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Earthquake damage assessment of water supply tunnels

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ABSTRACT

Assessment of potential seismic damage and repair costs for ten Greater Wellington Regional Council's (GWRC) water tunnels has been carried out. The assessment included inspections of the tunnels by an Opus geologist to characterise the tunnel construction type and condition, the ground type and its susceptibility to seismic damage. Analysis of the transient and permanent ground deformations and accelerations imposed on the tunnels from ground waves, fault rupture and landslide was performed.

Tunnel damage models have been developed and used for the assessment of seismic damage. Loss analysis was carried out using @RISK Monte Carlo simulation and taking account of uncertainties reflected in the probability distributions of the earthquake ground motion, structural response and cost variables, to find the deterministic repair costs under various earthquake scenarios. The probable maximum loss for earthquakes insurance purposes was taken as the 90th percentile loss and varied from $0.15M to a few million dollars for individual tunnels. In case of the Wellington Fault rupture, the Karori Raroa tunnel is expected to experience major damage and would need to be rebuilt.

Keywords: tunnel, earthquake, damage, Wellington greywacke, cost, seismic stress

1 INTRODUCTION

Assessments have been made of the expected level of seismic damage and costs of repairing earthquake damage to the GWRC wholesale water supply network tunnels.

For this study more engineering-based assessments of damage and repair costs have been made as follows:

- The tunnels were inspected by a geologist to characterise the tunnel construction type, condition, the ground type and its susceptibility to damage.
- The transient ground and permanent ground deformations and accelerations imposed on the tunnels from ground waves, fault rupture and landslide were analysed.
- Damage models for lined and unlined tunnels have been developed based on these analyses.
- The damage models were compared with the empirically-derived models published by the American Lifelines Alliance (ALA 2001).
- Expected level of seismic damage and costs of repairing earthquake damage have been assessed based on the adopted damage models.

This assessment covered ten tunnels through the Wellington region, these being: Karori Raroa, Maldive Street, Rocky Point, New Wainuiomata Tunnel, Hutt North 3, Hutt North 4, Orongorongo 1, Orongorongo 2, Kaitoke 1 and Kaitoke 2 tunnels.

The locations of these tunnels range from Karori to Kaitoke and Wainuiomata as shown in Figure 1.

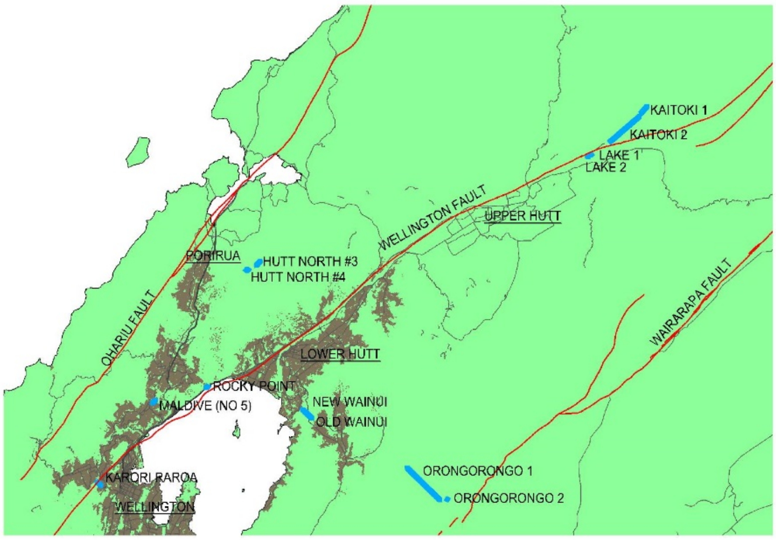

Figure 1. Tunnel locations and active faults

2 TUNNELS

Typical shafts of a lined and unlined tunnels and tunnel portals are shown on Figure 2.

