, moist						
1							
	Very stiff yellowish brown sandy SILT, low plasticity, moist. Sand is fine, pumiceous					2.8	
	becoming stiff, sand is fine to medium, pumiceous					3.3	
2						3.8	
	saturated						
	End of Hand Auger (Poor Sample Return)		EOH @ 2.70 m				

NOTES:

While groundwater was not encountered during hand auger drilling, the soil was saturated from 2.4m

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ALBANY, AUCKLAND

PH (09) 451 9044

SOIL TEST RESULTS SHEET Client: B.E. & C.G. DAVIES		Test Site ws9 of Lot 15 Ref: 111.1 Date: 22/08/2005
Project Location: 168-206 WITTY ROAD WHAKATANE		

Comments	Soil Sensitivity	Shear vane	Cu (kPa)	Scale (mm/blow)	Depth (m)	Soil Description
						TOPSOIL
					0.5	SAND- dark brown, fine, moist, traces silt at 0.5m- becomes fine to coarse, orange brown
					1.0	
					1.5	at 1.2m- minor medium gravel, cream
					2.0	SILT- brown, moist, minor sand
					2.5	at 2.3m- band of coarse sand, minor fine gravel (100mm)
					3.0	at 3.0m- becomes light brown, no apparent sand
					3.5	
					4.0	
					4.5	SAND- coarse, minor medium to coarse gravel, black speckling
					5.0	
					5.5	at 5.6m- band of dark brown fine sand (100mm)
					6.0	continued over

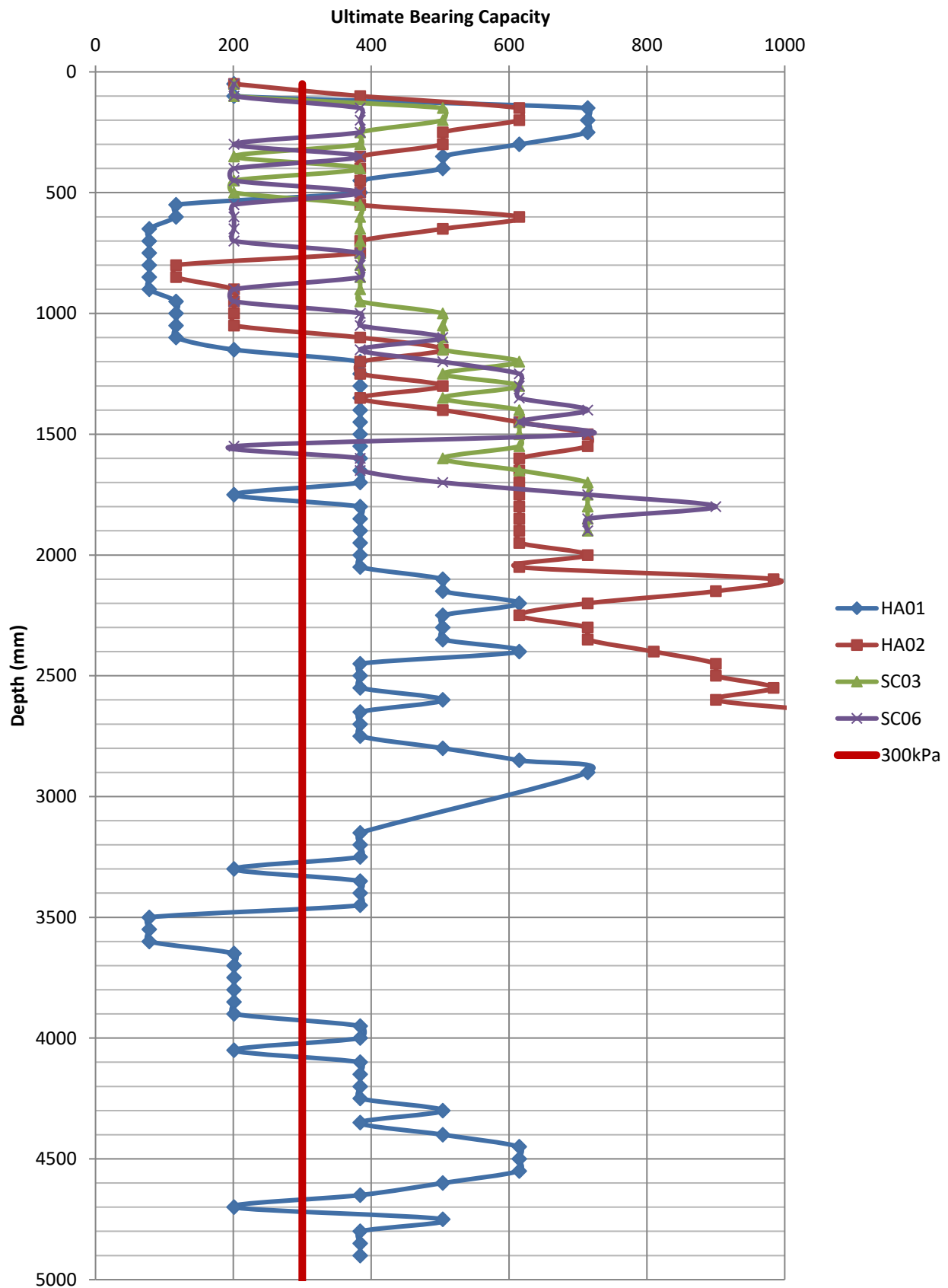
SOIL TEST RESULTS SHEET Client: B.E. & C.G. DAVIES		Test Site ws9 of continued Ref: 111.1 Date: 22/08/2005
Project Location: 168-206 WITHY ROAD WHAKATANE		

Comments	Soil Sensitivity	Shear vane Cu (kPa)	Scale (mm/blow)	Depth (m)	Soil Description
					continued from overleaf
				6.5	
				7.0	
				7.5	
				8.0	
				8.5	SAND- orange brown, moist, fine to medium, minor silt, occasional medium gravel, black manganese speckling
				9.0	
				9.5	SILT- cream, moist, traces fine sand, occasional black organic speckles, soft
				10.0	End of hole at 10m
				10.5	
				11.0	
				11.5	
				12.0	

APPENDIX C

UBC CHART

Summary of Ultimate Bearing Capacity vs Depth

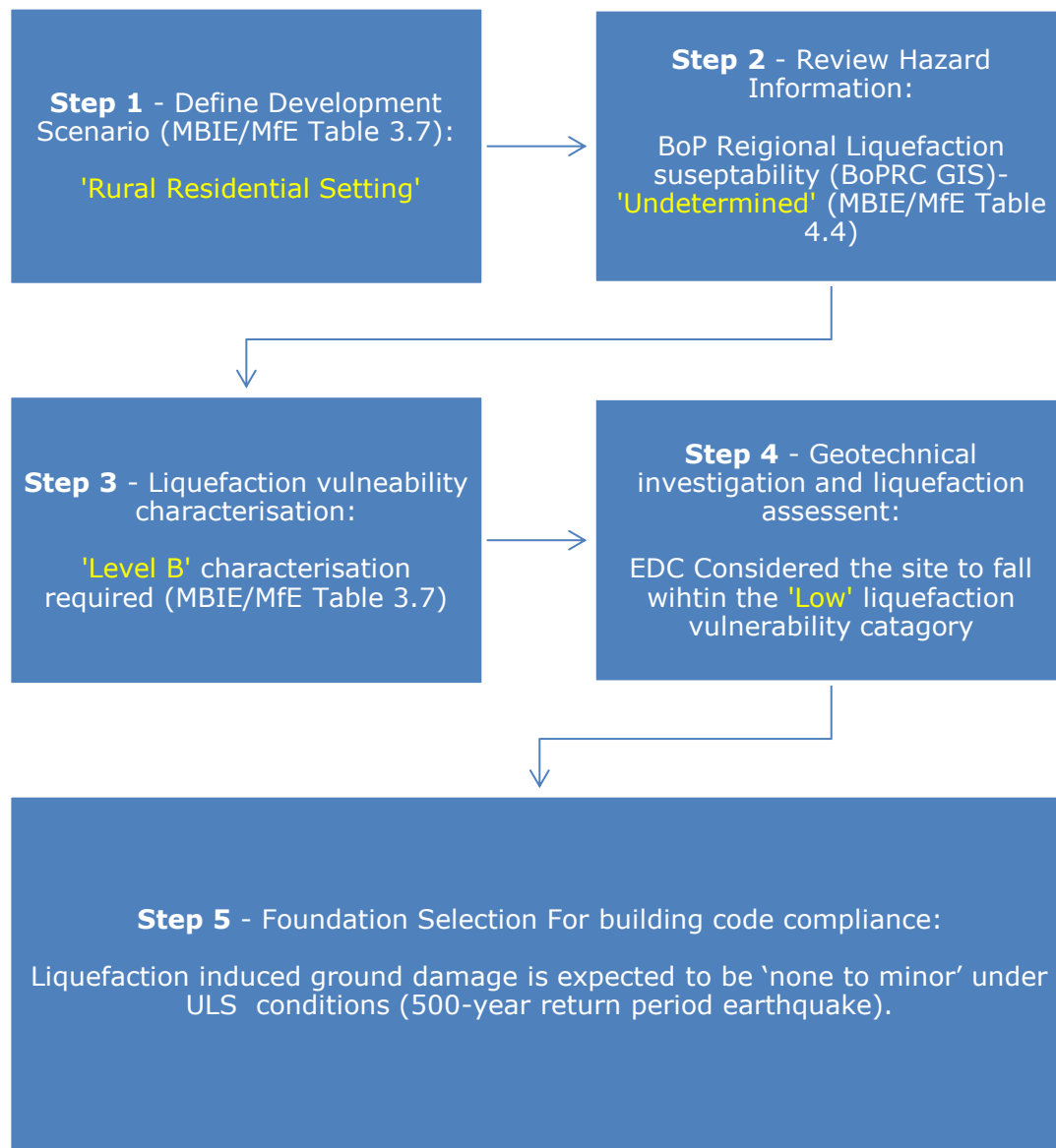


NOTES

1. Based on Stockwell, M.J., 1977: Determination of allowable bearing pressure under small structures. New Zealand Engineering (32:6), dated 15 June 1977, using a factor of safety of three to back calculate the UBC.

