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Design of the Sydney Metro West turnback tunnels

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ABSTRACT: Sydney Metro West is a new underground metro railway connecting Westmead to the Sydney CBD. The operation of the line requires the construction of a crossover cavern and turnback tunnels just beyond Hunter Street Station in the CBD. This network of tunnels allows the trains to switch tracks and reverse direction and provides train stabling for operational purposes.

The alignment is located within a relatively narrow corridor, with several tunnel and cavern profiles required to accommodate the various track geometries. The tunnels are constructed in a highly developed area of the CBD, below existing rail tunnels and high-rise buildings. The tunnels are also required to accommodate loads from future buildings.

Due to the close spacing of the tunnels combined, construction of the turnback tunnels required complex excavation sequencing and extensive use of reinforced concrete pillars.

This paper describes the challenges for the construction of the turnback tunnels and development of the design to satisfy the various constraints and construction requirements.

1 INTRODUCTION

Sydney Metro is Australia's biggest public transport project, building, operating and maintaining a network of four metro lines, 46 stations and 113 km of new metro rail. The Sydney Metro West (SMW) project comprises a new 24 km metro line with nine new stations to be constructed at Westmead, Westmead, Sydney Olympic Park, North Strathfield, Burwood North, Five Dock, The Bays, Pyrmont, and Hunter Street in the Sydney CBD.

The SMW tunnel and excavation works is being delivered in three contracts – the Western Tunnelling Package (WTP), Central Tunnelling Package (CTP) and the Eastern Tunnelling Package (ETP).

The John Holland, CPB Contractors and Ghella Joint Venture (JCG JV) is currently constructing the ETP, whose scope of works includes:

– 3.5 km TBM twin running tunnels from The Bays Station to Hunter Street Station.
– Excavation and civil works for two new station caverns at Pyrmont and Hunter Street in the Sydney CBD and a crossover cavern at Pyrmont Station.
– Turnback tunnels to the east of Hunter Street Station, allowing trains to travel back west towards Westmead.

This paper describes the constraints related to the design of the turnback tunnels, which included both geometric limits and consideration of existing and proposed buildings directly overlying the ETP alignment. These constraints as well as the relatively poor ground conditions led to unique design solutions which comprised a complex arrangement of variable sized caverns, tunnels, and adits. A complex excavation and construction sequence allowed for early installation of permanent reinforced concrete pillars between the closely spaced tunnels.

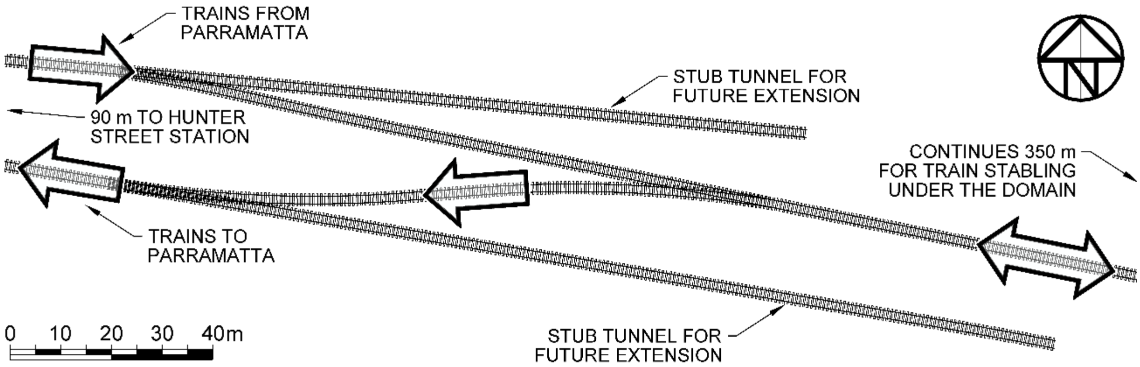


Figure 1. Schematic plan showing the functionality of the turnback tunnels, including stabling of trains and stub tunnels for future extension of the line.

2 KEY DESIGN CONSTRAINTS

2.1 *Alignment*

The required functionality of the turnback tunnels for the SMW ETP line, coupled with the location of Hunter Street Station, existing and proposed surface and underground infrastructure, and the potential future tunnel extensions, imposed significant constraints on the location, alignment and arrangement of the turnback tunnels.

