# Repair and Modification of NZ Railway Tunnels

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#### Summary

The current NZ rail network is approximately 4000km long, with 145 tunnels ranging in length from 50m to 9km and with a total length of 87km. The tunnels were opened between 1867 and 1980 and have been constructed in a diverse range of geological conditions. Tunnel cross-sections and lining arrangements have varied over the period and only two tunnels can be considered as being modern designs.

The tunnels have performed remarkably well and the major reasons for modifications have been invert lowering to provide greater clearances for larger rolling stock or to permit electrification. A few tunnels have required works to repair lining failures due to swelling ground pressures and, in a small number of tunnels, lining repairs have been required to remedy lining deterioration due to a low standard initial construction.

This paper describes the philosophy and approach taken in the repair and modification of a number of tunnels over the last 30 years.

#### 1. Introduction

The earliest tunnels on the NZ rail network were constructed in the 19th century and were lined after excavation with a masonry or unreinforced in-situ concrete lining.

Except in areas where swelling minerals are present in the ground or the tunnels were constructed through landslide slip surfaces the tunnel linings have been structurally adequate. In a few tunnels, poor initial concrete quality combined with the effects of corrosive steam locomotive exhaust has resulted in surface erosion sufficiently severe to warrant remedial works.

The major challenge has been to pass ever increasing sized rolling stock and containers through the tunnels and to provide for the installation of overhead electrification traction cables. To date this has been achieved by lowering the rock floor of the tunnels enabling lowered rails.

The construction of the railway network was largely undertaken by the Public Works Department (later Ministry of Works and Development (MWD)). The maintenance and modification of the network was the responsibility of New Zealand Government Railways (NZGR or NZR) until 1991. From 1993 to 2003 the network was in private ownership, after which it was purchased back by the government and became the responsibility of ONTRACK, which has become KiwiRail Network (KRN) with the formation of KiwiRail in 2008.

#### 2. Overview of NZ rail tunnels

#### 2.1 History

The development of the New Zealand railway network can conveniently be subdivided into three stages, with corresponding evolution of tunnel size, design and form of construction.

Downer (1977) provides a history of the construction of a number of the major tunnels but otherwise there is little published information on the history of the tunnels.

The following is a brief simplified chronology of the construction of the major lines still in operation, a significant number of local branch lines having been closed in the second half of the 20th century.

#### 2.1.1 Founding the network in the 19th century

The development of the network commenced in the 1860s with a number of short provincial railways heading from a port into the hinterland. Railway construction began in the South Island and the 2596m long Lyttelton Tunnel, built in 1867 was a significant achievement for the pioneers. The Main South Line (MSL) between Christchurch, Dunedin and Invercargill opened in 1879. In Auckland, the railway to Drury included a 340m long single track brick tunnel between Parnell and Newmarket that was opened in 1873.

In 1870, Julius Vogel set about creating a national network and standardising the track gauge. The private Wellington & Manawatu Railway Company (WMR) opened its line from Wellington to Palmerston North in 1886 which enabled trains to run from Wellington to New Plymouth and later to Napier when the difficult section through the Manawatu Gorge was completed in 1891. The Government's line from Wellington crossed over the Rimutaka ranges using a third rail Fell engine system. Other railways in the North Island went to Thames, Rotorua and Helensville. Progress continued on constructing the North Island Main Trunk (NIMT) which eventually met in the middle in 1908. The Government then purchased the WMR.

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#### 2.1.2 Early 20th century expansion

The first half of the 20th century saw the expansion of the network through the connection of isolated lines to form trunk routes and the construction of new lines including:

- The Midland Line from Christchurch to the West Coast including the 8565m long Otira Tunnel which took 11 years to complete and contained electric traction from when it opened in 1924
- The completion of the North Auckland Line (NAL) which was built progressively from 1880 - 1925
- The Stratford Okahukura Line (SOL) opened in 1933 providing a connection to the Marton – New Plymouth Line and also an alternative route to the central North Island section of the NIMT
- The extension of the Palmerston North to Napier Line onward to Gisborne (the PNGL) opened in 1943
- The central section of the Main North Line (MNL) between Christchurch and Picton opened along the Pacific coastline in 1945.

All of these lines entailed significant tunnelling that accounts for two thirds of today's tunnels.

In addition to completing the network, attention turned to providing deviations to eliminate difficult sections of line. This work involved the construction of the longer and wider tunnels. In 1915 a new double track tunnel was installed parallel to the original one near Newmarket. And, to provide an easier route into Auckland, the eastern line from Otahuhu to Parnell via Glen Innes opened in 1930.

In 1937, two double track tunnels totalling 5.5km eliminated the steep and winding access into Wellington. The Turakina – Fordell deviation opened in 1947 with three tunnels totalling 3.5km. A new record for length was set in 1955 when the 8789m long Rimutaka Tunnel was opened to eliminate the 1 in 15 Fell incline.

