# Design of an Innovative Single-Pass Tunnel Lining for Watercare's Central Interceptor Tunnel

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## ABSTRACT

The Central Interceptor (CI) main tunnel is a 14.7-kilometre-long, 4.5-metre-diameter sewer tunnel currently in construction under central Auckland. It will provide additional capacity in the network to meet planned population grown and development, provide a more resilient wastewater system, and mitigate west weather overflows. The tunnel runs from the Mangere Wastewater Treatment Plant, passes under central Auckland and finishes in Tawariki. It will be New Zealand's largest and longest bored wastewater tunnel. The single-pass tunnel lining is designed for a 100-year design life. It consists of 324.5-millimetre-thick concrete lining with an inner 3-millimetre high-density polyethylene (HDPE) corrosion protection layer. The thickness of concrete lining includes a sacrificial concrete layer per the client's specification.

As the tunnel will be excavated through a range of different ground conditions, going from very loose sand to competent rock with an overburden ranging from 15 metres to 110 metres above the tunnel crown, several segment reinforcement types have been designed for the CI main tunnel. In the area where the tunnel encounters soft ground with low overburden, three types of segments using hybrid reinforced concrete (rebar and steel fibre) are being utilised to take into account the highly variable design conditions. For the remaining alignment that is fully in rock, steel fibre reinforced concrete (SFRC) segments are being utilised. Because of the inherent uncertainty of ground conditions along the tunnel length, several strategies are being employed to align the segment type and associated design assumptions with the conditions encountered during tunnelling.

This paper will discuss the challenges, lesson learned and detailed considerations in designing the tunnel segmental lining from the structural, geotechnical and constructability perspectives. Furthermore, some of the observations and lessons learned from segment production and lining installation will also be discussed, with recommendations for improvements relevant to similar future projects.

## 1. INTRODUCTION

Watercare is building a 14.7 km-long underground wastewater tunnel called the Central Interceptor (CI) that will run from Māngere Wastewater Treatment Plant to Grey Lynn in central Auckland. The 4.5m diameter main tunnel is being dug by a large tunnel boring machine named *Hiwa-i-te-Rangi*. Two smaller link sewers (2.4m and 2.1m diameters for Link Sewers B and C respectively) being constructed as pipejacks will intersect the main tunnel to connect the current wastewater network to the main CI tunnel. The CI tunnel alignments are shown on a map of Auckland in Figure 1 along with the existing wastewater network and significant overflow locations.

Ghella Abergeldie JV is delivering the project for Watercare. The contractor's designer is Arup. Watercare's design team includes Jacobs, AECOM and Delve Underground. Construction is taking place at 16 sites across Auckland and involves the excavation of 17 shafts and associated infrastructure. Around 600 staff from the various project partners are working on the $1.2b project, which is due for completion in 2026. It is the largest wastewater infrastructure project in New Zealand history and will leave a legacy of cleaner waterways by reducing around 80 per cent of wet-weather overflows in central Auckland by capturing and storing combined stormwater and wastewater flows and sending them to Māngere Wastewater Treatment Plant for processing at a controlled rate (Watercare, 2016).

The CI main tunnel consists of a single pass segmental lining. This paper discusses the design development of CI main tunnel lining from the initial reference design stage through detailed design phase and construction. The considerations behind the adoption of various design elements are discussed along with lessons learned on the design during construction.

### 1.1. Project Geologic Conditions

The CI main tunnel TBM, *Hiwa-i-te-Rangi* was launched from a launch shaft at the Mangere Pump Station and will travel through the various shaft sites ending in Grey Lynn. The geologic conditions along the alignment are highly variable starting with, alluvial soils from the Tauranga Group and Kaawa Formation, which generally consists of soft to firm clay and silt as well as loose to dense sand. As the tunnel progresses, the geology turns to East Coast Bay Formation (ECBF), which is the material anticipated for the rest of the tunnel alignment until *Hiwa-i-te-Rangi* arrives in Grey Lynn. The ECBF in this project generally consists of interbedded mudstones, siltstones, and sandstones with varying degrees of cementation. A subunit of ECBF known as Parnell Volcaniclastic Conglomerate is also anticipated to be encountered further along the alignment. This is characterized by pebble to boulder size conglomerate and can generally be very permeable. Additionally, while there are no known active faults in the proximity of the project, the tunnel was expected to encounter a number of inactive faults (Kenny, Linsay and Howe, 2012) where heavily fractured/crushed rock can be expected. A geological profile of the CI main tunnel and the design geotechnical design parameters are shown in Figure 1 and Table 1 respectively.

The main CI tunnel runs approximately between 15 to 110m below the ground surface, including a 1500m undersea section, in which the tunnel crosses from Māngere Bridge to Hillsborough. The minimum overburden occurs below the Manukau Harbour while the maximum overburden occurs in between the PS23 site and Keith Hay Park shaft which is also where the maximum hydrostatic pressure of 8.7 bar was encountered.

