

Our Ref: 711369 LE 20181212 JAC

13 December 2018

GWRL
P O Box 11646
Wellington

Attention: Richard Noakes

Dear Richard

Waterloo Subway Seismic Assessment

1 SUMMARY

Calibre has completed a seismic assessment of the subway structure at Waterloo Station. ***The seismic rating of the subway is at least 68% NBS (IL3), limited by the capacity of the walls adjacent to the ramped entrances to the subway.*** As we were able to obtain the original drawings and calculations for the structure, and carried out some scanning of as-built reinforcement, we have a high level of confidence in this rating.

2 BACKGROUND

The Waterloo Station Subway passes underneath the station building, the railway and the road. The station building itself comprises a lightweight roof supported on a steel spaceframe/truss system, in turn supported on reinforced concrete columns. As the building was designed in 1987, the IEP score is high (95% according to the Aurecon IEP Report supplied).

The building columns are founded immediately adjacent to the subway walls and may surcharge the subway structure.

The subway itself is rectangular in cross section and appears to be constructed from reinforced concrete walls and floor slab with a reinforced concrete lid. It is assumed that it was formed using a 'cut and cover' methodology.

Calibre has been engaged by GWRL to complete a seismic assessment of the subway structure.

3 METHODOLOGY

3.1 Information gathering

An extensive archive search was undertaken which eventually yielded an incomplete set of original plans and calculations for the subway and canopy structure.

A Geotechnical Desktop Study was undertaken by Coffey Geotechnics to provide confirmed site subsoil class, determine the likelihood for liquefaction and provide input parameters for the structural assessment of the walls.

Scanning of the walls was undertaken to confirm the as-built reinforcement. There was strong agreement between the scanning and the original drawings, giving us a high level of confidence in the accuracy of the original drawings.

3.2 % NBS Rating

Tunnels and subways are specifically excluded from consideration of the New Zealand Earthquake Loading code NZS1170.5, the usual reference for a seismic rating expressed as a % NBS.

We have therefore adopted the loading parameters from the NZTA Bridge Manual for traffic loading and the NZTA Railway Bridge Guidance for loads from the railway.

Element capacities are calculated using section C5 of the MBIE Technical Guidelines.

3.3 Analysis

The subway structure was checked at critical locations relating to the expected worst case combinations of load. The locations chosen were as follows:

- Under the train tracks
- Under the road
- Ramp walls
- Void walls (walls adjacent to voids in the station floor slab)

The following analysis parameters were used:

Parameter	Value	Comment
Importance Level	IL3	
Probable concrete strength, $f'_{c_{prob}}$	45 MPa	As per MBIE Tech Guide C5
Probable reinforcement strength, $f_{y_{prob}}$	275 MPa	As per MBIE Tech Guide C5
Site sub-soil class	D	From geotechnical desktop assessment
Ductility (ULS)	$\mu = 1.25$	
Ductility (SLS)	$\mu = 1$	

Refer to the geotechnical desktop study report for details of the geotechnical parameters used in the analysis.

We completed conservative load analyses of the building columns and applied a surcharge load to the top of the walls to take account of the effect of the building column loads on the walls. This is somewhat conservative.

2d analyses were completed at each location and the calculated capacities of the elements were compared to the demands at ULS to generate a % NBS rating.

4 RESULTS

4.1 Element Capacities

Set out below are the results for the elements considered:

Subway under train

Element	% NBS	Comment
Roof slab	>100%	
Walls	>100%	
Floor/foundation	>100%	

Subway under road

Element	% NBS	Comment
Roof slab	>100%	
Walls	>100%	
Floor/foundation	>100%	

Ramp wall

Element	% NBS	Comment
East wall	>100%	
West wall	>68%	Moment demand calculation likely conservative

Void wall

Element	% NBS	Comment
Void wall	>76%	Probably a higher rating if allowed to span in 2 directions.

4.2 Geotechnical Effects

The soils at this location are susceptible to liquefaction and hence we can expect some liquefaction induced settlement to be expressed at the surface in a significant earthquake.

The geotechnical desktop study (full text appended to this letter) concluded the following:

Earthquake Event	Liquefaction induced settlement
Serviceability, 25 year return period	Negligible
Ultimate Limit State, 1,000 year return period	30-50mm differential settlement
500 year return period	30-50mm differential settlement

5 DISCUSSION

5.1 General

The influence of the columns adjacent to the walls is less than originally feared as the piles extend to some depth. Our approach to considering the columns is somewhat conservative as all load is applied at the top of the wall.

As expected, the cantilever walls in the ramped sections of the structure where there is no restraint at the top of the walls are the limiting elements.

5.2 Seismic Rating

The seismic rating is 68% NBS (IL3). Most of the elements of the subway structure actually achieve a higher rating, many at 100% NBS.

5.3 Performance and expected damage

The following events are considered:

5.3.1 Serviceability Earthquake (25 year return period)

Based on the analysis completed, no structural damage is expected in a serviceability earthquake.

No significant liquefaction induced settlements are expected.

5.3.2 Ultimate Limit State (Design Level) Earthquake (1,000 year return period)

Based on the analysis completed, the subway will remain largely elastic during a ULS event. Some damage can be expected to the ramp walls as the reinforcement in the walls yields and cover concrete spalls or crushes. It may be that this damage is not readily visible as the tension reinforcement is on the outside face of the wall. It is unlikely that this will result in collapse of the wall.

Some liquefaction induced settlements are expected with the potential for differential settlements of up to 50mm.

5.3.3 500 year return period event

No significant structural damage is expected in this event.

Some liquefaction induced settlements are expected with the potential for differential settlements of up to 50mm.

In summary, the levels of service after earthquakes are expected to be acceptable based on the proposed levels of service in our proposal letter, refer to the table below.

Earthquake Event	Level of Service	Acceptable Damage
Serviceability EQ (25 year return period)	Subway is immediately useable by the public.	No significant damage. No movements or settlements that would cause a trip hazard (ie, less than 20mm steps).
Ultimate Limit State (design level) EQ (1,000 year return period)	Subway structure does not collapse.	Significant damage may occur.
500 year return period event (a design level earthquake for many Wellington buildings)	Subway structure does not collapse.	Some damage may occur.

	Immediately accessible to emergency services.	
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6 CONCLUSIONS

A seismic assessment of the Waterloo Station Subway has been completed. We were able to obtain the original design drawings and calculations, which have been passed on to you for your records.

The seismic rating of the subway is at least 68% NBS (IL3), limited by the capacity of the ramp walls. The assessment is somewhat conservative and many elements achieve >100% NBS ratings.

Some liquefaction induced settlement can be expected in a significant earthquake, but the proposed levels of service can still be achieved.

Yours sincerely
Calibre Consulting Ltd

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Structural Engineer

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ATTACHMENTS

1. Geotechnical Desktop Study
2. Reveal Infrastructure Scanning Report
3. Structural Calculations

Calibre Consulting Ltd
Waterloo Station and Subway Geotechnical Assessment
773-WLGGE224216

30 November 2018



Pour trust
into your
foundations
and you
can build
anything

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Waterloo Station and Subway Geotechnical Assessment

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AA

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Figure 1: Ground shaking hazard for Waterloo Station and Subway (from GWRC Web Map Viewer)

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1. Introduction

1.1. Project Brief

Calibre Consulting Ltd (Calibre) have engaged Coffey Services (NZ) Ltd (Coffey) to undertake a desktop study to assist with the seismic assessment of Waterloo Station, Lower Hutt including the pedestrian subway. As part of this work Coffey will review the available information and provide two ground models with geotechnical parameters; one for the subway structure and one for the Waterloo station.

1.2. Site Description

Waterloo Station is located in central Lower Hutt, Wellington. The station is at the intersection of Oxford Terrace, Knights Road and Pohutukawa Street to the north west and on Cambridge Terrace to the south east. Waterloo Station is surrounded by roads, vehicle parking and Waterloo Water Treatment Plant to the north. The site is near flat. The Hutt River is 1.6km north west of the site and the eastern hills 1km away.

The subway runs from within the northern side of Waterloo Station and terminates at Trafalgar Square. It runs beneath the train tracks and Cambridge Terrace. The subway is ~2-3m below ground level.

The station is ~1m above road level with steps up to the building on the northern side. Along the south eastern boundary between a Waterloo Station carpark and Cambridge Terrace is a concrete crib retaining wall up to ~1m high, decreasing in height to the north and south. The neighbouring Waterloo Treatment Plant is at road level.

2. Reviewed Information

2.1. Received Information

We have received the following information from Calibre:

- "Waterloo Water Treatment Plant - Liquefaction Analysis Review" GHD, dated July 2018
- "Waterloo Water Treatment Plant – Liquefaction Mitigation Concept Design, GHD, dated July 2018

This information for the Waterloo Water Treatment Plant (WWTP) includes a simplified ground model as well as the interpretation of this for liquefaction assessment. The assessment in this GHD report was based on the site investigation conducted by Opus in 2016. A summary of the Opus investigation and liquefaction assessment results was included in the GHD report. The Opus report and associated logs and testing data was not included in the provided report. The locations of the testing data and the simplified ground model provided in the GHD report are included in appendices A and B, respectively.

We have also received the below table outlining the performance criteria recommended by Calibre.

Table 1: Performance Criteria for Waterloo Subway (provided by Calibre)

Design Case*	Earthquake Event	Level of Service	Acceptable Damage
SLS	Serviceability EQ (25 year return period)	Subway is immediately useable by the public.	No significant damage. No movements or settlements that would cause a trip hazard (i.e., less than 20mm steps).
ULS	Ultimate Limit State (design level) EQ (1,000 year return period)	Subway structure does not collapse.	Significant damage may occur.
SLS₍₅₀₀₎	500 year return period event (a design level earthquake for many Wellington buildings)	Subway structure does not collapse. Immediately accessible to emergency services.	Some damage may occur.

* design case reference added by Coffey for reference throughout this report

2.2. Greater Wellington Regional Council – Hazards/Risks

The site is not within the GWRC flood hazard area from the Hutt Valley Floodplain Management Plan. However, the mapped boundary of the 0.23% AEP (Annual Exceedance Probability) is immediately adjacent to Waterloo Station on the north western side.

The Greater Wellington Regional Council (GWRC) GIS Viewer maps show the earthquake related hazards for the Wellington Region. Table 2 below show the risk levels at this site.

Table 2: Earthquake related hazards at Waterloo Subway, Lower Hutt

Earthquake Related Hazards	Risk Level
Combined Hazard	Moderate
Ground Shaking	Low-moderate (east) and moderate-high (west) – refer to Figure 1 below
Liquefaction	Variable
Slope Failure	Low
Tsunami Evacuation Zone	None

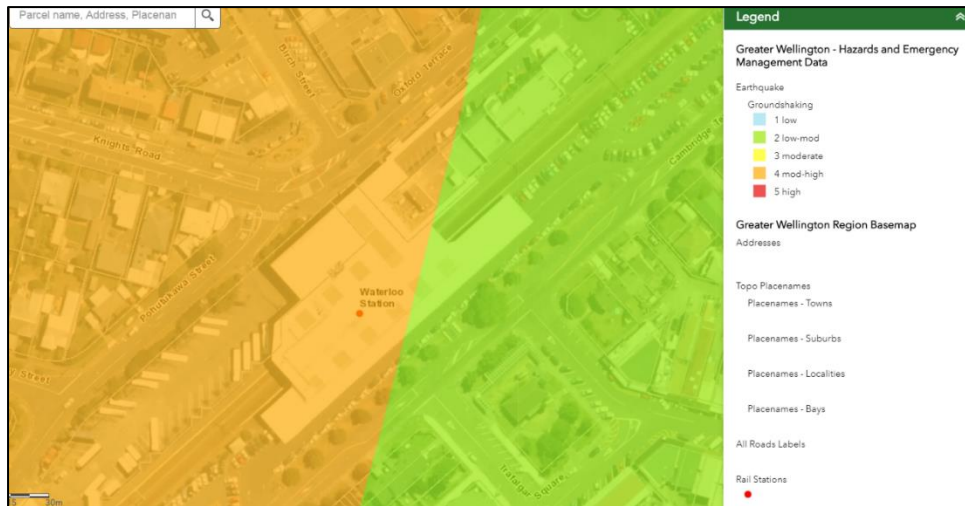


Figure 1: Ground shaking hazard for Waterloo Station and Subway (from GWRC Web Map Viewer)

2.3. HCC Archives

Calibre completed a search of the Hutt City Council (HCC) archives. We understand that no relevant records relating to the station or subway were available.

2.4. Geology

The geology of the site from the Geology of Wellington Area 1:50 000 Map is “Alluvium (fa), including Taita Alluvium” (recent alluvium).

From the GNS Active Fault Database, the nearest fault is the Wellington Fault which is mapped 1.7km to the north west.

2.5. NZGD

A search of the New Zealand Geotechnical Database (NZGD) returned three boreholes within 100m of Waterloo Station. These logs are from GWRC water bores. The ground conditions encountered in these are summarised below. The location of these boreholes is shown on the site location plan (Figure A-1) in Appendix A.

Borelogs R27/6926 & R27/6927 ~100m south west of Waterloo Station:

- 0 - 1m: Silty clay
- 1 - 4m: Sandy silt clay

Borelog R27/1176 ~80m north of Waterloo Station

- 0 - 1.9m: silt and clay
- 1.9 - 3.9m: sandy gravel
- 3.9 - 7.2m: sand and silty clay
- 13.5 – 19.2 gravel and clay (assumed to be the aquiclude)
- 19.2 – 32m: gravel, water bearing (assumed to be the aquifer)
- 32 - 65.4m: clay and gravel interbedded

The borelogs have been included in Appendix B for reference.

3. Ground Model

We have provided two ground models based on the available information about the ground conditions at and near the site.

The site is underlain by variable alluvial soils comprising sand, sandy gravel, silt and sandy silt. The aquiclude is anticipated at ~12 to 13m depth beneath which is the aquifer unit. The aquiclude is anticipated to be a firm silty clay based on our experience in the Hutt Valley.

It is assumed that Waterloo Station has been built up on engineered fill, as this is ~1m higher than the surrounding area. There is not enough nearby information to otherwise distinguish the natural ground profile between the station and the subway. The northern section of the subway that runs beneath the station (north of Cambridge Terrace) anticipated to be beneath the filled area. The area south of this is anticipated to be straight into natural ground. It is assumed that where the ground level is at road level that the area is on or very near natural ground.

The ground model is primarily based off the information from WWTP and in particular the closest information relating to the south-western section of the WWTP.

The Opus site investigation (referred to in the provided GHD report) included hydro-excavation of the upper 2.5 – 3.0m below ground level (bgl) of each of the investigation points, so there is limited data for this upper section of the ground profile. The Opus report noted that the hydro-excavated material was fill comprising lightly packed gravel. The site of the WWTP and Waterloo Station area and surrounds is near flat, so it would not be expected that this entire area is build up 2.5-3.0m. It is therefore assumed that there was localised over-excavation of the WWTP. Therefore, we look at the next closest available data which indicate silt and clay in the upper profile. Tables 3 and 4 are the conceptual ground models at Waterloo Station and Subway – south end respectively.

Table 3: Concept ground model at Waterloo Station

Layer No.	Depth (m)	Unit	Description	Consistency
0	0-1	Fill	Sandy/Silty GRAVEL	Dense
1	1-3	Alluvium	SILT and CLAY	Unknown – assume firm to stiff
2	3-8	Alluvium	SAND with interbedded SILT, sandy SILT and sandy GRAVEL	Loose to medium dense/ stiff
3	8-13	Alluvium	Sandy GRAVEL*	Dense
4	13	Aquiclude	Silty CLAY	Firm

Table 4: Concept ground model at Waterloo Station Subway – south end

Layer No.	Depth (m)	Unit	Description	Consistency
1	0-2	Alluvium	SILT and CLAY	Unknown – assume firm to stiff
2	2-7	Alluvium	SAND with interbedded SILT, sandy SILT and sandy GRAVEL	Loose to medium dense/ Stiff
3	7-12	Alluvium	Sandy GRAVEL*	Dense
4	12	Aquiclude	Silty CLAY	Firm

*the sandy gravel unit may be shallower than this at the site as it appears to be dipping down beneath the WWTP site.

The groundwater level at the site was measured by Opus in 2017 was 2.4-2.6m bgl after a dry period. The design groundwater level used by GHD was 2m. We recommend a design groundwater level of 2m is used.

Table 5 below presents the preliminary geotechnical parameters for this model. These are based on the received information discussed above and our knowledge of the area.

Table 5: Geotechnical Ground Model

Unit No.	Unit	Unit Weight (γ)	Friction angle (ϕ')	Cohesion (c')	Shear strength (C_u)	Youngs Modulus (E'_v)	Youngs Modulus (E'_h)
		kN/m ³	°	kPa	kPa	MPa	MPa
0	Fill	20	36	0	-	50	35
1	Silt/Clay	17	28	5	50	7	5
2	Sand with interbedded silt and gravel	18	30	1	-	20	14
3	Sandy gravel	20	36	0	-	50	35
4	Silty clay	19	27	5	40	6	4

Due to the limited information, conservative values have been used. Access to the field data, rather than the limited description of units and the interpretation, would enable these parameters to be further refined. Further, site specific data relating to the Waterloo Station and subway site, especially the south western section would enable the ground model and associated parameters to be refined. This is discussed further in Section 5 below.

4. Geotechnical Assessment

4.1. Subsoil Classification

The site is subsoil class D – Deep soil site based on the developed ground model and “1170.5:2004 Site Subsoil Classification of Lower Hutt” (Boon, Perrin, Dellow, Van Dissen, & Lukovic, 2011, p. 5).

Based on the “1170.5:2004 Site Subsoil Classification of Lower Hutt” site subsoil class map, Waterloo Station is at the edge of an area mapped as may contain localised subsoil class E sites.

4.2. Site Seismic Response

We understand that the station and subway are Importance Level (IL) 3 structures and have used this to estimate the horizontal ground accelerations. The ground motion parameters have been assessed utilising Guidelines from NZGS “Earthquake Geotechnical Practice: Module 3” (NZGS, 2016).

The following parameters are suitable for the assessment of peak ground accelerations and are based on un-weighted peak ground acceleration coefficients and effective earthquake magnitudes. The method utilised is Method 1 (MBIE and NSGS, 2016) which refers to the New Zealand Transport Agency (NZTA) Bridge Manual (NZTA, 2016). Table 6 below presents the ground motion parameters.

Table 6: Ground Motion Parameters

Design Life	Design Case	IL	Annual Probability ¹	C _{0,1000} ²	M _{eff} ³	R _u or R _s ⁴	f ⁵	PGA
50 years	SLS	3	1/25	0.45	6.2	0.25	1.0	0.09
	SLS ₅₀₀	3	1/500	0.45	7.1	1.0	1.0	0.35
	ULS	3	1/1000	0.45	7.1	1.3	1.0	0.45

¹ – Table 3.3 (Standards New Zealand, 2002)

² – Unweighted peak ground acceleration coefficients

³ – Effective Magnitudes - Table 6A.1 (NZTA, May 2016)

⁴ – Return period factor

⁵ – Site subsoil class factor

4.2.1. Ground Motion during previous earthquakes

Table 7 below provides a summary of the ground motions experienced at select Lower Hutt sites during the Kaikoura (2016) Earthquakes (NZ Earthquake Clearing House, 2016).

Table 7: Ground motions during Kaikoura earthquakes

Site	Location	Peak Ground Acceleration (Kaikoura Nov 2016)	Site Subsoil Class	Distance from Waterloo Station
SOCS	Lower Hutt St Orans College	0.173g	Class D	1.2km north west
LHES	Lower Hutt Emergency Management Office	0.168g	Class D	1.5km west

These recorded PGA values are greater than those calculated for a SLS design case. It is therefore assumed that the site has been tested to a SLS case.

4.3. Bearing Capacity

At this stage, the details of the foundations and subsurface structures are unknown. Bearing capacity for the structures can be provided based on this assumed ground model once details of the foundations of interest are provided.

If shallow foundations are within the upper fill layer at the station area, then a geotechnical ultimate bearing capacity greater than 300kPa will be available.

4.4. Liquefaction Potential and Effects

The nearest information about the ground conditions are from the GHD report for the WWTP site. GHD conducted a liquefaction analysis and review based on the earlier Opus ground investigation and liquefaction assessment for the WWTP site. Neither the field data or lab testing data from the ground investigation was included in this report. We must therefore rely on the summarised ground model developed in the report and the liquefaction assessments that have been completed for the WWTP site. The PGA values used for the WWTP are summarised in Table 8 below.

Table 8: PGA values used in liquefaction assessment for WWTP

Design Case	Opus (2016)	GHD (2018)
ULS ₍₂₅₀₀₎	0.8g	0.62g
SLS2 ₍₁₀₀₀₎	0.58g	0.45g

The PGA values used for the WWTP, an IL4 structure, are higher than those for the Waterloo Station, an IL3 structure.

The ULS case for our assessment at Waterloo Station is equivalent to the SLS2 case for WWTP by GHD (both with a return period of 1/1000 years). The SLS and SLS₅₀₀ cases for Waterloo Station both have lower than PGA values used in the liquefaction assessment for WWTP. The results of these liquefaction analyses are therefore summarised here and the applicability to the Waterloo Station site discussed.

For the south-western section of the WWTP, the GHD (2018) report states:

- Potentially liquefiable layers between 3.5m to 7 or 9.5m bgl, corresponding to the loose to medium dense sand layer based on the borehole data (BH1, BH2 & BH3).
- Potentially liquefiable layers between 3.6 and 6.8m bgl based on CPT9.
- Free field settlements of 60-100mm.

The Opus (2016) liquefaction analysis as summarised by GHD for the corresponding area ("southwest end") indicated that:

- Potentially liquefiable units between 2.5 to 4.5m; 6.0 to 6.5m and 7.5 to 8.0m
- Free field settlements of 100-150mm
- These results were based on boreholes BH1 and BH2.

Both these assessments found that the results for the ULS and SLS2 cases were similar.

At the Waterloo Station site, it is anticipated that liquefaction at the SLS case is negligible as no liquefaction was noted during the Kaikoura Earthquake (2016). The liquefaction risk during a SLS₅₀₀ and ULS case is considered high based on the above results and assumed ground model. The loose to medium dense sand unit likely has high liquefaction potential. The discrete units of sandy silt and silt within the interbedded unit are also at high risk of liquefying. The thickness and strength of these units will be a significant factor in the overall liquefaction risk at the site.

If the sandy gravel unit grades up to the south as is the general trend beneath the WWTP, then the liquefaction potential and free field settlement will likely be lower than the above calculated.

Based on the available information, it is anticipated that total free-field settlement at the Waterloo Station and subway site is on the order of 60-100mm. Differential settlement is likely ~50% of the total (~30-50mm).

5. Recommended Further Work

This assessment is based on nearby geotechnical data and no data was available specifically for this site. Alluvial soils are typically variable and review of the available data supports that this is the case in this area. The ground conditions may therefore vary considerably over the ~130m distance of the station and subway. There is little information about the upper 2.5 to 3 metres of the profile, a depth range which is likely critical for the assessment of the foundations.

To further refine the ground model, a site specific geotechnical investigation would be required. This may involve 1 to 2 boreholes to 10-12 metres depth with Standard Penetration Tests (SPTs) at 1m centres to allow for an SPT based quantitative liquefaction assessment. If it is determined that site specific information is required, this site investigation would be developed to target the information required.

Reviewing the site investigation and testing data from the Opus (2016) report would enable the geotechnical parameters for the site to be refined. In particular, the raw data would enable us to conduct a liquefaction assessment with the parameters appropriate for this site.