Figure 1 illustrates the functionality of the turnback tunnels, allowing trains from Westmead to depart Hunter Street Station and either continue into the stabling tunnel beneath the Domain, or reverse direction back west to Hunter Street Station and on to Westmead. Two bifurcations are also included in the track alignment for trains to run through the northern and southern stub running tunnels for a future extension of the line.

To minimise the length of the turnback tunnels and the time required for trains to reverse direction during operation, the horizontal alignment of the various tracks was constrained which resulted in closely spaced tunnels of varying span. The vertical alignment of the turnback tunnels was also significantly constrained by existing and proposed infrastructure directly above and below the tunnels, together with limitations on the track grade.

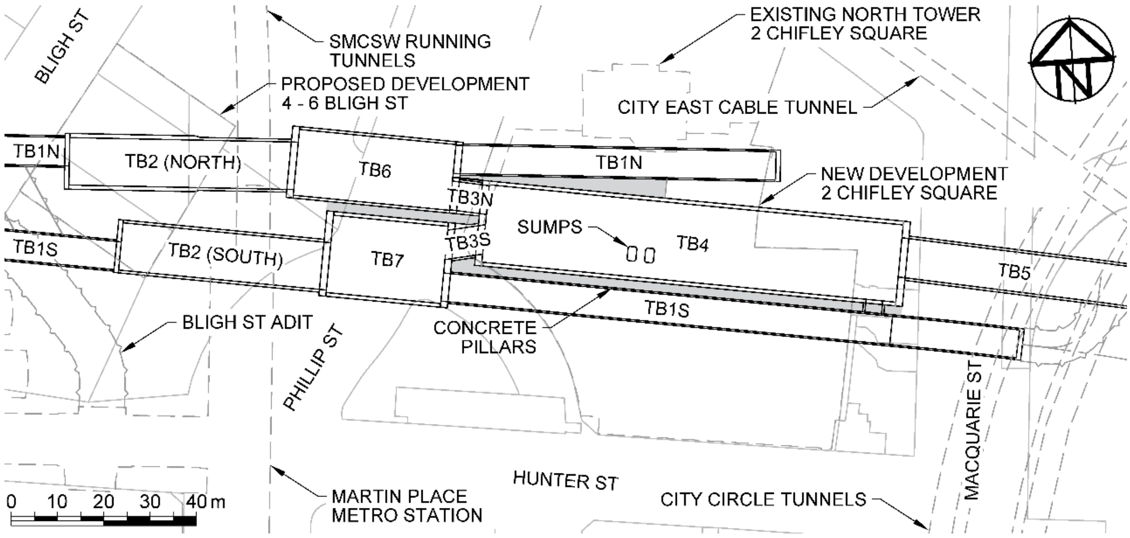
Alternative locations for the turnback tunnels west of Hunter Street Station and within the Domain were assessed during design development but the overall project requirements demanded the complex arrangement of large span tunnels be constructed in a heavily developed and highly constrained location of the CBD.

2.2 *Existing infrastructure*

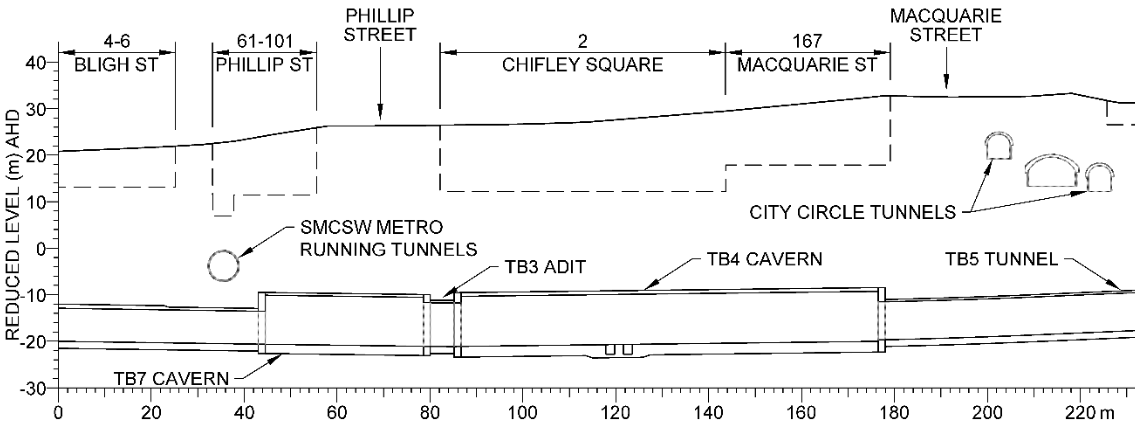
As a result of alignment and location constraints along with line-wide functionality requirements, the turnback tunnels were required to be constructed near significant and sensitive existing infrastructure including (Figure 2):