#### 2.1.3 Late 20th century

KRN's two newest tunnels show a step change in construction method and size. The Kaimai Tunnel is New Zealand's longest railway tunnel at 8879m. It was also built for a deviation that enabled a more direct route to the port of Tauranga and the Bay of Plenty when it opened in 1978. Finally, the 1272m long Poro-o-Tarao tunnel was opened in 1980 and was constructed to bypass a tunnel on the NIMT that had suffered major lining failures.

#### 2.2 Size and shape of tunnels

The tunnels dealt with in this paper were constructed in the 19th or early 20th centuries. Jones (1987) has set out the evolution of the tunnel profiles over this period. Tunnels built between 1881 and 1892 have the Vogel profile (named after Works Minister Julius Vogel) with an initial height of 4300mm and width of 3800mm. From 1901 through to the 1970s a larger Ward-Cadman profile (named after Ministers of Railways Ward and Cadman) was adopted with an initial height of 4630mm and width of 4530mm.

There was a rapid evolution of the size of locomotives and rolling stock during this period and some of the Vogel profile tunnels on the NIMT had their floors lowered soon after being opened.

One important feature was the shift in the 1930s from a predominant horseshoe shape of the 19th century and early 20th century tunnels to vertical straight walls following American practice. A number of tunnels with vertical walls have exhibited poor structural performance.

Practice in the 19th century was to retain a vertical centreline in curved tunnels rather than tilt the centreline to match the track cant. This creates challenges when determining the optimum track position to maximise clearances in tunnels that include both straight and curved sections.

#### 2.3 Lining form

The early 19th century tunnels and those on the WMR line to Palmerston North are generally either unlined (not common) or masonry (brick or local building stone). Brick linings with two or three layers and various bond arrangements have been observed. Later in the 19th century there was a general transition to in-situ concrete walls with a crown lining of precast concrete blocks. There is only one case of an invert lining.

In the early 20th century there was a move to fully concreted arch linings. In many cases the concrete was of low quality and the construction techniques were still in development. This resulted in poor quality concrete with segregation and variable thicknesses in the crown where concrete appears to have been pushed into place.

The tunnels constructed in the late 20th century have modern cast in situ concrete linings and have not presented any structural or serviceability issues.

#### 2.4 Geological setting

New Zealand is a geologically young country with a wide range of geological conditions which can be very generally subdivided into the following categories:

- Greywacke bedrock, a strong rock which has been heavily faulted, folded and sheared during major tectonic activity over the last 200M years. It can vary locally from massive to very closely jointed sandstone with compressive strengths up to 200MPa through to thick or thin beds of weak fissile argillite. Near the surface greywacke sometimes appears heavily weathered.
- Metamorphic rocks, schists and gneiss, formed by alteration of the greywacke / argillite basement rock. Predominantly found in the South Island.
- Sedimentary rocks laid down from the Tertiary (20M years ago) through to the Pliocene. These are commonly referred to as "papa" or "soft rock" and are characterised as massive thick deposits of mudstones, siltstones, sandstones and limestones all with low strength and many with low resistance to weathering.
- Volcanic deposits of various origins including lava flows, ignimbrite sheets and ash deposits. An important feature of geological settings is the presence of swelling clays (typically smectite or montmorillonite) locally in both volcanic and soft rocks.

#### 3. Tunnel performance

#### 3.1 General condition and records

For most tunnels there are no records of tunnel lining performance issues and no current indications of problems.

A small number of tunnels experienced lining problems soon after commissioning and have been the subject of ongoing review and/ or remedial work. In the case of the NIMT Poroo- Taroa tunnel, replacement has been necessary.

In a number of tunnels investigated for remedial works or invert lowering, some historical lining replacement is evident, although unrecorded. In some cases long-serving KiwiRail employees can recall the work being undertaken.

All tunnels have been subject to periodic inspections. However, these have generally been undertaken by tradesmen from a structures background who lack specific professional engineering or tunnelling expertise. As a result, the reports tend to focus on operational issues such as water falling on rails, invert drainage, track condition and signage and are often of limited value in terms of assessing the severity and history of development of lining issues. There are frequent reports of long standing issues adjacent to portals with water ingress and extensive cracking of the lining. The crack patterns are indicative of the tunnel barrels supporting the portal faces, which were constructed integral with the tunnel lining and without a footing capable of resisting the earth pressures on the back of the portal face. As discussed in the case histories below, measurements have been made in a small number of tunnels to monitor the rate of convergence of the walls.

#### 3.2 Tunnel 'failures'

Of the approximately 200 tunnels constructed on the original network, very few have failed to the extent of requiring bypassing or extensive remedial works. Two tunnels, PNGL 24 and WMR Tunnel 12 were constructed through active landslides and in both cases the tunnel lining suffered collapses soon after opening and the line was diverted around the tunnel. One of the earliest and longest tunnels on the NIMT, Poro-o-Tarao, suffered major lining distress from the 1920s onward due to wall bulging after early lowering and softening of the invert due to heavy water inflows. A deviation with a 1300m long tunnel was opened in 1980 to bypass the original tunnel.