If the subsoil conditions are critical in the seismic assessment or modifications to the foundations etc are required, we recommend an intrusive geotechnical investigation at the site to confirm the extrapolated ground model in this report.

6. Conclusions

This desktop study is based on nearby geotechnical data and assessments (predominantly from the neighbouring Waterloo Water Treatment Plant) and is indicative only. The available data also indicates that the alluvial soils are variable. Therefore, if the subsoil conditions are critical in the seismic assessment or modifications to the foundations etc are required, we recommend an intrusive geotechnical investigation at the site to confirm the extrapolated ground model in this report

The assessment based on the available nearby information, the following conclusions can be made:

- The site is underlain by variable alluvial soils, and the station is likely located on engineered fill overlying variable alluvial soils. The southern section of the subway (outside the station) is likely in natural alluvial soils.
- Groundwater level is anticipated at 2.0-2.5m bgl, with 2.0m recommended as the design level.
- The site is site subsoil class D.
- Liquefaction induced free field settlement is estimated to be on the order of 60-100mm under SLS₅₀₀ and ULS cases, with differential settlements potentially on the order of 30-50mm.
- Liquefaction under SLS case is assumed to be negligible.
- Bearing capacity for the structures can be provided based on this assumed ground model once details of the foundations of interest are provided.

7. Limitations

This report has been prepared solely for the use of our client, Calibre Consulting Ltd, their professional advisers and the relevant Territorial Authorities in relation to the specific project described herein. No liability is accepted in respect of its use for any other purpose or by any other person or entity. All future owners of this property should seek professional geotechnical advice to satisfy themselves as to its ongoing suitability for their intended use.

Please also refer to the enclosed *Important Information about Your Coffey Report*.

Important information about your Coffey Report

As a client of Coffey you should know that site subsurface conditions cause more construction problems than any other factor. These notes have been prepared by Coffey to help you interpret and understand the limitations of your report.

Your report is based on project specific criteria

Your report has been developed on the basis of your unique project specific requirements as understood by Coffey and applies only to the site investigated. Project criteria typically include the general nature of the project; its size and configuration; the location of any structures on the site; other site improvements; the presence of underground utilities; and the additional risk imposed by scope-of-service limitations imposed by the client. Your report should not be used if there are any changes to the project without first asking Coffey to assess how factors that changed subsequent to the date of the report affect the report's recommendations. Coffey cannot accept responsibility for problems that may occur due to changed factors if they are not consulted.

Subsurface conditions can change

Subsurface conditions are created by natural processes and the activity of man. For example, water levels can vary with time, fill may be placed on a site and pollutants may migrate with time. Because a report is based on conditions which existed at the time of subsurface exploration, decisions should not be based on a report whose adequacy may have been affected by time. Consult Coffey to be advised how time may have impacted on the project.

Interpretation of factual data

Site assessment identifies actual subsurface conditions only at those points where samples are taken and when they are taken. Data derived from literature and external data source review, sampling and subsequent laboratory testing are interpreted by geologists, engineers or scientists to provide an opinion about overall site conditions, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist, because no professional, no matter how qualified, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. For this reason, owners should retain the services of Coffey through the development stage, to identify variances, conduct additional tests if required, and recommend solutions to problems encountered on site.

Your report will only give preliminary recommendations

Your report is based on the assumption that the site conditions as revealed through selective point sampling are indicative of actual conditions throughout an area. This assumption cannot be substantiated until project implementation has commenced and therefore your report recommendations can only be regarded as preliminary. Only Coffey, who prepared the report, is fully familiar with the background information needed to assess whether or not the report's recommendations are valid and whether or not changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report there is a risk that the report will be misinterpreted and Coffey cannot be held responsible for such misinterpretation.

Your report is prepared for specific purposes and persons

To avoid misuse of the information contained in your report it is recommended that you confer with Coffey before passing your report on to another party who may not be familiar with the background and the purpose of the report. Your report should not be applied to any project other than that originally specified at the time the report was issued.

Interpretation by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a report. To help avoid misinterpretations, retain Coffey to work with other project design professionals who are affected by the report. Have Coffey explain the report implications to design professionals affected by them and then review plans and specifications produced to see how they incorporate the report findings.

Data should not be separated from the report

The report as a whole presents the findings of the site assessment and the report should not be copied in part or altered in any way. Logs, figures, drawings, etc. are customarily included in our reports and are developed by scientists, engineers or geologists based on their interpretation of field logs (assembled by field personnel) and laboratory evaluation of field samples. These logs etc. should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

Geoenvironmental concerns are not at issue

Your report is not likely to relate any findings, conclusions, or recommendations about the potential for hazardous materials existing at the site unless specifically required to do so by the client. Specialist equipment, techniques, and personnel are used to perform a geoenvironmental assessment. Contamination can create major health, safety and environmental risks. If you have no information about the potential for your site to be contaminated or create an environmental hazard, you are advised to contact Coffey for information relating to geoenvironmental issues.

Rely on Coffey for additional assistance

Coffey is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction. It is common that not all approaches will be necessarily dealt with in your site assessment report due to concepts proposed at that time. As the project progresses through design towards construction, speak with Coffey to develop alternative approaches to problems that may be of genuine benefit both in time and cost.

Responsibility

Reporting relies on interpretation of factual information based on judgement and opinion and has a level of uncertainty attached to it, which is far less exact than the design disciplines. This has often resulted in claims being lodged against consultants, which are unfounded. To help prevent this problem, a number of clauses have been developed for use in contracts, reports and other documents. Responsibility clauses do not transfer appropriate liabilities from Coffey to other parties but are included to identify where Coffey's responsibilities begin and end. Their use is intended to help all parties involved to recognise their individual responsibilities. Read all documents from Coffey closely and do not hesitate to ask any questions you may have.

Appendix A - Figures



Legend

WWTP Borehole Locations

NZGD Borehole Locations

WWTP CPT Locations



CLIENT:
Calibre Consulting NZ Ltd
PROJECT TITLE:
Waterloo Station and Subway

PROJECT: 773-WLGGE224216
DATE: 28/11/2018
REVISION: AA
SCALE: NTS

DESIGNED: SM
DRAWN: SM
CHECKED: KWH
STATUS: Final

FIGURE TITLE: Site Location Plan

FIGURE NO: A-1

NOTES: Aerial from HCC

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Appendix B – Available Site Investigation Data

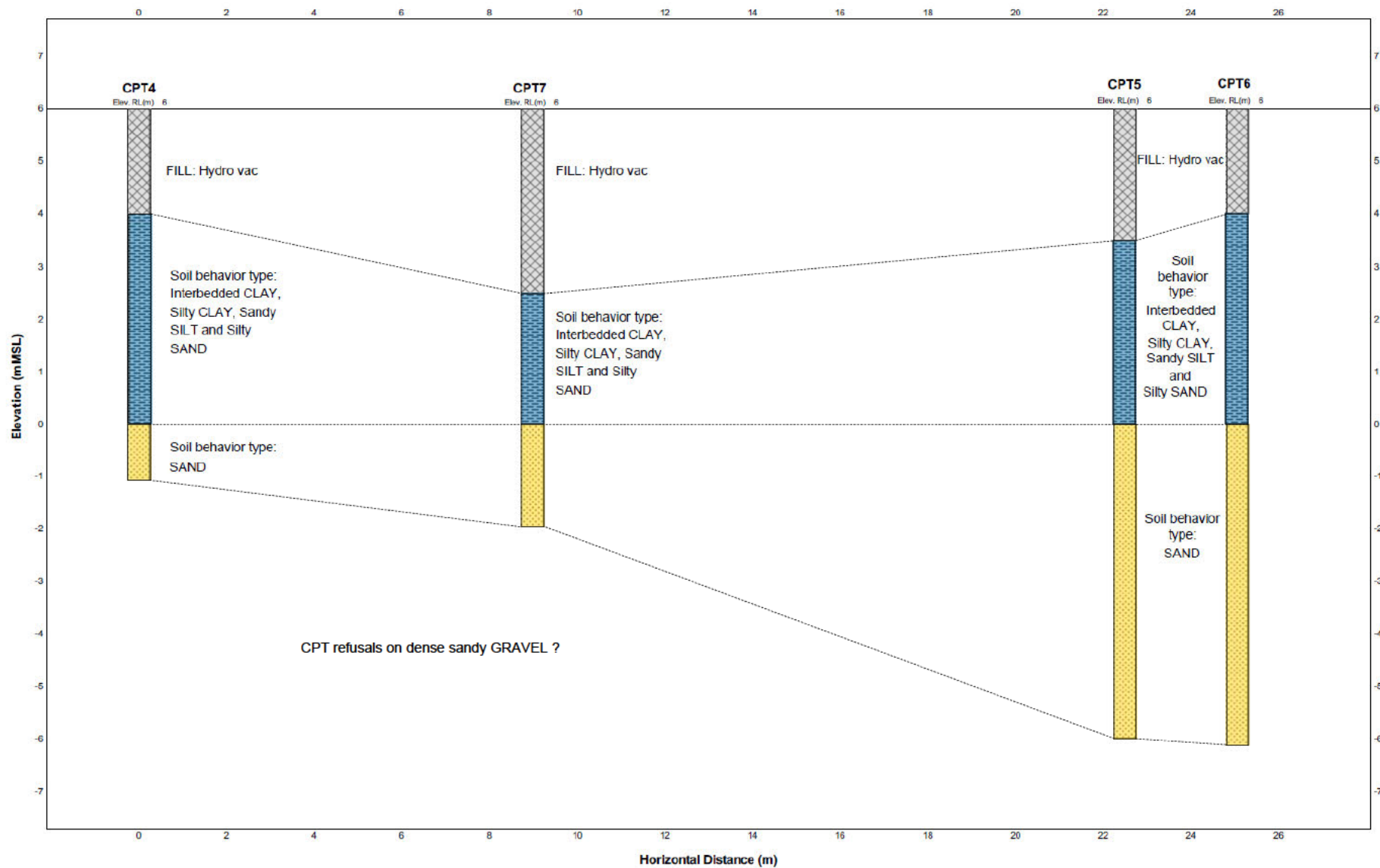


Figure 6 North-eastern Cross Section (Control Room, Pump and Motor Hall)

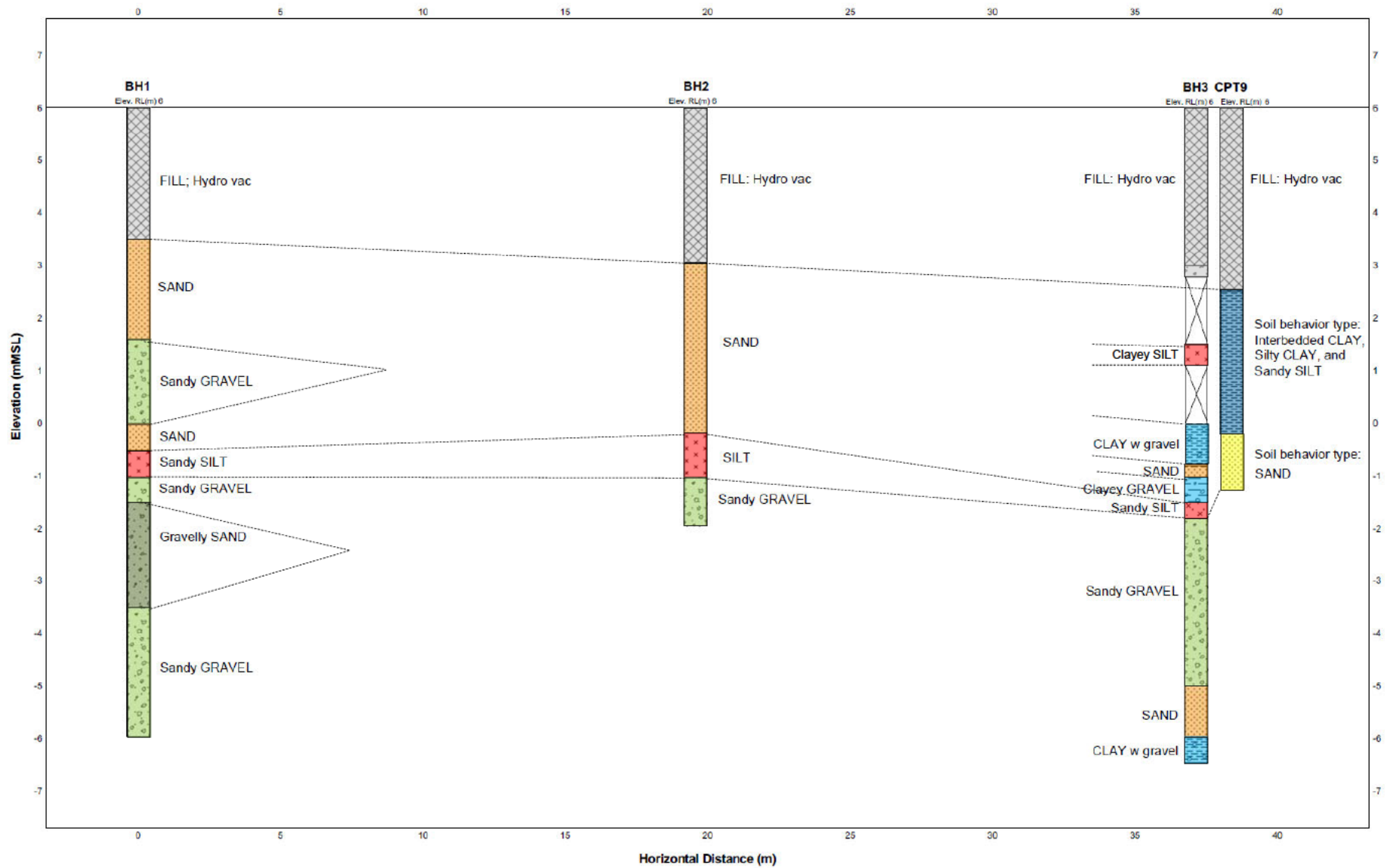


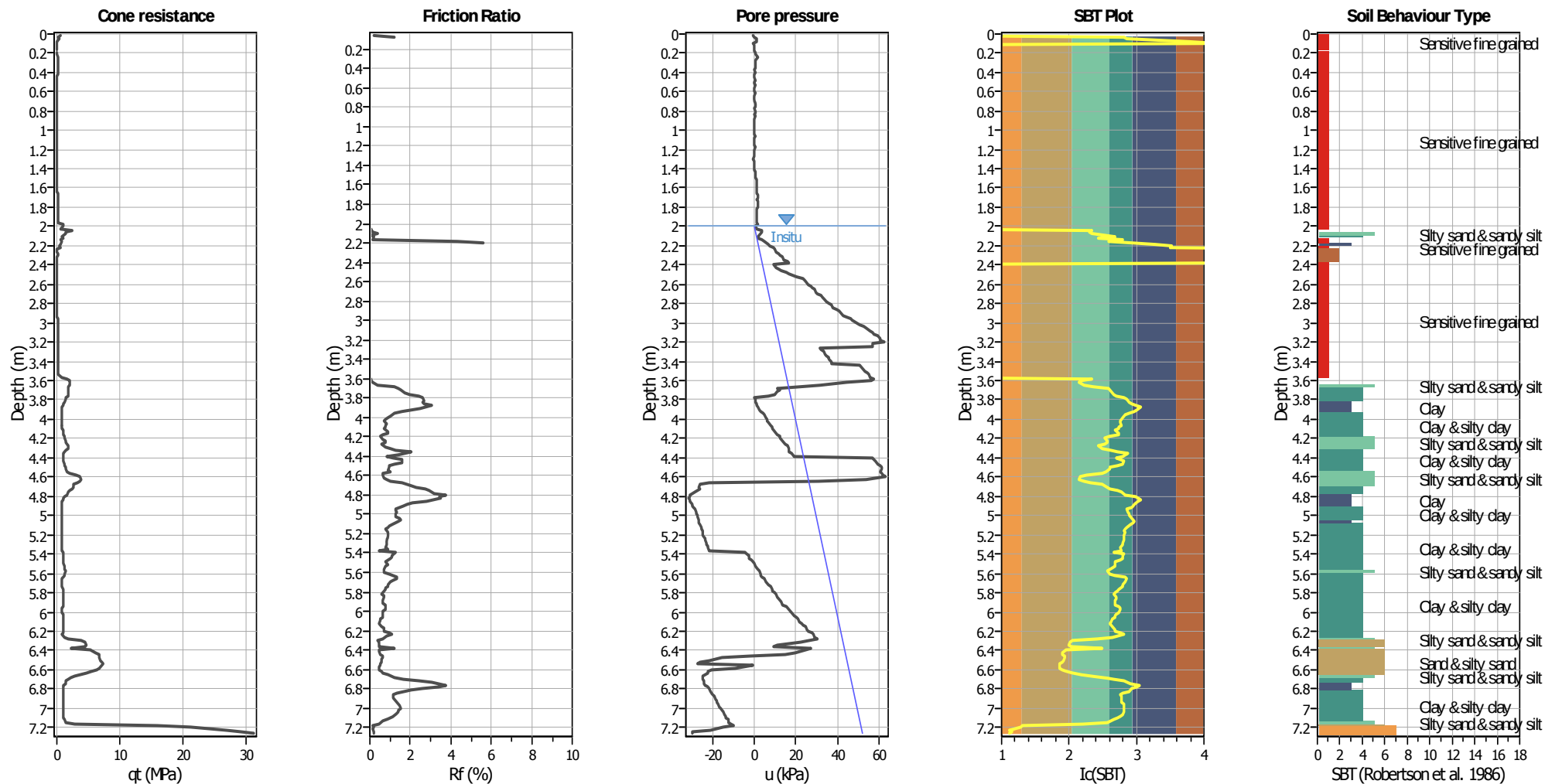
Figure 7 South-western Cross Section (Reservoir)

Project:

Location:

CPT: WWTP CPT9 SLS2

Total depth: 7.26 m



Analysis method: B&I (2014)
Fines correction method: B&I (2014)
Points to test: Based on Ic value
Earthquake magnitude M_w : 7.10
Peak ground acceleration: 0.45

G.W.T. (in-situ): 2.00 m
G.W.T. (earthq.): 2.00 m
Average results interval: 1
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT

Use fill: No
Fill height: N/A
Fill weight: N/A
Trans. detect. applied: No
 K_σ applied: .

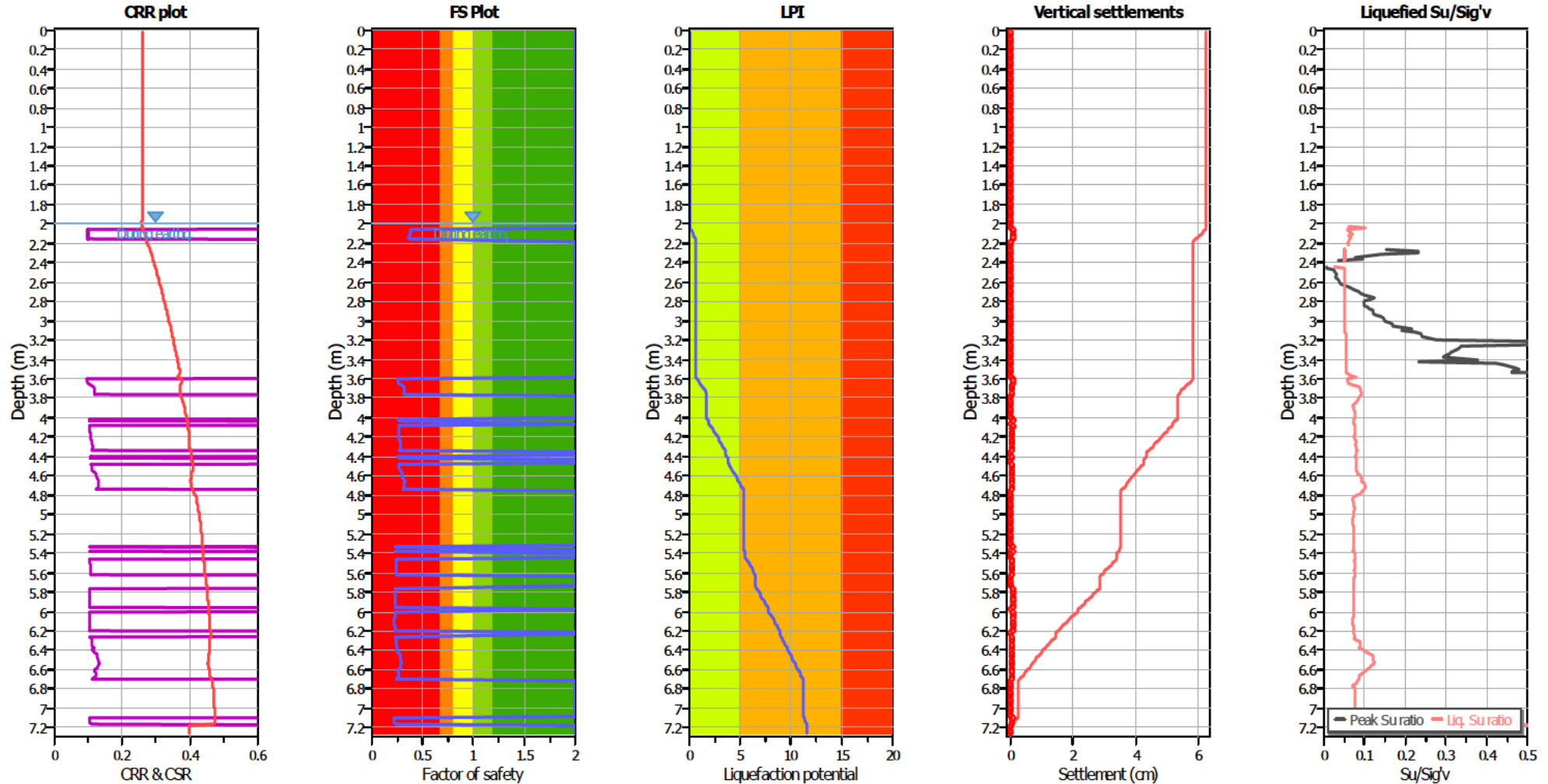
Clay like behavior applied: .
Limit depth applied: No
Limit depth: N/A
MSF method: Method based

Project:

Location:

CPT: WWTP CPT9 SLS2

Total depth: 7.26 m



Analysis method: B&I (2014)
Fines correction method: B&I (2014)
Points to test: Based on Ic value
Earthquake magnitude M_w : 7.10
Peak ground acceleration: 0.45
G.W.T. (in-situ): 2.00 m
G.W.T. (earthq.): 2.00 m
Average results interval: 1
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT

Use fill: No
Fill height: N/A
Fill weight: N/A
Trans. detect. applied: No
 K_σ applied: .
Clay like behavior applied: .
Limit depth applied: No
Limit depth: N/A
MSF method: Method based

Borelog for well R27/1176

Gridref: 1760989.5435796

Ground Level Altitude 6.30 +MSD

Driller : RICHARDSON DRILLING COMPANY LTD

Drill Method :

Drill Depth : m Drill Date : 1/01/1975 12:00:00 a.m.

