+64 4 385 0019
dtcwgtn@dunningthornton.co.nz

20 Customhouse Quay
PO Box 27-153, Wellington 6141

# Memorandum

| | |
|---|---|
| Title | **Subway Investigation** |
| Job Number | **8962** |
| To/Attn | **GWRC c/- RCP** |
| From | s7(2)(a) [redacted] |
| Date | **13/10/2025** |
| Attachments | **WSP (5pgs) T&T (7pgs)** |

## 1 Background

The existing concrete subway tunnel at Waterloo Station was built ~1989. Given the age of the tunnel, and the unlikely appetite to build a new tunnel as part of redevelopment works, two aspects of the existing tunnel needed to be investigated, namely,

- Durability performance of the tunnel
- Performance of the tunnel under a seismic event with widespread liquefaction.

## 2 Tunnel Durability

Investigations were conducted as the existing concrete tunnel was subject to durability concerns, given its age, low cover to reinforcing and continued water ingress. Two cores were taken from representative locations and analysed by WSP labs for carbonation of the concrete and chloride concentration.

As noted in the attached WSP report covering the analysis of the two concrete core samples taken, the carbonation front is within the concrete cover zone and the chloride concentration is relatively low and so degradation of the internal reinforcing is very unlikely to be accelerated in the near term. They did note that the chloride concentration is higher than typically expected, but still below the threshold of likelihood of corrosion developing 'some risk' as per Figure 1.

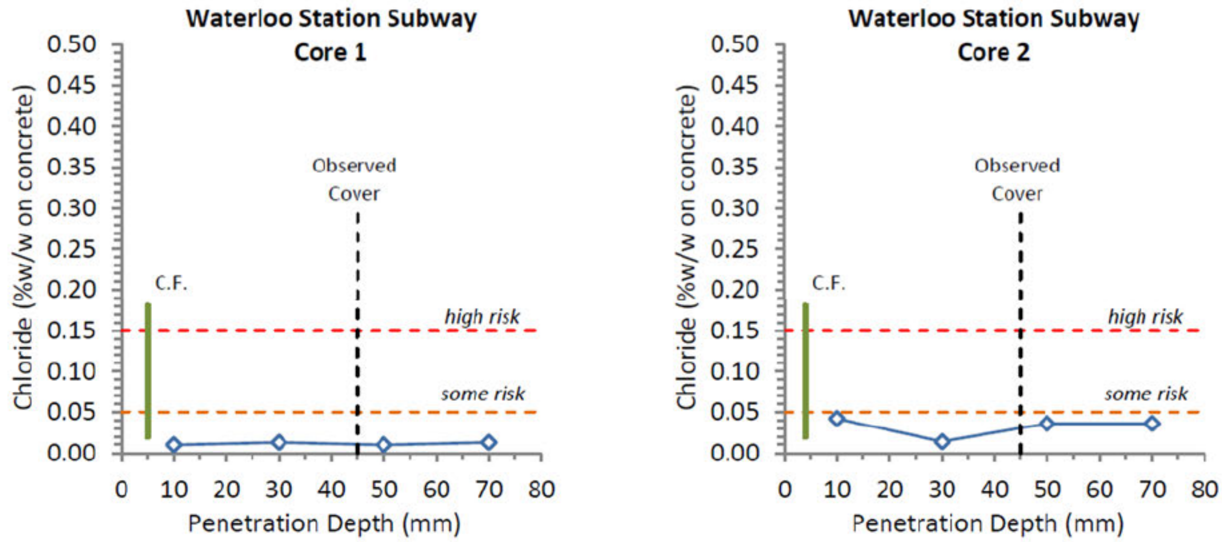



***Figure 2. Chloride contamination profiles and position of the carbonation front (from the external concrete face) measured on the provided core specimens for evaluation of the current and future reinforcement corrosion risk.***

Figure 1 Excerpt WSP Durability Assessment of Waterloo Station Subway Cores 21 Aug 2025

This was among the best possible outcomes, and implies that from a concrete durability perspective, there are no imminent concerns. Barring any material change to the tunnel itself (ie a medium-large seismic event or other cause of overloading of tunnel which results in substantial cracking to the tunnel) we would anticipate that the tunnel will continue to perform well from a durability perspective for the remainer of its original design life (ie another 14 years) and beyond.

Based on results of the investigation, it appears that the original tanking is performing well at preventing water ingress generally, and the current steady flow of water through the tunnel walls is occurring at discreet points.

# 3 Liquefaction Assessment

A Detailed Seismic Assessment (DSA) had previously been carried out (by others) on the tunnel but this did not address performance under liquefaction.

As per the attached T&T report, based on site investigation recently completed, large scale liquefaction is expected at an event corresponding to ~60%NBS if considering the tunnel as an Importance Level 2 structure (~45%NBS if considered IL3.) The North portion of the tunnel is expected to suffer worse imposed liquefaction pressures compared to the south portion.

Our analysis shows that under the loads anticipated by T&T once liquidation occurs, the majority of the tunnel has sufficient strength to ‘hold together’ as a box. However, it is likely that the tunnel will vertically displace (ie rise up toward the surface) under these liquefaction loads. As the liquefaction is either present or not, this is not presented as a gradual buildup of pressure (ie we cannot structurally pinpoint a lower %NBS that this ‘starts’ to occur, without fully occurring.) This heave will impact the road and rail lines above, but should be considered in the context of widespread liquefaction across the area (ie functionality is likely impacted away from the tunnel locale also.)

We would expect to see damage to the walls local to the stairs/ramps once this liquefaction is triggered, as in these areas the tunnel is no longer a box. Similarly, but to a lesser extent at the small lightwell locations, the upper portion of the walls has insufficient capacity to resist the full liquefaction load as predicted by T&T. It is likely that failure of these elements will locally relieve pressure, so this is not seen to be a failure which would lead to significant loss of life (ie global failure.) In such an event the tunnel is likely to require wholesale replacement.

Examination of the existing drawings show that generally the tunnel is independent of other surrounding structures, and should generally be free to move the ~160mm horizontally predicted by T&T. The existing drawings are ambiguous as to the exact setout of roof columns in relation to the tunnel, but generally there appears to be sufficient space between tunnel wall and pile supporting station roof columns such that they will act as two independent structures.

There are a small number of roof columns which are built into ramp walls, these are likely to also be subject to some lateral pressure under liquefaction and therefore impact on the roof structure. However, it is currently proposed that this roof structure will be removed, so this is not seen to be a long term risk.