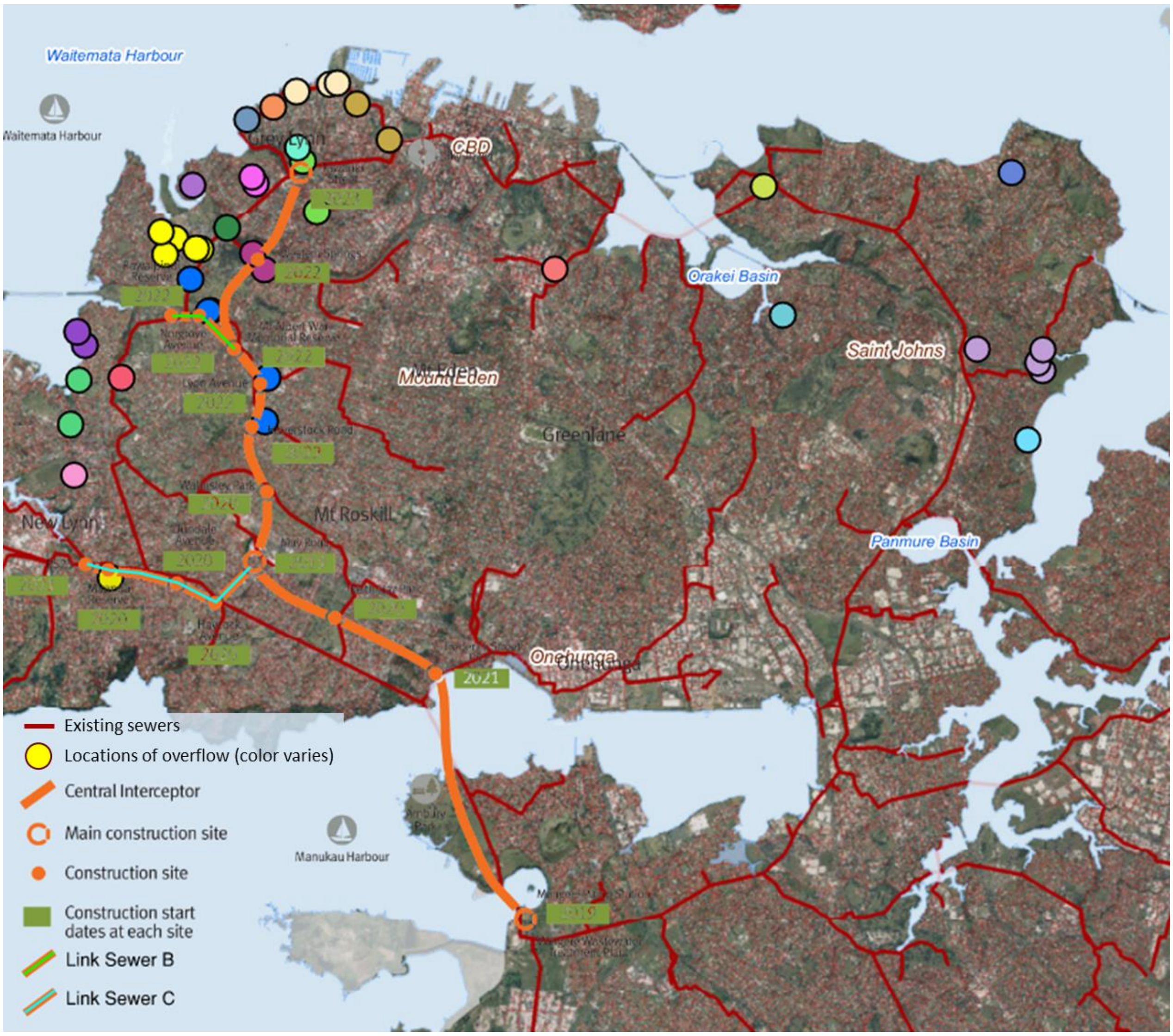


*Figure 1. Central Interceptor alignment with existing wastewater network and current significant overflow locations (Watercare, 2016).*

*Table 1. Geotechnical design parameters.*

| Geological Unit | Unit Weight, γ (kN/m$^3$) | Cohesion, c' (kPa) | Friction Angle, Φ' (°) | Soil / Rock Mass Modulus, E (MPa) | Poisson's Ratio, ν (-) | Unconfined Compressive Strength, UCS (MPa) |
|---|---|---|---|---|---|---|
| **Made Ground** | 15 | 1 | 32 | 25 | 0.3 | - |
| **Tauranga Group (Granular)** | 17 | 0 | 30 | 10 | 0.3 | - |
| **Tauranga Group (Cohesive)** | 16 | 7 | 28 | 7 | 0.4 | - |
| **Kaawa Formation** | 19 | 25 | 32 | 20 | 0.3 | - |
| **Basalt** | 27 | 200 | 50 | 3000 | 0.35 | 120 |
| **ECBF Residual Soil (Granular)** | 20 | 0 | 40 | 50 | 0.3 | - |
| **ECBF Residual Soil (Cohesive)** | 19 | 6 | 32 | 30 | 0.4 | - |
| **ECBF Rock** | 20 | 100 | 34 | 400 | 0.25 | 2 |
| **Parnell Volcaniclastic Conglomerate** | 20 | 140 | 40 | 700 | 0.1 | 10 |