#### 4. Tunnel modification or repair

#### 4.1 Reasons for modification or repair

The tunnel modification and repairs described in this paper arise from three primary reasons which have different timeframes and priorities.

#### 4.1.1 Structural failure

In a small number of tunnels structural failure of the lining has occurred evidenced by bulging of tunnel walls and cracking of the lining. Apart from in the two tunnels constructed through landslides, these failures have been gradual and have not resulted in an actual lining collapse. Because of the three dimensional geometry, determination of the degree of cracking and bulging which is acceptable before remedial action is required to prevent failures is largely a matter of judgement based on simple analyses. In most of the fully investigated case histories of structural failure the causative factor appears to be the presence of swelling ground rather than construction deficiencies or rock loading on the crown.

#### 4.1.2 Lining deterioration

There are a few places on the network where linings were constructed from inferior concrete. Exhaust gases from coal fired steam trains have attacked the lining to the extent that remedial works were undertaken.

#### 4.1.3 Inadequate operational clearances

By far the most common reason for tunnel modifications has been to allow the passage of larger rolling stock and containers or to permit electrification of the line.

#### 4.2 Design philosophy

A major consideration when developing designs for modification works is what design philosophy should be used given that the tunnels were not designed or constructed to modern standards.

#### 4.2.1 Building Act

Prior to the Building Act, 2004, design of public works in New Zealand was to standards established by the relevant government department. The Building Act applies to transportation tunnels, so new tunnels or significant structural modifications to existing tunnels may require a Building Consent. The three basic issues that must be addressed in a Building Consent Application are:

- Strength and structural adequacy
- Achievement of durability for a design life of 50 years
- Fire protection

The Building Code refers to general loadings and material codes, but it does not contain or refer to any specific standards relating to loadings on, structural design of, or fire protection for tunnels.

#### 4.2.2 Modern tunnel design approach

Modern tunnels requiring a supporting lining are generally designed and constructed with the assumption and objective that either:

- An internal support structure is provided (by a structural lining with or without primary support such as steel sets) with filling between the excavated ground structure so there is a positive transfer of loading and reaction between the ground and the support structure; or
- Creation of a reinforced arch in the ground around the opening by rock-bolting with an integrated structural surface membrane where required. In both cases there is a key assumption that there will be intimate load transferring contact between the lining/load supporting elements and the natural ground. With cast in place or precast linings this contact is established by grouting of the space between the lining and the excavated ground surface. The existence of the intimate lining ground contact makes it possible to theoretically analyse the load interaction between the lining and the ground.

#### 4.2.3 Practice in NZ in the 19th century

There is no formal recorded information about a design approach for the 19th century other than references to a rule of thumb that the lining should have 1 inch thickness per foot of width (which generally matches lining thicknesses observed on the WMR and NIMT tunnels). Apart from tunnels in massive self-supporting volcanic rock where a lining is clearly not required and was not provided, there does not appear to have been any systematic adjustment of lining thicknesses to reflect differences in ground conditions. Site investigations beyond observation of exposures were not standard practice. Loose backpacking between masonry tunnel walls and the ground has been observed in most cases but is not effective in creating positive load transfer between the tunnel lining and the ground. Where holes have been drilled through cast in-situ concrete walls, localised gaps between the lining and the ground have been observed but it is not clear whether this is due to poor concrete placement or the result of timber inclusions rotting out. Tunnel lining crown sections appear to have been laid on centering and while backpacking has been observed in some shoulder investigation holes it is unlikely this was extensive.

#### 4.2.4 Design philosophy for repairs and modifications

The modern tunnel design processes described in section 4.2.2 are not applicable to historic tunnels where the lining is not in contact with the ground. The case histories presented here have generally adopted one of the following design philosophies:

- Vertical and horizontal rock loads (see Terzaghi's paper on case histories from early tunnel construction)
- Replacement of like with like (i.e. replicating structural capacity) when replacing elements that have not failed in service over an extended period.

#### 4.3 Structural repairs to linings

#### 4.3.1 Lining replacement

Sections of lining in at least two tunnels (NIMT Tunnel 4 and MSL Tunnel 5 Mihiwaka) have been replaced but details of the reasons for the replacement, the design assumptions, or the construction methodology are unknown. In both cases the new lining is to the same horseshoe profile as the original. Further sections of the lining at Mihiwaka were replaced by KRN in 2009. The tunnel is constructed through volcanic deposits with some sections being unlined and the remainder lined with ashlar masonry in the walls and brick in the crown. Some sections within the lined length have exhibited significant bulging and longitudinal cracking.

Investigations and testing in the 1980s indicated the presence of swelling clay minerals within the breccia matrix. The bulging is generally confined within the walls and the inward movement is of the order of 200mm. It was deemed that if no action was taken the thrust line from the bulged walls would, at some time, pass outside the wall foundations with a risk of failure of the unreinforced masonry wall. KRN is replacing the bulging sections of wall in 1 metre wide hit and treble miss panels using a precast panel which performs two functions. Primarily it acts as formwork for in-situ concrete placed behind the panel and secondly as a component of the new structural wall. Details are shown in Figure 1.