Figure 2. Typical lined tunnel, roof of an unlined tunnel, and tunnel portals

As-built drawings for the tunnels were reviewed. The tunnels were inspected by an Opus geotechnical engineer who walked through the tunnels with a GWRC representative, observed and assessed the quality of the rock exposed in the roof and the walls of the tunnels (for unlined tunnels) and of the concrete linings (for lined tunnels). The portals of the tunnels were also inspected and the potential for seismic slope failures at the portals was assessed. To enable quantitative assessment of seismic damage to the tunnels, the rock quality was assessed in terms of the Rock Mass Rating (RMR) after

Bieniawski (1989). All tunnels are formed in rock comprising weathered Wellington Greywacke sandstone of variable quality. Therefore, wide range of rock parameters was used for the assessment of the potential earthquake damage. Some information about the tunnels is summarised in Table 1.

Table 1: Tunnel parameters

Location	Tunnel Shaft Size, m	Tunnel Length, m	Embedment of tunnels below ground surface, m	Lining	RMR	Assessed Shear Wave Velocity for Rock Mass, m/s
Hutt 3	1.7 x 1.7	430	5 to 33	Lined	N/A	600-1000
Hutt 4	1.7 x 1.7	220	1 to 2	Lined	N/A	600-1000
Karori Raroa	1.8 x 1.8	360	2 to 67	Partial	26-36	500-1000
Maldive St	1.7 x 1.7	350	6 to 40	Lined	N/A	400-1000
Kaitoke 1	1.8 x 1.8	690	2 to 141	Partial	45.5	600-1000
Kaitoke 2	1.8 x 1.8	2,780	3 to 275	Partial	45.5	600-1000
New Wainuiomata (3 sections)	7 x 3.6 3.9 x 2.7 2.4x 2.4	880	3 to 130	Lined	N/A	200-1000
Orongorongo 1	2 x 2	3,150	7 to 526	Partial	35-46	700 - 3500
Orongorongo 2	2 x 2	100	44 to 50	Partial	35-46	700 - 3500
Rocky Point	2.5 x 2	230	6 to 27	Lined	N/A	600-1000

The ranges of the shear wave velocities for the rock mass were assessed based on observed rock quality for unlined tunnels and on site geologies and limited rock exposures for unlined tunnels in accordance with the correlations developed by Perrin et al. (2010). The unconfined compressive strength of intact rock was estimated based on the quality of exposed rock. Strength data for the concrete linings was not available. Therefore, based on our observations during the inspections of the tunnels, the following properties have been adopted for the analysis of seismic damage: compressive strength of 25 MPa, tensile strength of 3.5 MPa, Young's modulus of 25,000 MPa and Poisson's Ratio of 0.2.

3 SEISMIC HAZARD

Available analytical and numerical analyses methods for estimating the earthquake induced stresses in the tunnel lining require information on the intensity of the ground shaking in terms of the peak ground acceleration (PGA) and the peak particle velocity (PPV). For any particular site, these parameters can be determined by a conventional seismic hazard study which involves identifying the earthquake sources within the region, determining the frequency and magnitude of the earthquakes arising on each source and establishing the attenuation of the earthquake waves as they travel from the source to the site. Results from a New Zealand wide seismic hazard study have been published by GNS (Stirling, 2002). In this document PGA's are presented as a function of earthquake return period for any location. While the hazard model involves a degree of averaging on a moderately coarse area grid and does not necessarily give precise ground motion predictions for all specific sites, the results of the study are considered to be appropriate for the assessment of the earthquake damage to the tunnels.

Our assessment of PPV was based on empirical relationships between PGA and PPV published by American Lifelines Alliance (ALA, 2001). ALA proposes that the ratio of peak ground velocity in m/s to peak ground acceleration in g units is about 1.0 for rock sites located within about 50 km of the source of earthquakes of magnitude between 6.8 and 7.7.

The seismic hazard that the tunnels are exposed to is dominated by the Wellington and Wairarapa Faults. The Wellington fault is in close proximity to the all but Orongorongo tunnels which are close to the Wairarapa Fault. The Wellington and Wairarapa faults have relatively short recurrence intervals of 600 years and 1500 years respectively. The PGA's at the tunnels calculated in accordance with the McVerry et al (2006) seismic wave attenuation relationship for the Wellington and Wairarapa faults are shown in Table 2.

