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Strengthening of heritage tunnel portals

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ABSTRACT

Wellington City Council (WCC) operates and maintains four heritage tunnels, constructed in the early twentieth century. WCC have developed a strengthening programme to successively investigate and (where necessary) design seismic strengthening of each tunnel. The Kariori Tunnel strengthening scheme was completed in 2013 and the design of the Hataitai Tunnel is currently ongoing. Both tunnels presented a unique set of challenges during the design and construction of the strengthening works. These include uncertainty in the condition and form of the structures, maintaining operation of the road and preserving the heritage value of the structures.

Keywords: ground investigations, slope stability, ground anchorages, load testing

1 INTRODUCTION

The Wellington City Council (WCC) road infrastructure team operates and maintains four heritage tunnels. In 2010, WCC developed a tunnel seismic strengthening programme. The objectives of the strengthening programme were to assess the condition and expected performance of the tunnels, and where necessary, design and construct a strengthening scheme. This paper presents a summary of the unique geotechnical challenges during the investigation, design and construction of the Karori and Hataitai Tunnels.

Both tunnels were very similar in some respects, and it was these similarities that dominated the development of the investigation, design and construction phases. These are highlighted below.

- The tunnels were constructed in the early twentieth century and are formed of mass concrete portals and wingwalls and brick and mortar lined tunnel barrel.
- A collapse occurred at the both tunnels during construction, resulting in loss of life. This highlights the challenging ground conditions and topography of both sites.
- A unique combination of site constraints such as steep heavily vegetated slopes, weak and highly fractured Greywacke rock and electrified overhead trolley buses.
- Both are iconic structures that are very visible to motorists and highly regarded by the local public.

This paper focuses on the design phase of the Hataitai Tunnel and then turns to the construction phase of the Karori Tunnel. Two approaches to investigating the tunnel conditions are presented, and their relative advantages and limitations are highlighted. Seismic strengthening of heritage tunnels requires careful consideration of cost and preservation of the heritage value and aesthetic of such structures.

2 HATAITAI TUNNEL – INVESTIGATION AND DESIGN

The investigation, seismic assessment and development of strengthening options were undertaken in 2012 and 2013. The detailed design of the preferred strengthening scheme is currently ongoing. Sample pictures of the tunnel portals are presented in Figure 1 and 2 below.

Figure 1. The current eastern portal with wingwalls either side

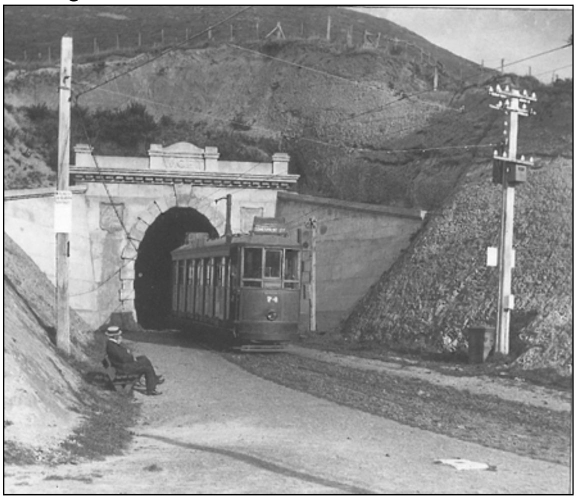

Figure 2. Historic photo of the western portal (courtesy National Archives)

2.1 Geological Setting

Published geology (Begg and Johnston, 2000) indicates the site is underlain by Greywacke and Argillite rock of the Rakaia Terrain. This unit is typically weak, steeply bedded and often shattered or sheared, all of which combine to considerably reduce the overall strength of the rock mass. The map identifies an unnamed inactive fault extending across the tunnel near the eastern portal.

The eastern portal is surrounded by steep (50 to 70 degree) cut slopes within highly weathered Greywacke. The insitu rock is very closely jointed with some unfavourable joints extending out of the slope. The rockmass is very dilated rock mass and a number of shallow translational failures extend along the southern cut slope.

The western portal comprises gently sloping to steep cut slopes within completely to highly weathered Greywacke. A thick wedge of typically weak non engineered fill extends from the rear of the portal and wing walls.

2.2 Ground Investigation and Seismic Assessment

The ground investigations and seismic assessment were undertaken in a number of stages. Each stage involved gathering progressively more data, resulting in a more detailed set of assessments and more accuracy and certainty in terms of the performance of the strengthening scheme.

A summary of the investigations and findings from each stage are set out in Table 1 below.