Page

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Scale	Depth	Drillers Description	Formation
	-0.80m	Topsoil	1
	-1.90m	Brown silt and clay	1
	-3.90m	Blue brown metal and sand. Water bearing	1
	-7.00m -7.20m	Fine blue sand and silty clay	1
	-9.40m	Brown and grey gravel Grey and brown gravel,sand,silt and clay	2
	-13.5m	Brown metal with fines	2
	-19.2m	Brown metal and clay	3
	-31.3m	Brown metal. Water bearing	h

Borelog for well R27/1176

Gridref: 1760989.5435796

Ground Level Altitude 6.30 +MSD

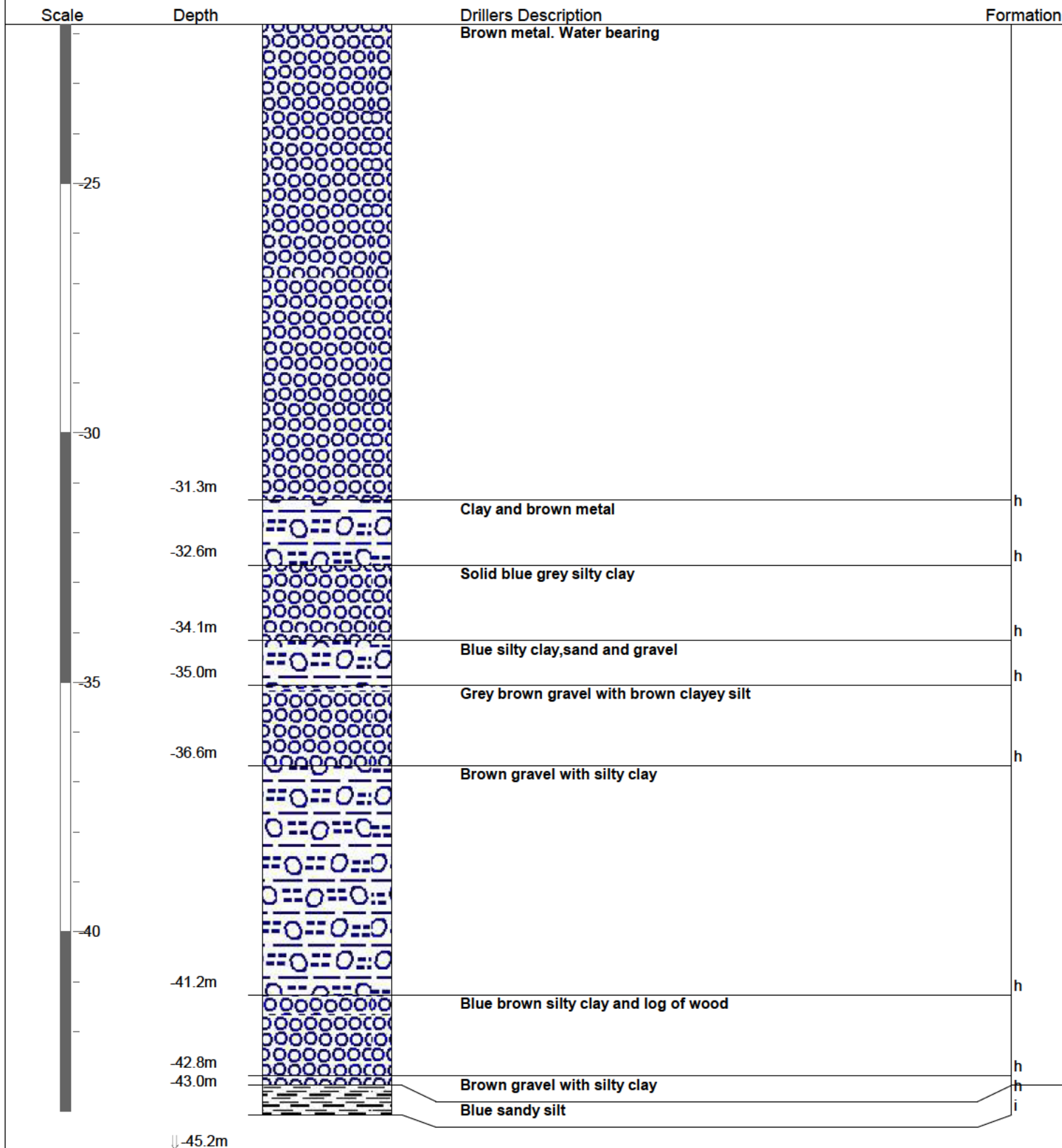
Driller : RICHARDSON DRILLING COMPANY LTD

Drill Method :

Drill Depth : m Drill Date : 1/01/1975 12:00:00 a.m.

Page

2 / 3



Borelog for well R27/1176

Gridref: 1760989.5435796

Ground Level Altitude 6.30 +MSD

Driller : RICHARDSON DRILLING COMPANY LTD

Drill Method :

Drill Depth : m Drill Date : 1/01/1975 12:00:00 a.m.

Page

3 / 3



Scale	Depth	Drillers Description	Formation
		Blue sandy silt	
-45	-45.2m		i
		Blue brown silty clay	
	-48.6m		i
		Pebbles	
-50	-50.3m		i
		No strata description filled in, though Lithology coded GR ie gravel	
	-52.1m		o
		No strata description filled in, though Lithology coded GR ie gravel	
-55	-56.3m		o
		No strata description filled in, though Lithology coded CL ie clay	
	-57.5m		o
		No strata description filled in, though Lithology coded GR ie gravel	
-60	-60.9m		o
	-61.5m		7
		No strata description filled in, though Lithology coded CL ie clay	
	-64.8m		7
-65	-65.4m		7

Borelog for well R27/6926

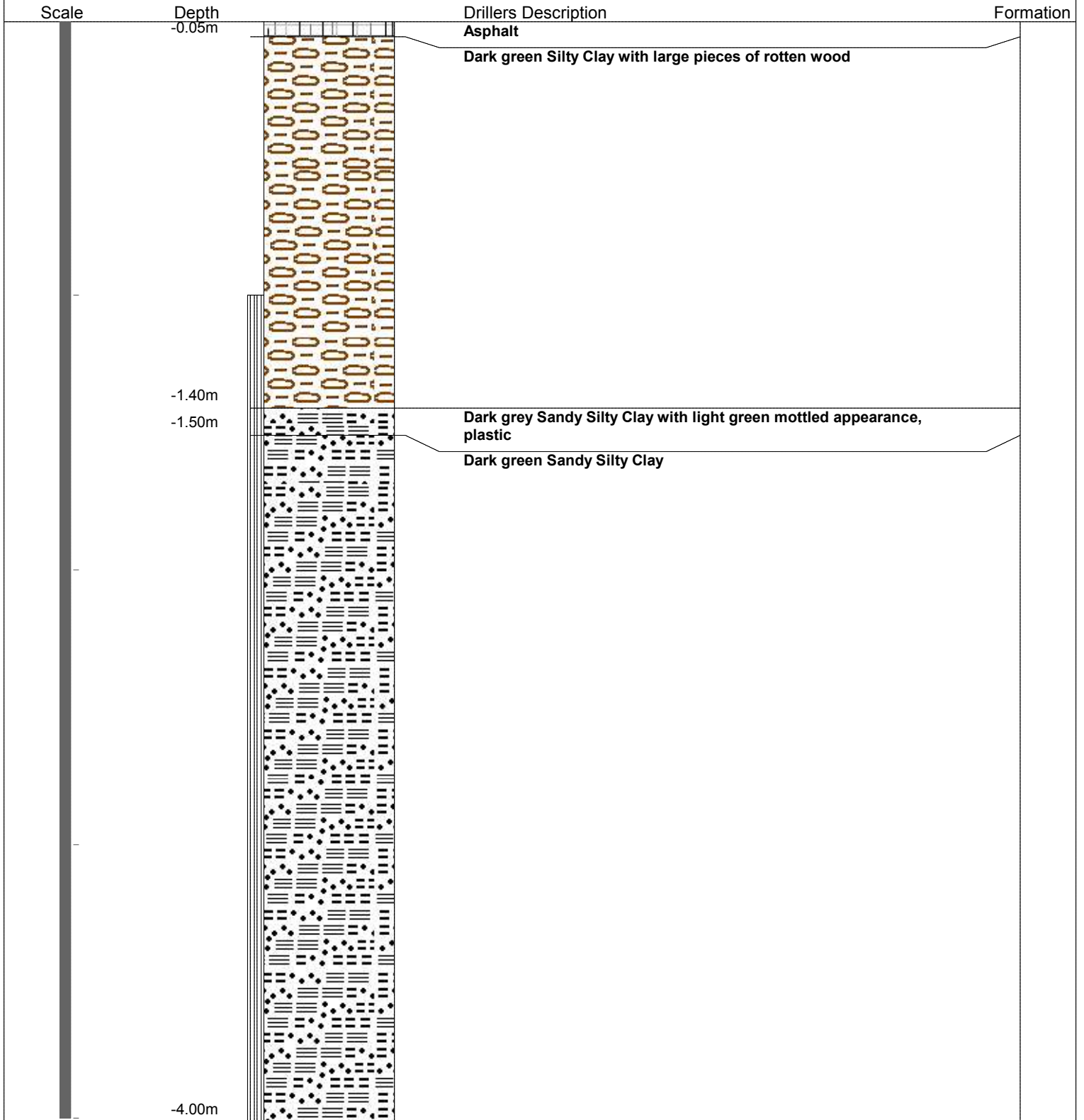
Gridref: 1760934.5435629

Ground Level Altitude 100.11 +MSD

Driller : WEBSTER DRILLING & EXPLORATION LTD

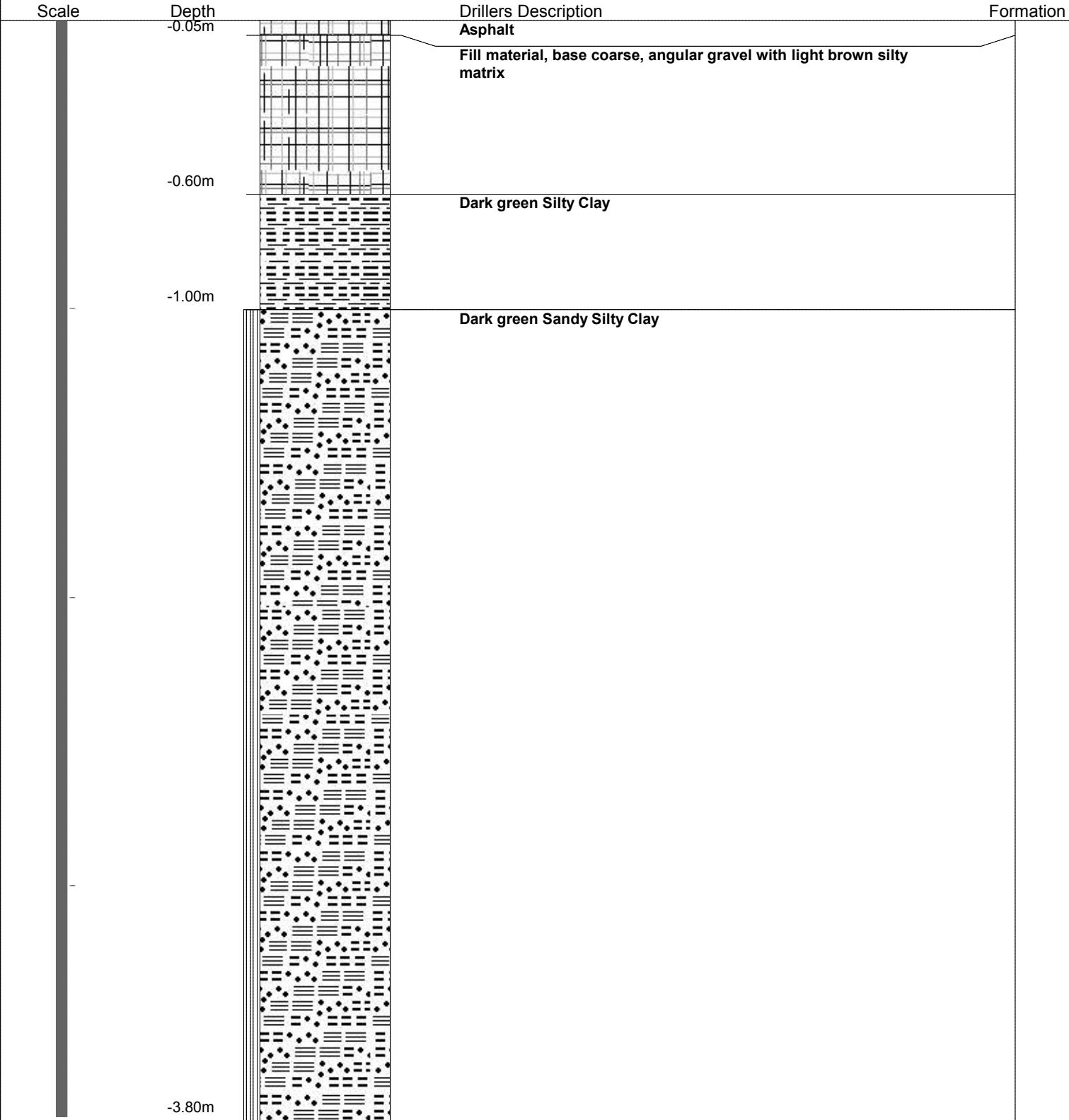
Drill Method : Auger Rig

Drill Depth : 4.00m Drill Date : 8/04/2003 12:00:00 a.m.



Borelog for well R27/6927

Gridref: 1760935.5435634
Ground Level Altitude 100.00 +MSD
Driller : WEBSTER DRILLING & EXPLORATION LTD
Drill Method : Auger Rig
Drill Depth : 3.80m Drill Date : 8/04/2003 12:00:00 a.m.

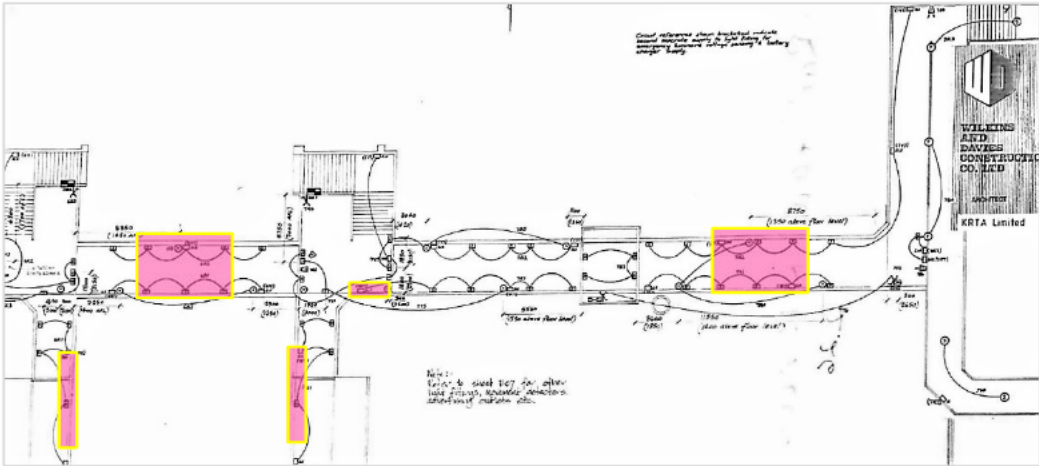


INSPECTION REPORT

PROJECT

REVEAL PROJECT: RVL004577

Waterloo Subway – Reinforcement Investigation

LOCATION	Waterloo Station, Lower Hutt	DATE	16/11/18
CLIENT	Calibre	CLIENT CONTACT	s7(2)(a)
REVEAL CONTACT	s7(2)(a)		
SCOPE	Locate, mark and report on internal reinforcing in the 5 scope areas highlighted below.		
			
	▼ Assess site features and details. ▼ Assess structural plans of known reinforcing and services. ▼ Undertake reinforcing bar location and marking. ▼ Produce report of findings.		
METHODOLOGY	▼ No near-misses or events were recorded. A site induction was carried out before work commenced.		
HEALTH & SAFETY	Investigations were undertaken using a 1.6 GHz antennae Ground Penetrating Radar.		
INVESTIGATION	The following limitations should be referenced when referring to the supplied plans: ▼ The scope of the investigation was limited to a maximum of Quality Level B (ASCE 38-02) ▼ Precise horizontal and vertical location of reinforcing bars will be required to increase accuracy to Quality Level A (ASCE 38-02), such as hydro excavation. ▼ Subsurface reinforcing surveys have known limitations and even after an appropriate and professionally executed survey not all reinforcing may be detectable.		

-
- ▼ Any depths provided are indicative only and can be affected by a range of factors including structure or ground movement, the material type of the reinforcing bars, the concrete composition and immediate environment, subsequent alteration of its position, as well as the proximity of the reinforcing bar to other bars nearby.

ASCE 38-02: <https://www.dropbox.com/s/2dlqv9uk2v6o6s6/ASCE%2038-02.pdf>

SUMMARY

- ▼ Areas that were too close to adjacent walls, corners or edges were unable to be scanned
 - ▼ A maximum height of 3m from ground level was only able to be scanned
 - ▼ The size of the reinforcing is unable to be determined
 - ▼ For this investigation a 2nd layer of reinforcing was unable to be detected with certainty
-



Figure 01

Northern wall, eastern end of subway – below the road.

Vertical reinforcing –Blue:

- ▼ Depth: 5-6cm
- ▼ Spacing: 20cm

Horizontal reinforcing – Pink:

- ▼ Depth: 9-10cm
- ▼ Spacing: 25cm

1. Reinforcing was seen at 10-12cm below the ex. lighting circuit panels – referred to in Figure 01 & 02 (dashed black).

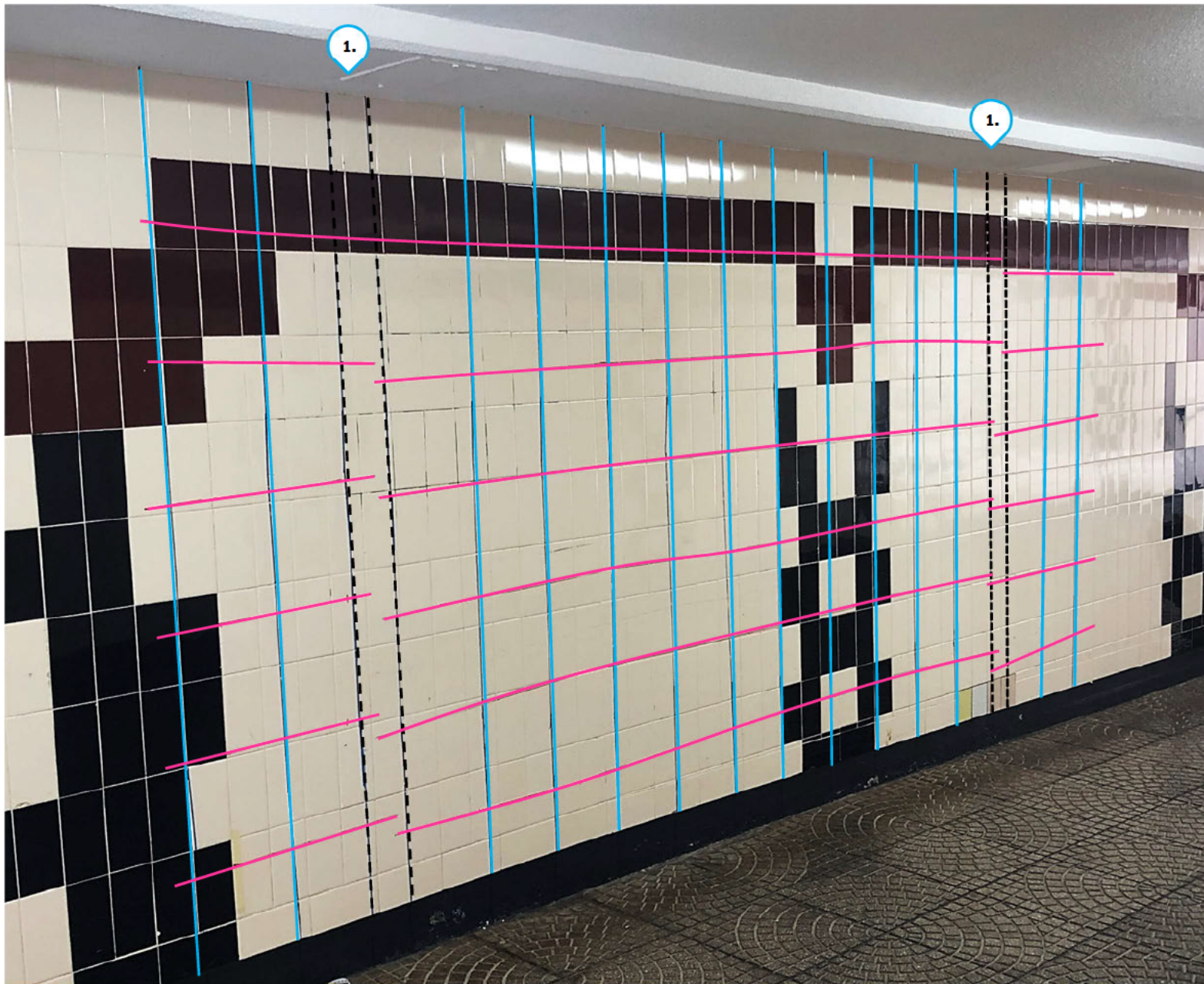


Figure 02

Southern wall, eastern end of subway – below the road.

Vertical reinforcing – Blue:

- ▼ Depth: 5-6cm
- ▼ Spacing: 20cm

Horizontal reinforcing – Pink:

- ▼ Depth: 9-10cm
- ▼ Spacing: 25cm

1. Reinforcing was seen at 10-12cm below the ex. lighting circuit panels (dashed black) – refer to Figure 03.

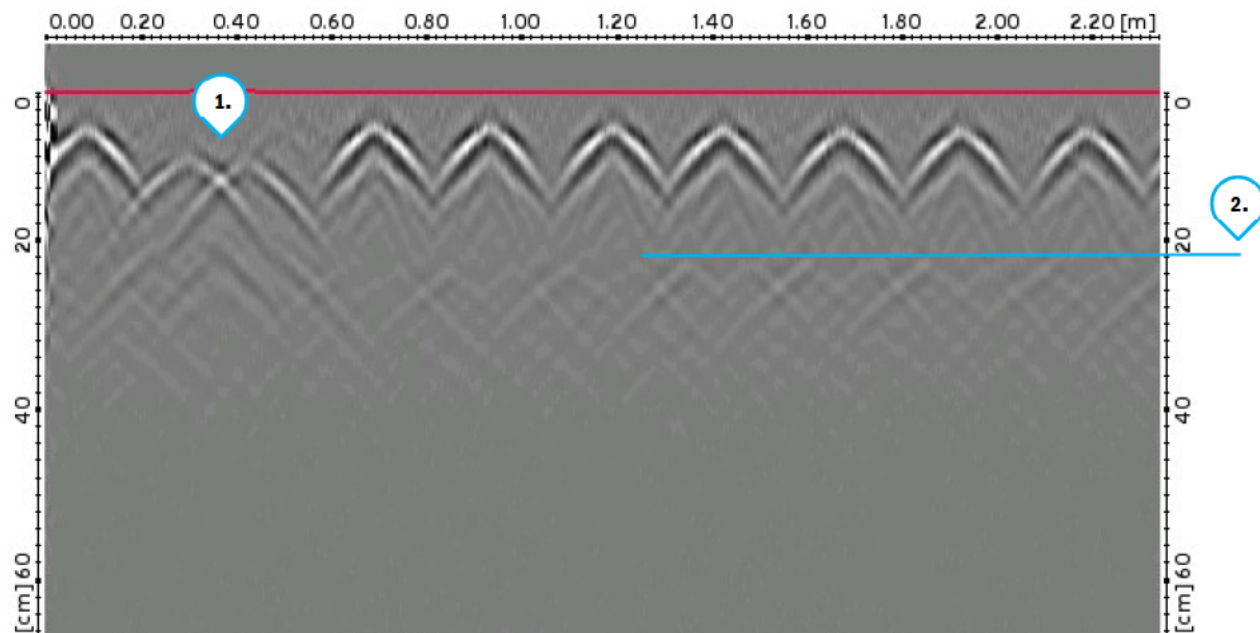


Figure 03

Vertical reinforcing:

- ▼ Depth: 5-6cm
- ▼ Spacing: 20cm

1. Reinforcing was seen at 10-12cm below the ex. lighting circuit panels – referred to in Figure 01 & 02 (dashed black).

2. It is possible that the 2nd layer of reinforcing sits between 22-26cm below the surface of the wall. However, this layer was not marked due to low visibility and inconsistency.