To facilitate construction handling, the thickness of the 3.1m x 1m panel was minimised to provide the flexural strength required for handling stresses and to resist the pressure of the placed concrete. The panel has shear transfer reinforcement on the back

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face to enable the panel to act compositely with the concrete backfill as an unreinforced vertical load bearing wall. It was noted during construction that there was intimate contact between the back of the ashlar lining and the volcanic breccia which is attributed to swelling of the breccia. It was also noted that sections of the ashlar lining on the opposite side of the tunnel had been replaced with in-situ concrete at some time in the past. There are no formal records but anecdotal recollections report that the lining replacement was done in the 1950s.

There are no indications of cracking or bulging of the replacement concrete sections after 50+ years of service. Consideration was given to placing a 'compressible' polystyrene panel behind the in situ concrete to accommodate any future ground swelling. However, the compression available from the polystyrene is limited and the acceptable performance of the 50 year old repairs on the other side of the tunnel was taken as a precedent for a 'like' with 'like' design approach.

#### Figure 1 Mihiwaka Tunnel repair



#### 4.3.2 Provision of support to linings

Major cracking indicating a potential for lining collapse was observed in four tunnels soon after their construction. These are:

- The 574m long Makarau Tunnel 9km north of Helensville on the NAL which has experienced major lining cracking over approximately 70% of its length
- The Fordell and Turakina Tunnels on the Marton New Plymouth line
- Tunnel 26 on the PNGL

#### Makarau Tunnel

Makarau Tunnel was constructed in part through Northern Allochthon, a geological unit of mass transported materials with difficult properties including low strength and swelling potential. The contractor experienced major problems to the extent that he successfully petitioned Parliament in 1897 seeking additional financial compensation. This was on the basis that the conditions encountered had been unexpected and that there were no boreholes that would have indicated the presence of the materials that collapsed during construction.

Within a year of completion of construction in 1897, longitudinal cracking was observed and sets and invert struts fabricated from rail were progressively installed over the next 20 years to support the lining over the section through the Northern Allochthon. While several studies have been undertaken for construction of a bypass cutting or tunnel, the existing tunnel is still in service with the rail lining support.

#### Fordell and Turakina Tunnels

The Fordell and Turakina tunnels were completed around 1947 as part of a 16km long deviation to bypass a section of the MNPL with very poor alignment. The tunnels had straight vertical walls and during construction the lining exhibited substantial cracking and distress leading to strengthening via the construction of a haunch in the lower walls (Kalaugher 1947).

#### **PNGL** tunnels

The tunnels between Wairoa and Gisborne on the PNGL were constructed of in-situ concrete on the same vertical wall profile used on the Fordell and Turakina tunnels. Construction photographs indicate the use of a large amount of temporary timber sets and lagging, which is likely to have been untreated timber and was probably left in place during the concreting. Soon after construction three tunnels on the line developed extensive longitudinal cracking and there were concerns as to whether the tunnel profiles were closing.

A number of cross-sections were instrumented with a pattern of eyebolts that were monitored with a tape extensometer. The monitoring did not disclose any ongoing movements except at one point in Tunnel 26. At that point there was a clear and significant inward movement of the walls (reaching a total closure of 300mm) and with a cracking pattern that could form a failure mechanism. Subsequent drilling investigations disclosed the presence of swelling clay minerals in the tertiary mudstone through which the tunnel was constructed at that point. In 1985, a 16m long section in Tunnel 26 was supported internally by steel sets formed by welding straight lengths of 200UC60 section. Invert struts and the wall sections of the sets were connected with tie bolts and encased in cast in situ concrete.

#### 4.4 Lining deterioration repairs

The Stratford Okahukura Line tunnels constructed in the 1930s have cast in situ concrete linings. The line traverses country with no road access and was constructed on two fronts from both ends. The tunnel concrete was batched using the aggregate available. At the eastern end sound river gravels were available but at the western end the available aggregate was "shell rock" which is a very weak lime cemented conglomerate of sand and shells. The concrete made from the shell rock was poor with low durability and very quickly started to break down under the attack from the acidic steam locomotive exhaust. As a result the surface of the concrete became chalky and began to exfoliate.

To halt this deterioration, remedial works were undertaken in the 1950s under contract by Downer Construction and Fletcher Construction. The works involved stripping the deteriorated concrete from the surface of the tunnels and applying a gunite (sprayed mortar) surface. These repairs were effective. The key activity was the removal of the deteriorated concrete and both contractors used an ingenious arrangement comprising a winch mounted on the last of a rake of wagons (personal comm. from R Foster, Fletcher contract manager). Chains were attached to the rim of the winch to act as flails to dislodge the deteriorated concrete and the winch position could be adjusted so that the chains could attack all parts of the tunnel perimeter. There have been a few incidents, fortunately rare and not necessitating extensive repairs, of concrete falling from the tunnel crown on both the SOL and PNGL. These falls have been from the crown where, due to inadequate placing equipment and techniques at the time, the cast in situ concrete thickness can be as little as 25mm adjacent to the construction joints.