APPENDIX D

MBIE/MFE SITE SPECIFIC LIQUEFACTION ASSESSMENT FLOW CHART, ASSESSMENT MATRIX & DEFINITIONS



STEP 1 – Define development scenario (refer to Table 3.7 of MBIE/MfE Guidance (2017))

		Increasing likelihood and severity of ground damage			
Increasing new capital investment and total exposure to a single event	DEVELOPMENT SCENARIO ²	LIQUEFACTION VULNERABILITY CATEGORY			
		LIQUEFACTION CATEGORY IS UNDETERMINED			
		LIQUEFACTION DAMAGE IS UNLIKELY		LIQUEFACTION DAMAGE IS POSSIBLE	
		Very Low	Low	Medium	High
	Sparsely populated rural area (lot size more than 4 Ha) eg A new farm building	Level A	Level A	Level A	Level A
	Rural-residential setting (lot size of 1 to 4 Ha) eg A 'lifestyle' property	Level A	Level B	Level B	Level B
	Small-scale urban infill (original lot size less than 2500 m2) eg Demolish old house and replace with four townhouses	Level B	Level B	Level B	Level D
	Commercial or industrial development ⁷ eg A warehouse building in an industrial park	Level B	Level B	Level C	Level D
	Urban residential development (lot size less than 1 Ha; typically <1000 m2) eg Home in a new subdivision	Level B	Level C	Level C	Level D

Table 4.4: Performance criteria for determining the liquefaction vulnerability category

LIQUEFACTION CATEGORY IS UNDETERMINED			
A liquefaction vulnerability category has not been assigned at this stage, either because a liquefaction assessment has not been undertaken for this area, or there is not enough information to determine the appropriate category with the required level of confidence.			
LIQUEFACTION DAMAGE IS UNLIKELY		LIQUEFACTION DAMAGE IS POSSIBLE	
There is a probability of more than 85 percent that liquefaction-induced ground damage will be None to Minor for 500-year shaking. At this stage there is not enough information to distinguish between Very Low and Low . More detailed assessment would be required to assign a more specific liquefaction category.		There is a probability of more than 15 percent that liquefaction-induced ground damage will be Minor to Moderate (or more) for 500-year shaking. At this stage there is not enough information to distinguish between Medium and High . More detailed assessment would be required to assign a more specific liquefaction category.	
Very Low Liquefaction Vulnerability	Low Liquefaction Vulnerability	Medium Liquefaction Vulnerability	High Liquefaction Vulnerability
There is a probability of more than 99 percent that liquefaction-induced ground damage will be None to Minor for 500-year shaking.	There is a probability of more than 85 percent that liquefaction-induced ground damage will be None to Minor for 500-year shaking.	There is a probability of more than 50 percent that liquefaction-induced ground damage will be: Minor to Moderate (or less) for 500-year shaking; and None to Minor for 100-year shaking.	There is a probability of more than 50 percent that liquefaction-induced ground damage will be: Moderate to Severe for 500-year shaking; and/or Minor to Moderate (or more) for 100-year shaking.

Table 3.1: Levels of detail for liquefaction assessment studies, and the key defining features

LEVEL OF DETAIL	KEY FEATURES	Increasing level of detail and decreasing degree of uncertainty
Level A Basic desktop assessment	Considers only the most basic information about geology, groundwater and seismic hazard to assess the potential for liquefaction to occur. This can typically be completed as a simple 'desktop study', based on existing information (eg geological and topographic maps) and local knowledge. Residual uncertainty: The primary focus is identifying land where there is a High degree of certainty that Liquefaction Damage is Unlikely (so it can be 'taken off the table' without further assessment). For other areas, substantial uncertainty will likely remain regarding the level of risk.	
Level B Calibrated desktop assessment	Includes high-level 'calibration' of geological/geomorphic maps. Qualitative (or possibly quantitative) assessment of a small number of subsurface investigations provides a better understanding of liquefaction susceptibility and triggering for the mapped deposits and underlying ground profile. For example, the calibration might indicate the ground performance within a broad area is likely to fall within a particular range. It may be possible to extrapolate the calibration results to other nearby areas of similar geology and geomorphology, however care should be taken not to over-extrapolate (particularly in highly variable ground such as alluvial deposits), and the associated uncertainties (and potential consequences) should be clearly communicated. Targeted collection of new information may be very useful in areas where existing information is sparse and reducing the uncertainty could have a significant impact on objectives and decision-making. Residual uncertainty: Because of the limited amount of subsurface ground information, significant uncertainty is likely to remain regarding the level of liquefaction-related risk, how it varies across each mapped area, and the delineation of boundaries between different areas.	
Level C Detailed area-wide assessment	Includes quantitative assessment based on a moderate density of subsurface investigations, with other information (eg geomorphology and groundwater) also assessed in finer detail. May require significant investment in additional ground investigations and more complex engineering analysis. Residual uncertainty: The information analysed is sufficient to determine with a moderate degree of confidence the typical range of liquefaction-related risk within an area and delineation of boundaries between areas, but is insufficient to confidently determine the risk more precisely at a specific location.	
Level D Site-specific assessment	Draws on a high density of subsurface investigations (eg on or very close to the site being assessed), and takes into account the specific details of the proposed site development (eg location, size and foundation type of building). Residual uncertainty: The information and analysis is sufficient to determine with a High degree of confidence the level of liquefaction-related risk at a specific location. However, the scientific understanding of liquefaction and seismic hazard is imperfect, so there remains a risk that actual land performance could differ from expectations even with a high level of site-specific detail in the assessment.	

Application of the vulnerability categories is at the discretion of the site engineer/reviewer, and should be taken into account, alongside all other site risks. The following table should be used as a **basic guide only**:

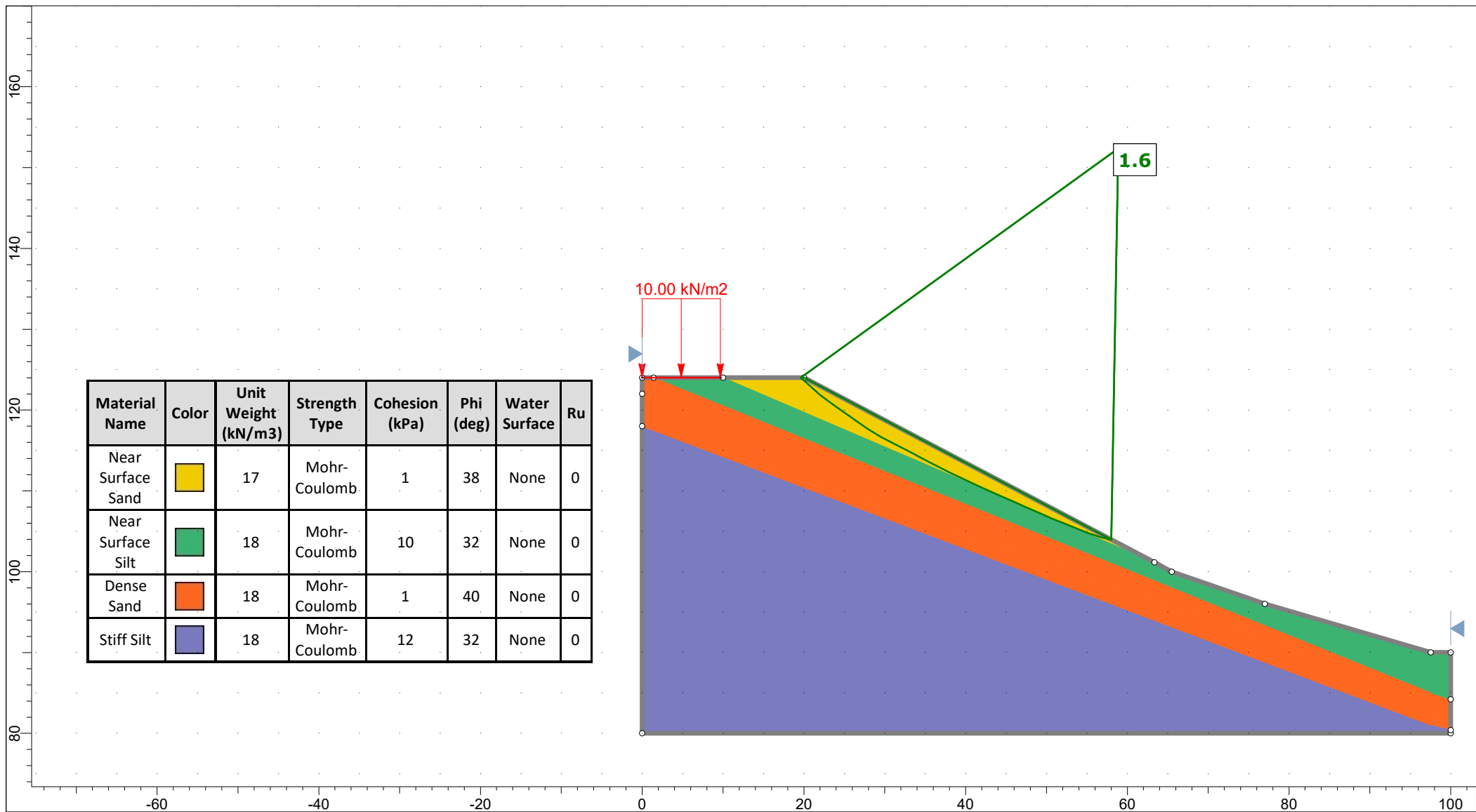
EDC Assessment	Structure Importance Level	Very Low	Low	Medium	High
Level B/C	1	NZS3604	NZS3604	NZS3604	MBIE TC2
	2		NZS3604 or TC2*	MBIE TC2 or TC2/TC3 Hybrid*	Detailed Assessment
Level D	1		NZS3604	NZS3604	MBIE TC2
	2			MBIE TC2	MBIE TC3

Table 8: Indicative foundation requirements based on liquefaction susceptibility and level of assessment

*Based on engineering judgement following assessment of risk.