Table 2: Site PGA's from Wellington and Wairarapa Faults

Location	Peak Ground Acceleration (g)			
	Wellington Fault		Wairarapa Fault	
	Median	84%ile	Median	84%ile
Karori-Raroa	0.50	0.72	0.26	0.38
Maldive Street	0.48	0.69	0.28	0.40
Rocky Point	0.49	0.72	0.30	0.43
New Wainuiomata	0.43	0.63	0.36	0.53
Hutt North 3	0.40	0.58	0.28	0.40
Hutt North 4	0.40	0.58	0.28	0.40
Orongorongo 1	0.31	0.44	0.51	0.73
Orongorongo 2	0.31	0.44	0.53	0.78
Kaitoke 1	0.46	0.67	0.37	0.53
Kaitoke 2	0.49	0.72	0.37	0.53

It should be noted that the Wellington Fault passes through the Karori - Raroa tunnel. Displacements in the order of 4m are anticipated on this fault. Therefore, the Karori - Raroa tunnel would need to be re-built if this part of the fault ruptures.

4 DAMAGE TO TUNNEL PORTALS DUE TO SEISMIC SLOPE FAILURES

The tunnels portals are located in generally steep terrain (Figure 2) and are potentially prone to damage from landslides induced by earthquakes. The potential damage due to slope failures from a Wellington Fault or similar earthquake has been assessed based on a previous study by GNS that gave recommendations on the assessment of slope failures as a function of slope steepness, rock quality and intensity of seismic shaking (Hancox et. al, 2002). For most of the tunnels the risks of deep-seated slope failures that would cause major damage at the portals is assessed to be low. For most of the tunnels portals the expected damage is relatively small amounts of ground slumping and rock fall (less than 5 m^3). The Rocky Point tunnel is within 100 m distance of a very large landslide triggered by the 1885 Wairarapa earthquake. A similar event at the site of the tunnel could cause substantial damage. Also, a high risk of a landslide at the south portal of the Kaitoke 2 tunnel has been identified.

5 CALLIBRATION OF DAMAGE MODELS

Analytical damage models for lined and unlined tunnels were developed by Opus specifically for this study as described in Sections 6 and 7 of this paper. The models were then calibrated by comparing the assessed level of tunnel damage using the models with the level of damage observed and recorded in previous earthquakes in New Zealand and elsewhere. The American Lifelines Alliance (ALA, 2001) has developed empirical earthquake damage models (also known as fragility models) for tunnels from the analysis of a database of 217 tunnels that have experienced strong earthquake ground motions. Some of the data used by ALA is presented in Figure 3.

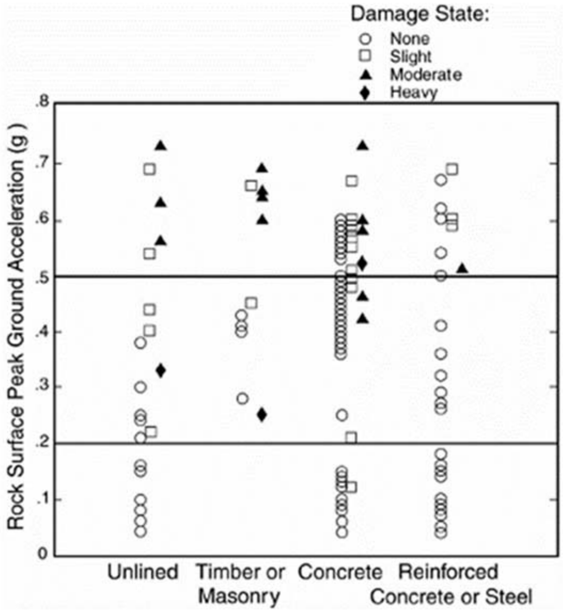

Figure 3. Earthquake Damage Data for Tunnels (adapted from ALA, 2001)

The ALA models use the following damage states: slight damage (small cracks in liners and minor rock falls that will not generally impact directly on the flow of water through the tunnel); moderate damage
(large cracks that could lead to dropouts into the water channel or damage the pipeline); major damage (major cracking and dropouts causing pipeline damage and restricting access to tunnels).