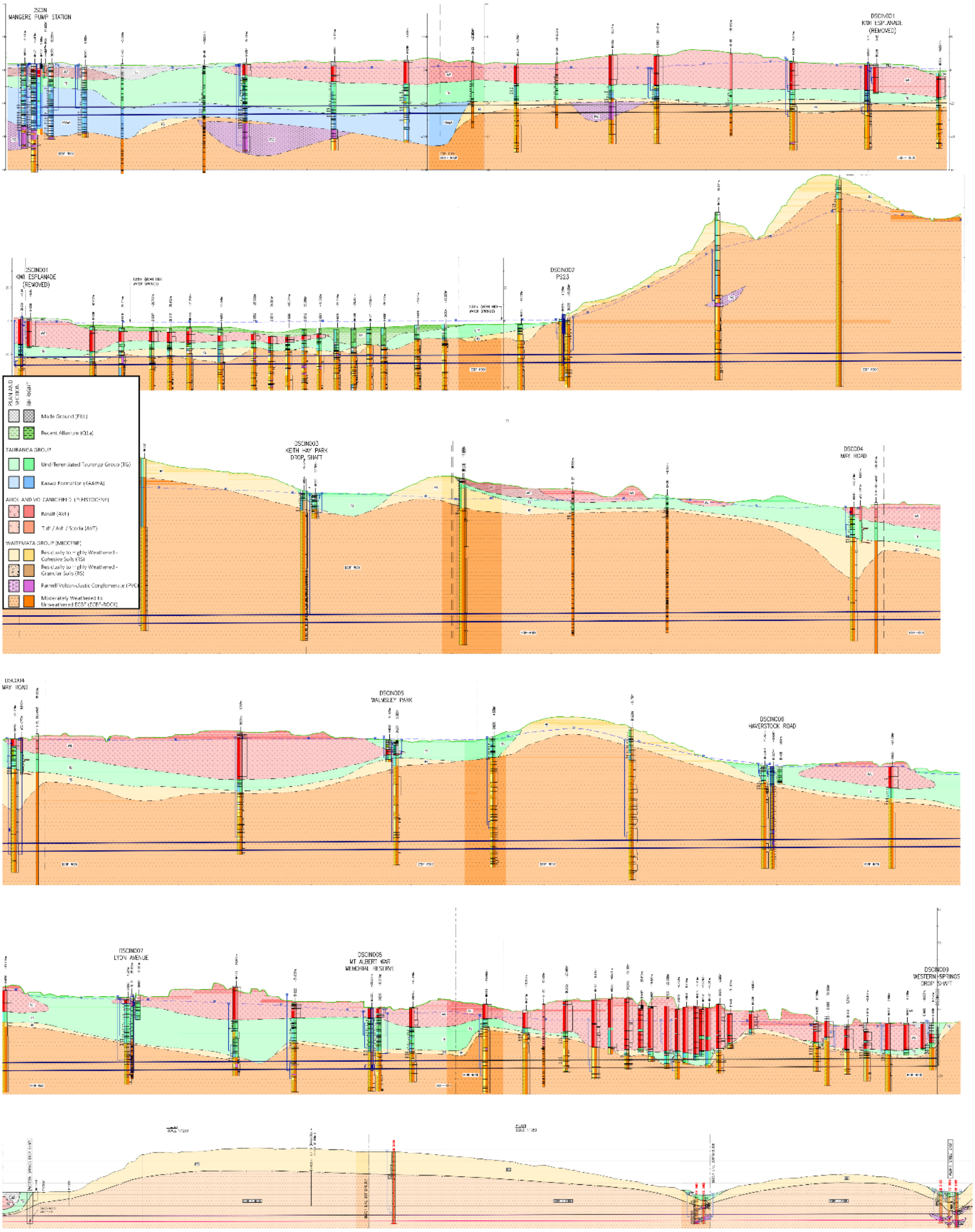


**Figure 2. Geological profile of CI main tunnel.**

## 2. REFERENCE DESIGN DEVELOPMENT

The CI main tunnel is anticipated to encounter highly corrosive conditions internally and fairly corrosive conditions externally throughout its design life. Therefore, during the reference design stage, one of the main design concerns was to provide a design scheme that can achieve the 100-year design life. Being the largest wastewater infrastructure in New Zealand, and considering the significant investment required, a large amount of desktop studies and testing were carried out as discussed in Grace, Kay and Goff (2019). In the end, a single pass lining solution with cast-in high density polyethylene (HDPE) lining was adopted for the main tunnel lining. This solution offers significant savings in terms of material usage and construction time as compared to a two-pass lining solution.

The 100-year design life is to be provided by the HDPE lining, with additional sacrificial concrete thickness provided behind the HDPE lining based on an estimated corrosion of 50 years. The additional sacrificial layer provides protection in the case of localised damage to the HDPE membrane with the damage to the membrane assumed to be identified and repaired within the 50-year timeframe.

For the external conditions, additional soil and groundwater testing was required to be conducted during the construction stage. Refer to further discussion on the external conditions in the Detailed Design Development section below.

As a follow-on design decision from selecting a single-pass lining with an HDPE membrane, dowels and guiding rods were specified in lieu of bolts in the reference design which eliminates the need to fill the bolt pockets after the completion of tunnelling. This removes the need to patch HDPE membrane at bolt pocket locations, which offers significant advantages to the construction productivity. In addition, lifting sockets were specified in place of a vacuum lifter to reduce the risk of damage to the HDPE lining during lifting.

## 3. DETAILED DESIGN DEVELOPMENT

The reference design was completed by Watercare's designers prior to bidding the project to establish the required performance criteria and project constraints. Upon the award of contract, the detailed design of the tunnel lining was further expanded by the contractor's designer to suit the contractor's construction methodologies. The detailed design development is discussed in the following sections.

Through discussions with Watercare during detailed design, it was made clear that sustainability is of paramount importance for the client. Therefore, sustainability initiatives were adopted in the tunnel lining design to minimize the carbon footprint. These initiatives are also discussed in the following sections where appropriate.

### 3.1. General Segment Geometry

The segment geometry was maintained from the reference design with a tunnel internal diameter of 4.5m and the use of a single pass lining arrangement including an HDPE lining and sacrificial concrete thickness.

For the HDPE lining, the critical consideration for design is drainage. As the HDPE lining is not capable of withstanding the external hydrostatic pressure in the case that there is leakage through the concrete behind the HDPE lining, segment joints at the tunnel invert are not to be welded to provide pressure relief. The 6+0 segmentation was maintained from the reference design to ensure that the unwelded joints are located within the dry weather flow as shown in Figure 3.