APPENDIX E

SLOPE STABILITY ANALYSIS



SLIDEINTERPRET 9.023

Project

Lot 15, 220 Withy Road

Group

Slope A-A'

Scenario

Normal

Drawn By

WW

Company

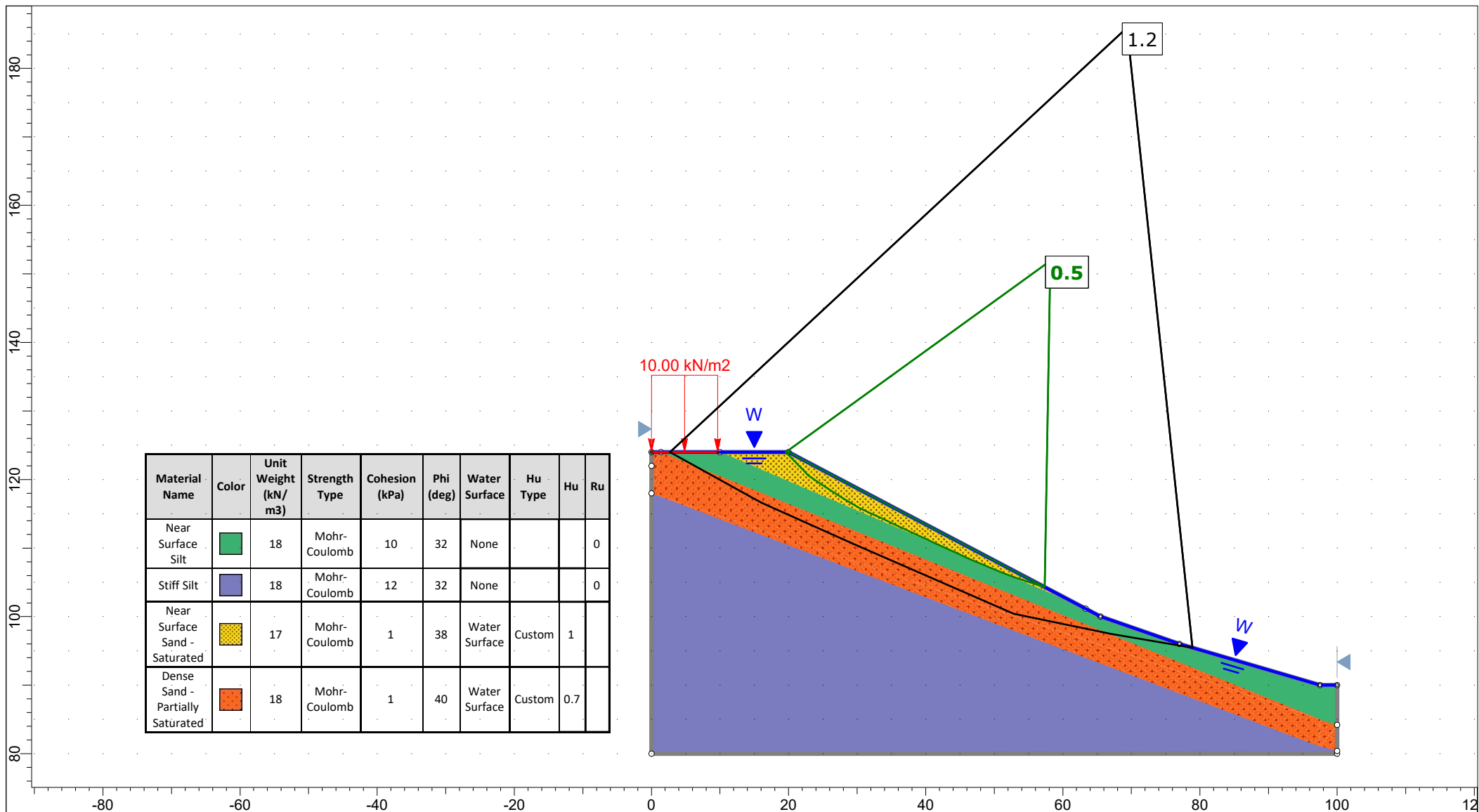
Engineering Design Consultants Ltd

Date

10/08/2022, 12:46:27 PM

File Name

20220810_50721_Slope Stability Analysis.slmd



SLIDEINTERPRET 9.023

Project

Lot 15, 220 Withy Road

Group

Slope A-A'

Scenario

Transient

Drawn By

WW

Company

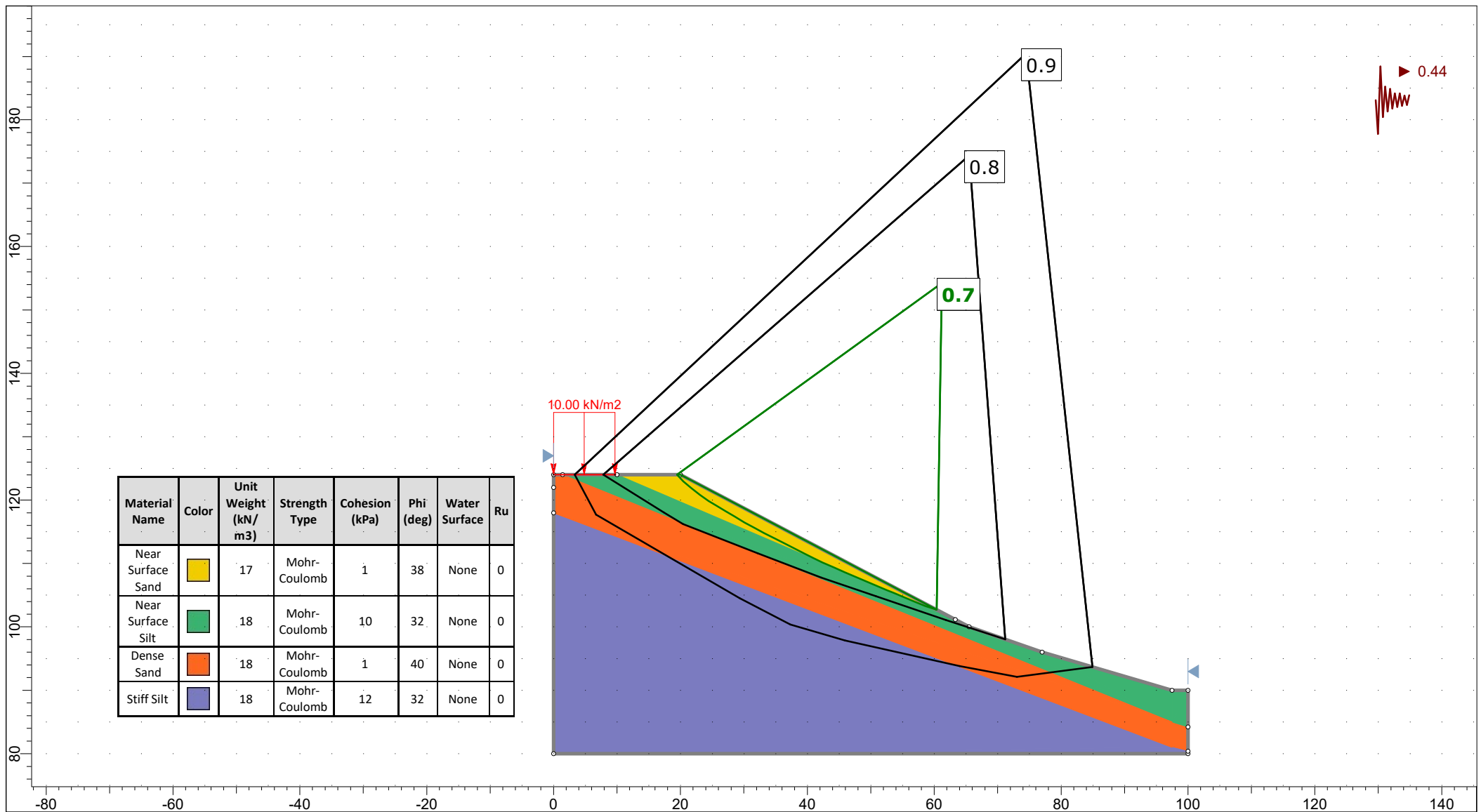
Engineering Design Consultants Ltd

Date

10/08/2022, 12:46:27 PM

File Name

20220810_50721_Slope Stability Analysis.slmd



SLIDEINTERPRET 9.023

Project

Lot 15, 220 Withy Road

Group

Slope A-A'