#### 4.5 Lining enlargement

#### 4.5.1 Original Poro-o-Tarao tunnel

A trial enlargement of the lining profile in a 12m length of the the Poro-o-Tarao tunnel was undertaken by NZ Government Railways (NZR) during the 1960s. Poro-o-Tarao was one of the smallest tunnels on the NIMT being constructed with a brick lining to the smaller Vogel profile between 1885 and 1891. As a result it was one of the first tunnels to have its invert lowered to allow larger rolling stock to pass.

The tunnel is constructed through weak mudstones with considerable ground water inflow. Steel props were installed to support the base of the walls which were above the lowered invert excavation. Failures of these props and of the tunnel dewatering system resulted in local areas of significant inward movement and cracking of the brickwork sufficient to allow insertion of an arm up to the elbow.

In 1934 a 45m bulging length was reconstructed to the original profile with a reinforced concrete invert concrete. In 1965 a further 12m bulged length was replaced both to improve

clearances and to investigate the cost and feasibility of enlarging the full tunnel length under service. The enlargement method is described by Webley (1970). A total of only 29 hours of occupation in 6 different occasions was available per week with the longest individual occupation being 10 hours. The trial was completed by NZR staff after two contractors failed to perform. The proposal to enlarge the existing tunnel under service was abandoned after it became apparent that it would take at least 9 years to enlarge the tunnel at twice the cost of a new tunnel. A 1300m long deviation tunnel was constructed to replace the original tunnel.

#### 4.5.2 Poro-o-Tarao monitoring plan

Towards the end of the construction of the replacement Poroo-Tarao tunnel, there were concerns that a collapse of some sections of the lining in the original tunnel might occur before the new tunnel was available. Closure of the tunnel would have closed the line from Auckland to Wellington, there being no alternative detour route available. Accordingly NZR asked MWD to instigate monitoring and prepare contingency plans (MWD 1976) to reinstate the tunnel in the event of a lining collapse. NZR has records from the 1920s of the progressive reduction in the tunnel cross-section. These records were obtained using a "sunflower" wagon and from direct measurement of the tunnel width.

The sunflower wagon provided a composite profile of the tunnel relative to the track position using a series of adjustable arms with a number of lead strips of different lengths at the outer end. The arms were spaced radially at 10 degree intervals. By towing the sunflower wagon through the tunnel a number of times and adjusting the position of the arms until one or more lead strips on each arm was bent over, it is possible to establish a composite profile. The closure records indicated that since the 1920s the tunnel appeared to have had two average closure rates: 2-3mm per year during periods when tunnel drainage was working and effective strutting was in place, and 8-10mm per year when tunnel drainage and lateral support to the lining footing had been allowed to deteriorate.

MWD installed instrumentation at a number of cross sections. This comprised both an array of points to be monitored by a tape extensometer to determine liming distortions and closures plus custom manufactured magnetic target borehole extensometers to measure absolutely the inward movement of the lining and movements in the ground behind the lining. These instruments were read monthly and had an assessed ability to monitor convergence with an accuracy of + or - 0.1mm. At one stage the monitoring over a three month period suggested that movements had increased from an average rate of 2-3mm per year to an annual rate of 6mm. An inspection established that the central invert drain flushing system had blocked and some of the steel props appeared to be buckling.

#### 4.5.3 Poro-o-Tarao contingency plan

The contingency plan for reopening the existing tunnel in the event of a collapse (MWD 1976) was based on immediately stopping work on the replacement tunnel and re-deploying equipment and resources into the existing tunnel. Designs were drawn up for a self-contained rail mounted work train supporting a road header excavator with muck conveyors and with shotcreting and rock bolting stations. NZR flat deck and well deck wagons suitable for forming the work train were identified and the necessary modifications designed.

The proposed reinstatement method was based on forming reinforced shotcrete arches in slots cut into the country and spanning between those with mesh reinforced shotcrete. The method was based on similar repairs carried out in European tunnels. Fortunately, the existing tunnel remained in service until the new tunnel was opened and it was not necessary to implement the contingency plan.

#### 4.5.4 NIMT electrification feasibility study

Prior to the electrification of the NIMT between Frankton and Palmerston North in the 1980s, consideration was given to enlarging a number of the tunnels under service as an alternative to lowering the inverts. MWD undertook a detailed feasibility study in 1982 which considered alternative means of enlargement. The brief specified one 3-day long occupation per week. The study concluded that a construction period of 2 years would be required for the longest (600m) tunnel. The work would also have a high cost as parallel simultaneous excavation and lining operations were proposed with a substantial workforce being paid a full week's work although the actual working time would be just 3 days. Neither the time nor the cost was acceptable and the enlargement did not proceed.