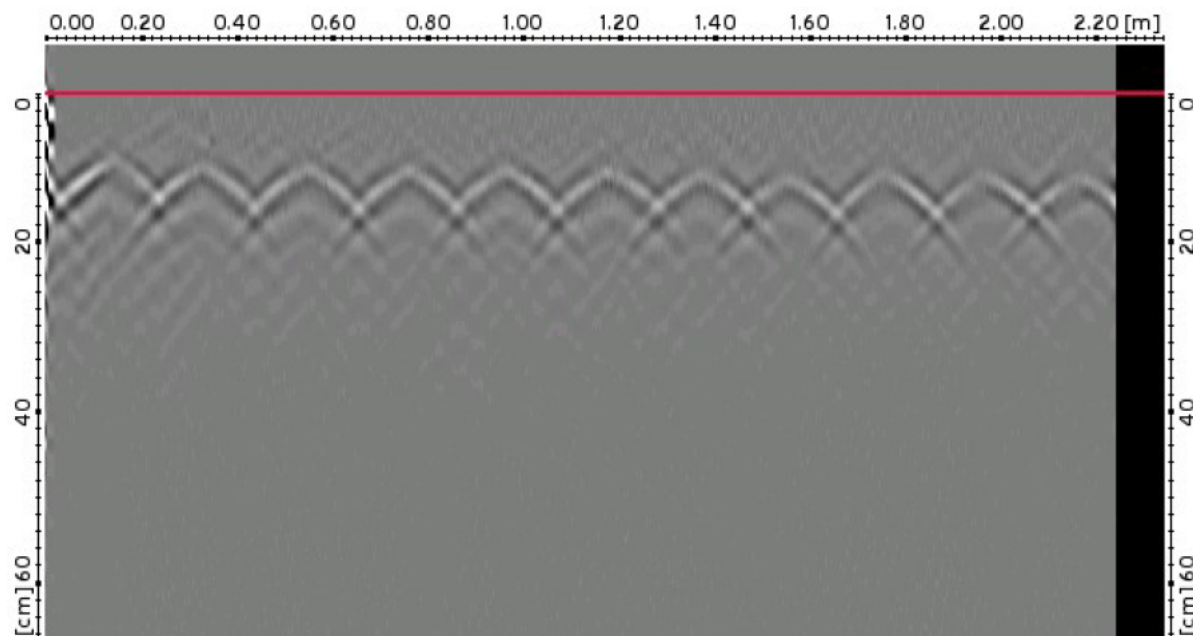


Figure 04

Horizontal reinforcing:

- ▼ Depth: 9-10cm
- ▼ Spacing: 25cm



Figure 05

Ceiling northern side, eastern end of subway – below the road.

Side to side (width of subway) reinforcing – Blue.

- ▼ Depth: 5-6cm
- ▼ Spacing: 20cm

End to end (length of subway) reinforcing – Pink.

- ▼ Depth: 9-10cm
- ▼ Spacing: 25-30cm

Side to side (width of ex. lighting channel) reinforcing:

1. Shallowest - Black

- ▼ Depth: 4cm
- ▼ Spacing: 30 - 70cm

2. Deepest - Blue.

- ▼ Depth: 5-6cm
- ▼ Spacing: 8cm

Reinforcing was seen at 10-12cm opposite the ex. lighting circuit panels.

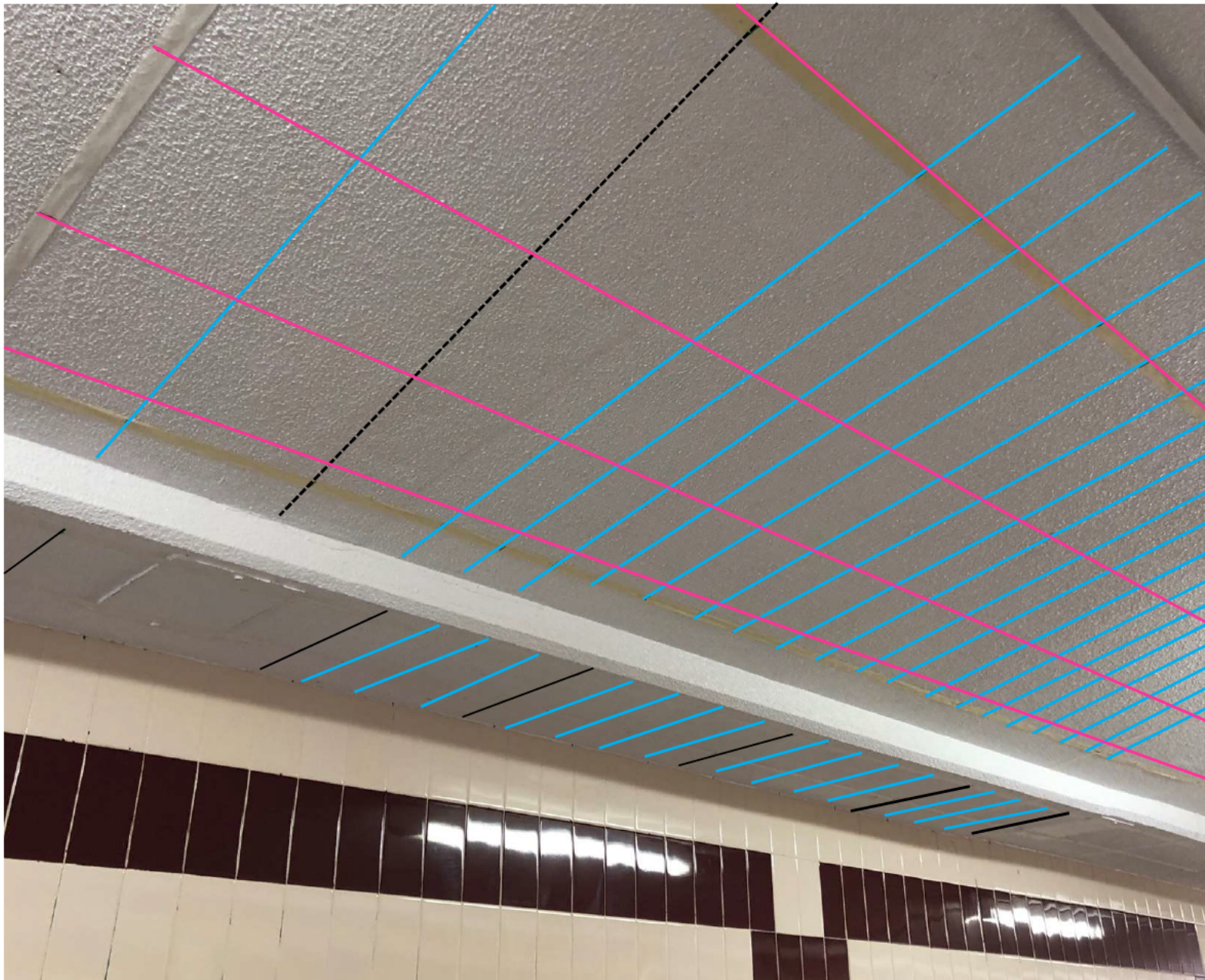


Figure 06

Ceiling southern side, eastern end of subway – below the road.

Side to side (width of subway) reinforcing – Blue:

- ▼ Depth: 6-7cm
- ▼ Spacing: 12cm

End to end (length of subway) reinforcing – Pink:

- ▼ Depth: 9-10cm
- ▼ Spacing: 25-30cm

Side to side (width of ex. lighting channel) reinforcing:

1. Shallowest – Black:

- ▼ Depth: 4cm
- ▼ Spacing: 30 - 70cm

2. Deepest – Blue:

- ▼ Depth: 8cm
- ▼ Spacing: 12cm

Reinforcing was seen at 10-12cm opposite the ex. lighting circuit panels.

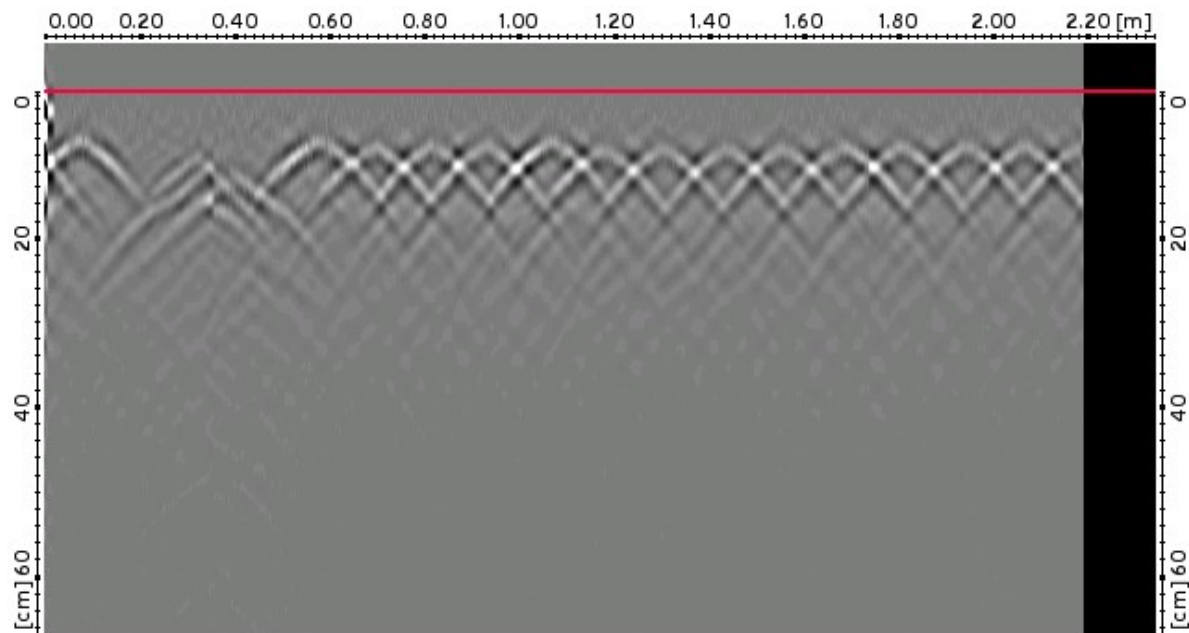


Figure 07

Side to side (width of subway) reinforcing
– Blue:

▼ Depth: 6-7cm

▼ Spacing: 12cm

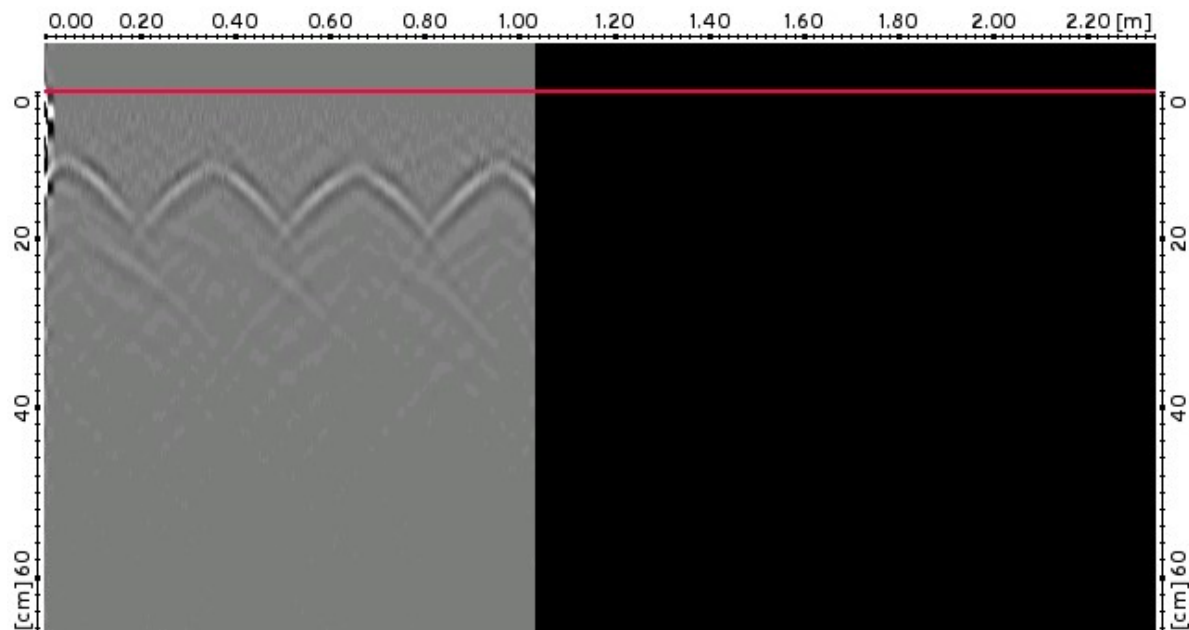


Figure 08

Horizontal reinforcing:

▼ Depth: 9-10cm

▼ Spacing: 25-30cm

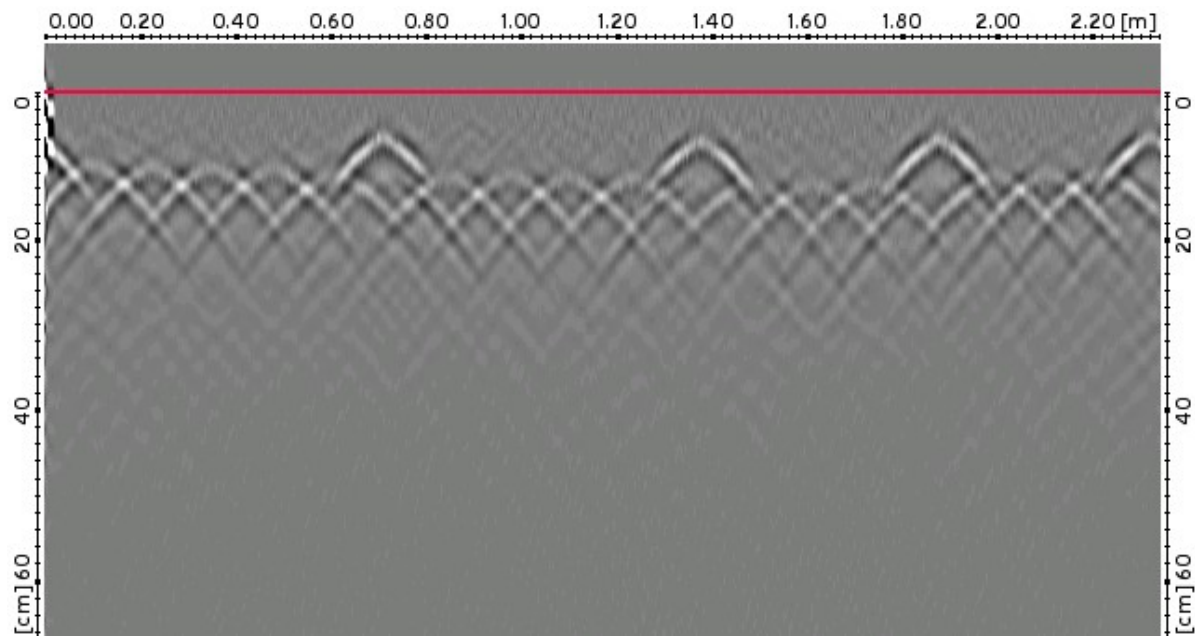


Figure 09

Side to side (width of ex. lighting channel) reinforcing:

1. Shallowest

▼ Depth: 4cm

▼ Spacing: 30-70cm

2. Deepest

▼ Depth: 8cm

▼ Spacing: 12cm



Figure 10

Southern wall, western end of subway – below the railway tracks.

This area was consistent for the most part, however, anomalies can be seen in various locations

Vertical reinforcing – Blue:

- ▼ Depth: 5-6cm
- ▼ Spacing: 20cm

Horizontal reinforcing – Pink:

- ▼ Depth: 8cm
- ▼ Spacing: 30cm

Reinforcing was seen at 10-12cm below the ex. lighting circuit panels.



Figure 11

Northern wall, western end of subway – below the railway tracks.

This area was consistent for the most part, however, anomalies can be seen in various locations

Vertical reinforcing - Blue:

- ▼ Depth: 5-6cm
- ▼ Spacing: 20cm

Horizontal reinforcing – Pink:

- ▼ Depth: 8cm
- ▼ Spacing: 30cm

Reinforcing was seen at 10-12cm below the ex. lighting circuit panels.

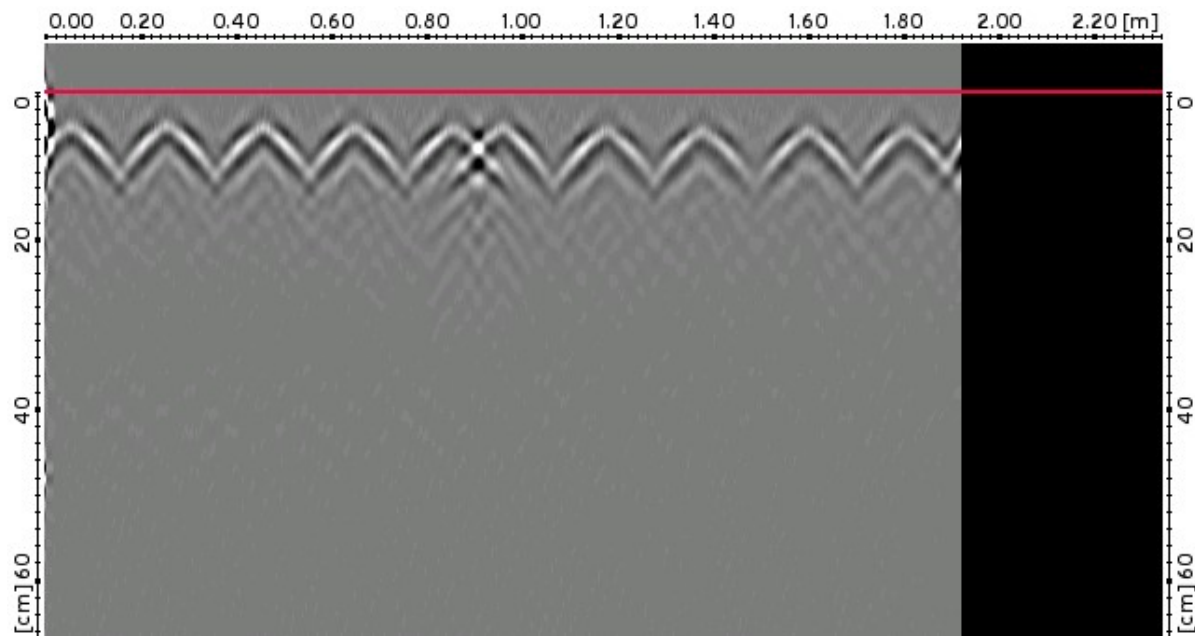


Figure 12

Vertical reinforcing:

- ▼ Depth: 5-6cm
- ▼ Spacing: 20cm

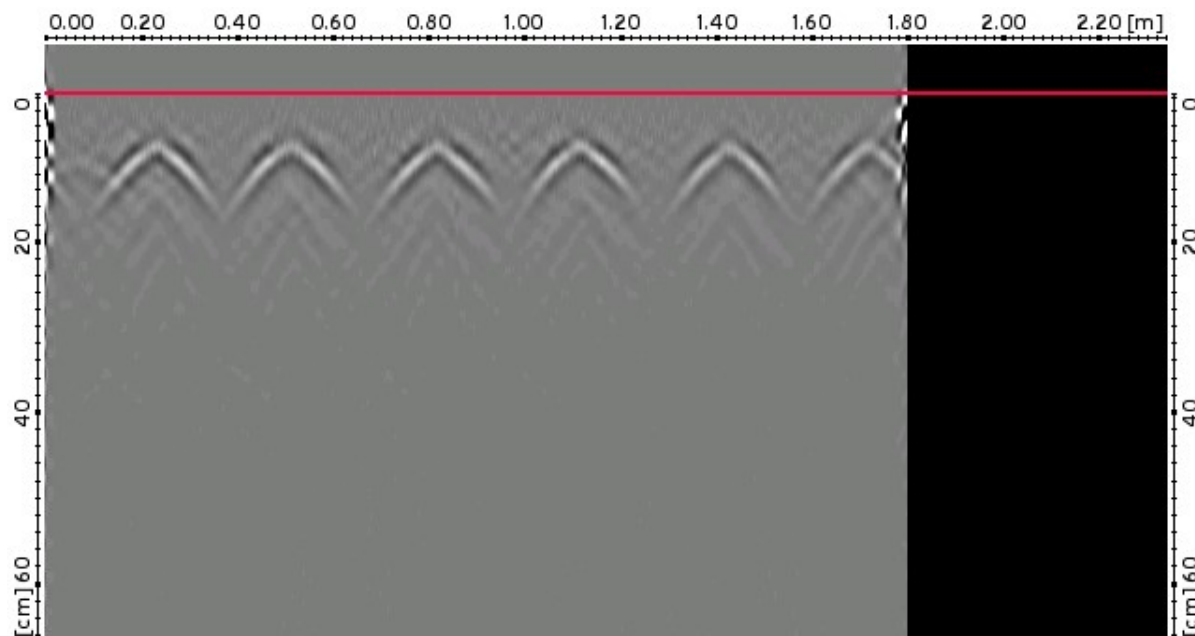


Figure 13

Horizontal reinforcing:

- ▼ Depth: 8cm
- ▼ Spacing: 30cm

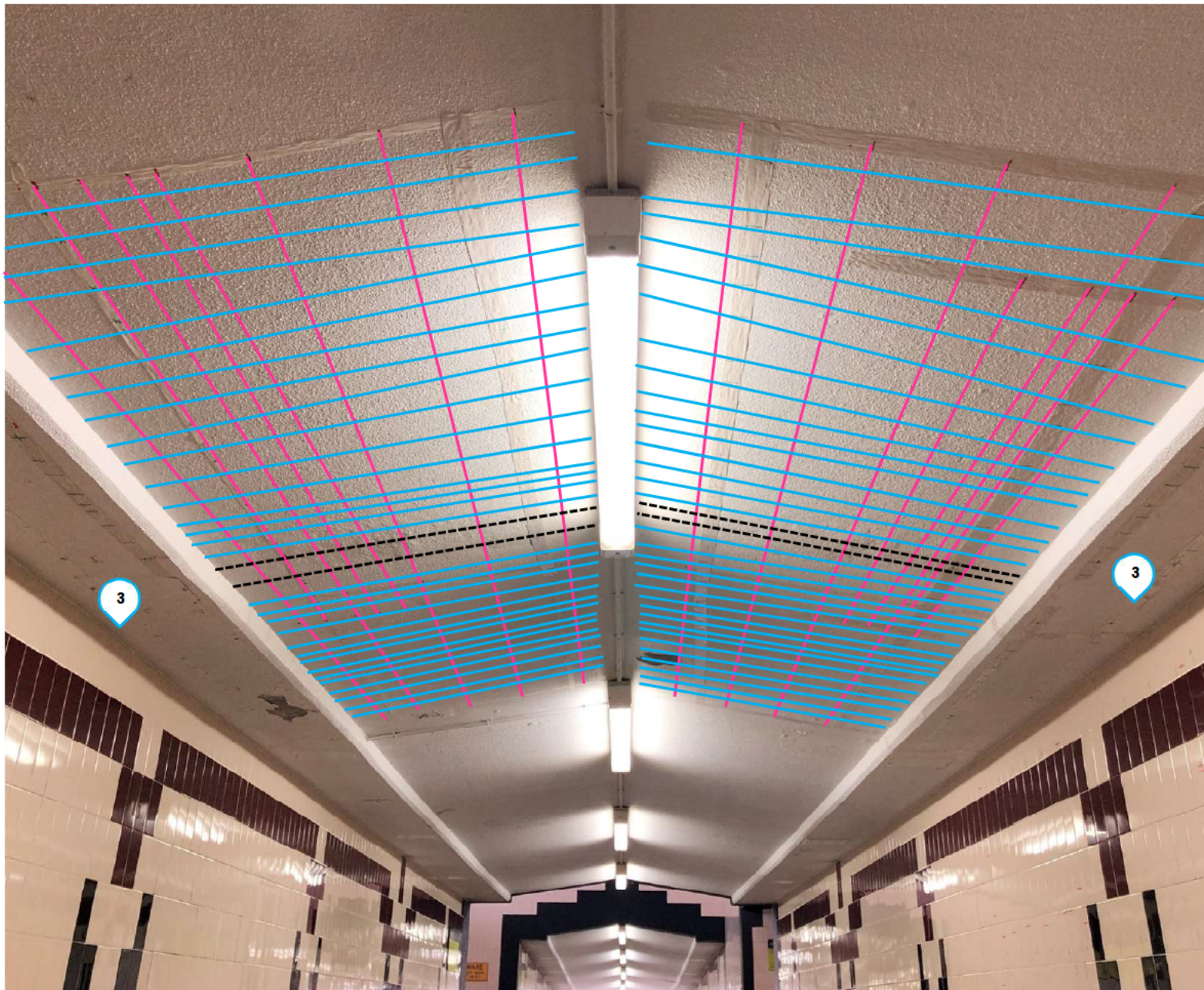


Figure 14

Ceiling, western end of subway – below the train tracks.