Scenario

Seismic

Drawn By

WW

Company

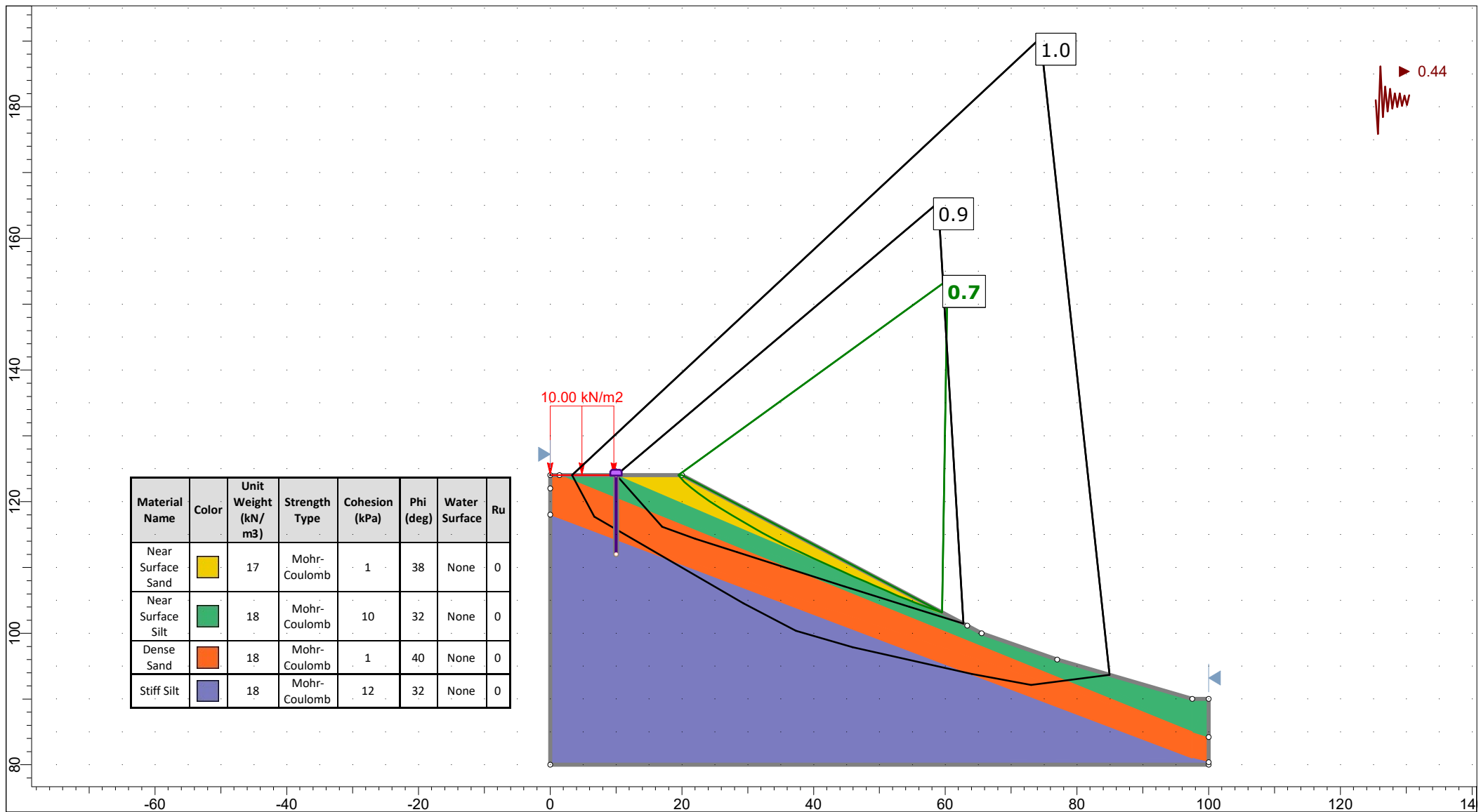
Engineering Design Consultants Ltd

Date

10/08/2022, 12:46:27 PM

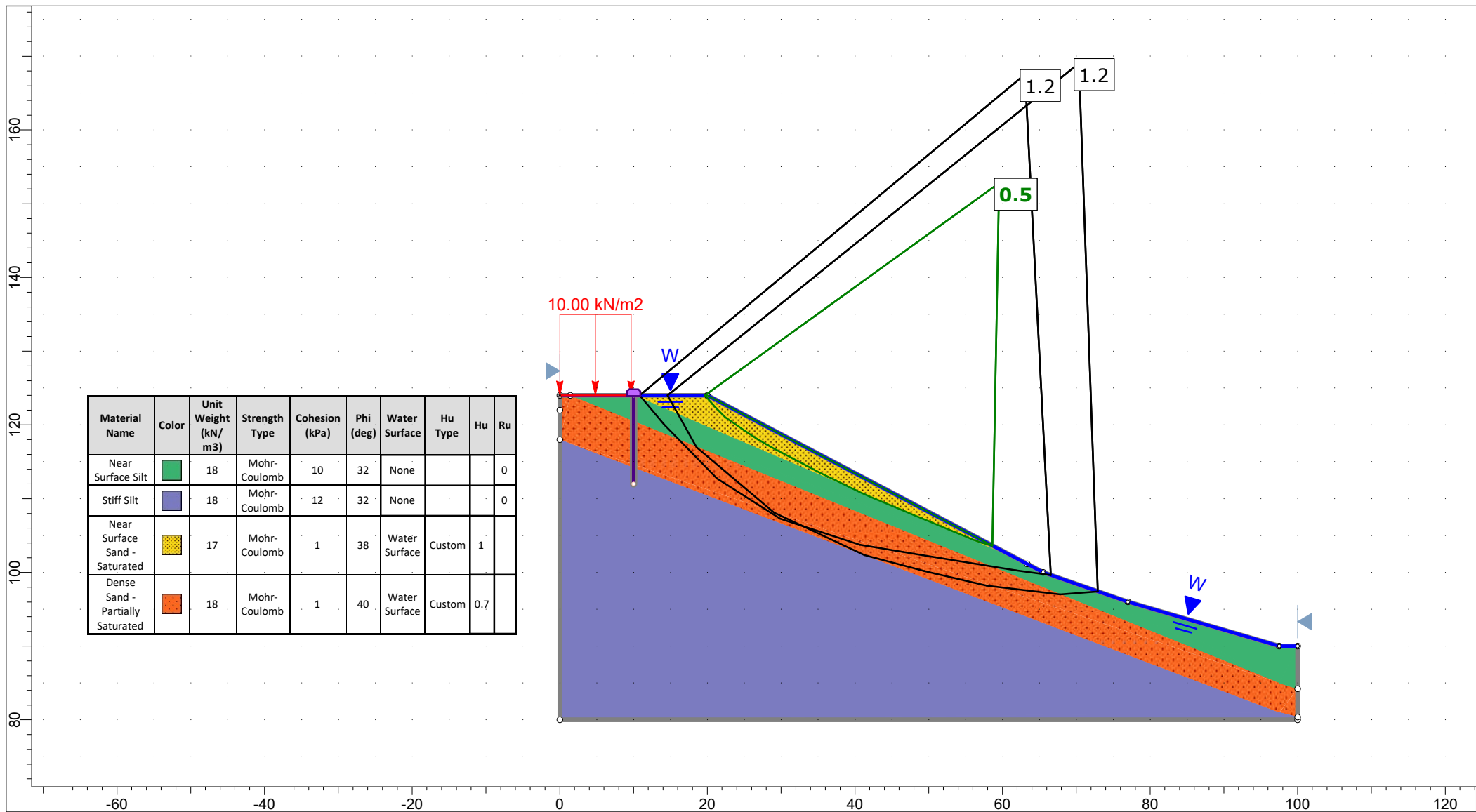
File Name

20220810_50721_Slope Stability Analysis.slmd



SLIDEINTERPRET 9.023

Project		Lot 15, 220 Withy Road	
Group	Slope A-A'	Scenario	Seismic with support
Drawn By	WW	Company	Engineering Design Consultants Ltd
Date	10/08/2022, 12:46:27 PM	File Name	20220810_50721_Slope Stability Analysis.slmd



SLIDEINTERPRET 9.023

Project

Lot 15, 220 Withy Road

Group

Slope A-A'