#### 4.5.5 Johnsonville tunnels

Part of the WRRP project (Gordon 2010) involves modifications to the Johnsonville commuter line to enable the new Matangi EMUs to pass through the tunnels. Tunnel invert lowering was identified as the primary means of achieving this. However, for programming reasons, modifications work needed to be undertaken before the characteristics of the new EMUs had been finalised. The initial analyses indicated that along approximately 210m of tunnel wall the clearances achievable with invert lowering alone would be insufficient and some wall modifications would be required. A contingency design was undertaken and Building Consents obtained for replacing the inclined lower brick wall with a precast panel with a recess using the arrangement shown in Figure 2. The panel and rock bolts were designed using the loads established for invert lowering as described in section 5.5.



When the final EMU characteristics became available it was established that the widening was unnecessary and the modifications did not proceed.

#### 5. Invert lowering

#### 5.1 Background

Over the last 50 years there has been a progressive increase in the height of wagons and their loads as container heights have increased from 8' 6" to 9' 6" and now to 10' and with the introduction of industry specific rolling stock such as milk tankers. The height of tunnels on a route has been a major constraint and there have been periods (Rails 1998) when it has been necessary to unload the larger containers from rail to tranship them by road past undersize tunnels. Lowering of a tunnel invert was established at an early stage as an effective means of gaining additional height. However, where the invert lowering excavation extends below the foot of the original lining there is no longer a lateral earth pressure load to support the base of the wall against inward movement. The electrification of the NIMT in the 1980ss and the upcoming electrification of the Auckland commuter network also requires track and invert lowering to accommodate the traction overheads.

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#### 5.2 Early approach to invert lowering

In early invert lowering cases restraint to the base of the wall's lower walls was restored either by installing a precast trough invert section as shown in Figure 1, or by propping the walls off each other using various arrangements of steel and cast in situ or precast concrete struts installed between the sleepers. One example is the strutting in the NIMT tunnels between Pukerua Bay and Paekakariki described in Rails (1996). The article also describes the placing of concrete in the floor of one tunnel to encase the sleepers and struts and effectively form a slab invert.

Some early trials with rock bolts were undertaken by NZR using the expansion shell anchors available at the time but these were unsuccessful. The major drawback of using struts between the walls is the complication of track and invert drainage maintenance because the struts prevent the use of normal ballast tamping equipment and severely hamper access to central invert drains.

#### 5.3 NIMT electrification invert lowering

The NIMT electrification required lowering of a number of tunnels which could not be daylighted or bypassed. MWD was asked to investigate the options and feasibility of enlarging the tunnels (described above) or lowering the invert with provision of either a slab track or ballast trough, the latter being the NZR preference. All work had to be undertaken in limited occupations.

#### 5.3.1 The options

After considering the slab track and invert trough options, MWD proposed an alternative of restraining the walls with the system of rock bolts and steel channel walers shown in Figure 3. The advantage of this system was the ability to install the wall support in a number of occupations without any need to disturb the track. The invert lowering preparation of the new formation and track installation could then be undertaken in a single operation. The proprietary Perfo rock bolt type selected had been successfully used by MWD on a number of other projects in materials similar to the siltstone and sandstone "papa" rocks at the tunnels to be treated.

From previous projects indicative design bond stresses had been established. It had been noted that for consistent achievement of the pullout bond stress careful drilling of the hole to the correct depth and diameter was essential. This is because the system relies on a measured amount of grout introduced in a perforated tube (formed from two half shells) being forced out into the surrounding annulus by the introduction of a threaded rock-bolt driven with a percussion hammer.





This system had the advantage of not relying on mechanical anchorages which had been found to be problematical in papa. After installation of the rock-bolts and time for the mortar to cure, the walers were installed and the nuts of the bolts installed and torqued to an equivalent 50kN bolt load.

#### 5.3.2 Design loadings

The determination of design loadings for the rock bolt system was approached using vertical and horizontal rock loadings proposed for different ground conditions (Terzaghi, 1968). The tunnels being treated were constructed with cast in situ walls and crown formed from concrete blocks and there were no indications of water inflow or lining distress. Further, there were a number of examples of single lane road tunnels in similar materials which had stood unsupported and unlined without any problems. It was therefore quite possible that the loadings on the tunnel linings might be small.

Modern tunnel design approaches did not appear to be useful for determining loadings on the linings as they are based on a tunnel analysis and design philosophy assuming intimate contact between the ground and the supporting lining, which was not the case with the NIMT tunnels. Application of Terzaghi's method with the assumption of a relatively competent ground class produced loads that appeared to be sensible and similar the working capacity of struts used up to that time. The loads could be resisted with a reasonable rock bolt size spacing and waler section. The construction is described by Jones (1987) and proceeded without difficulty.

There are no indications that the system has not been effective. However, the style of rock bolt used does not appear to still be in use. It does not provide the double corrosion protection required by modern practice and to comply with the Building Act.

#### 5.4 Manawatu Gorge tunnel

Lowering of PNGL Tunnels 1 & 2 in the Manawatu Gorge was undertaken in 2008 using an identical waler section and a similar bolting arrangement to the NIMT Electrification project. However, the Perfo bolt was replaced with galvanised Reidbar - a proprietary high strength deformed bar system with a matching nut and a range of termination hardware. The Reidbar was grouted into place with grout introduced to the base of the hole through a plastic tube taped to the bar. Bar to ground bond stress had been established with test anchors installed outside the tunnel in two different ground conditions.