From these data the median PGAs causing damage equal to or greater than the specified damage state were assessed. The probability distribution of the PGAs is lognormal with a dispersion of 0.5. The damage state median PGAs for the lined tunnels of good quality construction and in good condition are shown in Table 3 along with the maximum concrete stresses in the concrete lining of the typical water tunnel section corresponding to the median PGAs. These can be compared to likely compressive strengths in the 20MPa to 30MPa range and tensile strengths in the 3MPa to 5MPa range.

Table 3: Lined tunnel damage state median PGAs compared to analytically estimated lining stresses

Type of Tunnel	Slight Damage State	Moderate Damage State	Major Damage State
ALA median PGA (g)	0.61	0.82	-
Maximum concrete stress (MPa)	9.0	13.0	-

The damage state median PGAs for lined tunnels of poor quality construction are shown in Table 4 along with the maximum concrete stresses in the concrete lining of the typical water tunnel section corresponding to the median PGAs.

Table 4: Damage state median PGA's for poor quality lined and unlined tunnels compared to analytically estimated lining stresses

Type of Tunnel	Slight Damage State	Moderate Damage State	Major Damage State
ALA median PGA (g)	0.35	0.55	1.10
Maximum concrete stress (MPa)	5.0	8.0	17.0

It is expected that these PGA – damage relationships will give reasonable estimates of the maximum damage, but conservative estimates of the average damage.

6 EARTHQUAKE DAMAGE TO UNLINED TUNNELS

Currently there is no commonly accepted methodology for the assessment of earthquake damage to unlined tunnels. Therefore, the following methodology described below has been developed for this project. The damage to unlined tunnels has been quantified by estimating the volume of spalling (volume of rock failures) per 1 m length of the tunnel due to seismic shaking as follows:

1. PGA was assessed based on the considered earthquake scenarios.
2. The PGA was factored down based on the depth of the tunnel measured from the ground surface level. The factors recommended by FHWA (2009) were adopted for this purpose.
3. The unconfined compressive strength of the rock mass (this includes effects of rock defects) was calculated a function of the unconfined compressive strength of intact rock and RMR using a relationship given by Van & Vasarhelyi (2012).
4. The effective propagation shear wave velocity in the rock mass was calculated based on available correlations between the quality of the rock mass and the shear wave velocity given by GNS Science (Perrin et al., 2010).
5. The maximum axial strain generated by earthquake shaking was assessed as function of the effective propagation shear wave velocity as recommended by Wang (1993).
6. The seismic stress increment in the rock mass was calculated based on the maximum axial strain generated by earthquake shaking and the assessed rock stiffness.
7. The total stress in the rock mass was assessed as a sum of static stress and the seismic stress increment.
8. The volume of spalling (volumes of rock failures) per 1 m length of the tunnel due to seismic shaking was assessed as function of the maximum shear stress generated in the rock mass based on empirical correlations given in Martin et al. (1999).
9. Parameters in the developed model were also correlated against previous case studies (Section 5) and the observed volumes of failure in the Orongorongo tunnel in 2013 Seddon earthquakes.

Table 5 presents a summary of the 500 and 2500 year return period earthquake results for a representative section of unlined tunnel.

Table 5: Analysis Results for Representative Unlined Tunnel Section

Parameter	Value
Effective radius (m)	1.0
Tunnel depth (m)	200
Ground surface shear wave velocity (m/sec)	1000
Rock unconfined compressive strength (MPa)	30
Rock mass rating	40
500 year return period	
Peak ground acceleration (g)	0.4
Volume of spalling (m^3/m)	0.05
2500 year return period	
Peak ground acceleration (g)	0.72
Volume of spalling (m^3/m)	0.17

7 EARTHQUAKE DAMAGE TO LINED TUNNELS

Analytical procedures described by Wang (1993) were used to estimate the longitudinal stresses (tunnel axis direction) in the tunnel lining from the earthquake induced axial and curvature deformations. Two different methods can be used. A simplified procedure is to assume that the structure experiences the same strains as the ground in the free-field. A more complex and correct procedure is to consider tunnel-ground interaction effects. When the tunnel is stiff in the longitudinal direction relative to the surrounding soil or rock, it tends to resist rather than conform to the deformations imposed by the ground. The free-field simplified approach gives an upper-bound assessment of the tunnel response and has been adopted for this analysis. Wang (1993) also presents both free-field and lining-ground interaction solutions for circular tunnel sections subjected to earthquake induced ovaling deformations. For this study the more conservative free field method was used.