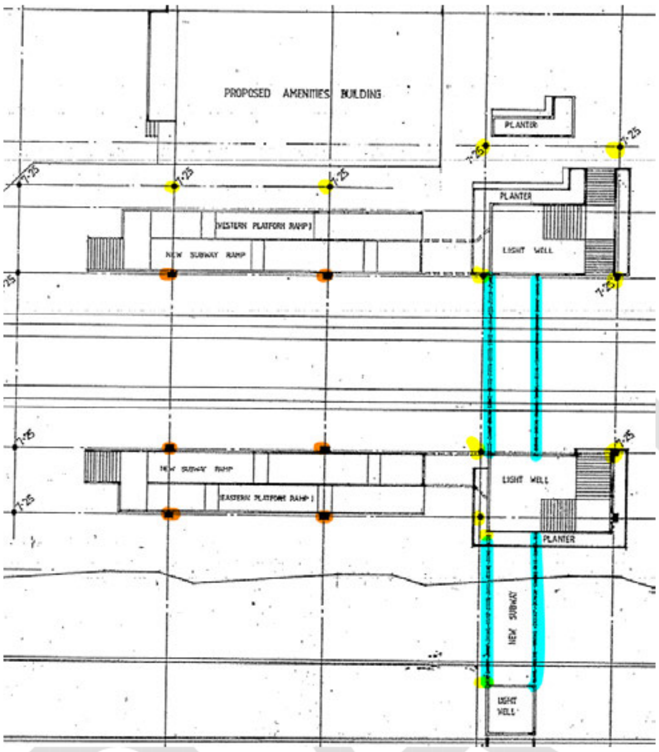



Figure 2 Excerpt Existing Drawing C55 Column Layout Plan with tunnel outline (blue), independent columns (yellow) inbuilt columns (orange)

## 4 Conclusions & Recommendations

Neither aspect of the further tunnel investigation has uncovered items which are structurally concerning in the immediate sense. If the tunnel is proposed to continue to be utilised in the longer term (+10-15 years) there may be advantages in considering strengthening work to the tunnel to relieve liquefaction pressures. However, this should be considered in the context of the wider rail/road network vulnerabilities. If liquefaction occurs sufficient to result in the tunnel uplift, the corresponding area is also going to be heavily impacted.

If the tunnel is to be decommissioned, but not removed, we would recommend that as part of the decommissioning work, work is undertaken to mitigate liquefaction uplift potential where possible. This may take the form of pressure relief holes in the structure or sufficiently dense infill to the tunnel.

We note that any alterations to the existing tunnel will require careful temporary works/ construction staging in order to manage the likely water ingress.

Similarly it would be prudent to consider likely lateral movement when setting out any new structure in the vicinity of the tunnel.

Hope this provides the information you require, please feel free to contact us for any further discussion.

[redacted] s7(2)(a)

[redacted] s7(2)(a)
Associate

**WSP Research & Innovation Centre**
**33 The Esplanade, Petone**
**PO Box 30 845, Lower Hutt 5040**
**New Zealand**

**+64 4 587 0600**
**www.wsp.com/nz**

21 August 2025

s7(2)(a) [redacted]
Senior Project Manager

RCP
Level 4
40 Johnston Street
Wellington Central
Wellington 6011

Ref: 5-24G23.G0

# Durability Assessment of Waterloo Station Subway Cores

Dear s7(2)(a) [redacted]

This letter provides the results of a durability assessment undertaken on two recently-received concrete core samples, reportedly extracted from the pedestrian subway at Waterloo Railway Station.[1]

It is understood that this structure was built in the late 1980s and, as part of a wider options report being completed for the station, testing was desired to ascertain whether there was any durability threat present that might compromise the foreseeable residual life of the structure.

Analyses were completed to measure carbonation depth and the extent of concrete contamination present; development of widespread reinforcement corrosion due to these deterioration mechanisms is considered to be the most probable limitation on the potential longevity of a reinforced concrete structure under New Zealand conditions according to NZS 3101: 2006 ‘Concrete Structures’.[2] At the request of Dunning Thornton,[1] risk analysis was primarily completed with respect to the exterior face of the concrete, which was unambiguously discernible by the remnants of a Bituthene® or similar asphaltic waterproofing membrane adhered to one end of each core.

## 1 Methodology

### 1.1 Carbonation

The cores were initially sectioned longitudinally with a water-cooled diamond saw to expose a fresh concrete surface suitable for carbonation testing that had not previously been directly exposed to the atmosphere. These cut surfaces were permitted to briefly air dry before being sprayed with a 1% w/v solution of phenolphthalein reagent.[3] Functioning as an acid-base indicator, phenolphthalein solutions are colourless below a pH of 8.5 but develop an intense pink to magenta colour where the pH is greater than ca. 9. The depth of concrete beneath the surface that is not stained by the application of the phenolphthalein solution is considered to be carbonated, i.e. depleted in alkalinity due to the complete consumption of calcium hydroxide within the cementitious matrix through reaction with atmospheric $CO_2$. Any mild steel reinforcement lying within this carbonated zone is no longer passivated by a highly-alkaline surrounding environment and hence becomes potentially susceptible to carbonation.

### 1.2 Chloride Contamination

Following completion of the carbonation testing, the split cores were then sub-sectioned transversely into four nominal 15 mm deep slices, representing depth increments from 0 – 20 mm, 20 – 40 mm, 40 – 60 mm & 60 – 80 mm through the cover region of the concrete, beneath the externally-exposed surface.

[1] Nicki Vance (Dunning Thornton), *personal communication*.

[2] Standards New Zealand. NZS 3101:2006. *Concrete Structures Standard. Part 1: The Design of Concrete Structures & Part 2: Commentary*. Wellington, New Zealand.

[3] RILEM Recommendation CPC-18. 1998. ‘Measurement of hardened concrete carbonation depth’. *Materials & Structures* 21 no 6, pp 453 – 455, November 1998.

The concrete slices were dried, crushed to a fine powder and analysed by XRF (x-ray fluorescence spectroscopy) to determine the total chlorine content as a percentage of the dry mass of concrete (%w/w). Use of XRF by a suitably experienced IANZ-accredited laboratory is a permitted method of chloride analysis for determining risk of reinforcement corrosion under NZS 3101.