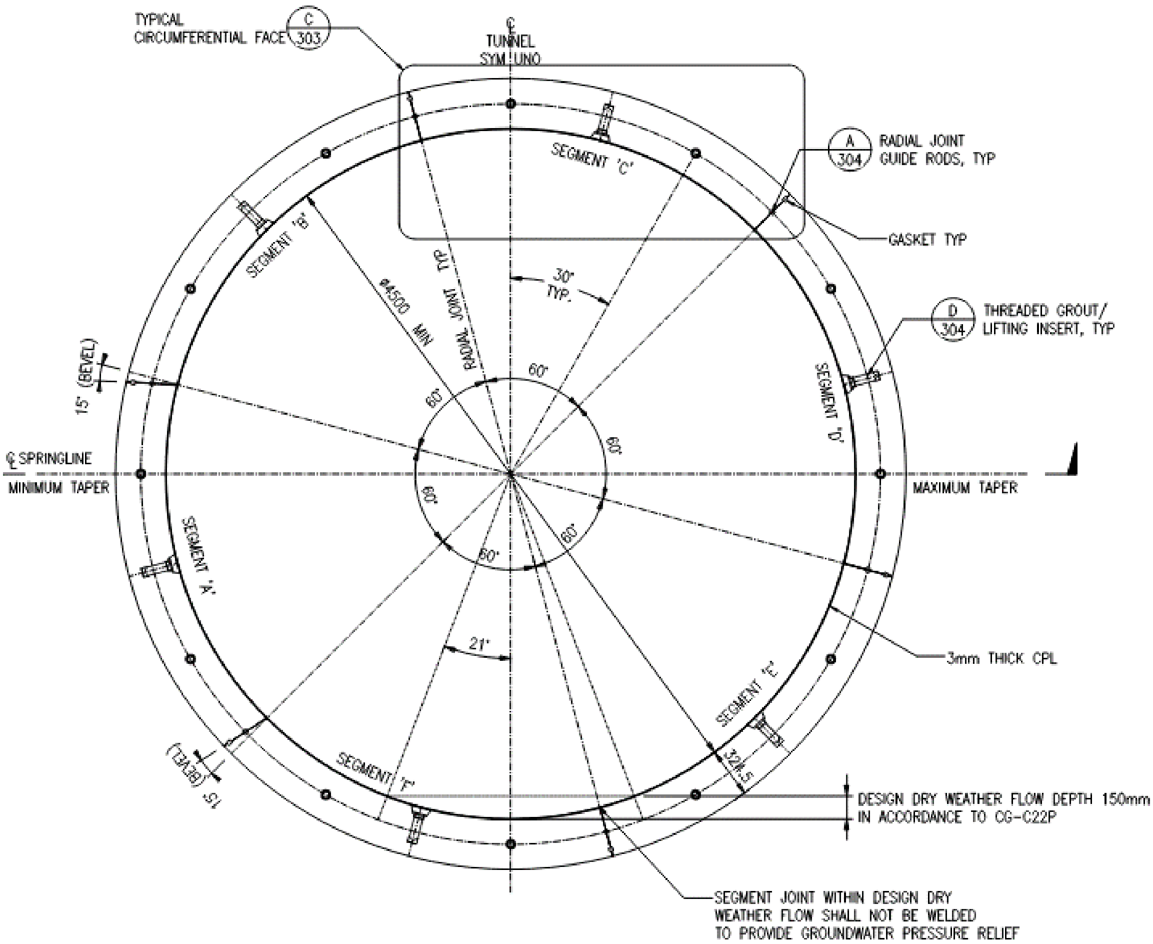


**Figure 3. Cross section view of CI main tunnel.**

For the sacrificial concrete thickness, two (2) different sacrificial concrete thicknesses were specified for the Southern and Northern tunnel stretches respectively as shown in Table 2. These thicknesses are based on the detailed study of the internal durability conducted by Watercare during the reference design (Grace, Kay and Goff, 2019). On top of the sacrificial thickness, an additional 20mm cover was specified to ensure there is sufficient concrete embedment for the reinforcements if the sacrificial concrete were to be corroded.

**Table 2. Sacrificial concrete thickness considered for CI main tunnel**

| Structure | Sacrificial thickness (mm) | Cover to reinforcement (mm) |
|---|---|---|
| Main tunnel (Southern section) | 78 | 98 |
| Main tunnel (Northern section) | 44 | 64 |

The 327.5mm single pass lining thickness consists of the following:

- 3mm-thick HDPE lining, which serves as the main corrosion protection layer of the tunnel lining, also identified as the corrosion protection lining (CPL)
- 44 to 78mm-thick sacrificial concrete layer (for Northern and Southern sections respectively, see Table 2) which serves as secondary corrosion protection
- 20mm remaining cover to reinforcement
- 226.5-260.5mm structural concrete layer

In order to suit the stroke length of the TBM, an additional 15-degree bevel angle was specified in the key segment ‘A’ as shown in Figure4.

Additionally, the segment accessories were further considered in the detailed design to optimise construction methodologies. Given the high water pressures expected to be encountered along the tunnel alignment, the tunnel gaskets have been designed to withstand the full hydrostatic pressure of approximately 9bar. To minimize the amount of segment handling on site, a cast-in gasket from FAMA was chosen.

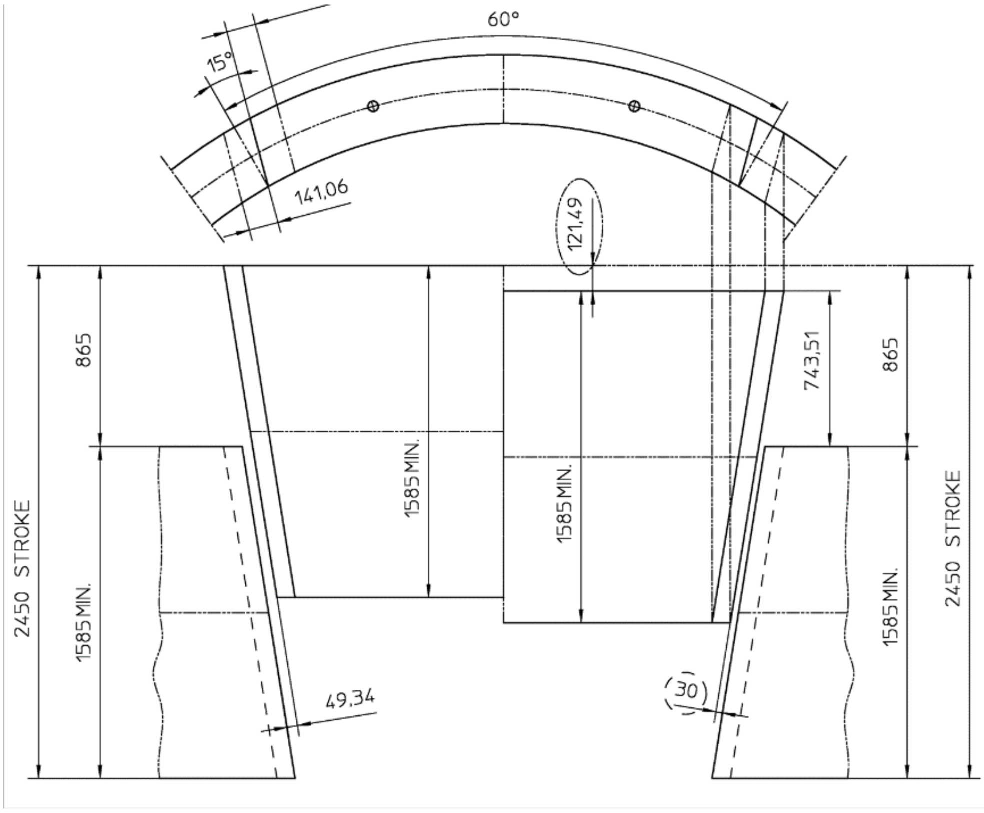


**Figure 4. Key segment bevel geometry check to ensure constructability due to limitation on stroke length of TBM (source: Herrenknecht).**

### 3.2. Structural Design

Due to the large sacrificial concrete thickness on the intrados in all segments, steel fibre reinforcement is adopted in the tunnel lining to control cracking in this thickness. The contribution of the steel fibre for crack control was calculated based on the method specified in NZS 3101:2006, where the tensile stress in the concrete due to fibre contribution was calculated as 0.45 $f_{R,1}$.