Scenario

Transient with support

Drawn By

WW

Company

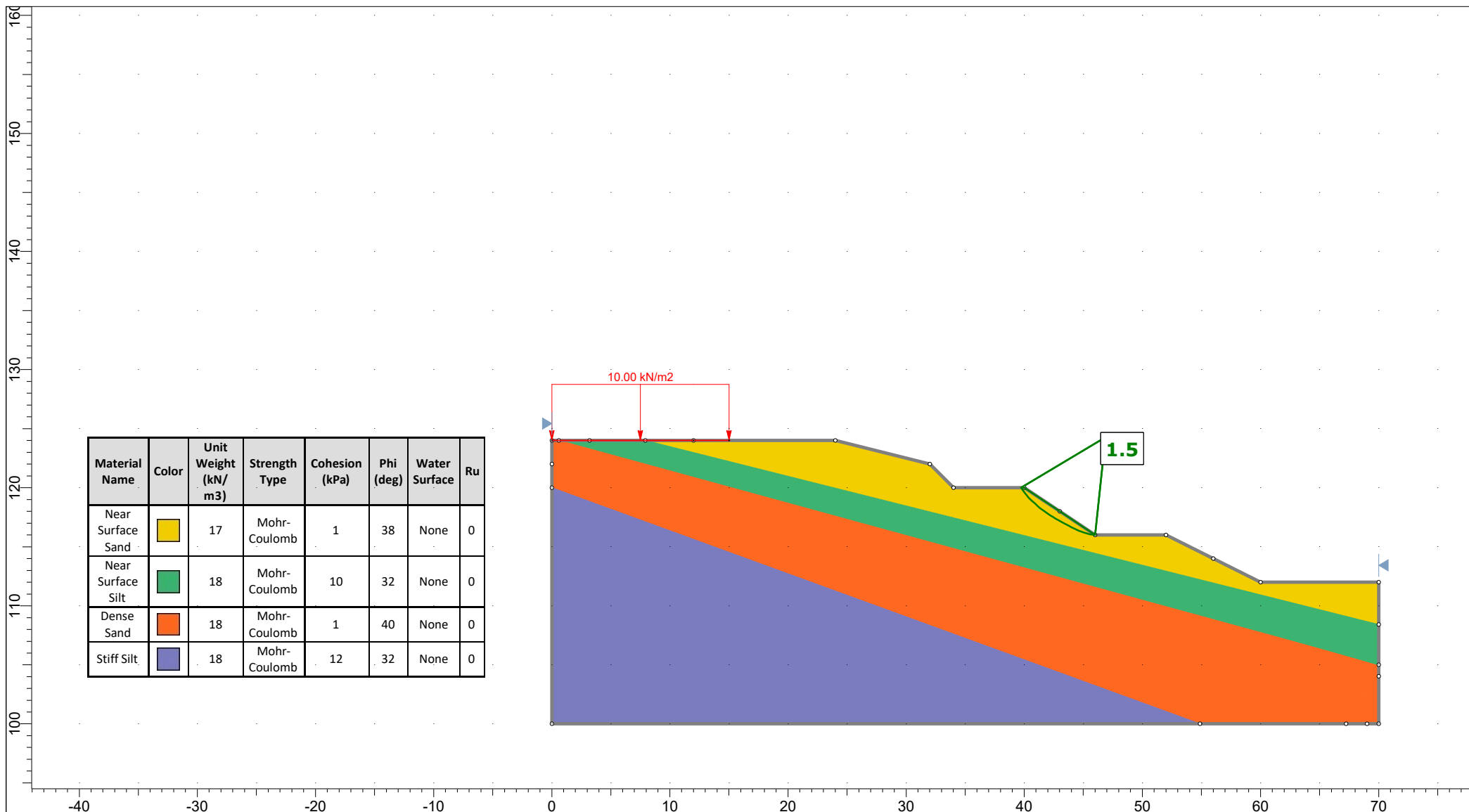
Engineering Design Consultants Ltd

Date

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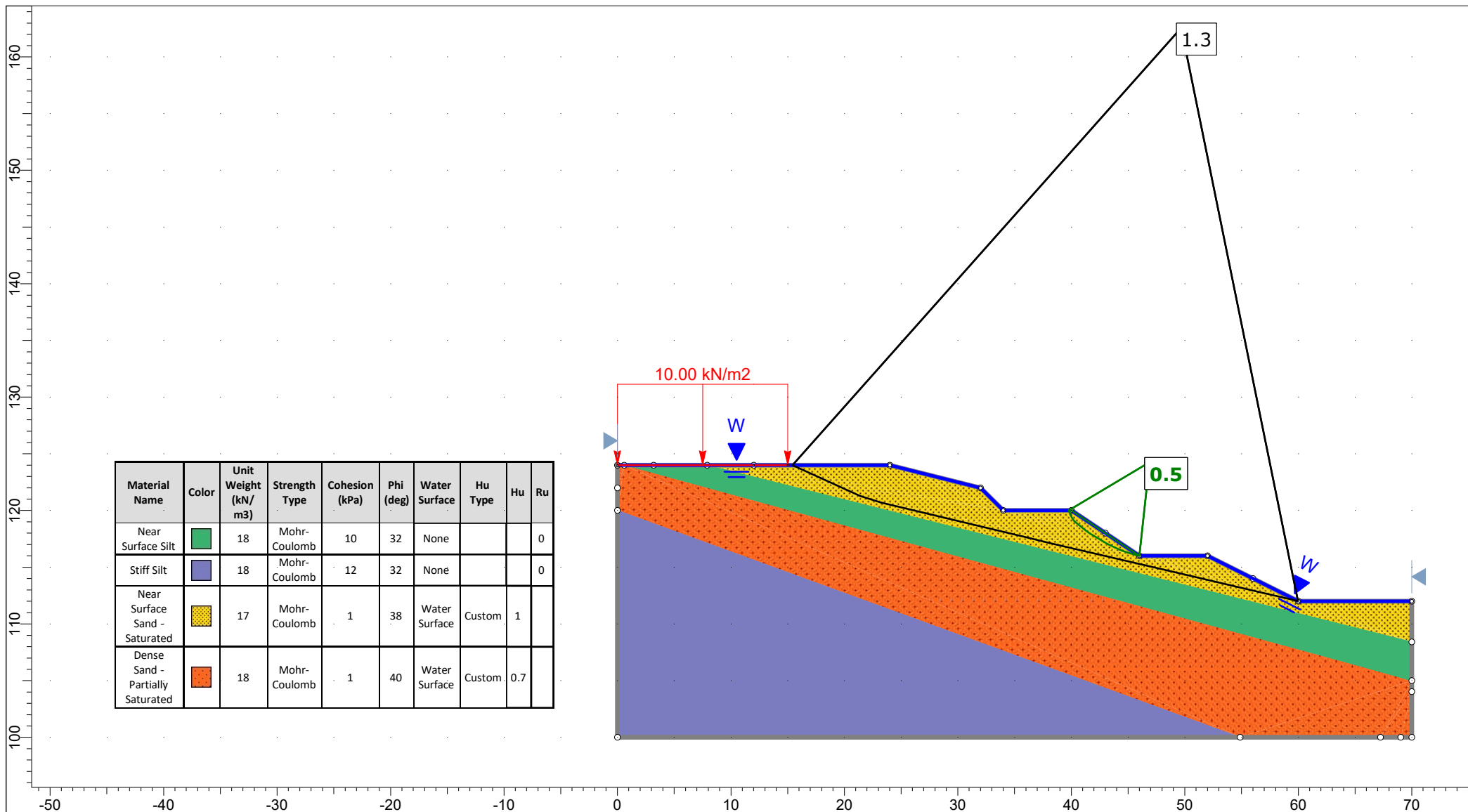
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20220810_50721_Slope Stability Analysis.slmd



SLIDEINTERPRET 9.023

Project		Lot 15, 220 Withy Road	
Group	Slope B-B'	Scenario	Normal
Drawn By	WW	Company	Engineering Design Consultants Ltd
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SLIDEINTERPRET 9.023

Project

Lot 15, 220 Withy Road

Group

Slope B-B'