Chloride ions in sufficient concentration are unique and specific agents for the corrosion of mild steel reinforcement; they destroy the protective surface oxide layer that ordinarily develops on steel surrounded by alkaline concrete, a process known as 'depassivation'. It is generally accepted that corrosion will only occur once the chlorides accumulate beyond a critical concentration in the concrete surrounding the reinforcement. Consequently it is important to understand the intensity and distribution of any chloride contamination present. Chloride ions can be introduced either through progressive infiltration by environmental sources or cast-in contamination that typically arises from incorporation of poorly-washed marine aggregate or calcium chloride based set accelerating admixtures in the original mix design.

In practice, because of the wide variety of possible concreting materials and exposure conditions, there is no unique concentration of chloride ions that invariably cause reinforcement depassivation.[4] Current thinking identifies a range of probabilistic risk levels for corrosion with particular concentrations, as shown in Figure 1;[5] the broad diffuse nature of the plot emphasises the range of uncertainty associated with these assertions. For simplicity, this report considers two discrete thresholds for corrosion suggested by the UK Concrete Society: Chloride ion concentrations above 0.05% by mass by mass of concrete at the depth of the reinforcement can be thought of as sufficient to present '*some risk*' of depassivation with subsequent corrosion developing, whereas concentrations above 0.15% indicate a '*high risk*'.[6]

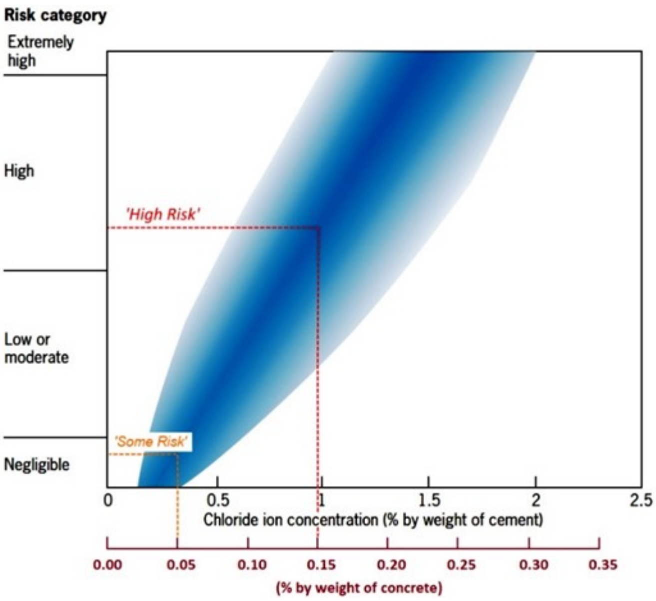



***Figure 1.  Estimated risk of steel reinforcement corrosion associated with ingress chlorides (adapted from BRE Digest 444 Part 2 to account for chloride analyses determined on percent concrete rather than cement).***

4 Glass, G. K. & Buenfeld, N. R. 1995. 'Chloride threshold levels for corrosion-induced deterioration of steel in concrete'. RILEM International Workshop on Chloride Penetration in Concrete, St-Remy-Les-Chevreuse, France, pp 429 – 440.

5 BRE. 2000. Digest 444 Part 2. *Corrosion of Steel in Concrete: Investigation and Assessment*. Watford, United Kingdom.

6 UK Concrete Society. 1984. *Technical Report 26 Repair of Concrete Damaged by Reinforcement Corrosion*. Camberley, United Kingdom.

## 2 Results

Table 1 summarises the results obtained from measurement of the carbonation depth and extent of chloride contamination in the cores.

To aid interpretation of this numerical data, the results for the individual cores are also depicted graphically as Figure 2. In these corrosion risk plots, the chloride profiles (i.e. the measured concentration vs. the midpoint of the sampling depth increment) are plotted in blue and the orange and red dashed horizontal lines indicate the UK Concrete Society's chloride concentration thresholds for various probabilities of the onset of corrosion. Chloride contamination levels that lie above the risk threshold at, or beyond, the depth of the concrete cover over the reinforcement indicates a potential threat of reinforcement corrosion.

The maximum depth of the carbonation front beneath the exterior surface of the concrete is illustrated by the position of the solid green vertical bar annotated with the label 'C.F.'. Where the carbonation front has progressed as far as, or beyond, the depth of the reinforcement, these bars lie in alkali-depleted concrete and are consequently also potentially vulnerable to corrosion.

***Table 1. Summary of durability analysis of supplied cores.***

| **Core Identifier** | **Reinforcement Cover** (mm) | **Maximum Carbonation Depth** (mm) | **Chloride Analysis** | |
|---|---|---|---|---|
| | | | **Depth Increment** (mm) | **Concentration** (%w/w on concrete) |
| 1 | 45 mm | 5 mm<br>*from exterior (membrane coated) face* | 0 – 20 | 0.010 |
| | | | 20 – 40 | 0.013 |
| | | | 40 – 60 | 0.010 |
| | | | 60 – 80 | 0.013 |
| 2 | 45 mm | 4 mm<br>*from exterior (membrane coated) face*<br>13 mm<br>*From interior face*<br>*Patchy carbonation visible along margin of apparent construction joint extending full length of core* | 0 – 20 | 0.042 |
| | | | 20 – 40 | 0.014 |
| | | | 40 – 60 | 0.036 |
| | | | 60 – 80 | 0.036 |