Otherwise however the segment reinforcement is designed to consider the variable geologic conditions along the alignment. When the tunnel encounters the soft soil layers of the Tauranga and Kaawa Formation prior to undercrossing the Manukau Harbour, steel bar reinforcement was employed in addition to the steel fibres to provide increased structural capacity in the lining. To minimise steel quantities, three types of bar reinforcement were adopted to address the different conditions anticipated within the soil. Steel Fibre Reinforced Concrete (SFRC) segments without bar reinforcement are adopted once the tunnel is fully within the ECBF rock. A summary of the segment reinforcement types is provided in Table 3.

**Table 3. Summary of CI main tunnel segment reinforcement types.**

| Reinforcement Type | Reinforcement |
|---|---|
| C/C-XP | 8 HD16 - 4 HD20 + SFRC |
| B | 12 HD16 + SFRC |
| A | 8 HD10 - 4 HD12 + SFRC |
| SFRC | SFRC |

Initially, forces in the tunnel lining due to the ground were derived based on analytical formula by Muir Wood (1975) with discussion by Curtis (1976). The calculation was derived based on circular opening in continuum, which enables the prediction of hoop thrust, bending moment and radial displacement of the tunnel lining. However, when the tunnel is driven fully in the ECBF rock, ground relaxation was considered for the tunnel lining design to enable the use of SFRC without bar reinforcement. On top of convergence confinement calculations modified for TBM by Almog, Mangione and Cachia (2015), 3D finite element analysis (FEA) was carried out to model the TBM excavation and ground movement prior to lining installation to verify the amount of ground relaxation. The maximum face pressure was applied in the model to capture conservative behaviour. The resulting relaxation was allowed for omission of the bar reinforcement including at the radial joints despite the high overburden and high hydrostatic pressure conditions.

The tunnel lining was also designed for a load case when the tunnel is subjected to net internal hydrostatic pressure, simulating a case when the tunnel is fully flooded in a heavy rainfall event at the location of minimum external hydrostatic pressure. Tension in the lining generated due to the internal pressure was modelled in a beam-spring structural model to take account of the confinement provided by the ground support.

#### *3.2.1. Seismic Design*

New Zealand is subject to high seismicity conditions and the CI main tunnel is designed for seismic design using the methodology outlined in ITA accredited material by Hashash (2001). Forces and deflection of the lining in the longitudinal and transversal direction were checked based on the design seismic load specified in the client's specification, which is derived in accordance with NZS 1170.5:2004 with importance factor of 3. Two design earthquake levels were determined for annual probability of exceedance of 1/25 (SLS) and 1/2500 (ULS). The SLS earthquake is checked against the serviceability limit states, to comply with the crack and deflection limit of the lining and the ULS earthquake is checked against the ultimate capacity of the lining.

Seismic actions result in a racking or ovalisation of the lining for which two methods are discussed in Hashash, et al (2001), that by Penzien (2000) and by Wang (1993). As outlined in the paper, the calculation of tunnel ovalisation using the two methods in the no-slip case results in very different forces. Hashash, Park and Yao (2004) did a further comparison between the two methods against finite element analysis and found that the no-slip case described by Penzien (2000) heavily underestimates the forces and should not be used. Therefore, the analytical design is for the no-slip condition is focused on the method provided by Wang (1993).

For the ovaling deformation analysis of CI main tunnel, FEA was carried out following the methodology outlined in Sedarat, et al (2009) to model the soil-structure interface numerically instead of assuming no-slip or full-slip based on the analytical methods. The results from numerical analysis were compared to the analytical solution and the following observations were made.

When the tunnel is deep and located in ECBF rock, the result of FEA matched very closely to the result of the analytical no-slip condition by Wang (1993). This result is expected. When there is no tension at the interface (due to the depth of the tunnel) and the soil interface behaves elastically during a seismic event (due to high rock strength), the result should be very close to the assumed no-slip condition in the analytical analysis as shown in Figure 5.

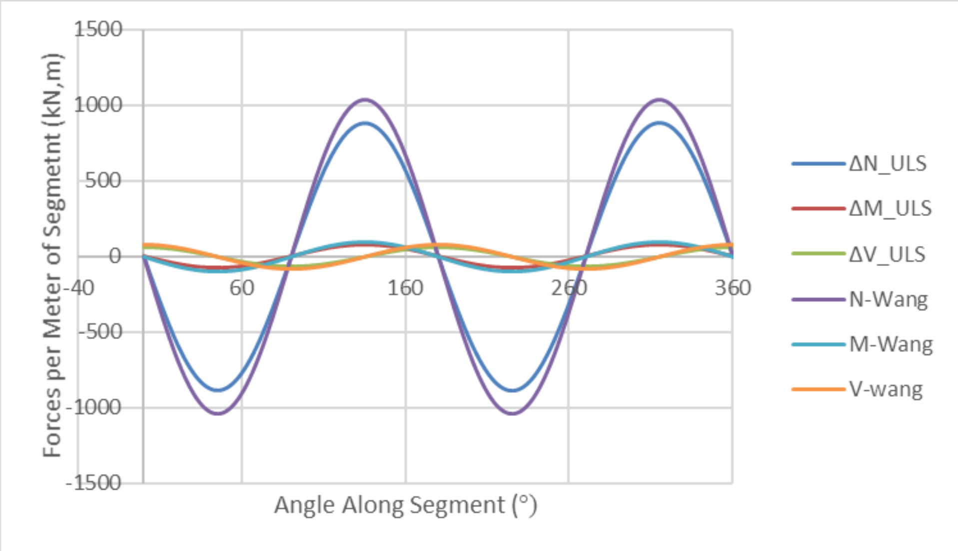


**Figure 5. Comparison of seismic forces between FEA and no-slip analytical solution by Wang (1993) at deepest tunnel location in rock.**

In the other case, when the tunnel is relatively shallow, it is observed that the analytical model is unable to adequately capture the design forces in the lining. This is demonstrated in Figure 6, where the design forces from FEA show an appreciable difference with the closest analysis from the no-slip condition by Penzien (2000).