Scenario

Transient

Drawn By

WW

Company

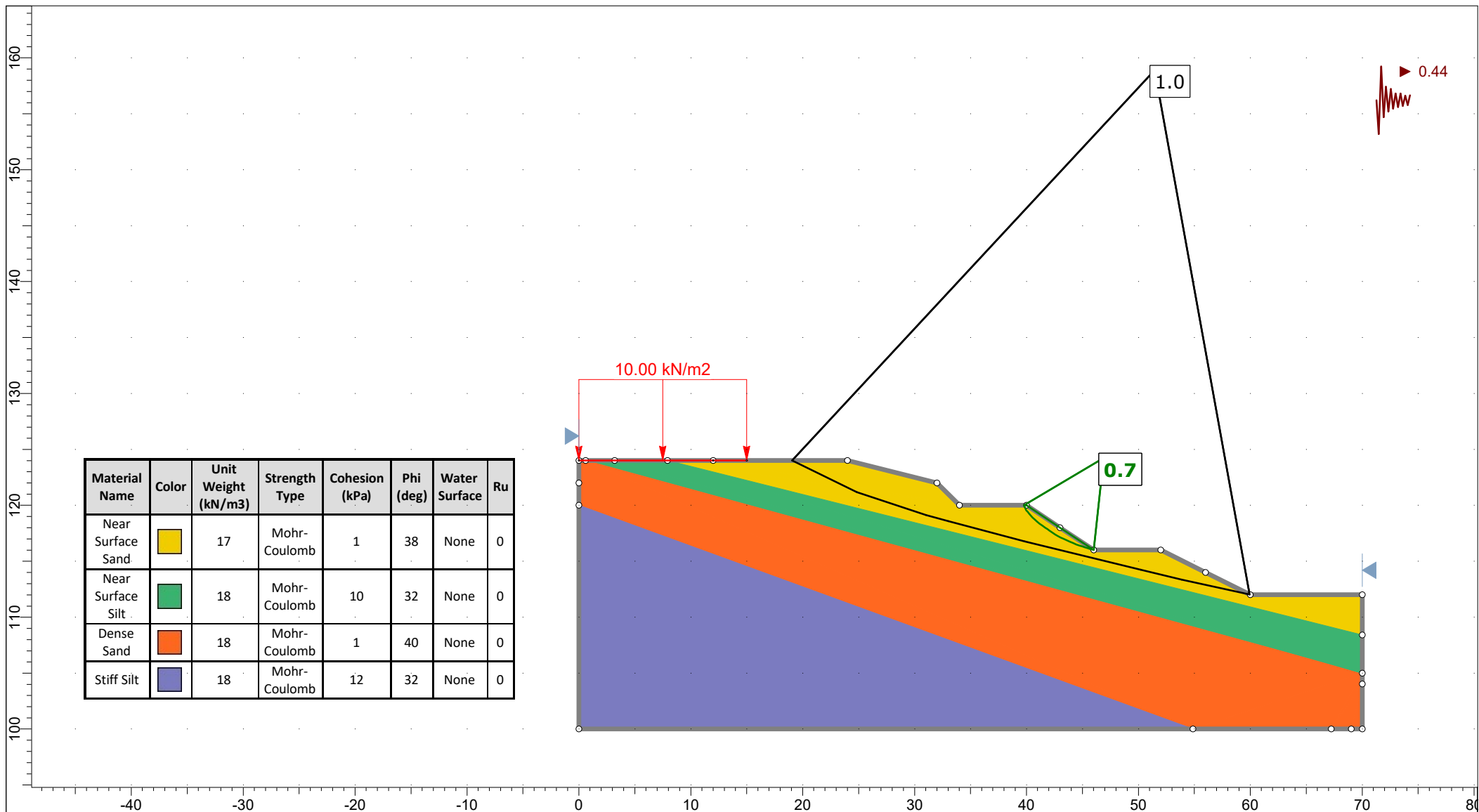
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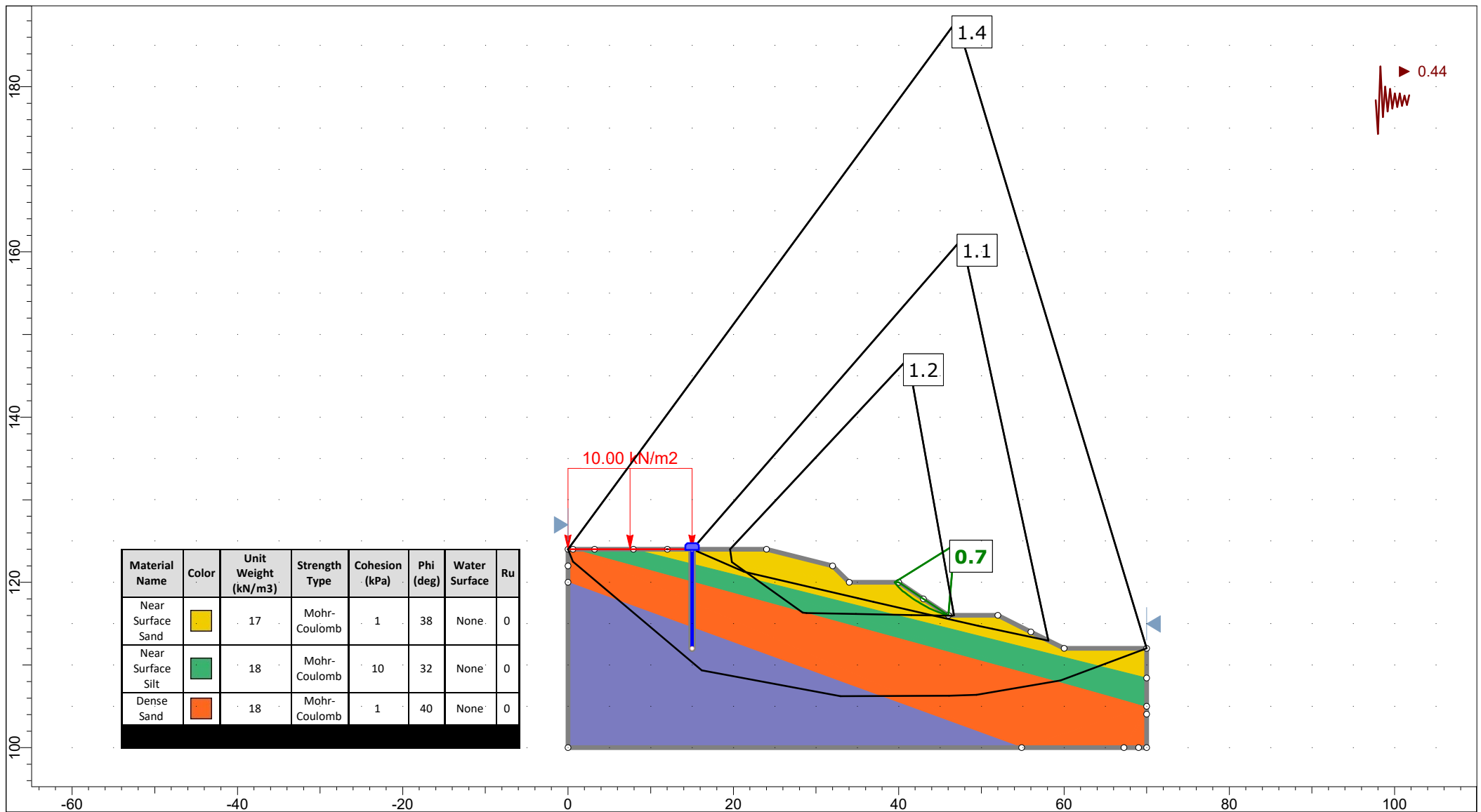
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SLIDEINTERPRET 9.023

Project	Lot 15, 220 Withy Road		
Group	Slope B-B'	Scenario	Seismic
Drawn By	WW	Company	Engineering Design Consultants Ltd
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SLIDEINTERPRET 9.023

Project

Lot 15, 220 Withy Road

Group

Slope B-B'

Scenario

Seismic with support

Drawn By

WW

Company

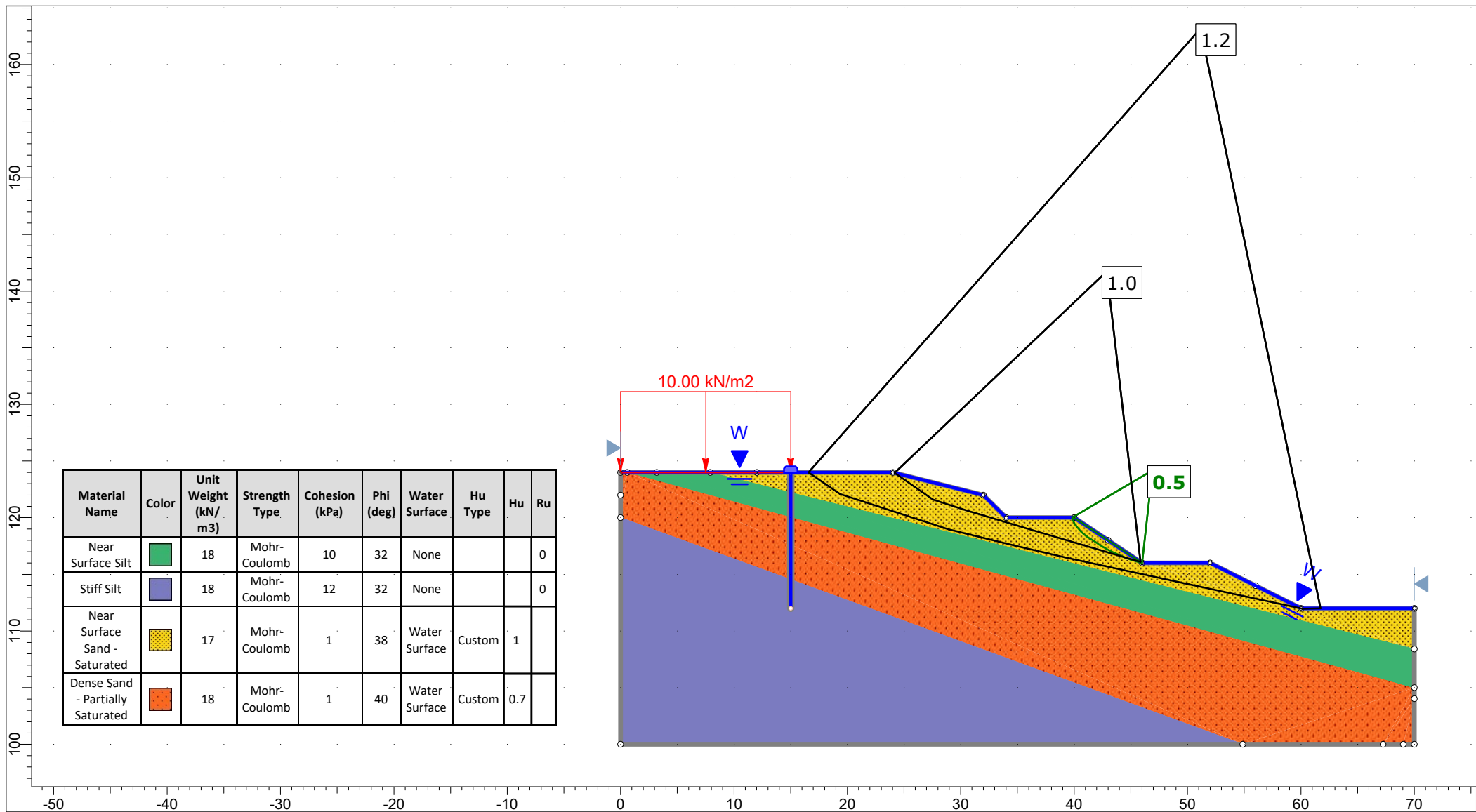
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SLIDEINTERPRET 9.023

Project

Lot 15, 220 Withy Road

Group

Slope B-B'