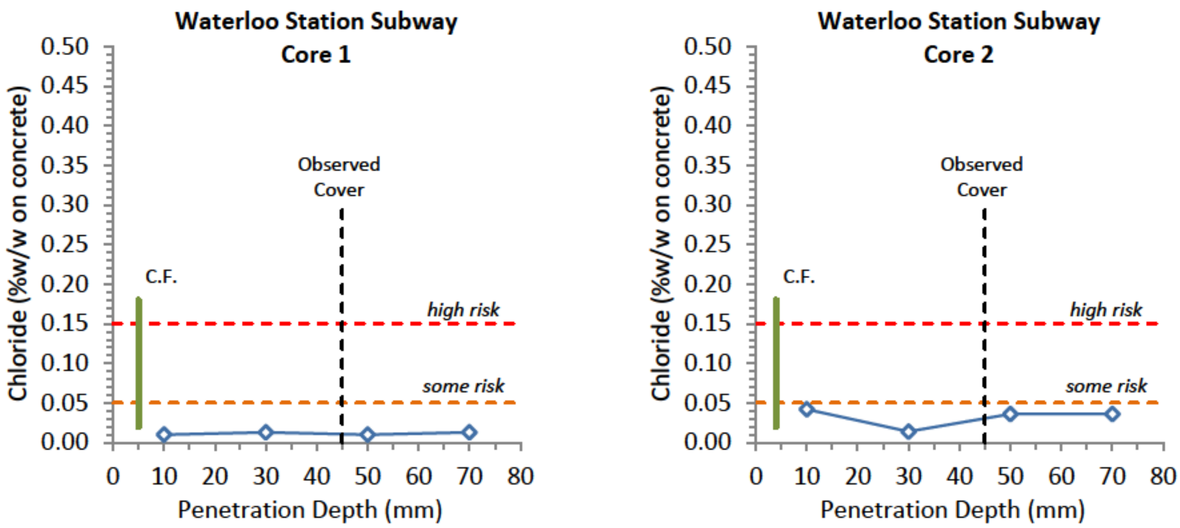



***Figure 2.*** ***Chloride contamination profiles and position of the carbonation front (from the external concrete face) measured on the provided core specimens for evaluation of the current and future reinforcement corrosion risk.***

Illustrative photographs of the longitudinal slices cut through the cores to determine carbonation depth recorded following application of the phenolphthalein indicator solution are reproduced as Figure 3. The coarse aggregate is a quite heavily-weathered alluvial greywacke with a maximum particle diameter of 13 mm. Core 2 has intersected a prominent construction joint extending the full length of the specimen. The reinforcement intersected in both cores is 20 mm diameter deformed bars with a measured cover of 45 mm from the exterior surface. The original interior face of Core 1 is not preserved with the sample delivered displaying a cut surface.

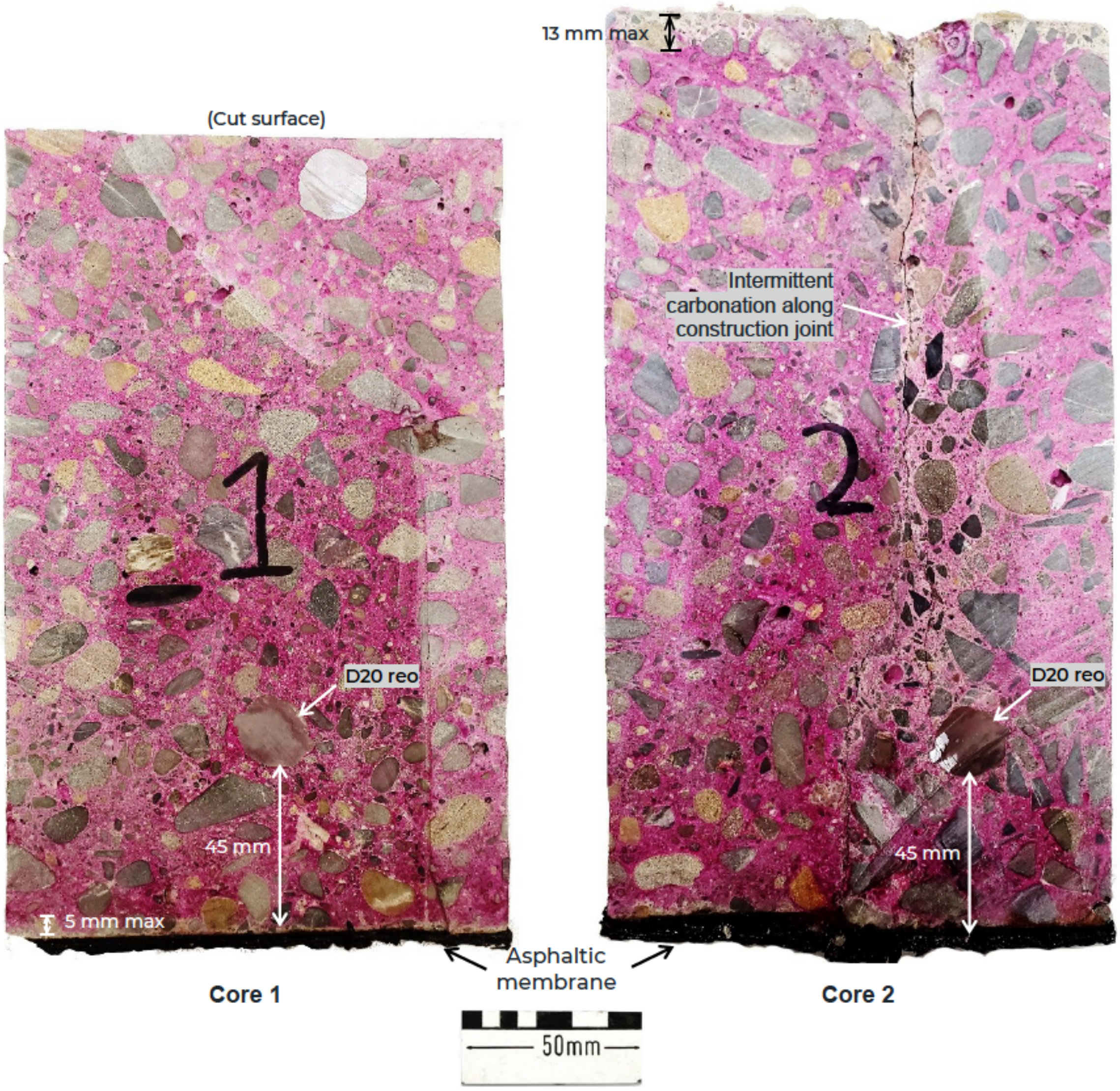



***Figure 3.*** ***Longitudinal slices through cores photographed after application of phenolphthalein indicator to ascertain carbonation depth; the exterior face of the core is at the bottom. Any regions of the concrete that remains colourless – rather than being stained a magenta colour – is effectively carbonated.***

## 3 Interpretation

It can be seen that:

i. Carbonation of the concrete is minimal with no appreciable loss of alkalinity from either the exterior or interior faces of the cores that would threaten the embedded reinforcement at the observed cover depth.

ii. There is no obvious chloride concentration profile through the cover concrete that would suggest permeation from an external environmental source that will accumulate further in the future. The background concentration in Core 2 is typically somewhat more elevated than would be expected from the normal contribution of routinely used Wellington ready-mix concrete constituents but at an observed maximum of 0.042% w/w is not sufficiently high as to pose a realistic risk for initiation of reinforcement corrosion providing the bars are thoroughly encapsulated within well-consolidated concrete. This appears to be the case, with no appreciable excessive voidage or gross compaction defects noted to affect the supplied cores.

iii. Consistent with these findings, there is no evidence for active corrosion initiation on any of the reinforcing bars intersected by the cores.