- Sydney Metro City & Southwest (SMCSW) running tunnels and Martin Place Station caverns, operational at the time of the turnback tunnel construction.
- Sydney Water heritage stormwater drainage tunnels.
- High rise buildings with deep basements and foundations directly above the tunnels, including the 53-storey North Tower at 2 Chifley Square over the TB4 cavern and TB1 North stub tunnel, noting that this was the tallest building in Sydney until 2020.
- Significant heritage buildings, including the State Library of NSW.
- City East Cable tunnel, a major power utility asset for the CBD, crossing four metres below the TB5 turnback tunnel, under the State Library building.
- Operational City Circle Line rail tunnels, and adjacent disused rail tunnels, crossing 15 m above the TB5 tunnel.

The assessment of impacts from the turnback tunnel construction on the adjacent existing infrastructure is beyond the scope of this paper, however this was a key design consideration to limit deformations and minimise effects on a wide range of sensitive infrastructure.



(a) Plan view



(b) Sectional elevation

Figure 2. Plan and sectional elevation of the western portion of the turnback tunnels.

2.3 *Future buildings*

In addition to the existing buildings and infrastructure surrounding the turnback tunnels, the design and construction was also required to accommodate development of proposed high-rise buildings above the alignment.

The new development at 2 Chifley Square involves construction of a 37-storey tower adjacent to the existing North Tower above the TB4 cavern and TB1 South stub tunnel (Figure 2). The permanent support of the turnback tunnels was designed to support loads from both Chifley Square towers. The tunnel primary support and excavation sequence was also heavily influenced by the existing and future towers, as these buildings imposed restrictions on the alignment and available substratum for ground support. The concurrent construction of the turnback tunnels and the new building necessitated detailed assessment of tunnel support design and tunnel construction effects on the buildings.

Additionally, the proposed development at 4-6 Bligh Street is located over the TB1 and TB2 turnback tunnels (Figure 2). The proposed development comprises a 59-storey building with a deep basement. Various foundation options for the new building were assessed during the design development to minimise impacts on the tunnel design. The turnback tunnel permanent linings were designed to support significant loads from the building footings, requiring modified tunnel profiles, lining thicknesses and reinforcement.