Side to side (width of subway) reinforcing - Blue

▼ Depth: 5-6cm

▼ Spacing: 10cm

End to end (length of subway) reinforcing - Pink

▼ Depth: 8cm

▼ Spacing: 10-30cm

3. This area is densely reinforced, mesh like – refer to Figure 15

Reinforcing was seen at 10-12cm opposite the ex. lighting circuit panels.

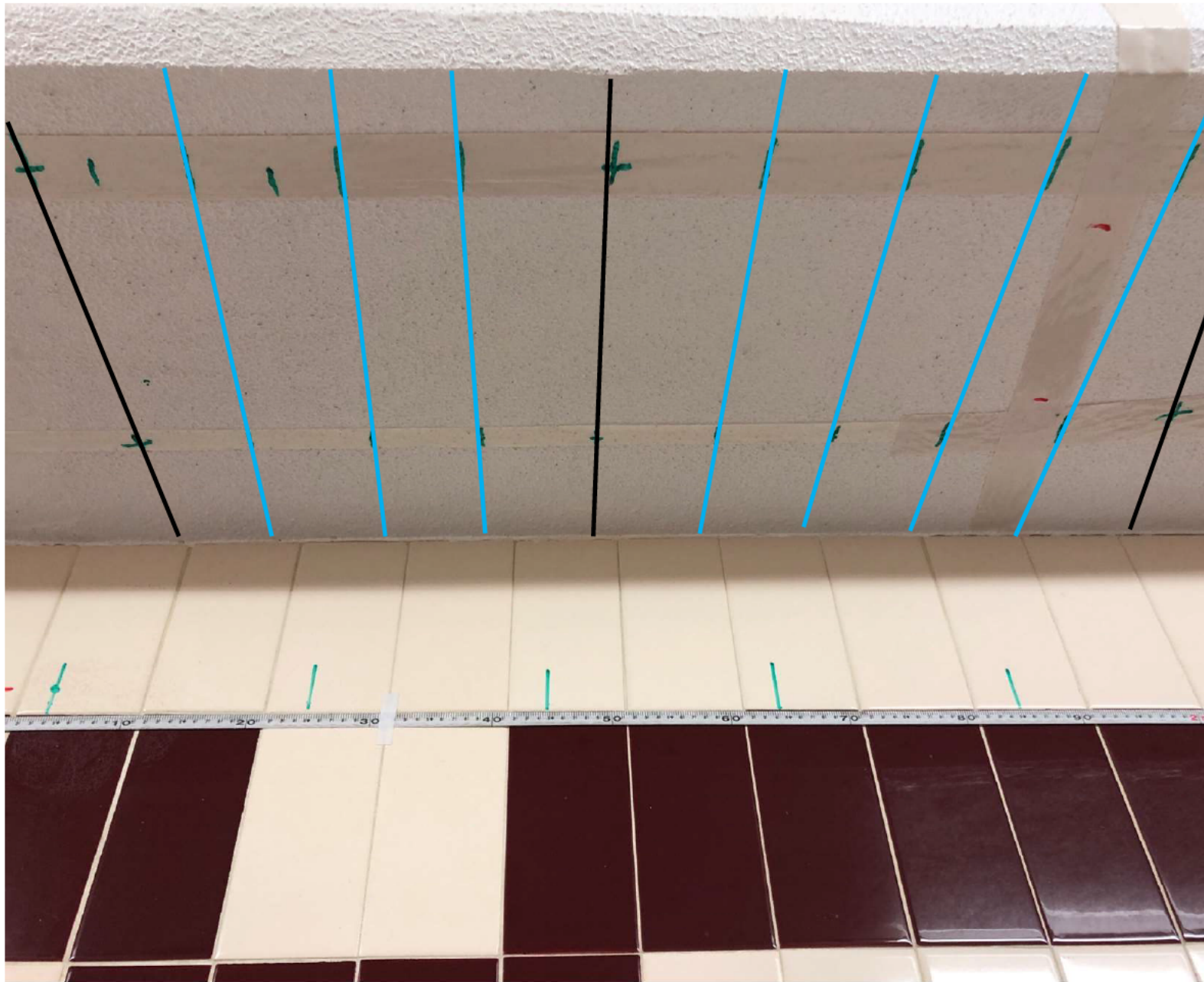


Figure 15

Ex. lighting channel reinforcing, western end of subway – below the train tracks.

Side to side (width of channel) reinforcing:

1. Shallowest (black)

▼ Depth: 4cm

▼ Spacing: 40cm

2. Deepest (blue)

▼ Depth: 5-6cm

▼ Spacing: 8cm

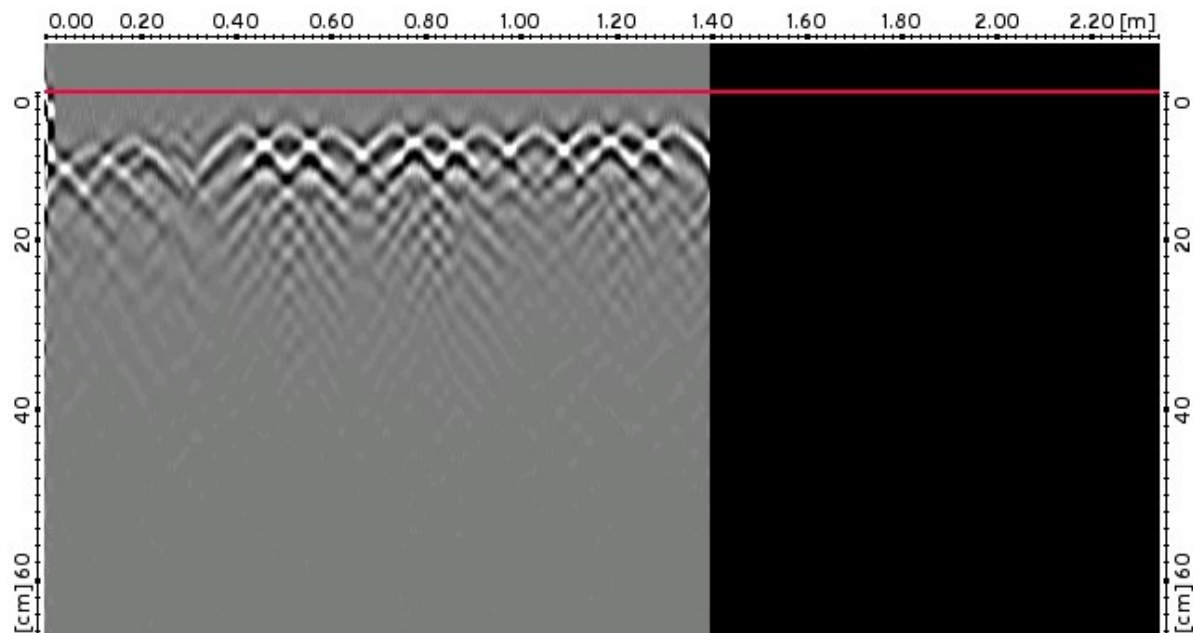


Figure 16

Side to side (width of subway)
reinforcing:

- ▼ Depth: 5-6cm
- ▼ Spacing: 10cm

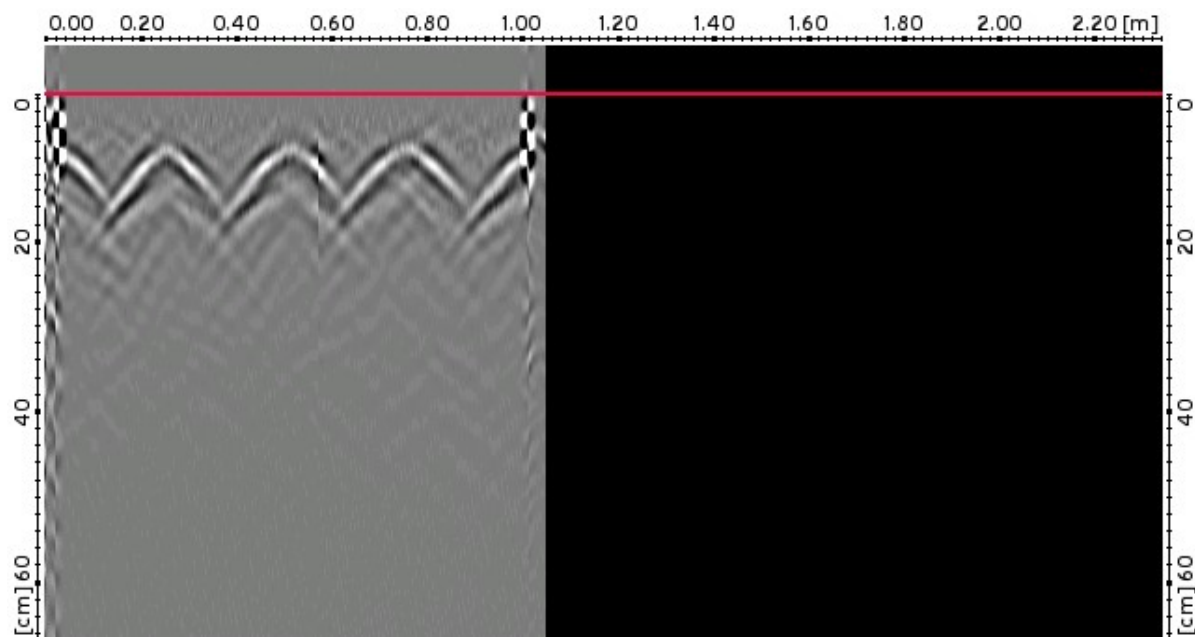


Figure 17

End to end (length of subway)
reinforcing:

- ▼ Depth: 8cm
- ▼ Spacing: 10-30cm

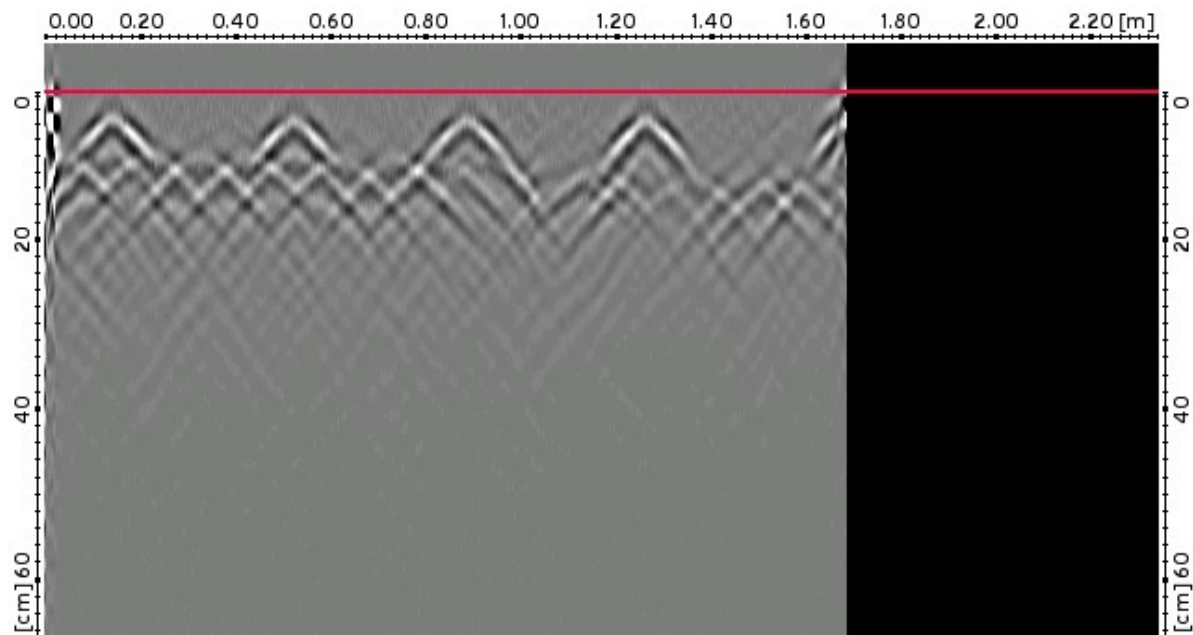


Figure 18

Side to side (width of ex. lighting channel) reinforcing:

1. Shallowest

▼ Depth: 4cm

▼ Spacing: 40cm

2. Deepest

▼ Depth: 8cm

▼ Spacing: 10cm

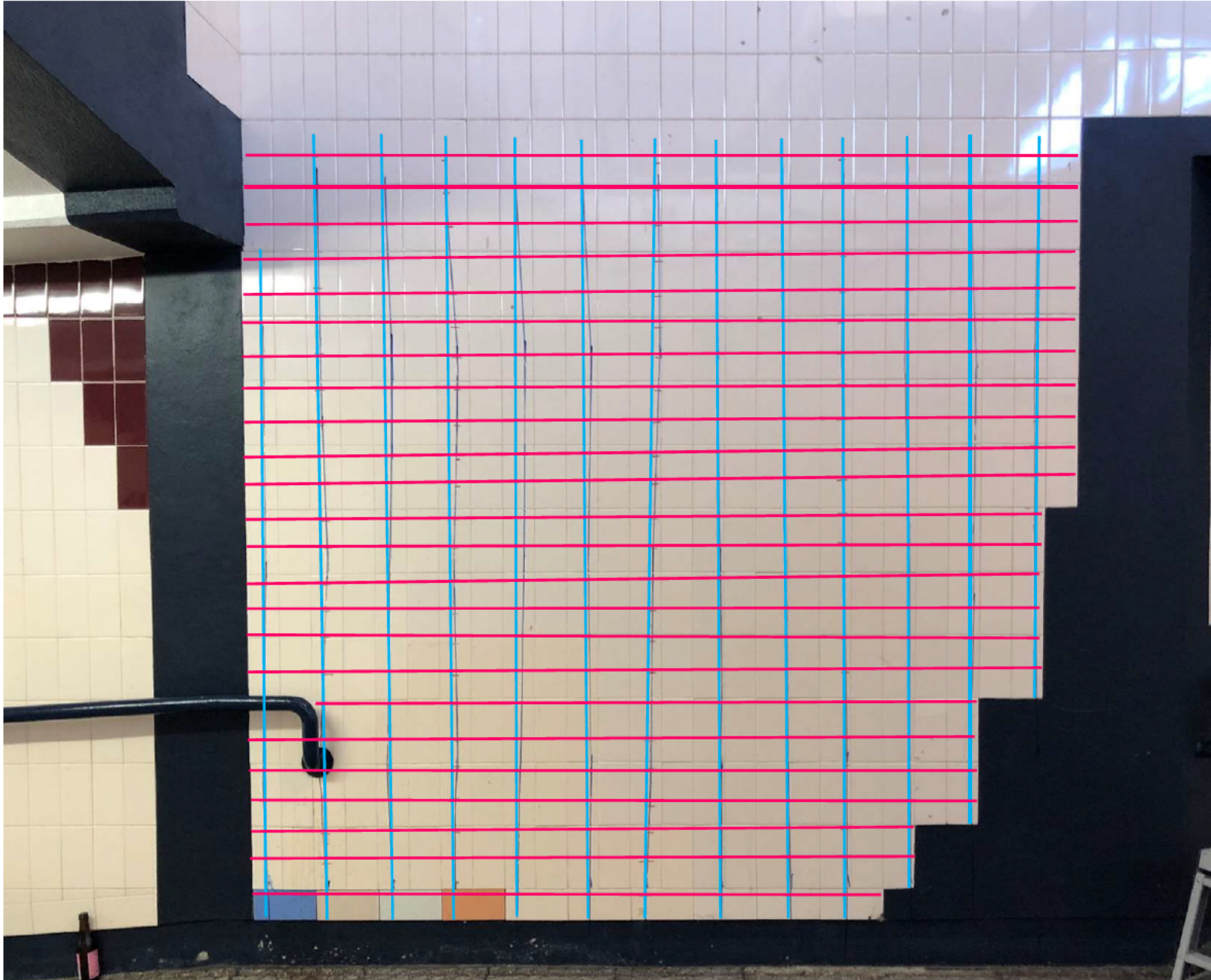


Figure 19

Southern wall, opposite stairs leading up to the platform.

Vertical reinforcing – Blue:

- ▼ Depth: 10cm
- ▼ Spacing: 20cm

Horizontal reinforcing – Pink:

- ▼ Depth: 12cm
- ▼ Spacing: 10cm

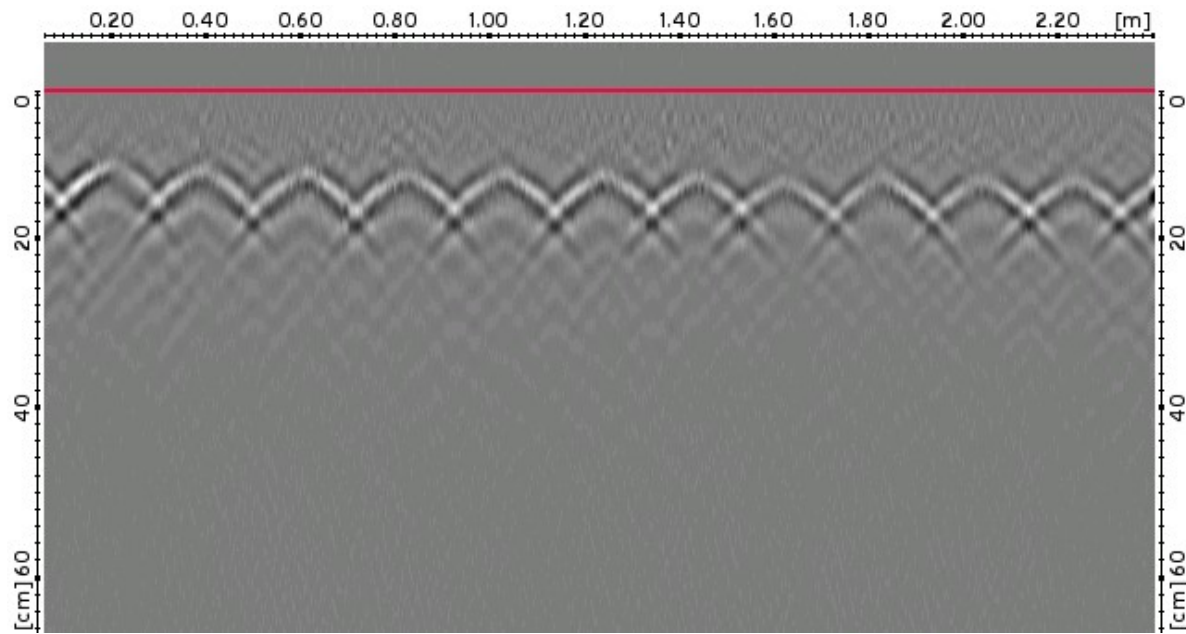


Figure 20

Vertical reinforcing:

▼ Depth: 10cm

▼ Spacing: 20cm

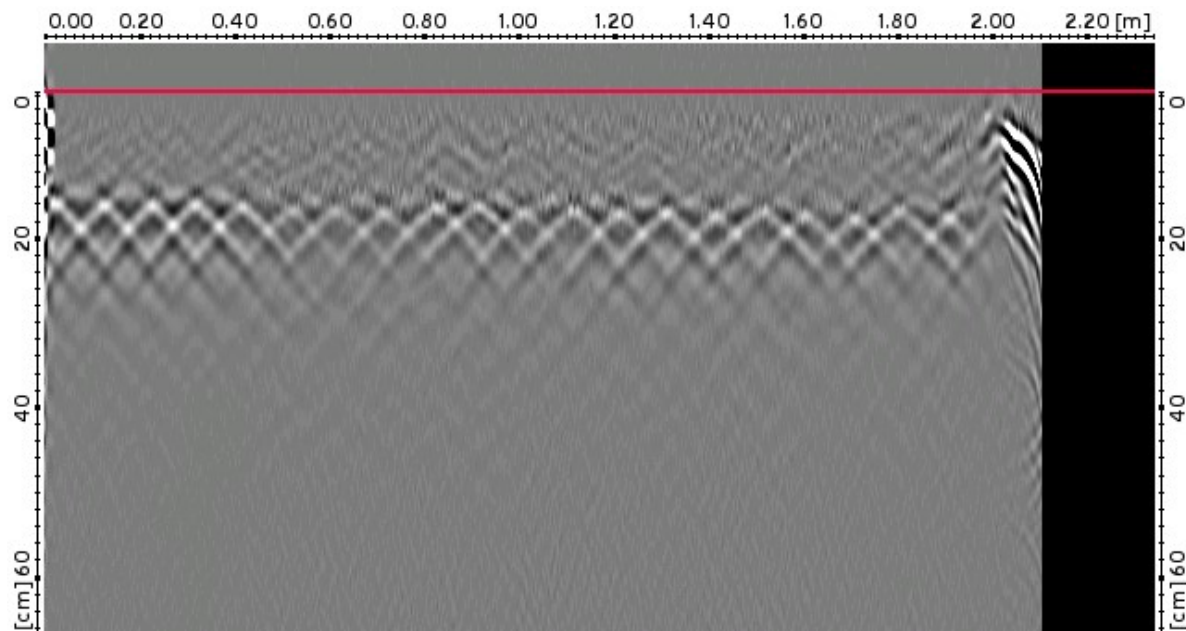


Figure 21

Horizontal reinforcing:

▼ Depth: 12cm

▼ Spacing: 10cm

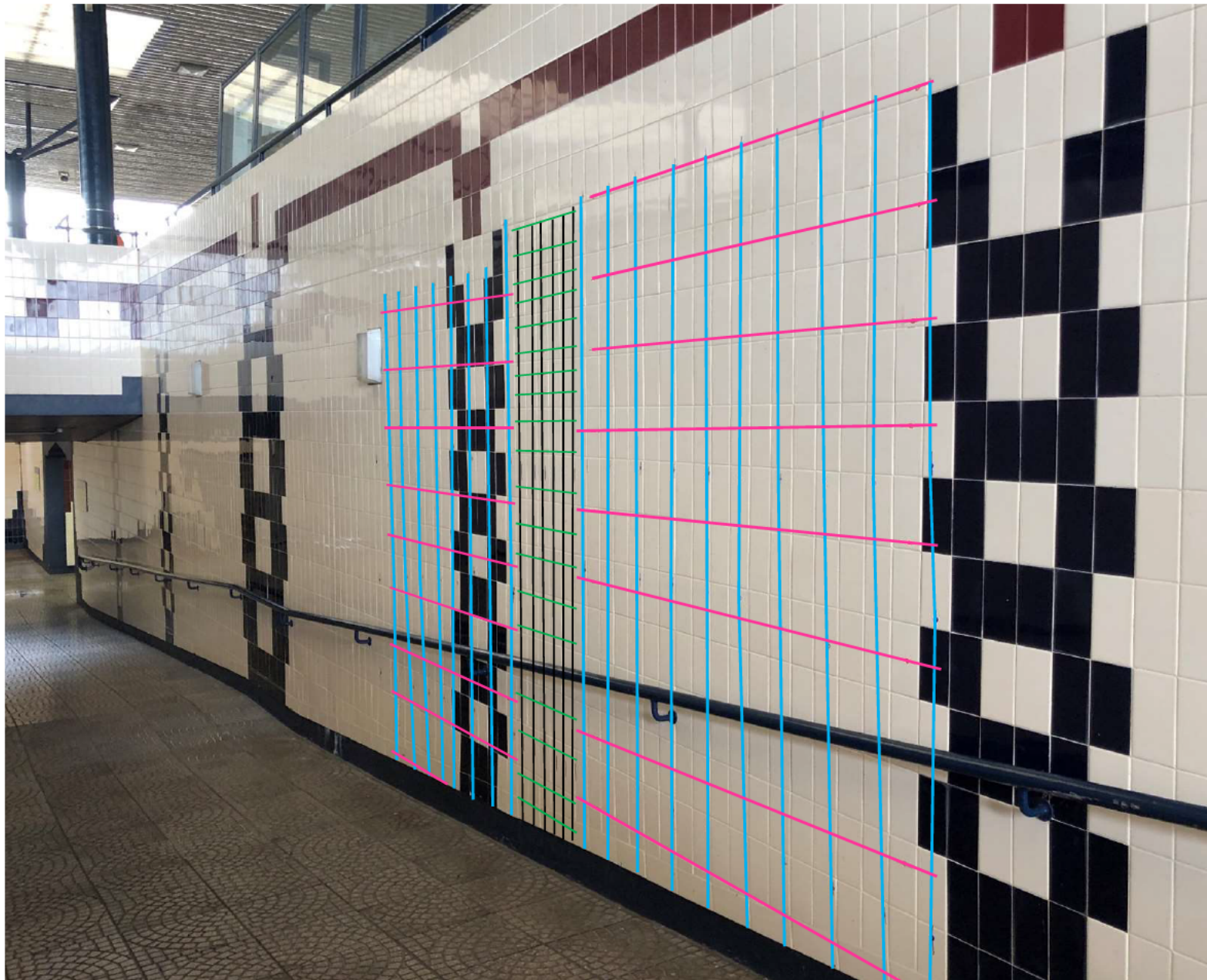


Figure 22

Wall below column on western side of station.

Below this column a reinforcing spacing change from 15cm – 10cm occurred at 1.8m from ground level.

Vertical reinforcing – either side of column – Blue:

- ▼ Depth: 10-11cm
- ▼ Spacing: 20cm

Horizontal reinforcing – either side of column – Pink:

- ▼ Depth: 10-11cm
- ▼ Spacing: 30cm

Vertical reinforcing – below column – Black:

- ▼ Depth: 10-15cm
- ▼ Spacing: 20cm

Horizontal reinforcing – below column – Green:

- ▼ Depth: 10cm
- ▼ Spacing: 10-15cm (top to bottom)

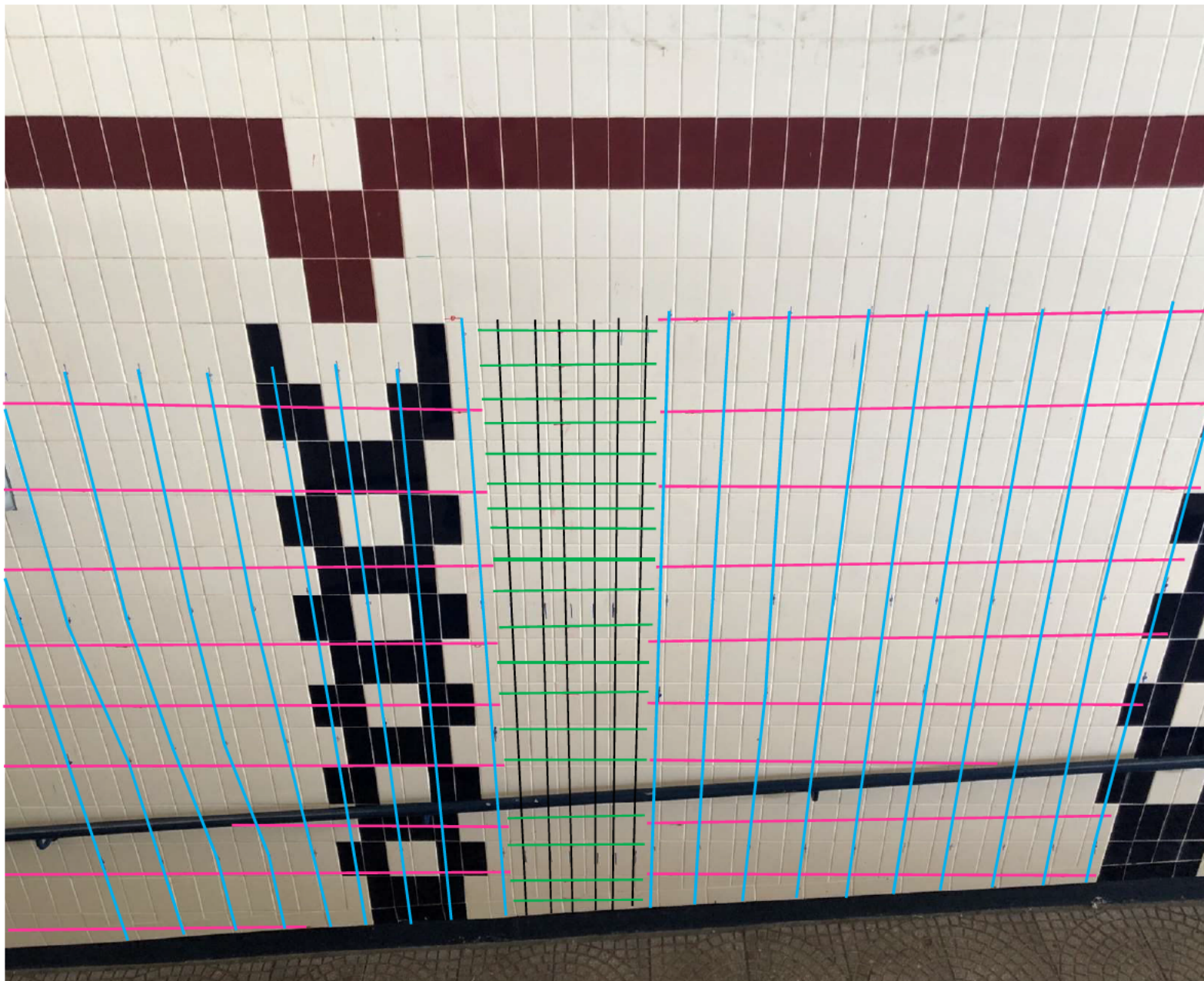


Figure 23

Wall below column on western side of station.

Below this column a reinforcing spacing change from 15cm – 10cm occurred at 1.8m from ground level.

Vertical reinforcing – either side of column – Blue:

- ▼ Depth: 10-11cm
- ▼ Spacing: 20cm

Horizontal reinforcing – either side of column - Pink:

- ▼ Depth: 10-11cm
- ▼ Spacing: 30cm

Vertical reinforcing – below column - Black:

- ▼ Depth: 10-15cm
- ▼ Spacing: 20cm

Horizontal reinforcing – below column - Green:

- ▼ Depth: 10cm
- ▼ Spacing: 10-15cm (top to bottom)

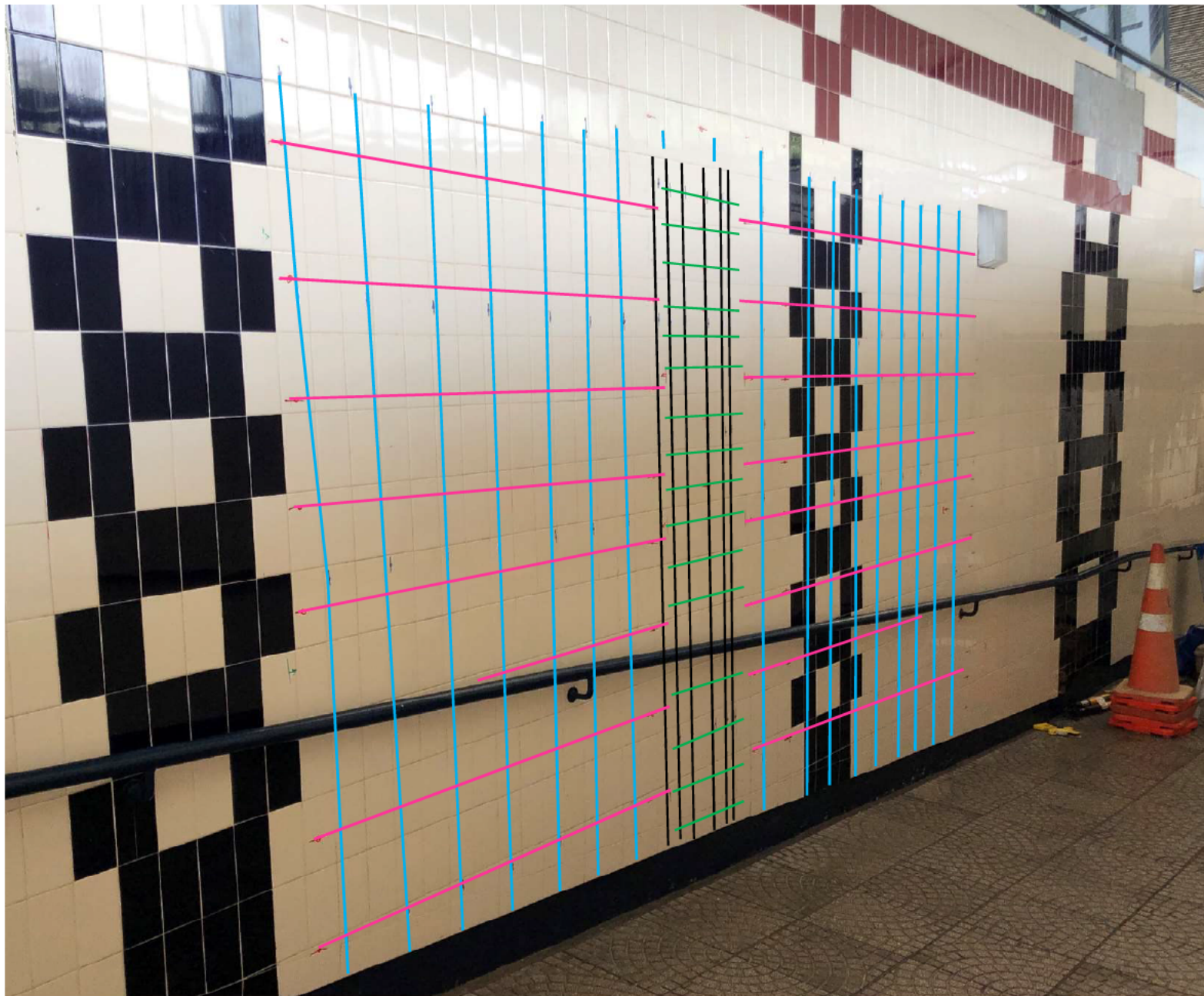


Figure 24

Wall below column on eastern side of station.

Below this column a reinforcing spacing change from 15cm – 10cm occurred at 1.6m from ground level.

Vertical reinforcing – either side of column - Blue:

- ▼ Depth: 10-11cm
- ▼ Spacing: 20cm

Horizontal reinforcing – either side of column - Pink:

- ▼ Depth: 10-11cm
- ▼ Spacing: 30cm

Vertical reinforcing - below column - Black:

- ▼ Depth: 10-15cm
- ▼ Spacing: 20cm

Horizontal reinforcing – below column - Green:

- ▼ Depth: 10cm
- ▼ Spacing: 10-15cm (top to bottom)

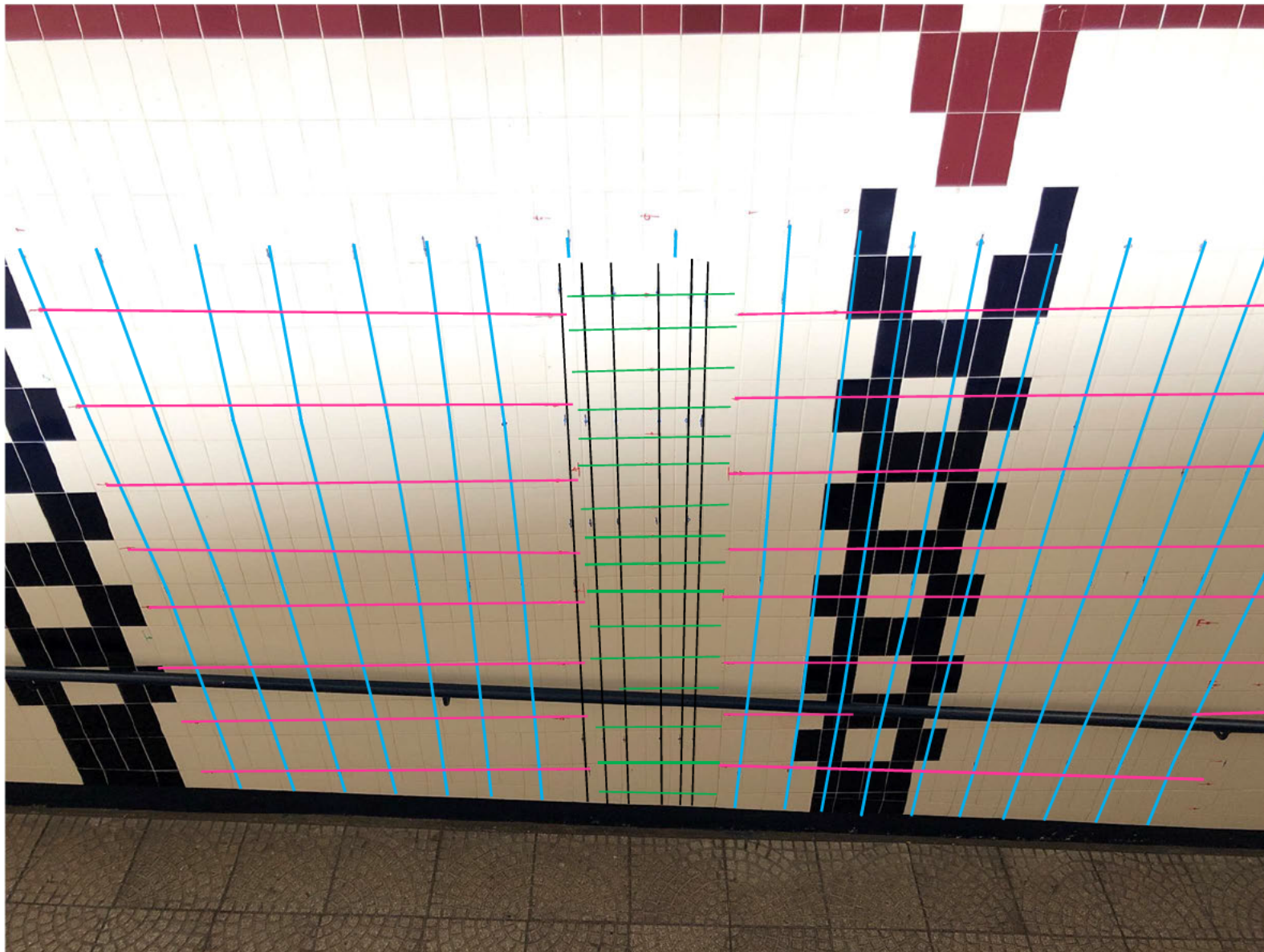


Figure 25

Wall below column on eastern side of station.

Below this column a reinforcing spacing change from 15cm – 10cm occurred at 1.6m from ground level.

Vertical reinforcing – either side of column - Blue:

- ▼ Depth: 10-11cm
- ▼ Spacing: 20cm

Horizontal reinforcing – either side of column - Pink:

- ▼ Depth: 10-11cm
- ▼ Spacing: 30cm

Vertical reinforcing - below column - Black:

- ▼ Depth: 10-15cm
- ▼ Spacing: 20cm

Horizontal reinforcing – below column - Green:

- ▼ Depth: 10cm
- ▼ Spacing: 10-15cm (top to bottom)

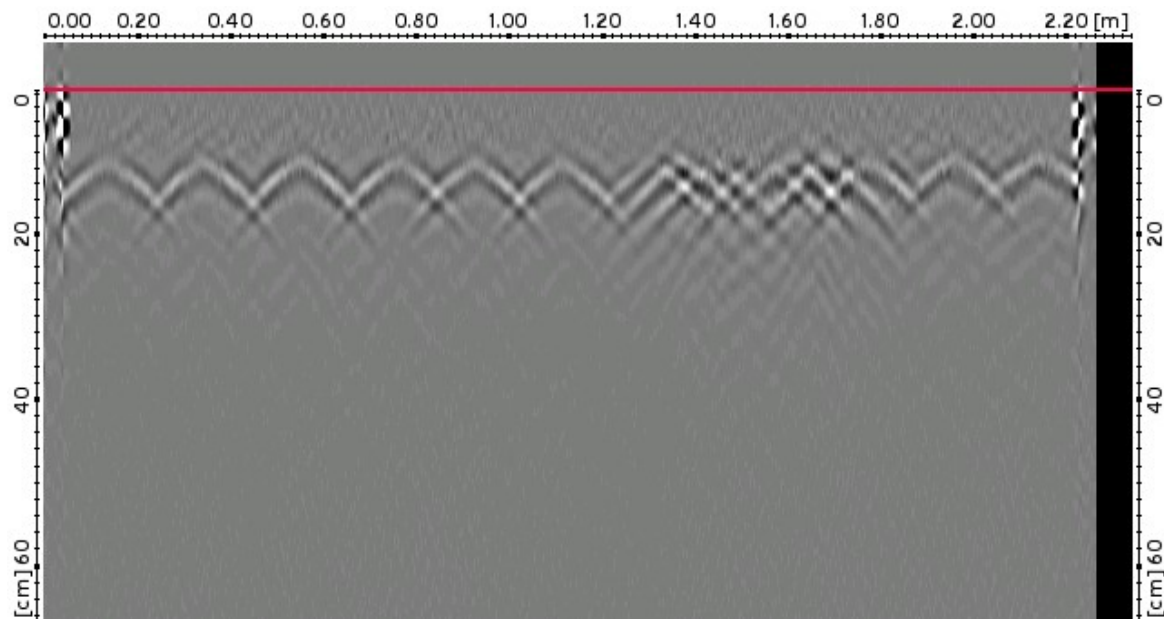


Figure 26

Vertical reinforcing – either side of column:

- ▼ Depth: 10-11cm
- ▼ Spacing: 20cm

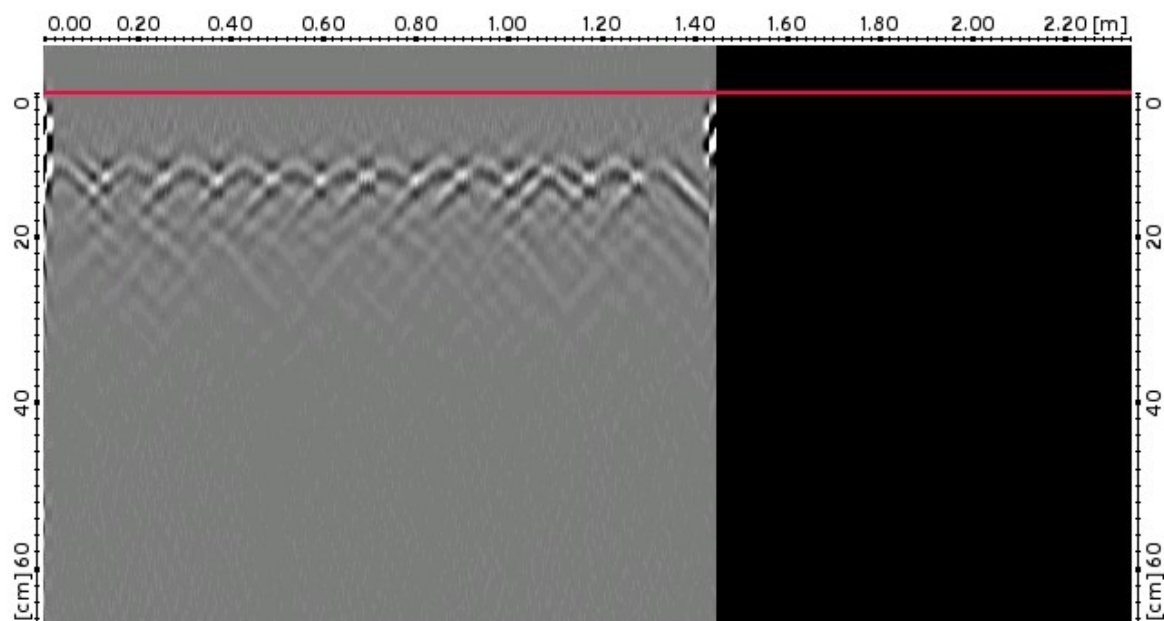


Figure 27

Horizontal reinforcing – below column:

- ▼ Depth: 10cm
- ▼ Spacing: 10-15cm (top to bottom)

* Below the western side column, the spacing change from 15cm – 10cm occurred at 1.8m from ground level.

* Below the eastern side column, the spacing change from 15cm – 10cm occurred at 1.6m from ground level.