Based on the durability analyses completed, there is no evidence for the presence of a systemic durability threat that would compromise the foreseeable future serviceability of the subway structure from which the cores were extracted. This statement is contingent upon the following caveats:

- WSP was not involved in the selection of the core locations and has no knowledge of how representative the particular specimens supplied for testing may be of the durability performance or exposure environment of the subway in its entirety.
- It is assumed that the environment to which the concrete has historically been exposed remain comparable in the future.

I trust this information is helpful. Please contact us if you have any queries regarding the contents of this report.

Prepared by: Reviewed by:

s7(2)(a) [redacted]

**Senior Concrete Technologist** **Laboratory Technician**

# 1. Existing Subway

In accordance with guidance published on the MBIE website, ground shaking hazard to be considered in geotechnical seismic assessment of existing building is based on Module 1 (Version 0, 2016). Peak ground acceleration (PGA) for the ultimate limit state (ULS) for both IL2 and IL3 structures is set out below.

- ULS(IL2, Subsoil Class D) PGA = 0.35g.
- ULS(IL3, Subsoil Class D) PGA = 0.45g.

The subway is founded within Layer 2. Widespread liquefaction has been identified within this layer in a >0.2g, M7.7 earthquake (approx. 60% ULS(IL2) or 45% ULS(IL3) earthquake shaking). Geotechnical consequences to the subway include:

1. Liquefaction induced uplift pressures on the base of the structure.
2. Large seismic earth pressures on the subway walls due to liquefied soils and cyclic displacement.
3. Post liquefaction free field settlement resulting in differential settlement of structure.

Geotechnical parameters for these are provided in the following section.

## 1.1 Uplift pressure on base slab

Liquefaction induced uplift pressures will be imposed on the base of the subway. Uplift pressures in the order of 85 kPa is possible. This pressure would likely reduce with upward movement. If there are no restraint to the subway, hundreds of millimetres of total and differential displacement of the subway is possible.

## 1.2 Seismic earth pressures on walls

Liquefaction induced lateral ground movements (cyclic displacement) is possible in an earthquake where widespread liquefaction is triggered. Three scenarios should be considered to represent the ground behaviour during different stages of an earthquake. These scenarios are described in Table 1-1.

**Table 1-1: Ground lateral behaviour during earthquake**

| Scenario | Description | Wall pressures |
|---|---|---|
| 1 | Start of earthquake. No liquefaction. | • Stiff wall earth pressures, see Figure B1 to B4 (1); plus<br>• 100% structure inertia load distributed over wall area. |
| 2 | Liquefaction triggered. No lateral ground movement | • Liquefied active earth pressures, see Figure B1 to B4; plus<br>• 100% structure inertia load distributed over wall area. |
| 3 | Liquefaction with cyclic (lateral) ground displacement. | • Liquefied passive earth pressures, see Figure B1 to B4. Pressures apply up to 160mm lateral displacement of the structure. Beyond 160mm, Scenario 2 earth pressures apply. |

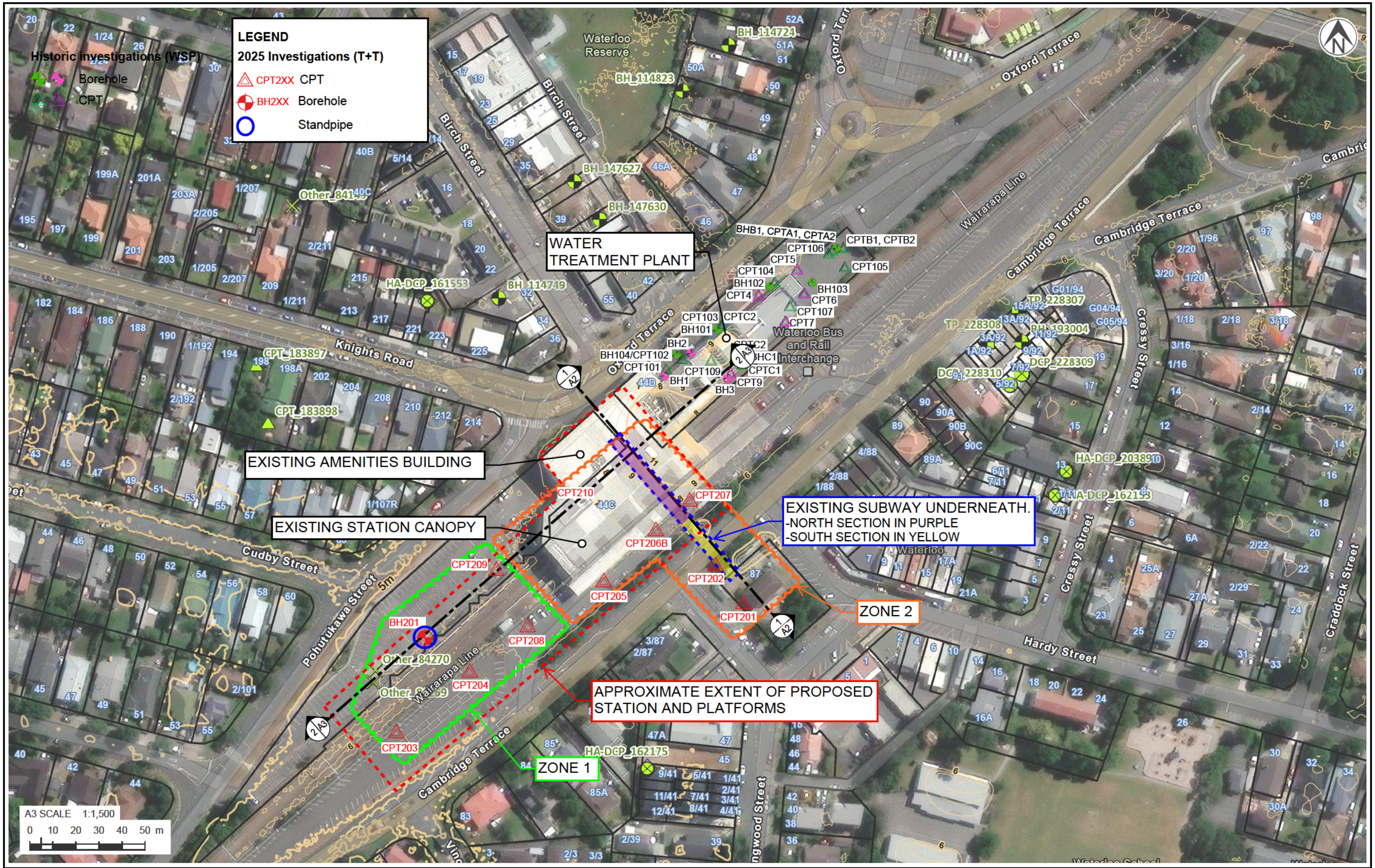
Note 1) Stiff wall earth pressures provided assume the structure is restrained by the station building. If not restrained, then only static earth and groundwater pressures apply (structure moves with the ground).