Table 1: Staged Investigation and Assessment

Stage	Scope of Investigations	Assessments	Key Outcomes
Preliminary Condition and Seismic Assessment	Site walkover and preliminary mapping of the rock faces. Review published and historical information	~	The tunnel barrel is generally sound, but some cracking and movement noted in portals. Further investigations required to confirm the site conditions and undertake seismic assessments.
Seismic Risk Assessment	Five machine boreholes, eighteen	Preliminary slope stability	The tunnel barrel is anticipated to perform adequately during a major

	concrete cores, four test pits	and structural assessments	earthquake. There is the potential for deep seated instability of the slopes behind both tunnel entrances, which could result in collapse of the portals and wingwalls. This presented a life safety risk to motorists and could result in closure of the tunnel for a considerable period.
Options Study	A sixth machine borehole and detailed rockface mapping.	Detailed slope stability and structural assessments. Construction cost estimates developed.	Four options were identified (refer to Section 2.4 below). Supplementary investigation data permitted refinement of the seismic stability of the cut slopes at the eastern portal, resulting in an overall reduction in the scope of work for three of the four options. Preferred option comprised new buttresses and a ground beam tied back with ground anchorages .

2.3 Seismic Assessment of the Portal and Wingwalls

Figure 3 below presents an extract from the historic construction drawings, highlighting the variable thickness of the wingwalls, and complex arrangement of the portal façade and wingwalls.

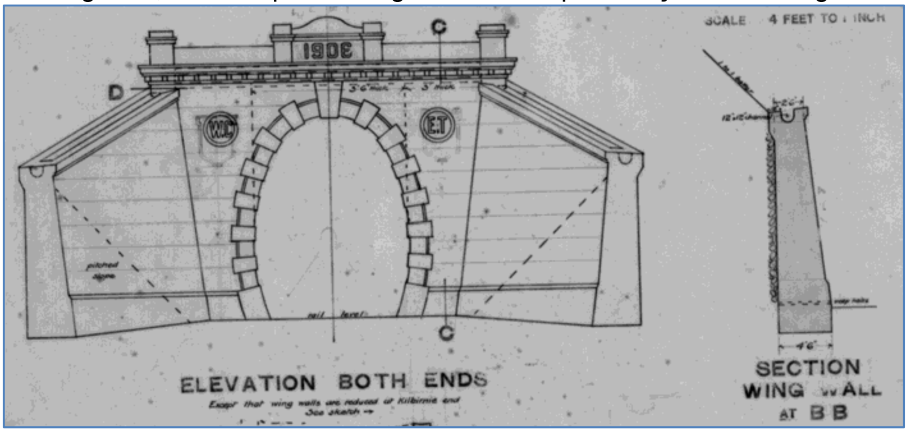

Figure 3.Typical Elevation and Section from historic drawings (courtesy WCC archives).

The seismic assessment of the portals focused on three failure modes as set out below.

- **Mode 1 -** Global slope stability of the tunnel entrance
- **Mode 2 -** Local stability immediately behind the portals and wingwalls
- **Mode 3 -** Shallow instability of the oversteepened Greywacke slopes (rockfall)

Static and seismic stability analyses were undertaken following two methods as set tout below.

- **Method 1 -** Slope stability analyses using GEO-SLOPE ‘Slope’W’™ software, adopting the limit equilibrium methods of Morgenstern and Price (1965) and Janbu (1954). Slope displacements were derived using the Newmark Sliding block;, following the method of Jibson (2007)
- **Method 2 –** Stereonet analyses to identify the defect sets, and kinematic analyses with Rocscience Swedge™ software to assess the behaviour of the dominant defect set.

The analyses indicated that during the ‘ultimate limit state’ earthquake event (1000 year return period, peak ground acceleration of 069g and 0.52g for the west and east portals respectively) there is the potential for deep seated instability at both portals. Such instability would impart a large load on the portal and wingwall structure, resulting in potential displacements in the order of 100mm to 400mm. As the portals are unreinforced and not well tied together this would likely lead to collapse.

Analyses and site observations also indicated the potential shallow translational slides to occur during periods of heavy rainfall and minor earthquakes. Rockfall would accumulate at the base of the portal presenting a hazard to motorists.

A supplementary ground investigation was undertaken at the eastern portal. The investigation comprise a machine borehole above the crest of the portal and detailed rock face mapping. This data allowed refinement of the ground model and geotechnical parameters, resulting in a considerable reduction in the load imparted on the read of the portal and wingwall structure.

2.4 Strengthening Options Assessment

Four strengthening options were developed, as summarised below.