Moreover, when the soil exhibits highly plastic behaviour during the seismic case, it was observed that the forces obtained from the analytical solution are inaccurate as shown in Figure 7, where no-slip condition heavily over-estimates the forces, and the full-slip condition heavily under-estimates the forces (not shown for clarity, however estimated axial thrust from the full-slip condition is 40.7kN/m). It is also worth noting that the analytical analysis is not able to capture asymmetrical forces in the lining, which is expected as the soil-structure interface behaves asymmetrically in tension and compression.

While the analytical method provided good upper and lower bound values for the design forces based on a no-slip or full-slip assumption, the analytical method produced a very large range of results, especially for the thrust in the lining. As the large thrust values from the analytical method taken as tension in the lining can control the overall lining design, it was decided a more detailed analyses using FEA would be adopted for CI tunnel lining design for a more realistic, optimised and sustainable design solution.

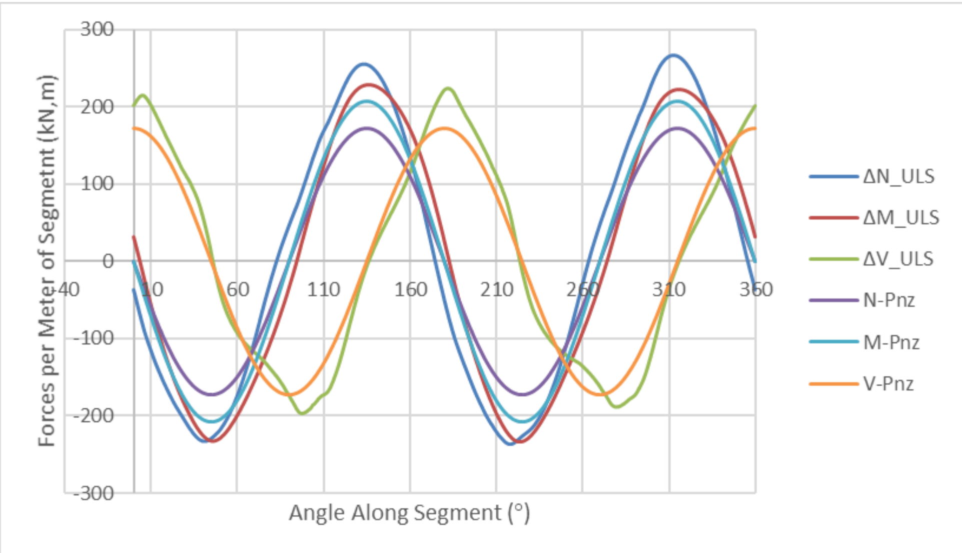


**Figure 6. Comparison of seismic forces between FEA and no-slip analytical solution by Penzien (2000) at shallow tunnel location in soil (Tauranga Formation).**

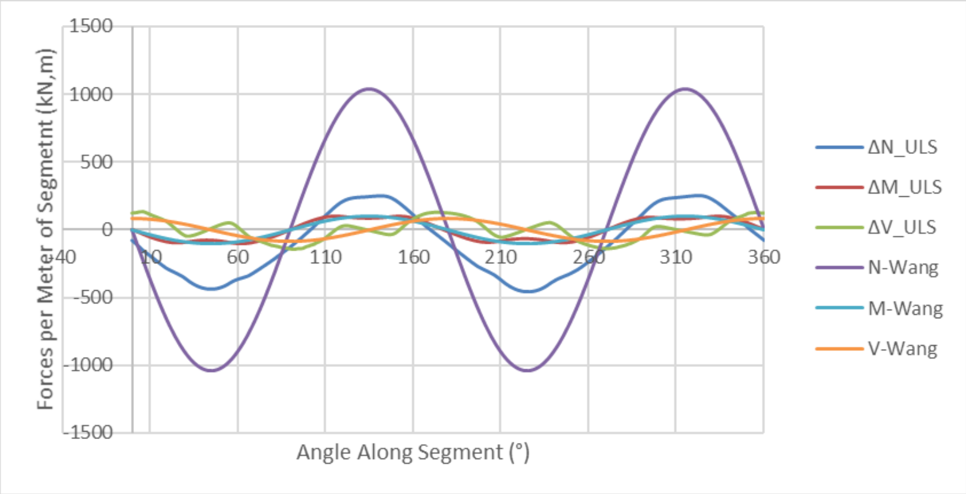


**Figure 7. Comparison of seismic forces between FEA and no-slip condition by Wang (1993) at Shallow Tunnel Location in Soil (Residual ECBF)**

### *3.2.2. Temporary Loading Design*

On top of the permanent loading cases, the tunnel segments were also checked against the temporary loading conditions during production, delivery and installation of the segments. Through close coordination with the contractor, design was carried out for 11 segment handling cases ranging from handling at the precast factory to handling during the installation at the TBM gantry. To accurately assess the segment loading, structural models were created for the different support and loading conditions, as shown in Figure 8. The resulting forces from structural analysis were checked against the allowable stress in the segment which varies depending on the segment age and corresponding concrete strength. This ensures all stages of the segment handling are covered, therefore providing a robust design.

Somewhat unique to this project, the temporary loading conditions including due to TBM ram thrust was generally found not to be a governing design condition due to the relatively thick lining that includes the sacrificial concrete layer.