## 1.3 Post liquefaction free field settlement

The structural design should consider a total post-earthquake free field settlement of 200mm and a differential settlement of 150mm over 5m due to re-consolidation of liquefied soils.

COPYRIGHT ON THIS FIGURE IS RESERVED DO NOT SCALE FROM THIS FIGURE

Tonkin + Taylor Map Viewer (NZ) Date: 25/02/2025 12:13 PM




LOCATION PLAN

Tonkin+Taylor

*Exceptional thinking together* www.tonkintaylor.co.nz

NOTES:

CRS: NZGD 2000 New Zealand Transverse Mercator Credits: Tonkin & Taylor Group , Esri Community Maps Contributors, LINZ, Stats NZ, Esri, TomTom, Garmin, Foursquare, METI/NASA, USGS, Earthstar Geographics, Maxar, LINZ, Stats NZ, Esri, TomTom, Garmin, Foursquare, METI/NASA, USGS

| | | |
|---|---|---|
| PROJECT No. | 1098196 | |
| DESIGNED | BHR | FEB.25 |
| DRAWN | -WEB- | FEB.25 |
| CHECKED | | |
| APPROVED | | DATE |

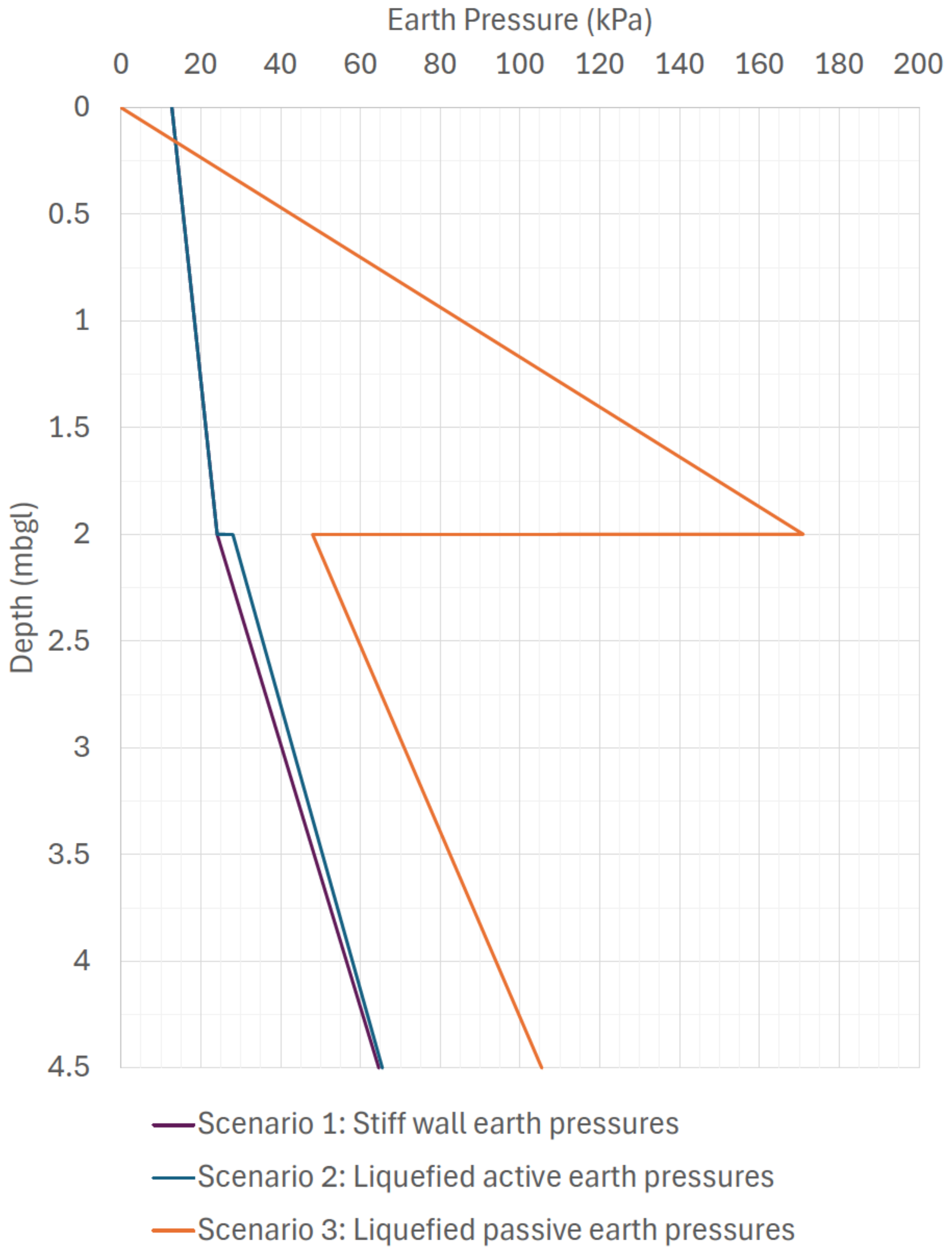
CLIENT GREATER WELLINGTON REGIONAL COUNCIL

PROJECT WATERLOO STATION

TITLE SITE INVESTIGATION PLAN

SCALE (A3) 1:1,500 FIG No. FIGURE A1. REV 0

| | | |
|---|---|---|
| Tonkin+Taylor | **Client:** | GWRC |
| | **Project Name:** | Waterloo Station |
| | **Project No.:** | 1098196.1 |
| | **Structure:** | Existing Subway |




| | |
|---|---|
| **Figure B1** | **Earth Pressures on Subway Walls - North Section (IL2)** |
| Calc/Chk: BHR/ELC | |
| Date: 16/09/2025 | |

| Tonkin+Taylor | | |
|---|---|---|
| | **Client:** | GWRC |
| | **Project Name:** | Waterloo Station |
| | **Project No.:** | 1098196.1 |
| | **Structure:** | Existing Subway |