Option 1 – Reinforced concrete portals and wingwalls cast against the existing structure, supported with ground anchorages. This arrangement was adopted for Karori Tunnel.
Option 2 – Reinforced concrete buttresses, a ground beam tying together the top of the buttress, supported with ground anchorages.
Options 3 – Reinforced concrete portals and wingwalls founded on large diameter bored piles socketed into the insitu rock.
Option 4 – A large region of soil nails and steel mesh, stabilising the soil and rock mass above the portals and wingwalls.

The advantages and limitations of each option were assessed on the basis of cost, programme, constructability and effect on key local stakeholders (public transport and recreation). The key consideration was the need to balance the efficiency of the solutions with the desire to minimise the impact on the existing portals, and in doing so preserve their heritage value.

A number of geotechnical considerations influenced the options assessment. These are set out below.

- Options 1 to 3 required supplementary catch fences, positioned at the crest of the portal and wingwall, in order to mitigate the risk of rockfall.
- Option 3 comprised large diameter bored piles, which would require a considerable piling platform, blocking the access of public transport into the tunnels.
- Option 4 could be constructed entirely from above the portals, providing continuity of service of the public transport.

3 KARORI TUNNEL – CONSTRUCTION AND TESTING

The investigation, design and construction of the Karori Tunnel seismic strengthening scheme was undertaken over a three year period from 2011 to 2013. The tunnel portals are illustrated in Figure 4 and 5 below.

Figure 4. Portal post strengthening

Figure 5. Historic photo of portal (Courtesy of National Archives)

Limited ground investigation data was available to inform the design stage. Consequently, the design was progressed by making a number regarding the soil profile and the strength and condition of the rock mass. The design assumptions were validated during construction by undertaking investigation and acceptance anchorage load testing and carefully monitoring the anchorage drill records as they proceeded.

3.1 Geological Setting

The published geology (Begg and Johnston, 2000) indicates the site is underlain by Greywacke and Argillite rock of the Rakaia Terrain. Refer to Section 2.1 above for further commentary.

Published geology and other recent research indicates the active Welllington Fault lies in the order of 20m from the eastern portal (Perrin, 2004 and Begg and Johnston, 2000). The Wellington Fault is a steeply dipping subvertical, dextral strike-slip fault, striking approximately north-east to south west. The rock is brecciated with extremely closely spaced steeply inclined defects, resulting in a friable material that appears as a weak angular gravel.

Both portals were back filled with non-engineered fill. Historic photos indicate a considerable amount of construction debris was discarded in the fill, such as wooden planks, drums, concrete and general waste.

3.2 General Arrangement of the Seismic Strengthening Scheme

The City Council and local stakeholders were keen to maintain the appearance of the heritage structure. The strengthening scheme comprised construction of an entirely new reinforced concrete portal, buttresses and ground beams, with a series of ground anchorages extending from the buttresses and ground beams through the uncontrolled fill and grouted into the insitu rock. The general arrangement is illustrated in Figures 6 and 7 below.

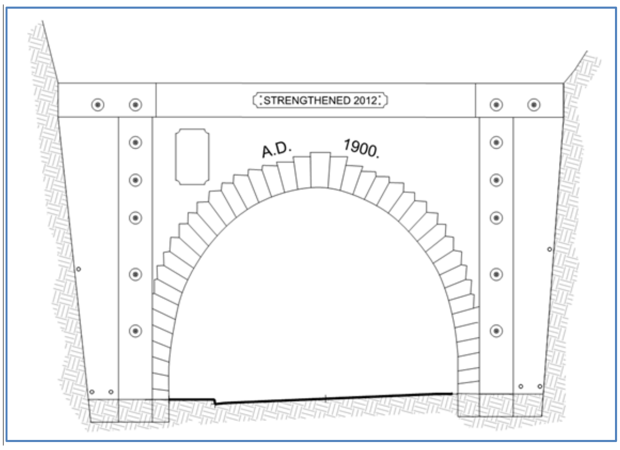

Figure 6. Design Elevation

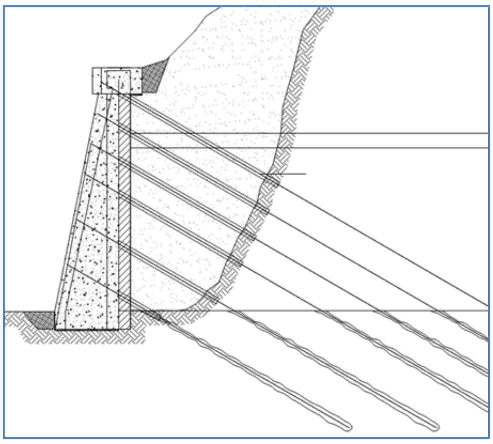
Figure 7. Design Section

A summary of the portal wall and ground anchorage arrangement is provided in Appendix A

3.3 Investigation Load Testing

Three investigation load tests were specified to confirm the bond capacity of the insitu rock. The design had adopted a geotechnical ultimate bond capacity of 400kPa. Each test anchor was drilled vertically from Raroa Road, which extends over the tunnel barrel. The test anchor depths were designed in order to target the region of insitu rock that would provide the ground anchorage bond zone.