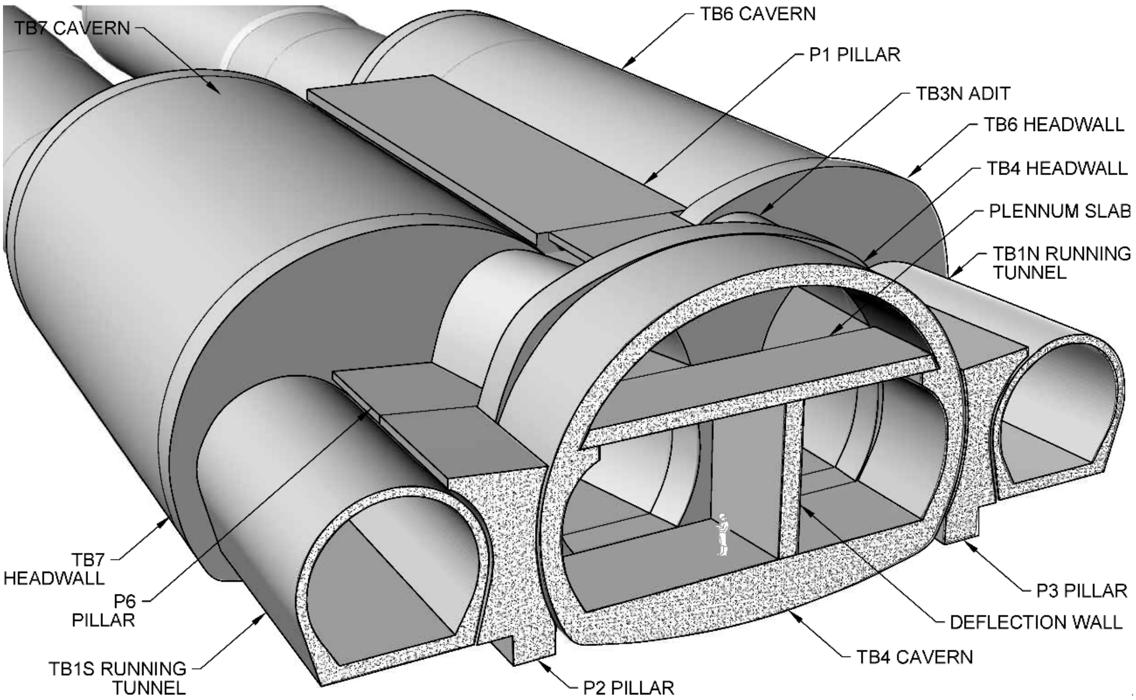


Figure 3. Perspective view of the caverns, headwalls, and adits required where the configuration changes from two adjacent caverns to a single cavern flanked by running tunnels.

2.4 *Construction methods*

The construction methods and tunnel permanent support specified for the project also constrained the design and configuration of the turnback tunnels. The use of permanent shotcrete tunnel linings was not permitted, preventing the use of tapered tunnel profiles to match the gradually diverging track alignments.

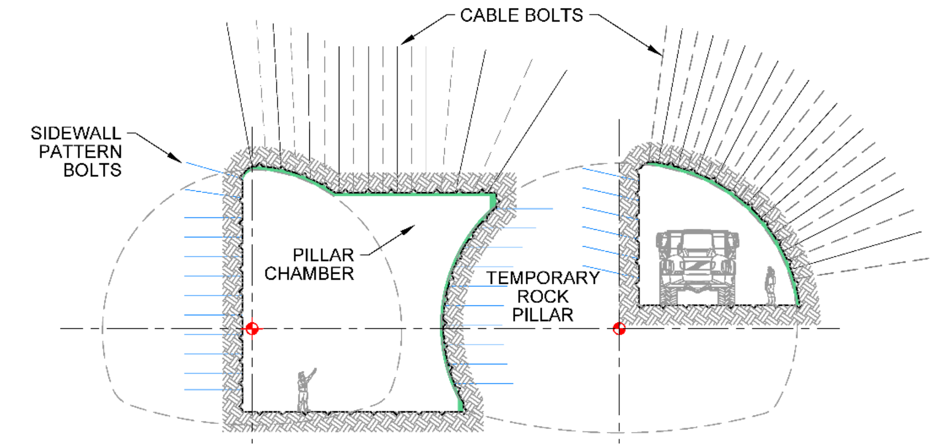
Instead, cast in situ reinforced concrete permanent linings were mandated, resulting in the complex configuration of progressively larger caverns to suit the diverging rail tracks. Six different tunnel profiles were required with multiple headwalls at profile junctions and pillar replacement between the closely spaced tunnels and caverns. The geometry and configuration resulted in complicated construction sequencing of the tunnel excavations, primary support, permanent pillars, waterproofing and permanent lining of the turnback tunnels (Figure 3).