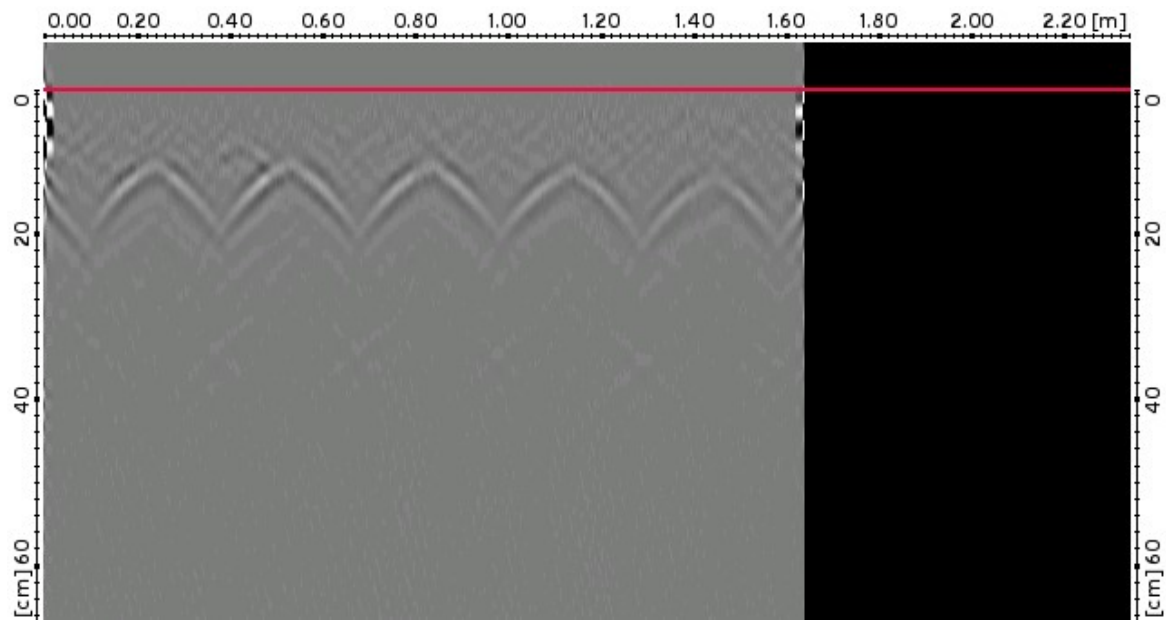


Figure 28

Horizontal reinforcing – either side of column:

- ▼ Depth: 10-11cm
- ▼ Spacing: 30cm

Background

- GWRL require a better understanding of the likely seismic performance of the pedestrian subway at the Waterloo interchange.
- Concrete wall structure with some walls as cantilevers, others are tunnels.
- Designed in 1987
- Station structure above has some columns very close to some walls, this may surcharge the structure and does not appear to have been considered in the original design
- Importance Level 3 structure
 - SLS Return period 25yrs
 - ULS Return period 1000yrs

Material Properties

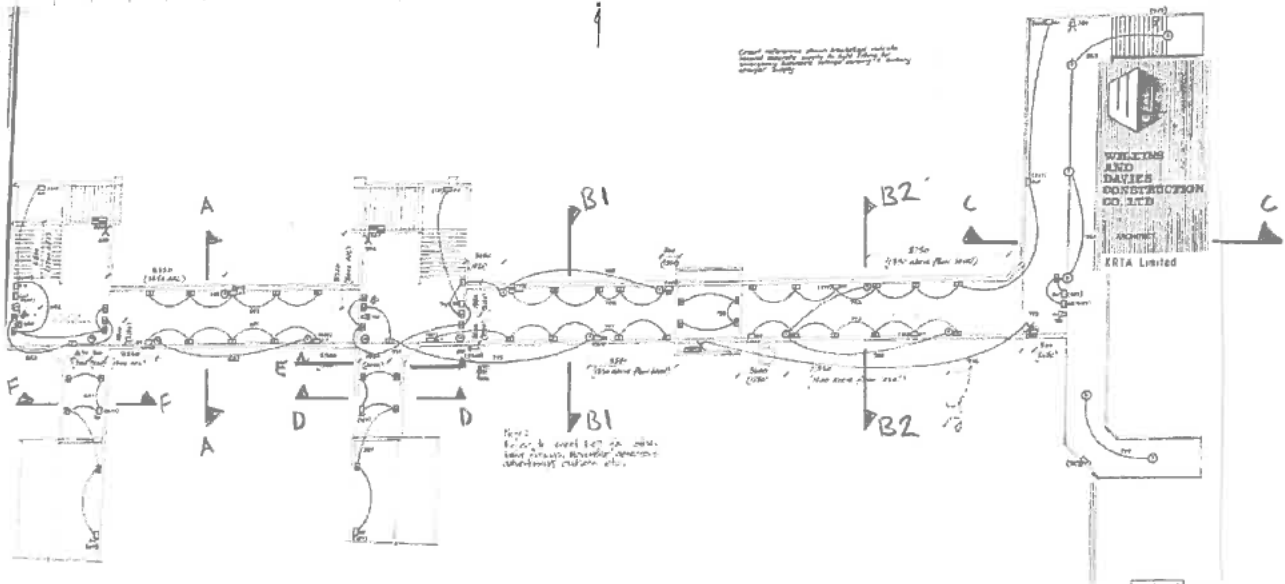
- From the original calculations
 - $f'_c = 30 \text{ MPa}$
 - $f_y = 275 \text{ MPa}$
- Probable strengths as per NZSEE Guidelines C5
 - $f'_{c\text{prob}} = 1.5 f'_c = 1.5 \times 30 \text{ MPa} = 45 \text{ MPa}$
 - $f_{y\text{prob}} = 275 \text{ MPa}$

Approach

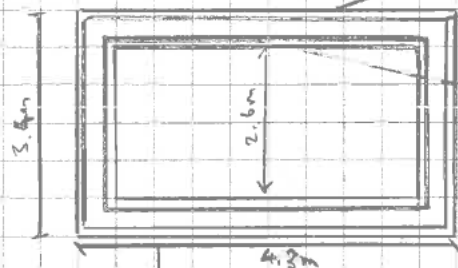
- Determine loads on walls for earthquake load cases
- Compare to wall capacities at critical locations
- Use original codes to determine reo layout, TBC by scanning. Note that for "walls" that are cantilever the scanning will be picking up compression steel which appears to mostly be nominal DLBs @ 200-300mm c/c from original calc. → Original DWG's Dated 22/11/18 → Update

Structure

Note: Scanning appears to confirm this geo layout



• Section A-A



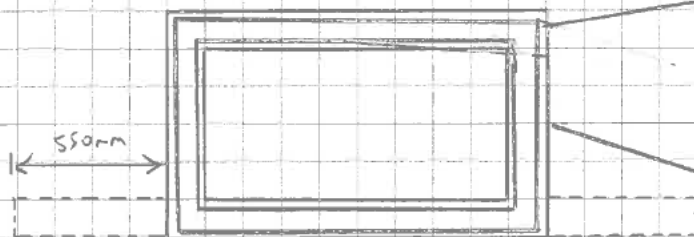
Roof: 350mm, 20mm cover
D20s @ 100 bottom
D20s @ 200 top (D20s @ 100 Top near corner)
R10 U bars @ 100 (S legs / 10 bars)
D16s @ 250 Transverse T+B

Wall: 350mm, 20mm cover
D20s @ 100 outside
D20s @ 200 inside
D16s @ 300 Horiz each face

Floor: 425mm, 60mm cover
D20s @ 100mm Top
D20s @ 200mm Bottom
D20s @ 200mm transverse

SR12 U bars @ 200 near corners

• Section B-B



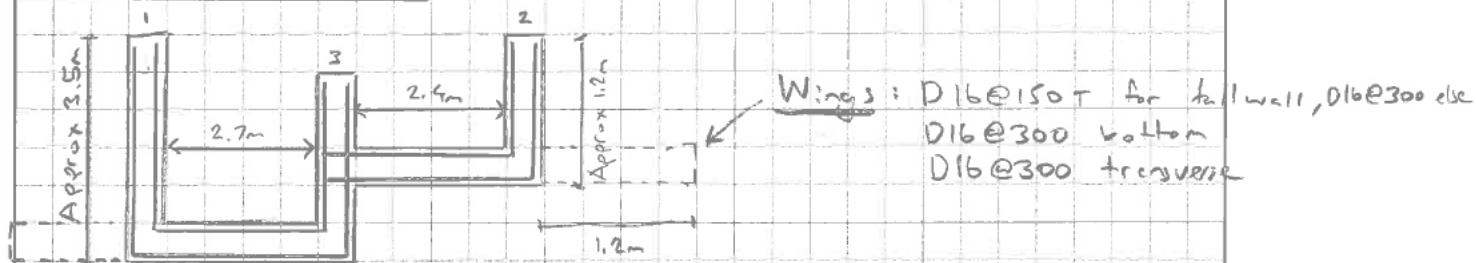
Roof: 250mm, 20mm cover
D20s @ 100mm bottom
D20s @ 250mm top (D20s @ 125 near corner)
D16 @ 300 transverse T+B
R10 U bars @ 100 (S legs / 10 bars)

Walls: 250mm, 20 cover
D20s @ 125 outside
D20s @ 250 inside
D16s @ 300 each face transverse

Floor: 250mm, 20mm cover
D20s @ 125 T
D20s @ 250 B
D20s @ 250 transverse T+B

Wings (where present): D20s @ 200mm T+B

Section C-C



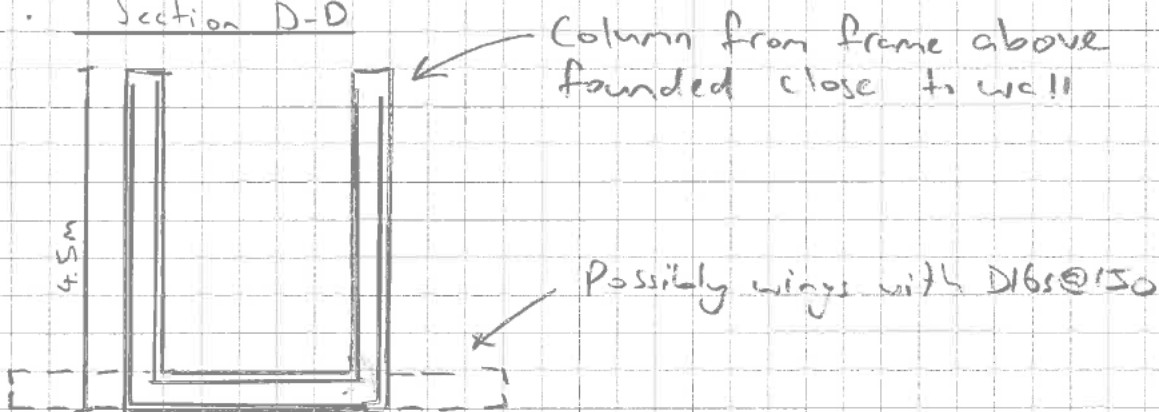
Walls 1 and 3

- 300mm thick with 50mm cover
- D16s @ 75mm c/c outside, D16s @ 300mm inside (same for floor)
- D16s @ 300mm transverse both faces

Wall 2

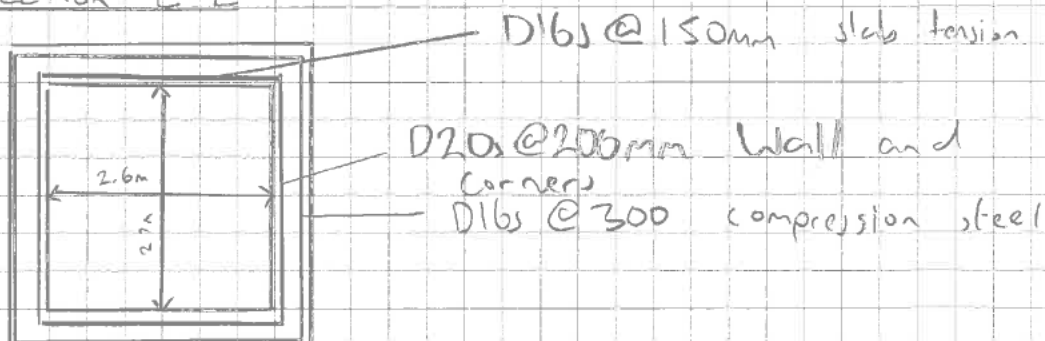
- 300mm thick with 50mm cover
- D16s @ 750mm outside, D16s @ 300 inside, same for floor
- D16s @ 300mm transverse both faces

Section D-D



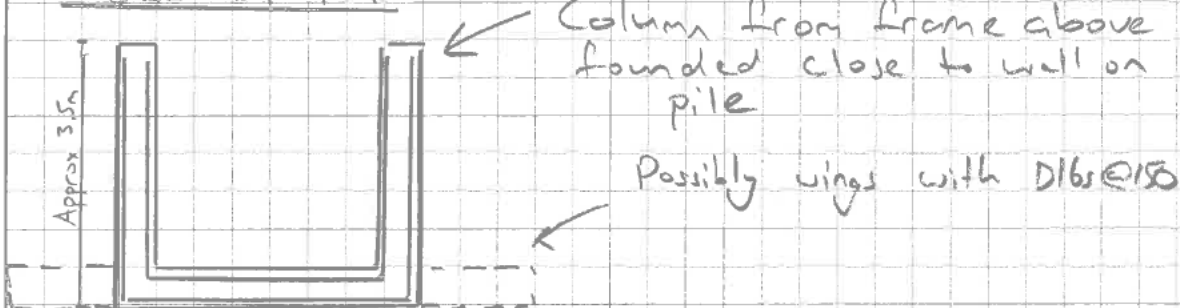
- 350mm thick, 60mm cover
- D24s @ 100mm c/c tension, D16s @ 300 compression steel
- D16s @ 300mm transverse

Section E-E

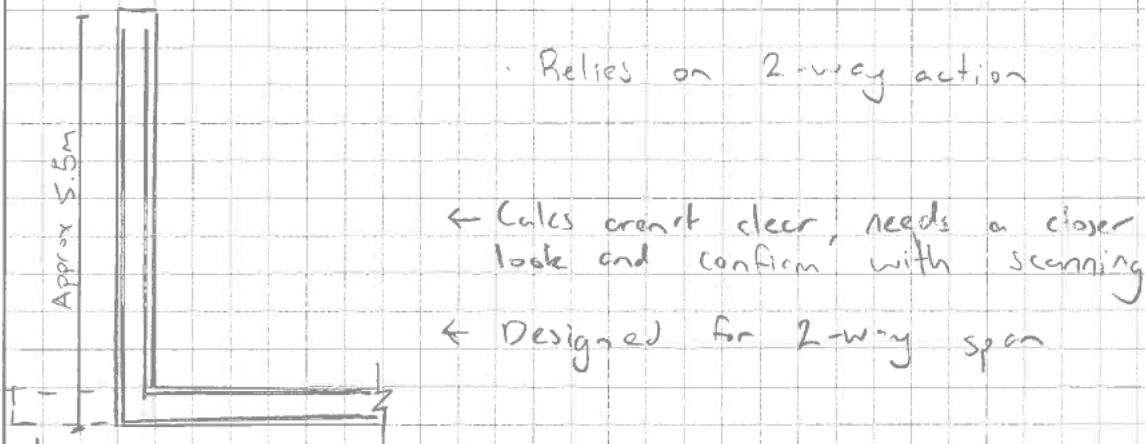


- 200mm thick, 60mm cover
- Transverse D16s @ 300mm

Section F-F



- 300mm thick, 60mm cover
- D20s @ 100mm $\updownarrow$, D16s @ 300mm $\updownarrow$ transverse
- Highest platform void wall (where stairs are)



- wing: 350mm thick, 60mm cover
- D24s @ 100mm $\updownarrow$ vertical, D20s @ 100 horizontal
- Wings: D20s @ 200, transverse D16s @ 300mm
- wings might be D16s @ 150
- Vertical might be D20s @ 100mm $\updownarrow$ or D24s @ 150mm $\updownarrow$

Geotechnical Parameters

- From original 1987 calcs

$$\rho = 1.8 \text{ t/m}^3 = 17.7 \text{ kN/m}^3$$

$$\rho' = 0.9 \text{ t/m}^3 = 8.8 \text{ kN/m}^3$$

$$\phi = 30^\circ$$

$$R_a = 0.33$$

$$R_o = 0.5$$

$$c = 15 \text{ kN/m}^2$$

- Ground water at RL = 5.5m
- Ground level about 5.8mRL - 7.5mRL
- varies wall to wall

From pumping station geotech report (next door)

- Wellington fault $D = 1.6 \text{ km}$ ← closest fault
- Moderate levels of ground shaking expected
- Lateral spreading negligible
- Potential liquefaction risk
- Sub-soil class D
- $PGA = 0.62g$ for UL5 (1/2500yr), $0.45g$ for SL2 (1/1000yr)

↑ Based on IL4 for pump station

- Ground water level $2.4\text{m} - 2.6\text{m}$ bgl
→ conservatively assumed as 2.6m bgl
- Soil weight of 18 kN/m^3 assumed in liquefaction assessment
- Soil profile appears to be relatively uniform over site:

0 - 2.5m : Sensitive Fine-grained
2.5m - 6m : Clay and silty clay / clay and sandy silt / clay
6m - 12.5m : Various layers of: Sand and silty sand, Silty sand and sandy silt, Clay and silt, Sand, Clay and silty clay

- From this could infer:

Layer 1:	$\rho = 18 \text{ kN/m}^3$
Layer 2: $\phi = 25^\circ$	$\rho = 18 \text{ kN/m}^3$
Layer 3: $\phi = 30^\circ$	$\rho = 18 \text{ kN/m}^3$

- Similar to original design, can probably use these values.

S/S → Use parameters
from Geotech Report

Section Capacities

- Base on NZSEE Assessment Guidelines section C5
- Probable capacities so $\phi = 1.0$
- For now base on reo layout and thicknesses from original calcs, update based on scans if required.
- Bars are deformed, no indication of where laps are assume not in PPHZ
- Take $\mu = 1.25$ for ULS, $\mu = 1$ for SLS
- Reinforcement around corners appear to be anchored by 90° hooks (see P31 of PDF)
- Check development first:

- Largest bars which need to be developed with 350mm are D20s

$$L_{db} = (0.5 \alpha_b f_y / \sqrt{f'_c}) \times d_b$$

f_y should be 324MPa
→ can scale up capacities accordingly

$$= (0.5 \times 1.0 \times 275 \text{MPa} / \sqrt{30 \text{MPa}}) \times 20 \text{mm} = 502 \text{mm}$$

$$\alpha_b = 1.0 \quad (\text{assume this, conservative})$$

$$\alpha_c = 1 + 0.5 (L_m / d_b - 1.5)$$

$$= 1 + 0.5 \times (60 \text{mm} / 20 \text{mm} - 1.5) = 1.75$$

$$\alpha_d = 1.0 \quad (\text{conservative, don't know if transverse reo is on inside or outside of bars})$$

$$L_d = (\alpha_b / \alpha_c \alpha_d) L_{db} \geq 300 \text{mm}$$

$$= (1 / 1.75 \times 1) \times 502 \text{mm} \geq 300 \text{mm}$$

$$= 300 \text{mm} \geq 350 \text{mm} - 60 \text{mm} = 290 \text{mm}$$

Can almost develop without hooks

$$L_{dn} = 0.24 \times \alpha_b \alpha_c \alpha_d f_y d_b / \sqrt{f'_c} \geq 8 d_b$$

$$= 0.24 \times 1 \times 0.7 \times 1.0 \times 275 \text{MPa} \times 20 \text{mm} / \sqrt{30 \text{MPa}} \geq 160 \text{mm}$$

$$= 169 \text{mm} < 290 \text{mm} \rightarrow \text{developed fully}$$

Flexural Capacities

- For now estimate M_{prob} as:

$$M_{prob} = A_s f_y (d - d')$$

- estimate $(d - d')$ as $(H - C) = \text{depth} - \text{cover}$

- IF sections are failing or close to failing then perform a more detailed calc for capacity

Section A-A (floor stronger as 425 thick)

D20s @ 100mm $M_{prob} = 3141 \text{ mm}^2 \times 275 \text{ MPa} \times (350 \text{ mm} - 20 \text{ mm}) = 285 \text{ kNm}$

D20s @ 200mm $M_{prob} = 1570 \text{ mm}^2 \times 275 \text{ MPa} \times (350 \text{ mm} - 20 \text{ mm}) = 143 \text{ kNm}$

Section B-B

20cvr	D20s @ 125	$M_{prob} = 2513 \text{ mm}^2 \times 275 \text{ MPa} \times (250 - 20) =$	159 kNm
	D20s @ 250	$M_{prob} = 1256 \text{ mm}^2 \times 275 \text{ MPa} \times (250 - 20) =$	79 kNm
50cvr	D20s @ 125	$M_{prob} = 2513 \text{ mm}^2 \times 275 \text{ MPa} \times (250 - 60) =$	131 kNm
	D20s @ 250	$M_{prob} = 1256 \text{ mm}^2 \times 275 \text{ MPa} \times (250 - 60) =$	66 kNm

Section C-C

D16s @ 75mm $M_{prob} = 2613 \text{ mm}^2 \times 275 \text{ MPa} \times (300 \text{ mm} - 50 \text{ mm}) = 180 \text{ kNm}$

D16s @ 150mm $M_{prob} = 1306 \text{ mm}^2 \times 275 \text{ MPa} \times (300 \text{ mm} - 50 \text{ mm}) = 90 \text{ kNm}$

Section D-D

D24s @ 100mm $M_{prob} = 4523 \text{ mm}^2 \times 275 \text{ MPa} \times (350 \text{ mm} - 60 \text{ mm}) = 361 \text{ kNm}$

D16s @ 150mm $M_{prob} = 1206 \text{ mm}^2 \times 275 \text{ MPa} \times (350 \text{ mm} - 60 \text{ mm}) = 96 \text{ kNm}$

Section E-E

D16s @ 150mm $M_{prob} = 1206 \text{ mm}^2 \times 275 \text{ MPa} \times (200 \text{ mm} - 60 \text{ mm}) = 46 \text{ kNm}$

D20s @ 200mm $M_{prob} = 1570 \text{ mm}^2 \times 275 \text{ MPa} \times (200 \text{ mm} - 60 \text{ mm}) = 60 \text{ kNm}$

Section F-F

D20s @ 100mm $M_{prob} = 3141 \text{ mm}^2 \times 275 \text{ MPa} \times (300 \text{ mm} - 60 \text{ mm}) = 207 \text{ kNm}$

- Highest platform void
D24s @ 100mm vert $M_{prob} = 361 \text{ kNm}$
D20s @ 100mm horiz $M_{prob} = 250 \text{ kNm}$

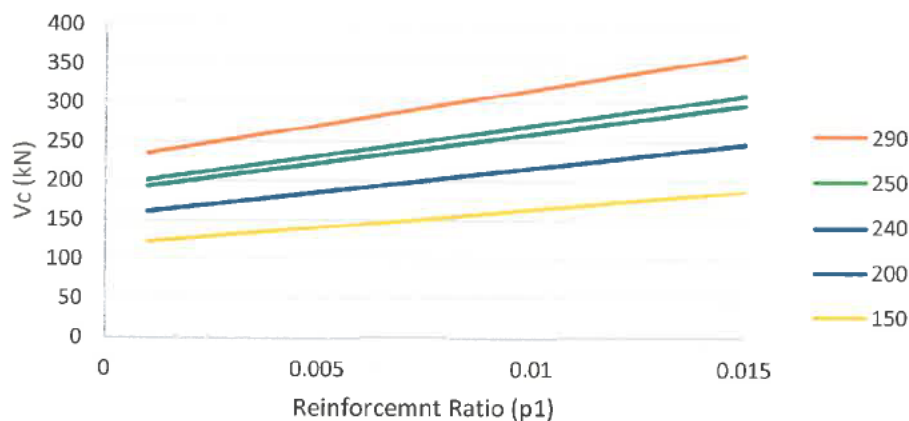
Shear Capacities

- From NZSEE

$$V_{prob} = 0.85 (V_c + V_s + V_N)$$

- Will ignore axial load contribution as this is small (V_N)
- Original calcs seem to imply there is shear reinforcing but probably not at joints (relies on shear friction here)
- Shear Friction may not be possible at joint due to bars already being utilised by moment, need to check in more detail.
- First determine V_c for each thickness

V_c for various reinforcement ratio and effective depths



$$V_c = \alpha \beta \gamma \sqrt{f'_c} (0.8 A_g)$$

- Take $\alpha = 1.0$ (conservative)
- Take $\gamma = 0.29$ (displacement ductility demand is low)

- Determine V_s for where shear steel present

$$V_s = A_v f_y + d/s$$

- Section A-A

$$\text{Roof: } V_s = 392 \text{ mm}^2 \times 275 \text{ MPa} \times 350 \text{ mm} / 100 \text{ mm} = 377 \text{ kN}$$

$$\text{Floor: } V_s = 565 \text{ mm}^2 \times 275 \text{ MPa} \times 425 \text{ mm} / 100 \text{ mm} = 660 \text{ kN}$$

- Section B : None

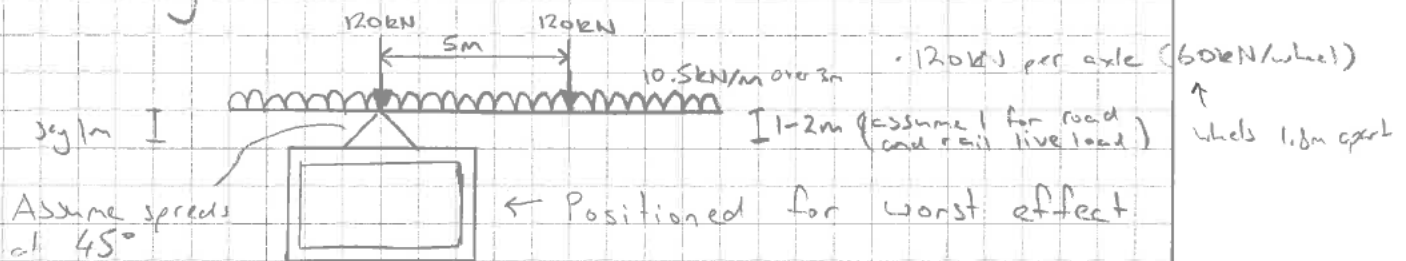
- Drawing of wall details for C-C show no shear reo $\rightarrow$ unlikely to be reo for other walls

- Determine $V_{\text{prob}} = 0.85 (V_c + V_s)$

Section	t (mm)	ρ_s	V_c (kN)	V_s (kN)	V_{prob}
A-A					
Roof	350	0.018	441	377	695
Wall	350	0.013	390	0	332
Floor	425	0.011	369	660	314
B-B					
Roof	250	0.022	336	0	286
Wall	250	0.015	286	0	243
Floor	250	0.015	286	0	243
C-C					
Wall 1+3	300	0.011	249	0	212
Wall 2	300	0.007	280	0	238
D-D					
Wall	350	0.015	361	0	307
E-E					
Wall	200	0.011	168	0	143
Roof	200	0.010	163		139
F-F					
Wall	300	0.013	284	0	241
Platform Wall					
Wall	350	0.015	299	0	254

Load Take-down

- Consider subways as closed boxes
- Consider ramp walls as retaining walls
- No need to check global stability as walls on other side will resist sliding or global overturning through passive pressure.
- Loads to consider:
 - Dead load and earth pressure
 - Live load (surcharge from train, road, columns)
 - Hydro-static pressure
 - Earthquake
- Max load from column near wall in original calc is 14.7 kN (pg. 26 of PDF)
- Use NZTA bridge manual for traffic loads below the road. Use normal loading (HN) as overload unlikely to be concurrently with EQ.