The key requirements for each test are set out below.

- Test each anchorage to geotechnical ‘failure’
- A 200mm diameter borehole, with a 1.5m long bonded zone
- Creep performance of the anchorage to be assessed over a 6 hour period

The results from the three investigation load tests are presented in Figure 8 below.

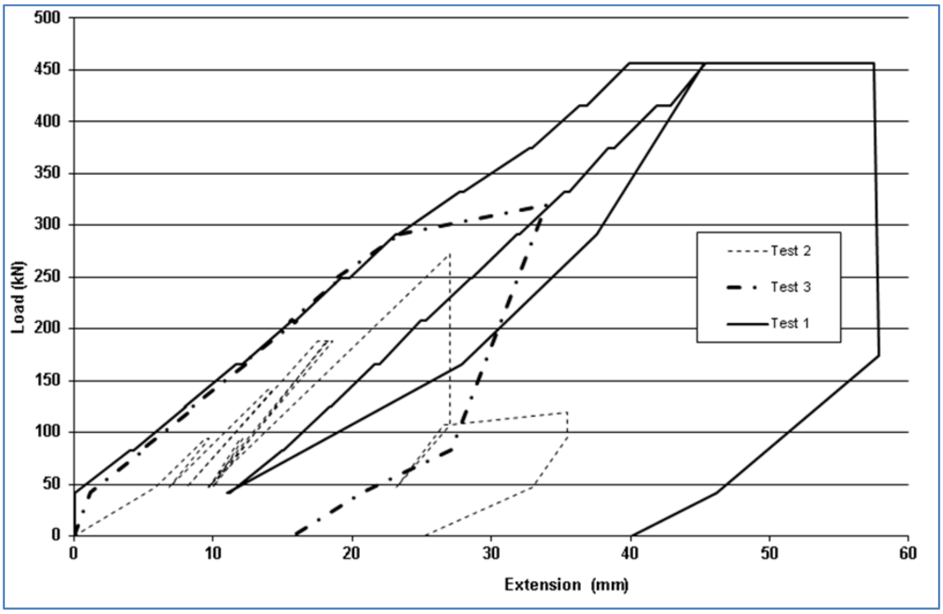

Figure 8. Load Test Plot

Two of these results were considerably less typically less than the 400kPa ultimate capacity adopted in the design. Drill logs indicated the bonded zone was likely to have been founded within the brecciated Greywacke, a highly fractured and weak rock mass, resulting in the low grout bond stress. A summary of performance of the tests anchors is provided in Table 2 below.

Table 2: Investigation Load Tests

Test ID	Load at 'failure' (kN)	Estimated stress at 'failure' (kPa)	$S_{a.}$(mm)	$S_{b.}$ (mm)
Test 1	457	486	n/a	40
Test 2	273	290	4.05	25
Test 3	320	340	5.66	16

[a] Apparent movement of the bonded length at the onset of creep (total movement less elastic extension of bar)
[b] Residual movement following release release of the jack (0kN)

Other factors may have contributed to the poor performance of the anchorages. These include smearing of debris at the hole annulus. Test 1 and Test 2 were positioned in a similar region and may not have been representative of the entire rock mass.

Further activities were considered, such as undertaking additional testing with increased bonded lengths and exhuming test anchors 2 and 3 to confirm the condition and extent of the grout annulus. It was not possible to do this due to programme and cost constraints.

The solution was to adopt a lower geotechnical ultimate bond capacity of 300kPa, which resulted in an additional 1m to 2m bonded length to each anchorage. A dependable bond strength of 150kPa was adopted for the design (allowing a strength reduction factor of 0.5). The dependable bond strength formed the basis of the acceptance load testing, refer to Section 3.5 below.

3.4 Ground Anchorage Construction

A number of challenges were overcome during the construction of the ground anchorages, these are summarised below.

- **Depth and condition of the insitu rock.** This was found to be highly variable and was difficult to determine with the down the hole hammer (DTH) drilling technique. The contractor took readings of the drill penetration rate and carefully monitored the drill flushings in order to determine the depth to the insitu Greywacke rock.