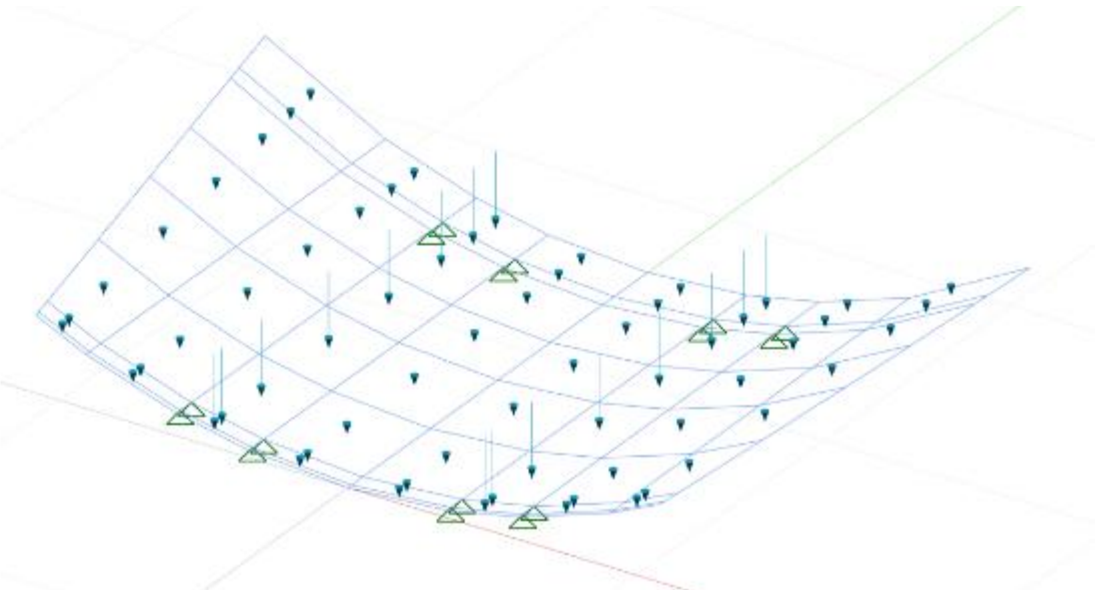

**Figure 8. Sample structural model used to analyse the temporary loading condition of the segments.**

### 3.3. Durability Considerations

Following the client's specifications, the CI main tunnel was designed for a 100-yearsdesign life. As the largest wastewater asset in New Zealand, it is imperative that the structure performs as intended throughout its design life.

As discussed above, a detailed study on the internal durability of the single pass lining was carried out during the reference design stage, and the detailed design of the lining fully complied with the findings from the reference design. Ordinary Portland Cement (OPC) concrete was chosen as the concrete material for the tunnel segments with an HDPE corrosion protection lining and 50-year sacrificial concrete thickness provided at the intrados of the segments.

In order to ensure the segments work as intended throughout its design life, external durability of the segments was assessed by concrete durability specialist OTB Concrete. The segments were assessed against chemical attacks from ground and groundwater, based on more than 100 soil chemistry test results carried out along the tunnel and shafts.

Soil and groundwater chemistry tests were carried out prior to construction to ascertain the impact of the ground and groundwater conditions on the concrete durability. Based on these tests, the conditions were categorized as aggressive to concrete with a pH generally in excess of 7.0, maximum ground chloride and sulphate content of 7700 mg/kg and 950 mg/kg respectively, and a minimum groundwater softness based on Langelier Saturation Index (LSI) -5.4.Based on these assessment, concrete exposure class of XA1 in accordance with NZS 3101.1:2006 was deemed to be appropriate and a concrete design mix recommendation was then developed to ensure sufficient protection for the segments to perform throughout its design life.

### 3.4. Design Review Process

A clear design review process was established at the beginning of the project to ensure that all concerns from the different project parties were addressed in the design. This provides for a collaborative environment between the contractor, designer and owner for a smooth detailed design process.

The detailed design stage started with a kick-off meeting between the contractor and designer to discuss the main constraints, construction methodologies, envisaged construction materials as well as any other issues that could affect the design, such as timeline, client's requirements, supplier availability, etc. Based on these considerations, a general design concept and strategy was agreed before commencement of the first design submission. Any pending information that could affect the design was raised as an RFI (Request for Information) with a timeline requested to make sure the information was procured in a timely manner.

The first design submission was then prepared, which contained developed design from the owner's reference design and incorporated further details based on input from contractor. The overall reinforcement scheme, indicating the initial proposed extents of hybrid segments and SFRC segments was also submitted. Soon after the first submission was reviewed by the owner's engineer, a meeting between the contractor, designer and owner was arranged to discuss and clarify the design.

Unfortunately, as the tunnel segment design was progressing earlier compared to the other design elements based on procurement schedule, an external peer reviewer was not yet appointed at the first submission. The design was progressed based on comments and clarification from the owner's engineer and additional discussions were arranged as required to solve any critical issues. The external peer reviewer was invited to the discussion after being appointed.

During the review process, one of the main concerns from the owner's engineer on the design was with regard to the transition between the different segment reinforcement types. Considering the interpretative nature of the geological profile, contingency plans were needed in case the actual geology encountered was worse than expected in the transition area.

Upon discussion and agreement with the owner's engineer, it was agreed that additional hybrid segments were to be produced to mitigate the risk of encountering more adverse geology over a longer extent and the actual geological condition at the segment transition were to be confirmed through face inspection. A photo of the face inspection at the transition location is shown in Figure 9.

Overall the detailed design and design review process of the tunnel segment proceeded relatively smoothly, through open discussion with all parties having started from the early stage of the design. Safe, robust and efficient design was deemed to be critical for a project of this scale, and this was successfully achieved through the collaboration of all the parties involved. The design included workable and effective measures to monitor and verify the design assumptions during construction where the assumptions are critical for the lining performance.


**Figure 9. Confirmation of ECBF rock at the tunnel face during face inspection at Ch. 11+300**

## 4. LESSONS LEARNED DURING CONSTRUCTION

At the time of writing, approximately 7.5 km of the tunnel has been completed by the main CI tunnel TBM *Hiwa-i-te-Rangi* with over 4500 segment rings having been successfully built. Some of the main lessons learned during the segment production and installation are discussed below.

### 4.1. Single Pass Lining with HDPE Corrosion Protection Lining

While generally most of the produced segments are found to be satisfactory, some issues during the initial production were encountered associated with the adoption of a single pass lining with HDPE corrosion protection lining (CPL).

During the early stages of the segment production, several instances of HDPE lining undulations were observed due to incorrect fixing of the HDPE lining on the mould as shown in Figure 9. It was learned that some spaces are required when fixing the HDPE lining on the mould to allow thermal expansion of the HDPE lining when the segments are steam cured. By adjusting the mould and taking necessary precautions when placing the HDPE lining during casting of the segments, occurrence of this defect was able to be significantly reduced.

To confirm the use of the affected segments, some of the segments affected were sawcut to check if any voids were formed behind the HDPE lining. No voids were formed, however, as the concrete fully fills the space up to the HDPE lining, the affected segments effectively have less sacrificial concrete thickness available in the lining. Therefore, the affected segments are proposed to be installed in the northern tunnel section, where less sacrificial concrete thickness is required based on the durability investigations.