Table 6 presents a summary of the 500 and 2500 year return period earthquake results for longitudinal and ovaling strains and corresponding stresses calculated in a typical section of lining using the simplified free-field approaches. The imposed strains are inversely proportional to the rock stiffness, represent by the shear wave velocity. The expected value of the estimated shear wave velocity range has been used in this analysis. The predicted longitudinal compressive stresses from deformations associated with the 500 and 2500 year return period ground shaking are significantly less than the compressive strength, so concrete crushing and spalling is unlikely to occur in competent lining. Longitudinal axial strains are predicted to exceed the tensile strength of the concrete so concrete cracking is likely to occur. However, this cracking will be transitory and will close up as the ground wave passes along or across the tunnel leaving only fine hair-line cracks. The ovaling flexural stresses are unlikely to exceed the tensile strength given the significant compressive stresses in the lining from overburden pressures. Our analysis indicated that damage to lined tunnels is only likely to occur if there are sections of defective lining that are marginally stable under static loads requiring relatively small increases in stress to trigger a failure.

Table 6: Analysis Results for Typical Lined Tunnel Section

Parameter	Value
Effective radius (m)	1.0
Lining thickness (mm)	200
Ground surface shear wave velocity (m/sec)	800
500 year return period	
Peak ground acceleration (g)	0.4
Longitudinal axial strain	0.00024
Longitudinal curvature strain	0.000009
Diametric (ovaling) strain	0.00062
Max. longitudinal concrete stress (MPa)	6.0
Max. ovaling flexural stress (MPa)	1.0
2500 year return period	
Peak ground acceleration (g)	0.72
Longitudinal axial strain	0.00043
Longitudinal curvature strain	0.000017
Diametric (ovaling) strain	0.0011
Max. longitudinal concrete stress (MPa)	11.0
Max. ovaling flexural stress (MPa)	2.0

8 LOSS ANALYSIS

Based on the described methodology of damage assessment, two loss estimation models were developed - a lined and an unlined models. Generally the tunnels were assessed using either a single model or a combination of the two models in different sections. To account for risk and uncertainties reflected in the probability distributions of the earthquake ground motion, rock quality, structural response and cost variables, Monte Carlo Simulation was carried out for the loss estimation analysis using @Risk simulation software. The Monte Carlo simulations calculated 10,000 iteration of each considered earthquake scenario. @Risk simulation was applied to the following input parameters of the model:

- Earthquake event magnitude: this allowed earthquake Magnitudes to be entered as a range, e.g. a M7.5 event was presented as M7.3 – M7.7.
- Repair cost rates: ranges of repair cost rates from recent projects were used.
- Rock parameters: a geologist made an assessment of the rock quality during tunnel inspections; ranges of rock parameters were developed based on the observed variability of the rock.

The outputs of the simulations included:

- Volume of spalling for unlined tunnels or tunnel sections
- Damage states for lined tunnels or tunnel sections
- Distribution curves of repair cost for each tunnel and all tunnels

The assessed 90%ile repair costs for individual tunnels for the considered earthquake scenarios varied from $0.15M to a few million dollars. In case of the Wellington Fault rupture, the Karori Raroa tunnel is expected to experience major damage and would need to be rebuilt.

9 CONCLUSIONS

In this study, models for seismic damage to lined and unlined GWRC water tunnels have been developed. The available information, inspections, engineering analysis and judgement have been used to assess earthquake damage to the tunnels. Monte Carlo simulations for both lined and unlined tunnels have been used to estimate the cost to repair earthquake damage to the assets. The probable maximum loss for earthquakes insurance purposes was taken as the 90th percentile loss and varied from $0.15M to a few million dollars for individual tunnels. In case of the Wellington Fault rupture, the Karori Raroa tunnel is expected to experience major damage and would need to be rebuilt.

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