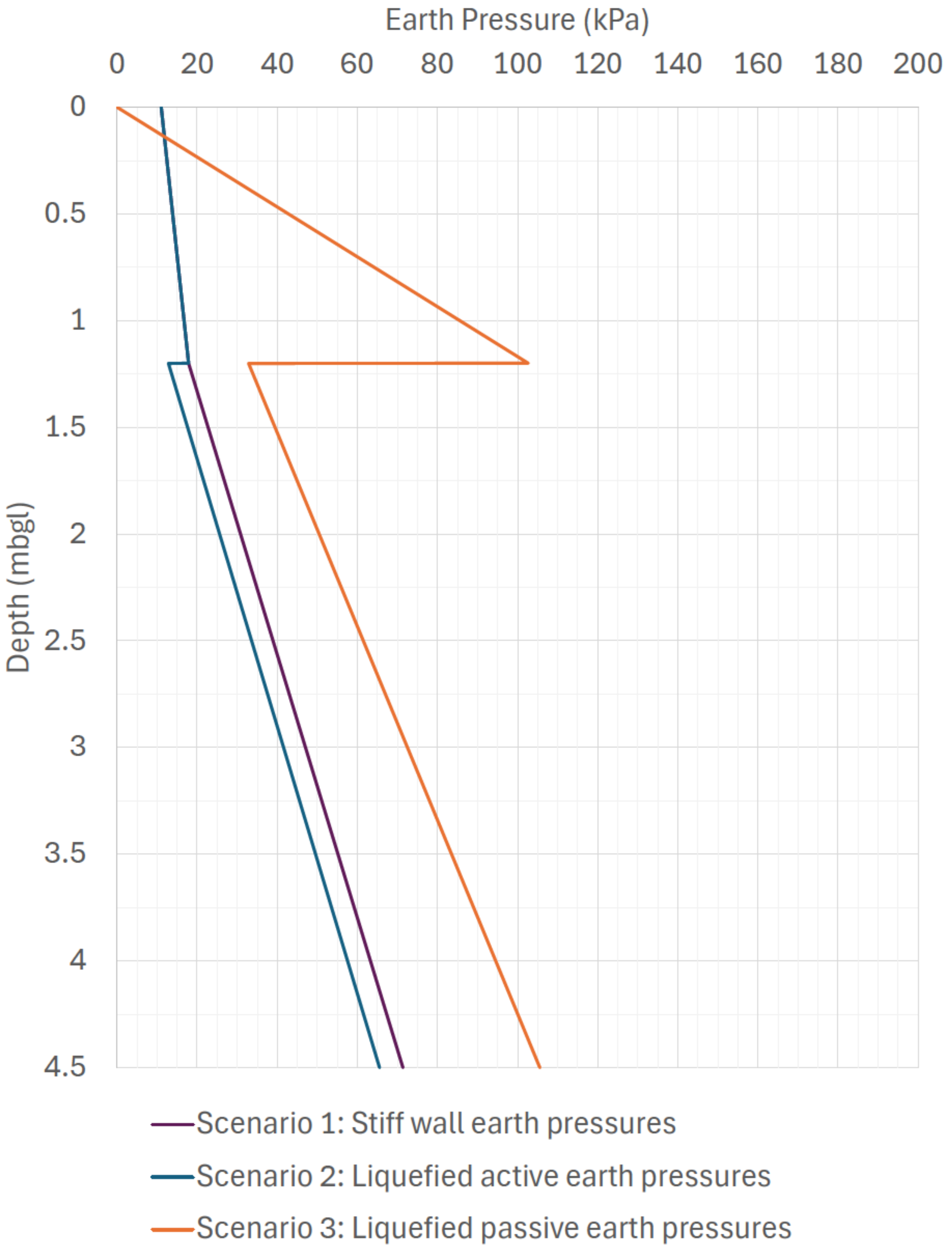
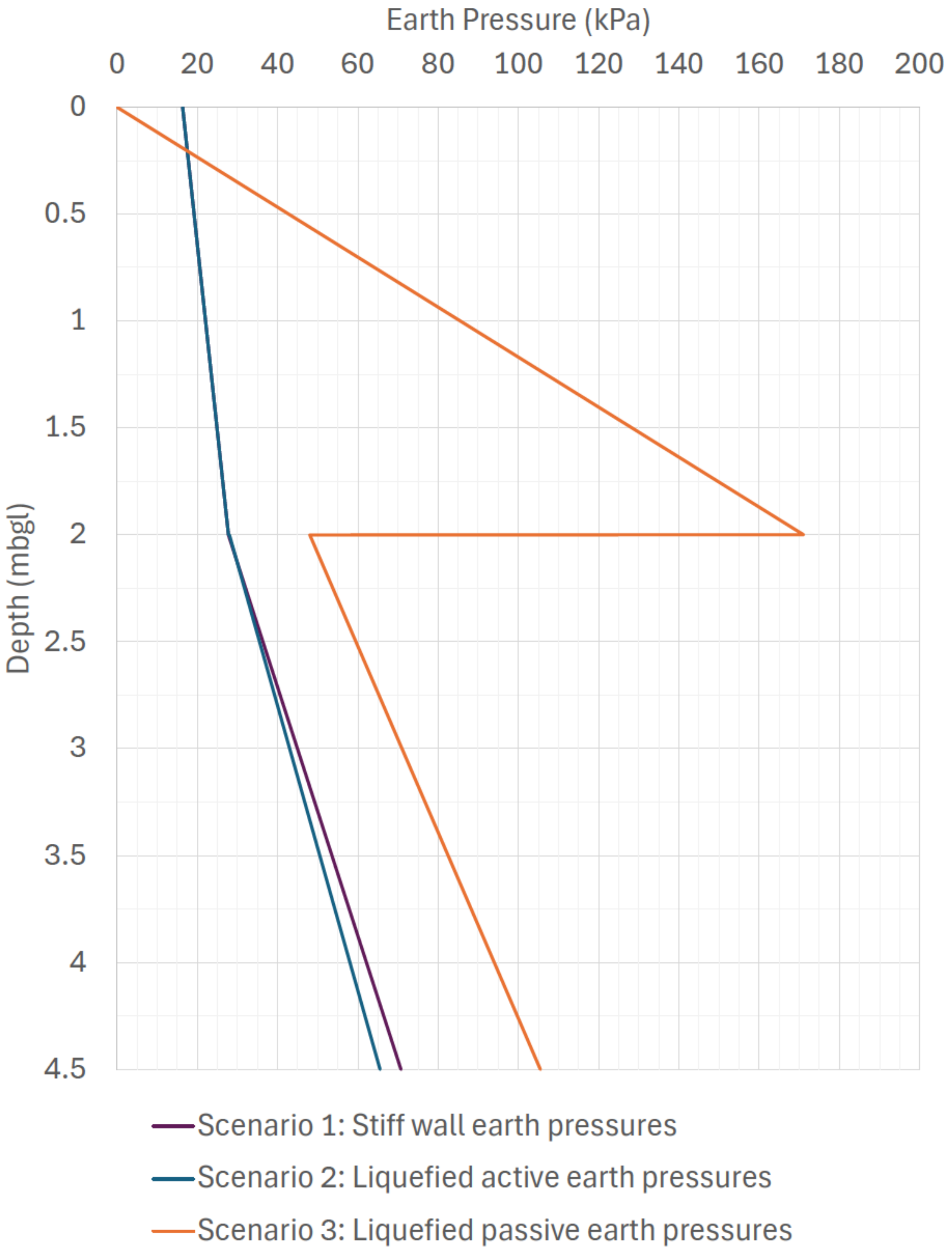