Different HDPE lining colours were used for different segment reinforcement types for easy recognition of segment type to ensure the correct segment types were installed in the correct locations within the tunnel. The lining colours were used in addition to a barcode identification system that tracks segments from production through installation.

Based on site observations, it was found that lifting/grout socket detail where the HDPE lining is directly welded to the grout socket (see Figure 8) is performing well. As the grout sockets were already welded during segment casting in a more controlled environment, the weld quality is better and the required welding of the HDPE on site is reduced.

A quality weld in this location reduces the potential flow path and any water that might flow behind the grout socket does not cause local bulging of the HDPE lining.

*Figure 10. Photo of HDPE lining bulging (photo courtesy of Wilson Tunneling)*

## 4.2. Segment Inserts

During the design process, cast-in gaskets were chosen to reduce the amount of segment handling required at the precast facility or on site. In general, reduced handling reduces the possibility of damaging the segments on site and there was very low reported damage on the CI segments caused by handling.

Nevertheless, some issues were identified during production, such as voids detected behind the gasket due to air entrapment or gasket dislocation during casting. Eventually, the risk to segments was found to be sufficiently mitigated through a robust review of the production process, by making sure the gaskets were properly fixed on the segment moulds prior to the casting process and the segments were adequately vibrated. A comprehensive review and repair process was also in place to ensure any affected segments were properly documented and repaired in accordance with the gasket supplier's recommendations. This system also includes tracking of the segments from the precast facility through installation so that the segment history can be included in the review of any issues.

Dowels and guiding rods were also successfully employed in the project, with ring build tolerances generally found to be within the limits specified.

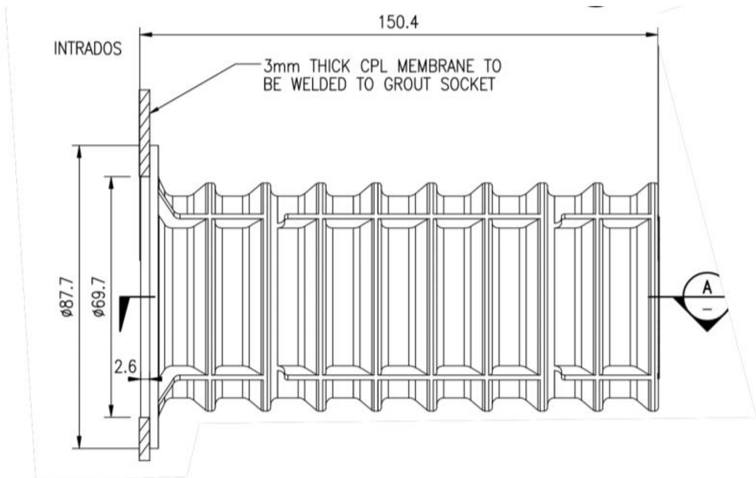


*Figure 11. Detail of grout socket in main tunnel with HDPE lining welded directly to the grout socket.*

## 5. CONCLUSIONS

Through comprehensive review and study process with a collaborative design environment, a safe, efficient and innovative design solution was developed for the Central Interceptor, the largest wastewater tunnel in New Zealand.

Design of the innovative single pass lining was successfully completed, through robust design development process starting from the reference design, detailed design through construction. Through establishing a clear design review process, design of the tunnel lining was completed smoothly with concerns of all project parties adequately addressed.

By ensuring smooth communication and good client-contractor relationship through design and construction, robust control and mitigation measures were also specified and smoothly applied on site to verify the design assumptions. Continuous review of the lessons learned through the construction ensures the improvement of the tunnelling works, with significant reduction in defects being recorded as the tunnel advances.

## REFERENCES

Almog, E., Mangione, M., Cachia, G., 2015. Ground Relaxation in Segmental Lining Design Using the Convergence-Confinement Method. Underground Design and Construction Conference 2015. Hong Kong.

Curtis, D.J., (1976). Discussion on The Circular Tunnel in Elastic Ground. Geotechnique 26 No. 1

Grace, S., Kay, N., Goff, S., 2019. Developing a Corrosion Strategy to Protect New Zealand's Largest Wastewater Asset. Water New Zealand Conference & Expo 2019. New Zealand.

Kenny, J.A., Linsay, J.M., Howe, T.M., 2012. Post-Miocene Faults in Auckland: Insights from Borehole and Topographic Analysis. New Zealand Journal of Geology and Geophysics 55(4):323-343.

Muir Wood, A.M., (1975). The Circular Tunnel in Elastic Ground. Geotechnique 25 No. 1

Hashash, Y.M.A, Hook, J.J., Schmidt, B., Yao, J.I.C., (2001). Seismic design and analysis of underground structures. Tunnelling and Under-ground Space Technology 16 (2001): 247-293.

Hashash, Y.M.A, Park, D., Yao, J.I.C, (2005). Ovaling deformations of circular tunnels under seismic loading, an update on seismic design and analysis of underground structures. Tunnelling and Underground Space Tech-nology 20 (2005): 435-441.

Penzien, J., (2000). Seismically -induced racking of tunnel linings. International Journal Earthquake Engineering and Structure Dynamics 29: 683-691.

Sedarat, H., Kozak, A., Hashash, Y.M.A., Shamsabadi, A., Krimotat, A., (2009). Contact interface in seismic analysis of circular tunnels. Tunnelling and Underground Space Technology 24 (2009): 482–490

Standards New Zealand. (2006). Concrete Structures Standard Part 1: The Design of Concrete Structures (NZS 3101.1:2006). Standards New Zealand. https://www.standards.govt.nz

Standards New Zealand. (2006). Concrete Structures Standard Part 2: Commentary on The Design of Concrete Structures (NZS 3101.2:2006). Standards New Zealand. https://www.standards.govt.nz/

Wang, J.N., (1993). Seismic Design of Tunnels: A State-of-the-Art Approach, Monograph, monograph 7. Par-sons, Brinckerhoff, Quade and Douglas Inc, New York.

Watercare Services Ltd., 2016. Significant Discharge Locations. https://www.watercare.co.nz/Faults-outages/Plumbing-and-wastewater/Wastewater-overflows/Wet-weather-overflows