Scenario

Transient with support

Drawn By

WW

Company

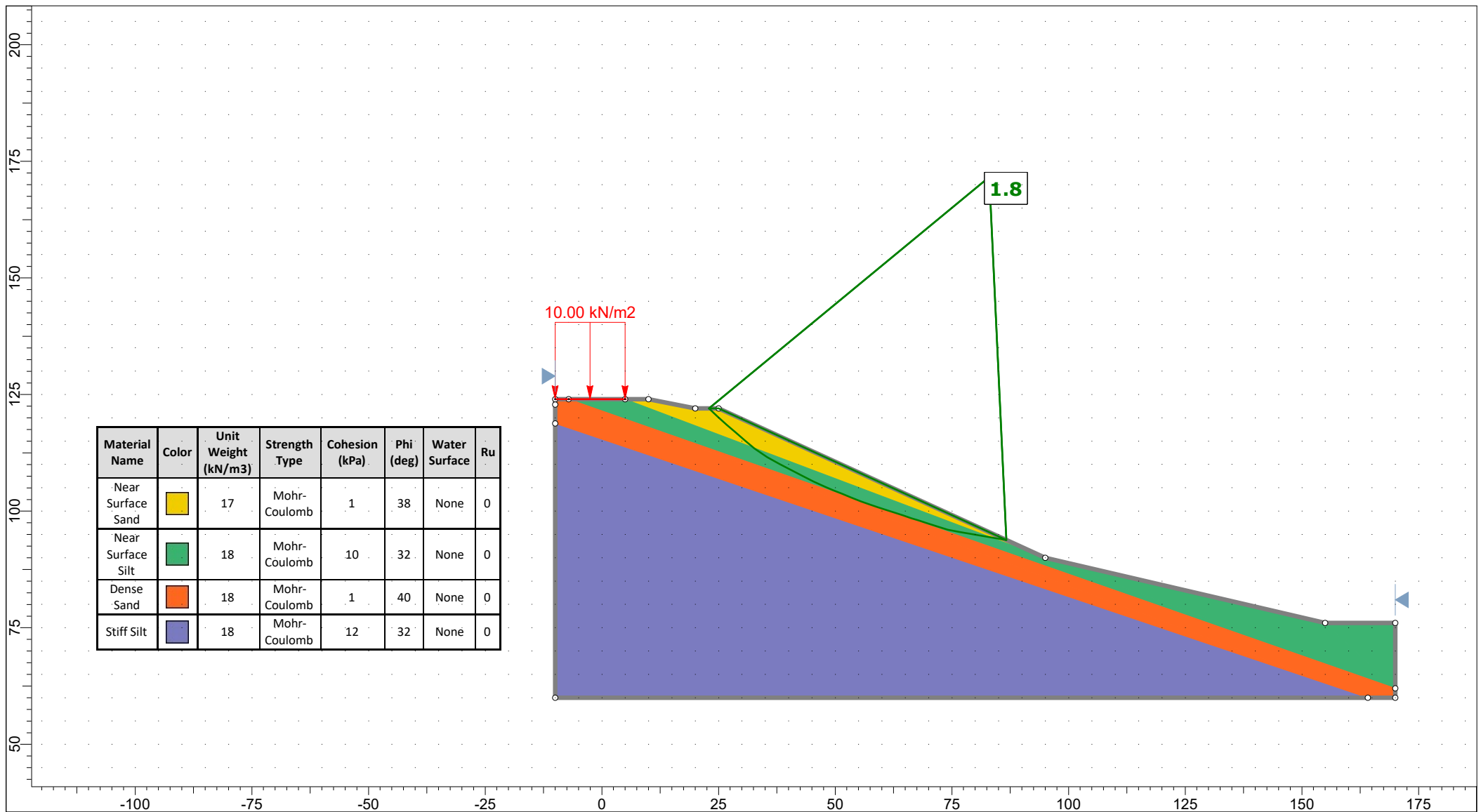
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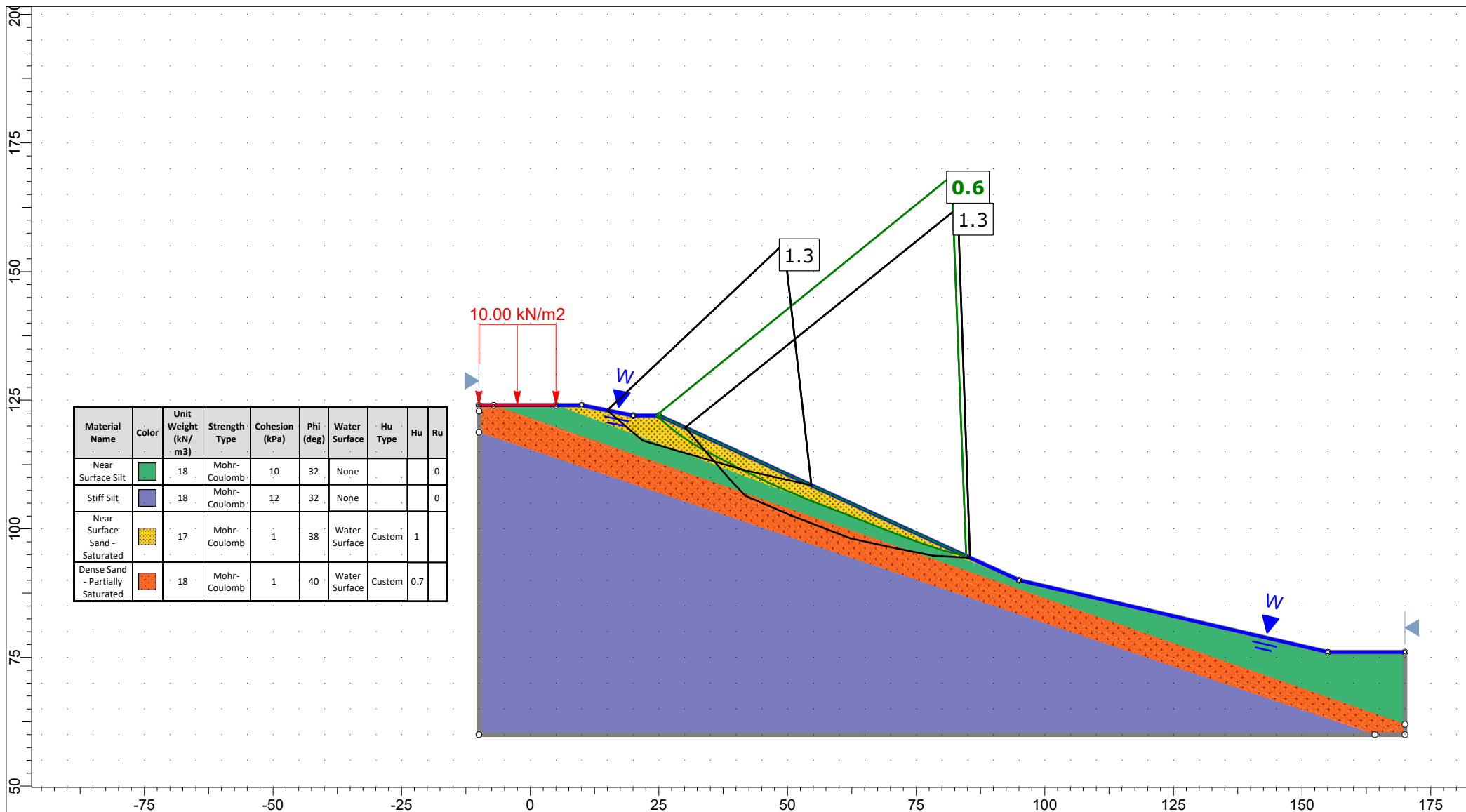
SLIDEINTERPRET 9.023

Project		Lot 15, 220 Withy Road	
Group	Slope C-C'		Scenario
Drawn By	WW		Company
Date	10/08/2022, 12:46:27 PM		File Name
			20220810_50721_Slope Stability Analysis.slmd

Normal

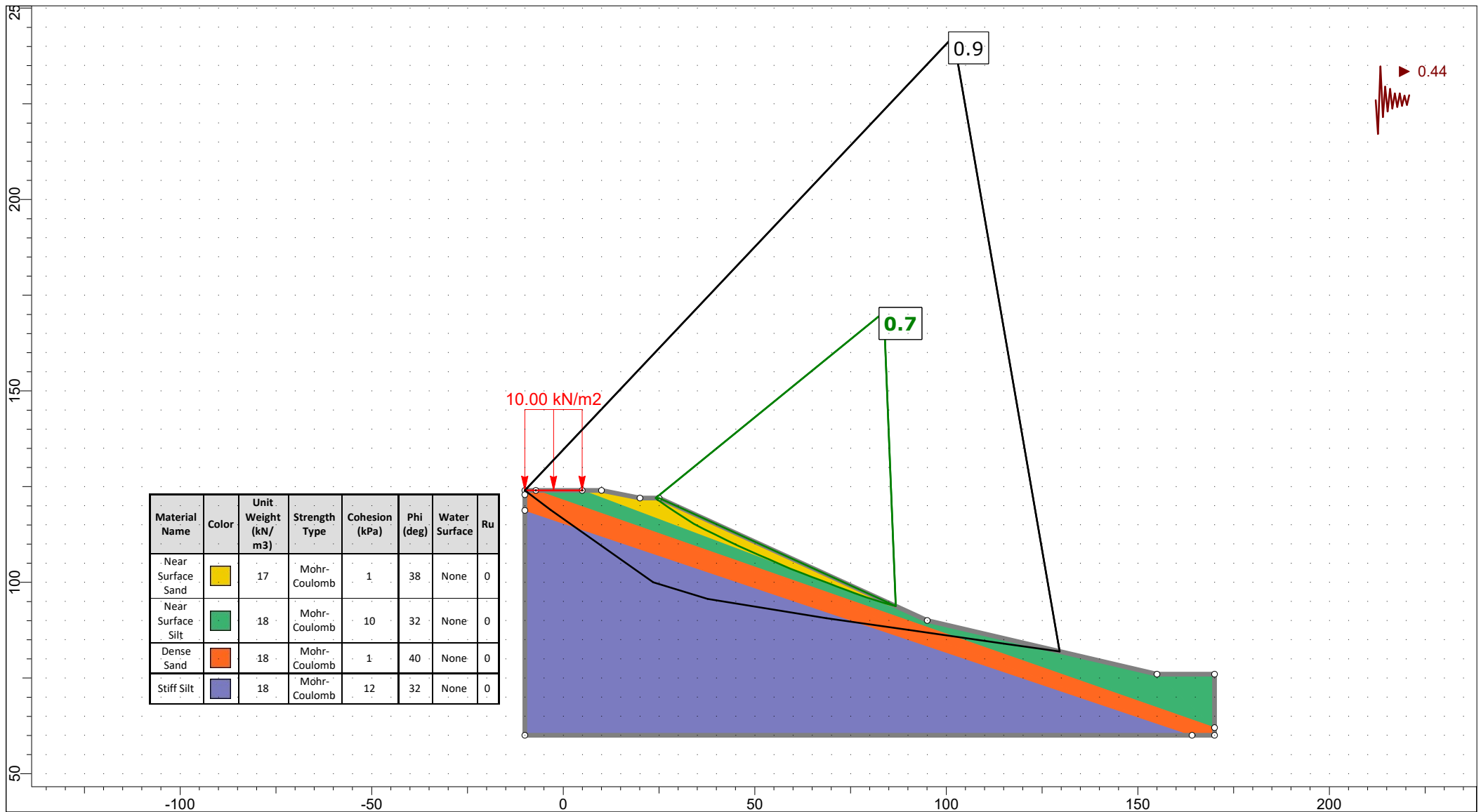
Engineering Design Consultants Ltd

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SLIDEINTERPRET 9.023

Project	Lot 15, 220 Withy Road		
Group	Slope C-C'	Scenario	Transient
Drawn By	WW	Company	Engineering Design Consultants Ltd
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SLIDEINTERPRET 9.023