The only significant difference between the NIMT and Manawatu Gorge applications was the use of a grout rather than a mortar for fixing the rockbolt. This led to the possibility of both grout flowing out into any gaps behind the cast in situ wall (which was observed) and also of the grout flowing out to the lining surface and removing the ability to tension the bar. The latter was addressed by wrapping the outer 500mm length of the bar with Denso tape to de-bond it. No problems were experienced with installing the bolts and walers, though the curvature in the tunnel did present some challenges.

#### 5.5 Johnsonville tunnels

#### 5.5.1 Background

The Wellington Rail Rejuvenation Project (WRRP) described by Gordon (2010) includes the replacement of 1950s English Electric EMUs with new Matangi EMUs. The 1970s Ganz Mavag EMUs units currently in service will be retained but cannot be used on the Johnsonville branch commuter line because of power limitations, and future services will be provided with the new Matangi units.

The Johnsonville line was constructed by the Wellington Manawatu Railway Company in the 1880s as part of their line to Longburn near Palmerston North. It was taken over by the government and became part of the NIMT in 1908. In 1938 it was truncated at Johnsonville and became a branch line when the twin track Tawa deviation tunnels were opened. The line has seven single track brick lined tunnels with a total length of a 944m and is constructed through the regional greywacke bedrock formation. In 2008 KRN established the likely need to modify the track position to allow passage of the new Matangi EMUs and engaged Connell Wagner (now Aurecon) to undertake laser surveys of the tunnels and create a digital model of the internal tunnel surface. Aurecon used this to establish the track level to achieve vertical clearances and the horizontal rail position which optimised the clearances from the moving EMU to the tunnel lining.

The initial analyses indicated that to achieve the dynamic clearances sought by KRN the track would need to be lowered by around 550mm and that the horizontal clearances varied throughout the tunnels because of track cant in curved sections. In most sections horizontal clearances within the permissible range could be achieved, although the amount to which surface mounted walers plates and rockbolts could protrude beyond the lining face varied.

#### 5.5.2 Lowering design

In June 2008 KRN appointed Beca in association with Parsons Brinkerhoff (PB) to determine concepts for lowering the inverts and tunnel widening (where required) and to prepare designs and consenting and construction documentation to enable the tunnels to be lowered in a scheduled closedown over the following December – January.

Investigations indicated the walls to be two brick thickness (alternating header and stretcher courses) contrary to file records which suggested a three brick thickness. Subsequent investigations indicated the crown to be also two bricks thick in a stretcher bond. A preliminary scoping report was prepared largely based on the studies and designs for the tunnel works during the 1980s NIMT electrification.

However it was necessary to recognise and allow for the major difference between the Johnsonville tunnels (with a brick arch with loose spall back packing behind the walls) and the NIMT tunnels which had in situ concrete walls cast against the country. The proposals therefore included low pressure void filling grouting of the gap behind the lower wall sections to provide a resistance for the rock bolt tensioning. Also, to reflect the varying permitted protrusion of hardware beyond the lining surface, a number of concepts were developed including surface mounted and recessed UC walers and a "top hat" recessed plate concept identified by Novare a consultant to KRN.

A workshop was held which concluded there was neither justification nor finances and time for overall enlargement and relining of the tunnels. Key concerns expressed and noted during the workshop included the possibility of extensive grout loss

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through and underneath the lining and the potential for grouting pressures to "blow" the lining off. PB had experienced problems on tunnel refurbishments at Folkestone in the UK where the grouting operations were abandoned as a result. Also a tunnel engineer who had supervised construction of a two lane road tunnel in Wellington expressed concerns that ground conditions could vary significantly over short distances and that difficulties might be experienced with rock bolt drilling and anchorage.

After the workshop, construction drawings and specifications, and building consent applications were developed for the general arrangements shown in Figure 4. To address the concerns of variable ground conditions, alternative designs were developed for two classes of ground, "Good" and "Poor", with loadings based on Terzaghi (1968). It was assumed that lateral support would be initially provided assuming "Good" ground conditions given that there was no evidence of significant poor ground or water inflow from the, albeit widely spaced, investigation bore holes or from lining inspections. The design included the ability to increase the amount of lateral support (closer spaced rock bolts where walers were used) if inspections of the invert after exposure for lowering indicated "Poor" ground conditions.

A major consideration was the need to obtain Building Consents and the requirement for all structural components to have a design life of 50 years. For the rock bolts this implied provision of a double corrosion protection system. The rock bolt types previously used on the NIMT and Manawatu Gorge lowering had only a single protection system (galvanising). A proprietary bolt system (Strata Control CT - similar in principle to the Dywidag DCP system) was identified which provided double corrosion protection, the first layer being provided by a HDPE sheath and the second by grout confined between the sheath and the bolt. Potential drawbacks recognised with this system were that when the bolt is installed initial tension is applied (prior to grouting) using a mechanical expansion shell anchorage and, once the bolt has been grouted, it is not possible to re-tension it or remove any hardware between the head of the bolt and the wall.