3 GROUND CONDITIONS

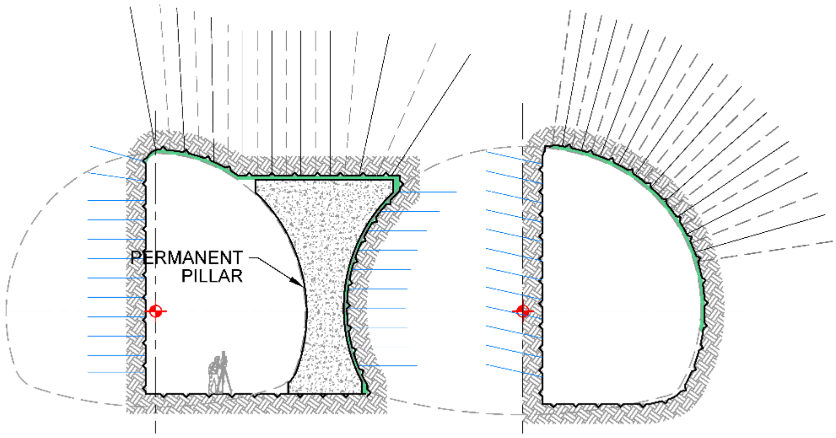
The turnback tunnels were expected to be excavated within Class I/II Hawkesbury Sandstone, classified in accordance with Pells *et al* (1998), with localised reduced rock mass class (Class III Sandstone with occasional Class IV/V) in proximity to significant geological structures. The rock cover above the crown of the tunnels ranged from 18 m to 44 m, with the minimum cover occurring beneath the existing 2 Chifley Square basement.

A significant laterally continuous, ramping, low angle fault was anticipated to be encountered by the turnback tunnels. The damage zone around the fault was inferred to extend 2 m to 4 m either side of a brecciated fault core, with the extent defined by sub-horizontal clay-infilled or extremely weathered features. Based on the expected ground conditions, relatively shallow rock cover, and site-specific stress test results, the caverns were assessed to be within the lower end of the typical in-situ stress range.

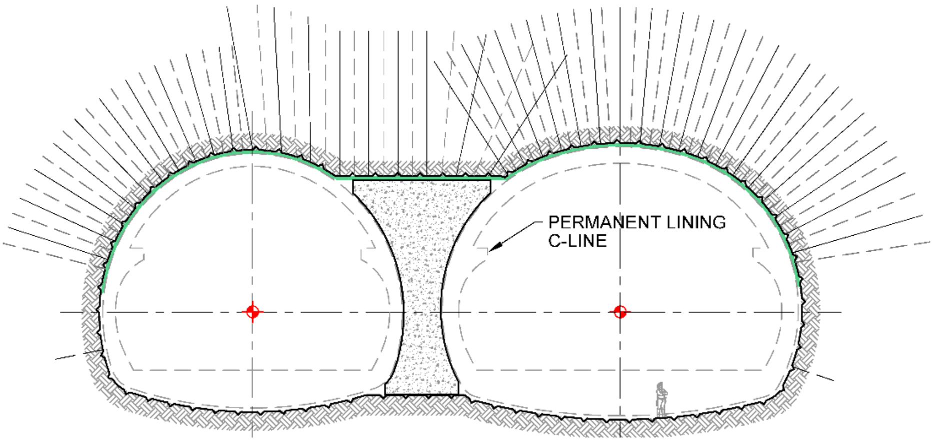
Groundwater levels prior to tunnelling were typically 12 m to 20 m above the tunnel crown level.



(a) Excavation and support of TB6 side heading, bench and invert, and pillar chamber.



(b) Construction of permanent reinforced concrete pillar and excavation of TB7 bench and invert.



(c) Excavation and support of remaining caverns.

Figure 4. Simplified sequence of the excavation and installation of primary support for the TB6 and TB7 caverns and permanent pillar P1.

4 PRIMARY SUPPORT

4.1 *Bolted support*

The majority of the turnback tunnels employed rock bolts combined with thin shotcrete for primary support. For the running tunnels and smaller tunnel profiles, the primary support comprised steel double corrosion protection (DCP) bolts with an ultimate capacity of 310 kN and lengths of 3 m to 4.5 m, typically spaced in either a 1.25 m or 1.5 m grid pattern depending on ground conditions. DCP bolts were not required for durability but were adopted because they were easier to grout than other bolt types. Bolt lengths for the turnback cavern were also constrained by the proximity of the permanent ground anchors and substratum boundary beneath the existing 2 Chifley Square tower.