Project

Lot 15, 220 Withy Road

Group

Slope C-C'

Scenario

Seismic

Drawn By

WW

Company

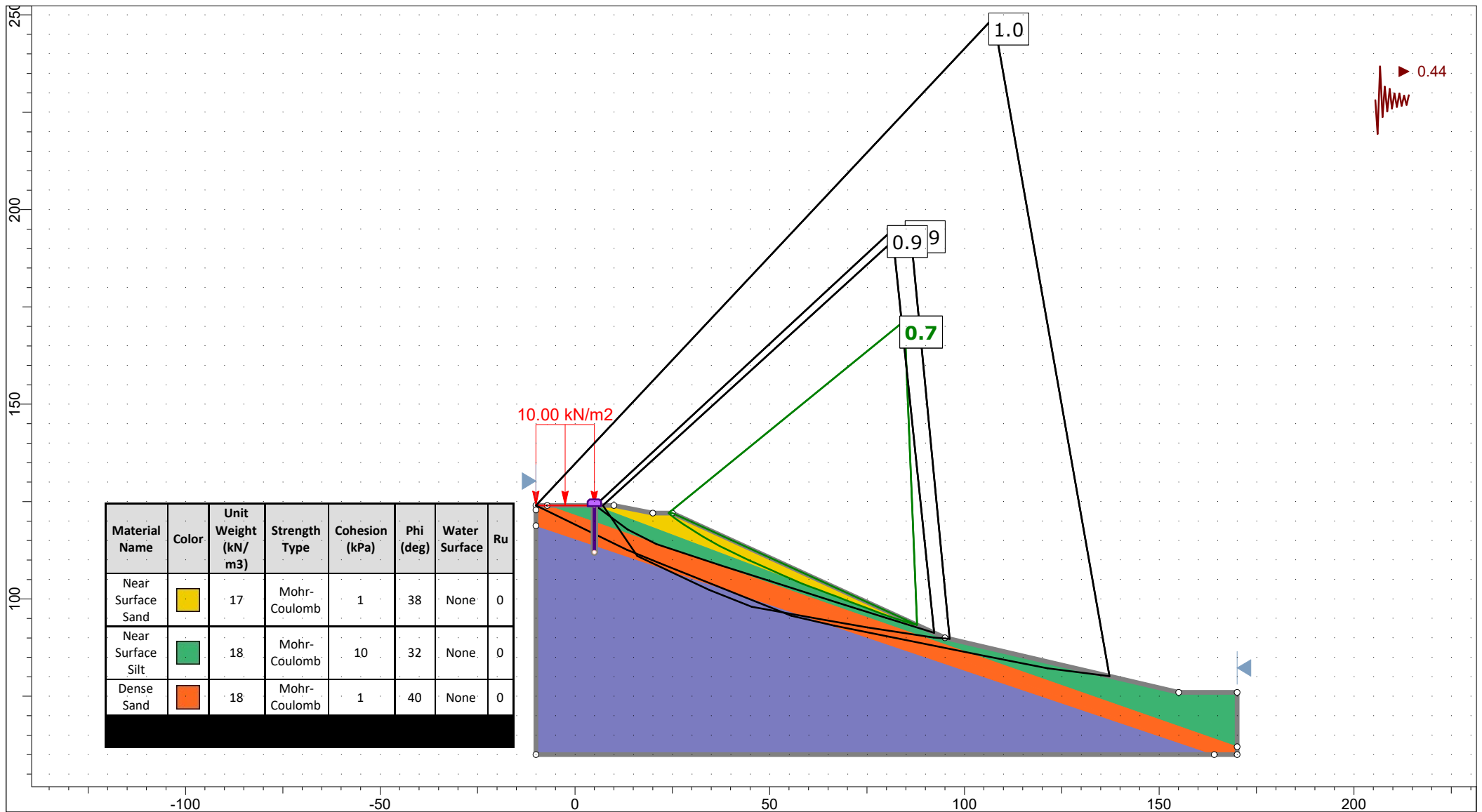
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File Name

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SLIDEINTERPRET 9.023

Project

Lot 15, 220 Withy Road

Group

Slope C-C'

Scenario

Seismic with support

Drawn By

WW

Company

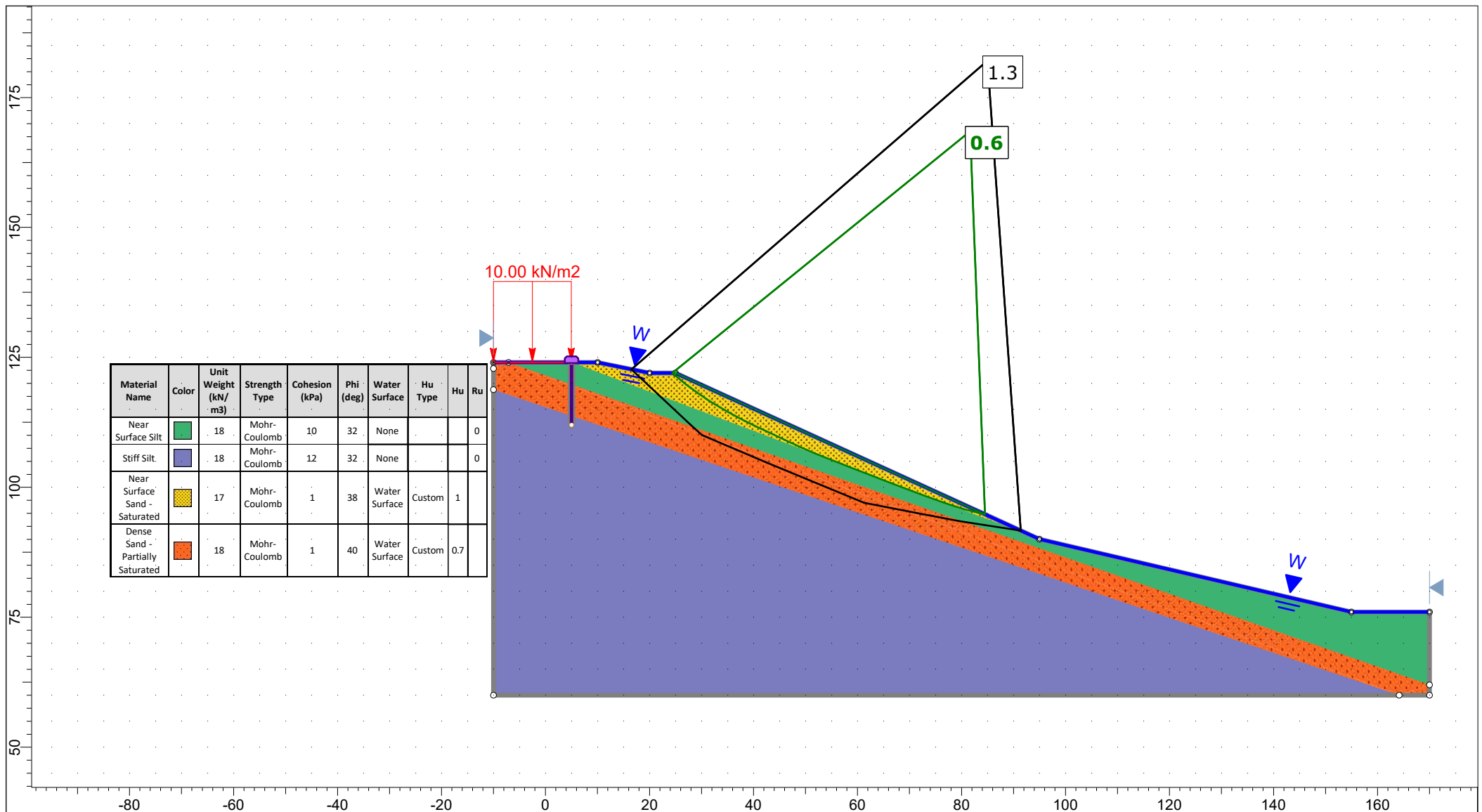
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SLIDEINTERPRET 9.023

Project	Lot 15, 220 Withy Road		
Group	Slope C-C'	Scenario	Transient with support
Drawn By	WW	Company	Engineering Design Consultants Ltd
Date	10/08/2022, 12:46:27 PM	File Name	20220810_50721_Slope Stability Analysis.slmd