The rock bolt specification included a range of preproduction proving tests, production quality control tests and the installation of sacrificial witness bolts (installed to the production specification) which can be drilled out in future if required to check the bolt condition. For the "good ground" condition, a 50kN preload in the rockbolts was sufficient to allow the vertical load on the lining predicted using the Terzaghi approach to be resisted in friction in the lining grout ground system. With the higher loadings in "poor ground" and the lower friction values, friction alone was insufficient and some additional lateral support to the base of the lining was required. Accordingly, it was proposed that in "poor ground" a wall of 100mm diameter 2m long grouted stainless steel mini-piles at 180mm centres would be provided immediately alongside the foot of the lining. The mini-piles were prefabricated and comprised a Dywidag bar grouted inside a steel tube.

#### Figure 4 Johnsonville tunnel invert lowering



### 5.5.3 Programming of building consents and contractor engagement

Because of tight programme requirements, the contact grouting and rock bolting contractors were separately engaged. The grouting, which was required to be complete before rock bolts could be installed, proceeded in parallel with the obtaining the Building Consents for the rock bolting systems restoring lateral support to the foot of the linings. After the actual extent of lowering required had been established, a separate Building Consent application was lodged for the actual invert lowering (and resistance of vertical loads on the lining). This was processed while installation of the rock bolts was proceeding.

#### 5.5.4 Construction of lateral support

The grouting of the void behind the lining and the installation of rockbolts and walers or top-hats over the 900m of tunnels was undertaken by two separate contractors in four weekday evening occupations of 10 hours and occasional full weekend occupations. This was possible as the line is a passenger only branch line with moderate passenger numbers and hence trains could be replaced by buses.

The grouting proceeded without incident, the contractor opting to grout in three lifts rather than in a single lift with provision of temporary lateral support. No grout migration through the lining was observed and an atypical high grout take was observed at only one location adjacent to a man refuge. During ballast removal and invert excavation prior to the invert lowering there was one report of a small volume of "conglomerate" being encountered and this would have been grout impregnated ballast.

During the rock bolt contract tender period one tenderer expressed severe reservations regarding the ability to drill the holes and a successful above ground trial was undertaken in an adjacent quarry. The rockbolting and waler/plate installation was completed within the limited construction period but only after additional occupations were made available and the contractor had augmented the originally proposed single drilling rig with two further rigs.

In initial trials and production there were problems with low drilling production rates and with some failures of the mechanical anchorages to grip, meaning the required 50kN preload could not be applied, and necessitating recovery and replacement of those bolts. The ability to achieve anticipated production rates was exacerbated by the short contract establishment and mobilisation period available to the contractor.

To assist in overcoming initial production difficulties, PB arranged for a professional tunnel construction adviser experienced with the drilling equipment and rock bolt installation to visit the site and advise the contractor and KRN supervision staff on equipment settings and drilling and bolt installation practice.

#### 5.5.5 Invert lowering operation

The invert lowering operation was planned and undertaken by KRN staff and contractors. The adopted process after removal of the track and ballast was to use a large 1.2m diameter tungsten carbide tipped saw to cut a vertical face into the bedrock 100mm out from the tunnel lining face and to the required depth. The rock to be excavated was then broken up with a hydraulic breaker mounted on a digger, pushed up into heaps with a dozer and removed with rubber tyred loaders.

After the new invert had been exposed and before the drainage blankets and new ballast was placed, the invert was logged and recorded by a Beca engineering geologist. It was then inspected by the Beca tunnel designer to ascertain whether any "poor ground" had been encountered and whether any remedial works were required to repair rock falls from beneath the foot of the lining. In the event no "poor" ground was observed and no remedial works were required.

The total operation of track removal lowering and track replacement was achieved within the programme, the only significant issue being that where the rock saw encountered a metal item, such as an old rail spike, the tungsten carbide tips were stripped from the blades. After discussion it was agreed to obtain replacement blades rather than proceed without the vertical saw cuts and run the risk of over break disturbing rock beneath the tunnel lining footing.

#### 6. Conclusions

This paper reports on the generally acceptable performance of the large number of railway tunnels built in NZ through the latter half of the 19th century and first half of the 20th century. Despite being built to simple designs well below current "good practice" they have performed remarkably well. However these simple designs did not prove adequate where swelling ground conditions are present. The replacement in the 1930s of the traditional horseshoe profile with a profile with vertical walls was unsuccessful. A significant number of tunnels with the new profile have exhibited extensive longitudinal cracking.

This paper discusses a number of successful tunnel lining repair and invert lowering operations which have however, frequently taken longer and cost more than anticipated. It is suggested that this has often been a consequence of tight programmes, inadequate planning, and an unwillingness to develop specific equipment and procedures for establishing and disestablishing operations in the very limited occupation times generally available.

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