**Figure B2**
Calc/Chk: BHR/ELC
Date: 16/09/2025

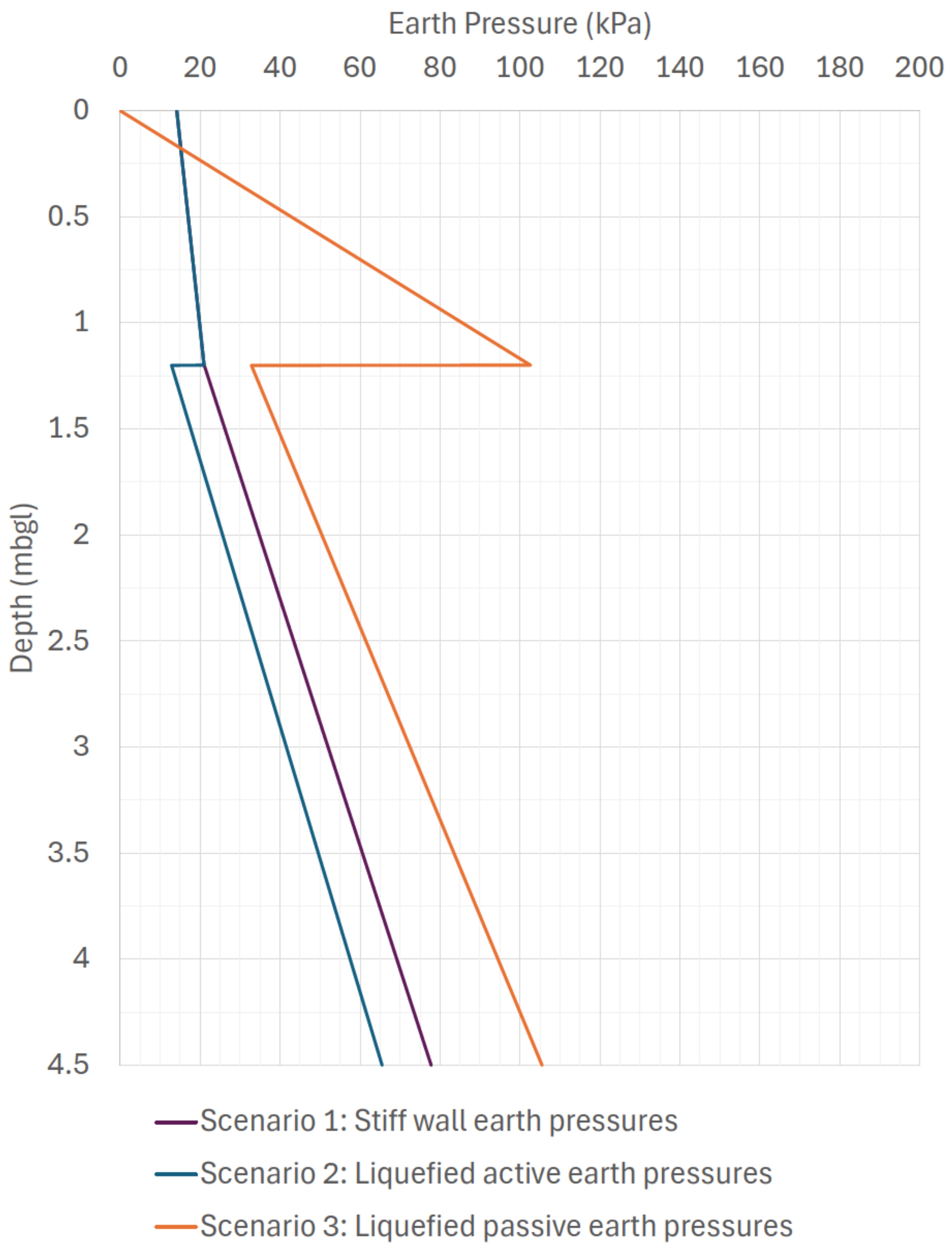
**Earth Pressures on Subway Walls - South Section (IL2)**

| | | |
|---|---|---|
| Tonkin+Taylor | **Client:** | GWRC |
| | **Project Name:** | Waterloo Station |
| | **Project No.:** | 1098196.1 |
| | **Structure:** | Existing Subway |




| | |
|---|---|
| **Figure B3** | **Earth Pressures on Subway Walls - North Section (IL3)** |
| Calc/Chk: BHR/ELC | |
| Date: 16/09/2025 | |

| | | |
|---|---|---|
| Tonkin+Taylor | **Client:** | GWRC |
| | **Project Name:** | Waterloo Station |
| | **Project No.:** | 1098196.1 |
| | **Structure:** | Existing Subway |




| | |
|---|---|
| **Figure B4** | **Earth Pressures on Subway Walls - South Section (IL3)** |
| Calc/Chk: BHR/ELC | |
| Date: 16/09/2025 | |