- Train load from W201 Railway Bridge Design Brief (003)
 - 250 kN/axle, 1.5m spacing → will essentially apply a UDL

$$250 \text{ kN} / 2 = 125 \text{ kN} / \text{track}$$

- Say tracks are 2m apart, should be able to consider tracks as independent

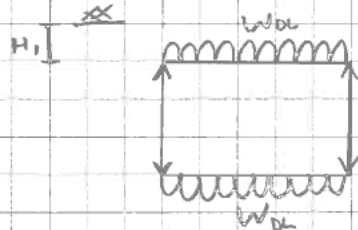
$$125 \text{ kN} / (1.5 \text{ m} \times 2 \text{ m}) = 42 \text{ kPa}$$

↑
1.5m spacing between loads

↑
spreads 1m either side into page

- Subway Loads, consider 1m strip

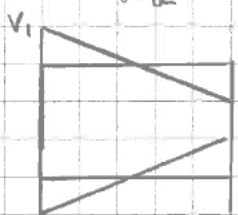
- Dead load



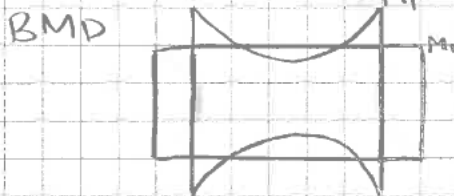
$$W_{DL} = \gamma_{soil} \times H_1 \times (1m \text{ width})$$

$$= 20kN/m^3 \times 2m \times 1m$$

$$=$$



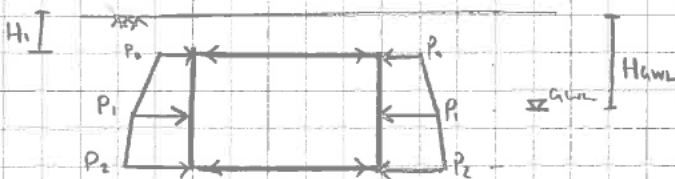
$$V_1 = W_{DL} L / 2$$



$$M_1 = W_{DL} L^2 / 12$$

- Ground water table approx 2.5m below ground level. Max depth above subway is about 2m, min about 1m. GWL needs to be confirmed by geotech → confirmed as 2m BGL

- Earth Pressure, Assume rigid wall



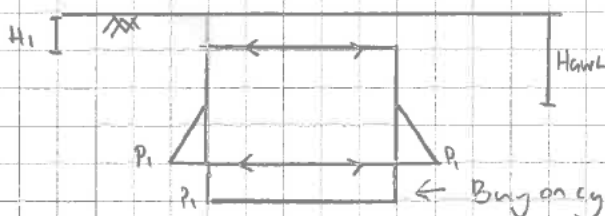
$$P_0 = K_0 H_1 \gamma_{soil}$$

$$P_1 = K_0 H_{aew} \gamma_{soil}$$

$$P_2 = K_0 (H_1 + 3.2m - H_{aew}) (\gamma_{soil} - \gamma_{water}) + P_1$$

- Solve for SFD and BMD once geotech parameters confirmed

- Water pressure

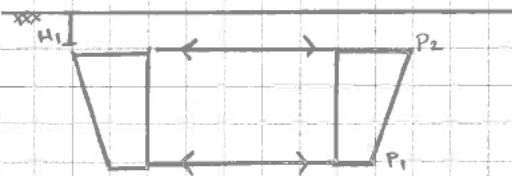


$$P_1 = (H_1 + 3.2m - H_{aew}) \gamma_w$$

- No need to check buoyancy as obviously OK because structure still buried

- Solve for SFD and BMD once geotech known

• Earthquake loads



$$P_1 = 0.5(10)\delta_s(H_1 + 3.2m)$$

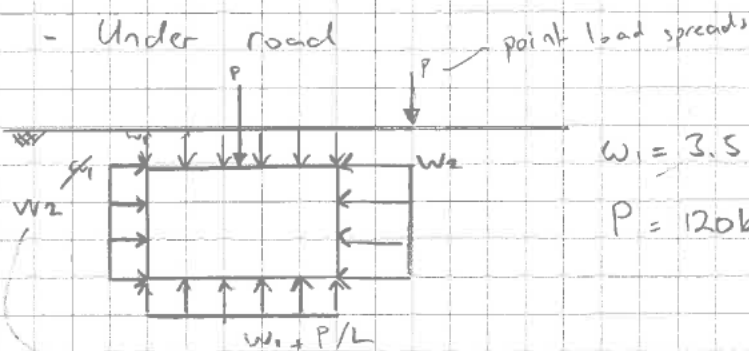
$$P_2 = 1.5(10)\delta_s \times 3.2m$$

• Ignore water effect for now, check if critical

- Solve for SFD and BMD once geotech known

• Live loads

- Under road



$$w_1 = 3.5 \text{ kN/m}$$

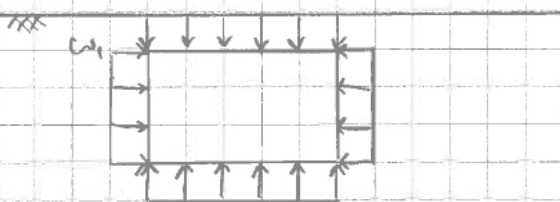
$$P = 120 \text{ kN} / 2m = 60 \text{ kN for 1m strip, will actually spread but for now treat as point load}$$

must = w_2 for equilibrium

$$w_2 = 3.5 \text{ kN/m} + 120 \text{ kN} / 4m^2 \times 1m = 33 \text{ kN/m}$$

assume point load spreads over $2m \times 2m$ area

- Under train



$$w_1 = 42 \text{ kN/m}$$

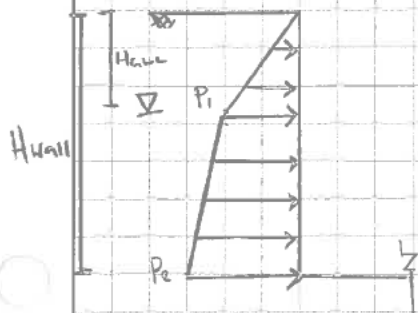
• NZTA Railway bridge Design EQ combination is

$$D + EP + EQ \text{ (Dead + Earth Pressure + EQ)}$$

→ But will include live load as this is conservative

• Wall loads (probably flexible)

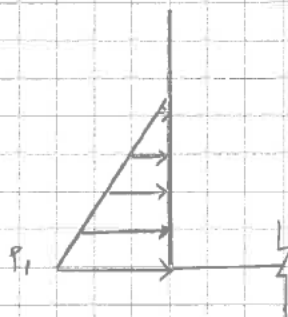
• Earth pressure



$$P_1 = K_a \gamma_{soil} H_{wall}$$

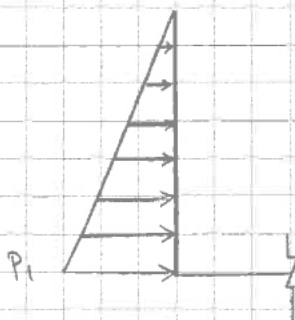
$$P_2 = P_1 + K_a (\gamma_{soil} - \gamma_w) (H_{wall} - H_{water})$$

• Water Pressure



$$P_1 = \gamma_w (H_{wall} - H_{water})$$

• Earthquake



$$P_1 = \Delta K_{ae} \gamma_{soil} H_{wall}$$

$$\text{where } \Delta K_{ae} = K_{ae} - K_a$$

• Load from column next to wall

- Columns founded on piles that likely go below wall. → Will have minimal surcharge from vertical load

- Max lateral load appears to be 30kN each direction from original piles. This was due to wind load which was critical. Conservatively apply 30kN as point load at top of wall.

• Seismic Parameters class D - TBC - confirmed by geotech
 $(1.0) = 1.12 \times 0.4 \times 1.3 \times 1.0$ ← lower limit ← 1/1000 Return

$= 0.58g$ for ULS (0.38 in original calcs, 50% higher)

$= 1.12 \times 0.4 \times 0.25 \times 1.0$

$= 0.112g$ for SLS

$A_{topo} = 1.0$ → not near cliff or ridge top

$W_d = 1.0$ for subway as rigid

$= 0.5$ for walls as flexible and supporting station structure foundation (Case 3 from MBIE Guidance)

$R_n = (1.0)$ for subway

$= 0.5(1.0)$ for walls

• Geotech Results

Layer	γ_s (kNm)	ϕ (°)	c' (kPa)	Max Depth (m)
Fill	20	36	0	1m
Silt/Clay	17	28	5	3m
Sand with silt	18	30	1	8m
Sandy gravel	20	36	0	13m
Silty Clay	19	27	5	713m

→ These are north end results which are most likely more relevant to the subway

→ Cohesion is very low → ignore

→ For now use most conservative parameters, as all layers relatively similar (deepest wall is < 8m)

$\gamma_s = 20 \text{ kNm}$, $\phi = 28^\circ$

→ GWL to be 2m below ground level

→ From original drawings it appears subways are 1m under the road and railway, with a max of 2m under the platform. → For now assume 2m everywhere as this is conservative

$$K_0 = 1 - \sin \phi$$

$$= 1 - \sin(28)$$

$$= 0.53$$

$$K_a = \frac{1 - \sin \phi}{1 + \sin \phi}$$

$$= \frac{1 - \sin(28)}{1 + \sin(28)}$$

$$= 0.36$$

Seismic Active Earth Pressure

$$k_h = 0.5 \times 0.58$$

$$= 0.29$$

$$0.5 = W_d$$

$$R_v = (v(T_v) S_p = 1.12 \times 0.4 \times 1.3 \times 1.0 \times 1.0$$

$$= 0.58$$

$$\psi = 0.012 \approx 0$$

$$\delta = 2/3 \times \phi = 2/3 \times 28^\circ = 18.6^\circ$$

$$\alpha = 0^\circ \quad - \text{vertical wall}$$

$$\beta = 0 \quad - \text{no backfill slope}$$

$$K_{ae} = \frac{\cos^2(28^\circ)}{\cos(28^\circ + 18.6^\circ) \left(1 + \sqrt{\frac{\sin(28^\circ + 18.6^\circ) \sin(28^\circ)}{\cos(18.6^\circ)}} \right)^2}$$

$$= 0.44$$

$$\Delta K_{ae} = 0.44 - 0.36 = 0.08$$

Earthquake Lateral Load from Column

- Re-check lateral load
- Dead load from original calcs = 1740 kN
- V_b using current Cal assuming $m=1.25$,
probably conservative, as columns likely more ductile.
 $V_b = 0.58 \times 1740 \text{ kN} = 1009 \text{ kN}$
- About 36 columns supporting roof
- Tallest columns next to walls $\rightarrow$ least stiff so
will pick up less load assuming roof acts as
diaphragm.
- For now assume equal load to all column,
 $1009 \text{ kN} / 36 = 28 \text{ kN}$
- Will be 100 ER-X + 30 ER-Y
 - $\rightarrow$ Still less than original wind load
 - $\rightarrow$ Much less than 50 kN
 - $\rightarrow$ Apply 40 kN at top of wall as point
load, should be very conservative.
 - $\rightarrow$ Load will not all be applied at top and
will spread over more than 1m of wall

Capacity vs Demand

- For load case $G+EP+WP+EQ+Q$
→ Conservative as Q not likely to be present during 1000 yr event

Subway under train

	M^*	V^*	M_{prob}	V_{prob}	% NBS
Roof					
Middle	71 kNm	0 kN	285 kNm	695 kN	100
Corner	104 kNm	178 kN	285 kNm	695 kN	100
Wall					
Middle	26 kNm	0	143 kNm	332 kN	100
Corner	104 kNm	176	285 kNm	332 kN	100
Floor					100

← Thicker so works by inspection

Subway under road

	$M^*(kNm)$	$V^*(kN)$	$M_{prob}(kNm)$	$V_{prob}(kN)$	% NBS
Roof					
Mid	68	30	131	286	100
Corner	86	160	131	286	100
Wall					
Mid	30	0	66	243	100
Corner	86	160	131	243	100
Floor (same as roof)					100

Ramp Wall

	$M^*(kNm)$	$V^*(kN)$	M_{prob}	V_{prob}	% NBS
EAST	304	138	361	307	100
WEST	304	138	207	241	68

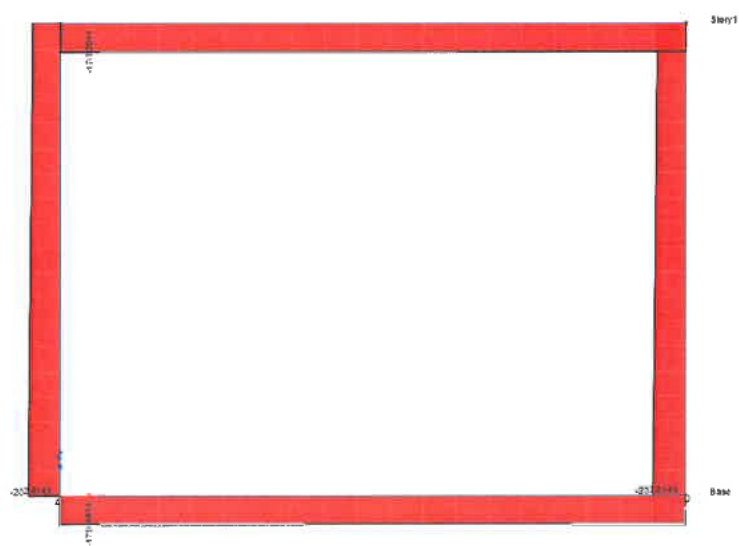
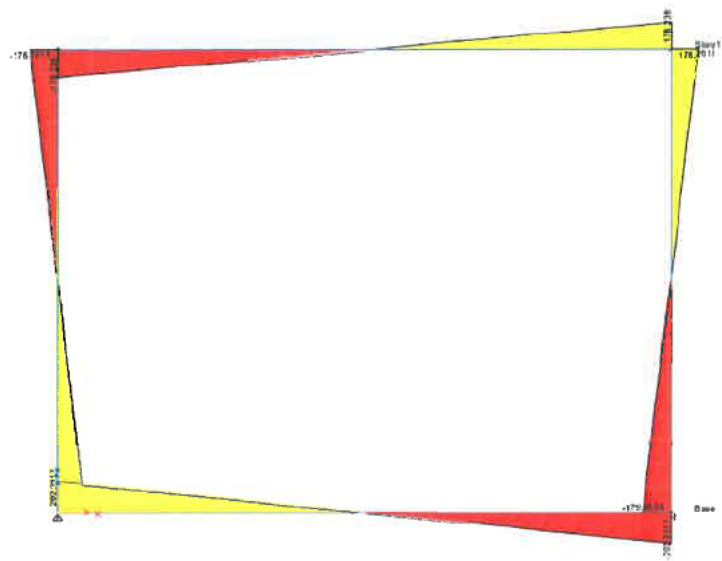
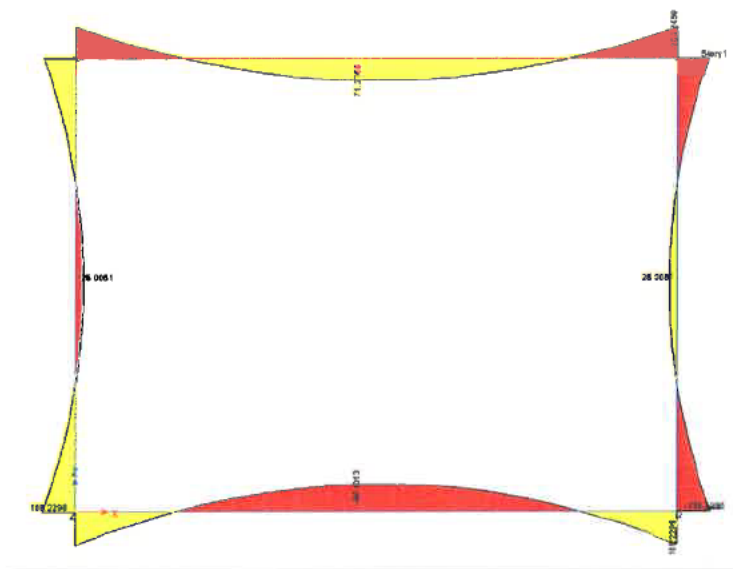
- Note: Estimate of moment demand is conservative, wall definitely > 68% NBS

Void Wall	$M^*(kNm)$	$V^*(kN)$	$M_{prob}(kNm)$	$V_{prob}(kN)$	% NBS
	472	199	361	254	76%

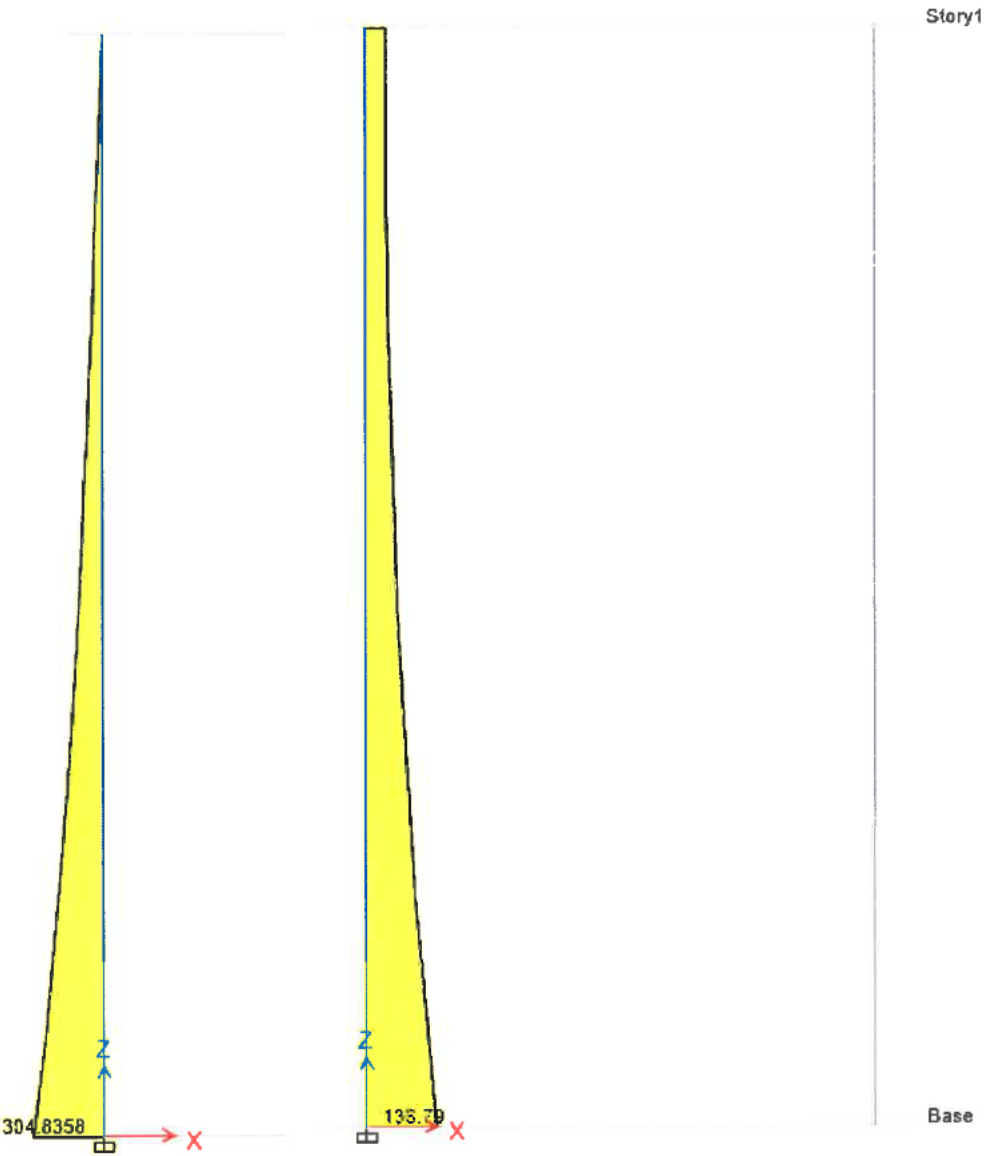
→ This wall has been designed to span in 2-directions

→ Check if 25% of load can span in the other direction

Subway under Train



Ramp Wall Actions



Void Wall Actions