- **Monitoring Grout Take.** Grout volumes recorded by the contractor indicated considerably higher grout take when compared to the theoretical volume, typically ranging from 120% to 160%. This was surprising given that the permanent casing (socketed a minimum of 0.5m into the insitu rock) was specified in part to prevent grout loss. The bore and grout logs were reviewed and discussions were held with the contractor. The exact cause of the additional grout take was not clear, although it may have been caused by a combination of a poor casing socket, overbreak from the DTH and a loss of grout through the weak and brecciated Greywacke.
- **Difficulty Flushing the Anchorage Hole.** The DTH regularly became blocked with drill flushings. This may have been caused by the saturated, weak breccicated Greywacke becoming a very thick pug, blocking the hammer and annulus of the hole. Increasing the frequency of flushing the hole appeared to reduce the frequency of the blockages.

3.5 Acceptance Load Testing

Three acceptance load tests were undertaken in order to assess the performance of the anchorages. Ideally the test would have targeted 150% of the working load, but this was not possible due to the capacity of the steel bar and the concern that the reaction block would cause excessive movement in the original portal buttress (the new buttress had not been constructed at this stage). Therefore, the anchorages were loaded up to 864kN, which is equivalent to 110% of the working load of 785kN (seismic load case). The 785kN load generates bond stress in the order of 150kPa for a fixed length of 9m to 10m, which is consistent with the modified dependable bond capacity (refer to Section 3.3 above).

A summary of the acceptance load tests is provided in Table 3 below.

Table 3: Acceptance Load Tests

Anchor ID	Free Length (m)	Bonded Length (m)	$S_{a.}$			$S_{b.}$ (mm)		
			Cycle 1	Gauge 2	Gauge 3	Gauge 1	Gauge 2	Gauge 3
WF	1.75	10.00	22.56	22.10	22.95	0.95	1.76	2.45
WJ	4.00	10.00	23.84	25.56	26.00	3.32	2.33	3.18
EN	5.00	9.00	7.88	10.19	10.65	5.86	5.68	6.14

[a] Apparent movement of the bonded length at the working load (total movement less elastic extension of bar)
[b] Residual movement following release of the jack (0kN)

All load tests exhibited adequate performance during a 30 minute hold period at 110% of the working load. Creep movement was typically less than 0.5% of the theoretical bar extension and the average creep was considerably less than 0.5mm over the 30 minute period.

These results generally indicate an elastic response of the ground anchorage at the working load. This testing validated the modified dependable bond strength of 150kPa.

4 CONCLUSION

Unreinforced concrete and masonry tunnel portals are expected to exhibit poor performance during a major earthquake. It is difficult to reliably predict the performance of the tunnel portal, due to their complicated arrangement, variable soil/rock profile and the behaviour of the rock mass.

Two approaches to investigation and assessment of seismic performance are presented. The first approach was adopted for the Hataitai Tunnel, and comprised a staged process of investigations and assessments. The strengthening scheme is then refined at each stage. The second approach was adopted for the Karori Tunnel and comprised a number of assumptions during the investigation and assessment. These assumptions were then validated during construction with investigation and acceptance load testing.

The Karori Tunnel design adopted a geotechnical ultimate bond capacity of 400kPa for the highly weathered Greywacke. Investigation load tests confirmed that a geotechnical ultimate bond capacity of 300kPa was representative of the brecciated Greywacke rock mass.

A number of seismic strengthening options are presented. These can be compared by a number of factors such as constructability, cost, programme and their effect on local services and stakeholders. However, the key consideration of both projects was the need to balance the efficiency of the solutions with the desire to minimise the impact on the existing portals, and in doing so preserve their heritage value.

5 ACKNOWLEDGEMENTS

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APPENDIX A

KARORI TUNNEL – STRENGTHENING GENERAL ARRANGEMENT

Portal Walls

- 400mm thick cast insitu reinforced concrete (40MPa) portal walls
- 900mm wide by 2200mm to 700mm deep reinforced concrete buttresses and a 1400mm wide by 900mm deep capping beam across the crest of the portal
- High yield strength (500MPa) reinforcing steel with minimum 50mm cover

Ground Anchorages

- 200mm diameter installed with a down the hole percussion hammer drill, fully encased with 40Mpa cementitious grout
- 8m to 10m bonded zone within the insitu rock (weathered Greywacke)
- 40mm diameter high strength steel bars, 1030MPa ultimate stress
- Sacrificial steel casing extending from the portal walls, keyed a mimimum of 0.5m into the insitu rock. The casing prevented hole collapse through the uncontrolled fill.
- Double corrosion protection provided with factory applied PVC sheathing and grout
- 21no. anchorages at the east portal and 14no. anchorages at the west portal