Cable anchors with an ultimate capacity of 570 kN were adopted for the larger caverns. These had lengths of 6.5 m to 7.5 m and were spaced at 1.0 m to 1.5 m centres, noting that the spacing of some profiles varied between the transverse and longitudinal directions.

Handlebar plates were adopted for both rock bolts and cable bolts to ensure an adequate structural connection between the shotcrete and the bolt heads.

Shotcrete was reinforced with synthetic fibres (SFRS) and typically sprayed to a design thickness of 100 mm to provide temporary ground support between rock bolts.

Pattern sidewall bolts were adopted in the support designs due to the known variable rock quality and likelihood of intercepted poor-quality bedrock, including faults and joint clusters. Sidewall bolts were installed for significant portions of the turnback tunnels.

Short sections of four of the turnback tunnels crossing below the operational SMCSW tunnels and Martin Place Station with low cover of 4.6 m to 5.3 m. These locations required additional support to minimise ground movements, comprising rock bolts and a 250 mm to 350 mm thick passive shotcrete arch.

4.2 *Pillar chambers*

A unique challenge of the primary support design was accommodating the unusual geometry required for the reinforced concrete pillar chambers. These included convex rather than concave sidewalls and flat crowns, all in the context of relatively poor rock conditions. The sidewall geometry restricted the installation of cable bolts across the crown of the chamber and necessitated the adoption of flatter anchor angles and use of reaming tools for the anchors located near the sidewalls.

The necessity of excavating the caverns to full height to allow construction of the pillars meant that the more conventional approach of heading, bench, then invert excavation could not be employed across the full span of the TB6 and TB7 caverns (Figure 4).

Construction of the permanent pillars part-way through excavation of the turnback tunnels also imposed additional programming and scheduling challenges.

4.3 *Monitoring*

In tunnel monitoring during construction included routine use of optical survey prisms, progressively installed with the tunnel heading excavations. Multi-point rod extensometers were installed at 10 m intervals within the larger caverns to monitor crown sag for comparison against design predictions. The highest anchor was located approximately 6.5 m above the crown. Endoscopes were used to assess ground conditions above crown level and monitor shear deformations along horizontal defects in the rock mass. These were also used for assessing ground conditions for pillars at critical locations through probe holes.

Comprehensive real-time optical survey monitoring was required for the operational SMCSW running tunnels and City Circle rail tunnels. Bi-axial tiltmeters were employed to monitor overlying sensitive buildings.

Discussion of the results of the construction monitoring are beyond the scope of this paper, though it is noted that the results fell within the range of expected movement and permitted construction to proceed without the need of having to stop work to review design assumptions or primary support details.

4.4 *Construction sequencing*

The need to complete the pillar replacements prior to opening up the caverns, combined with the unusual cavern layout involving multiple profiles of differing sizes, resulted in a complex construction sequence which was developed in conjunction with JCG to improve the construction program.

Of particular importance was gaining access to the 350 m long stabling (TB5) tunnel located at the eastern end of the project. Without timely access, tunnelling works would have been stalled due to the design constraints preventing commencement of the excavation in the other tunnels prior to the adjacent pillars' completion. This requirement was met by changes to the transition tunnels located between the two caverns to the west (i.e. TB6 and TB7) and the single cavern (TB4) to the east, as well as by excavation of a temporary access adit from the southern stub tunnel to the TB5 tunnel, allowing works within TB5 to proceed independently, thereby avoiding delays to the overall program.

5 PERMANENT CONCRETE PILLARS

5.1 *Purpose*

Owing to the closely spaced track alignments within the turnback tunnels, the minimum clearance envelope around each track, the permanent lining thickness, and construction tolerances, the remaining pillar width between most of the tunnels ranged from as little as 0.8 m to 2.7 m. These rock pillars were replaced by reinforced concrete.

Due to the overburden thickness, existing and proposed buildings, and the substantial cavern spans, the pillars were required to support substantial loads for both temporary and permanent conditions. Ground load assessments using a range of 2D and 3D numerical approaches indicated vertical working loads for the various pillars ranging from 9 MN/m run to 13 MN/m run, with mean compressive stresses at the neck (i.e. narrowest width) of the pillars ranging from approximately 3 MPa to 11 MPa.

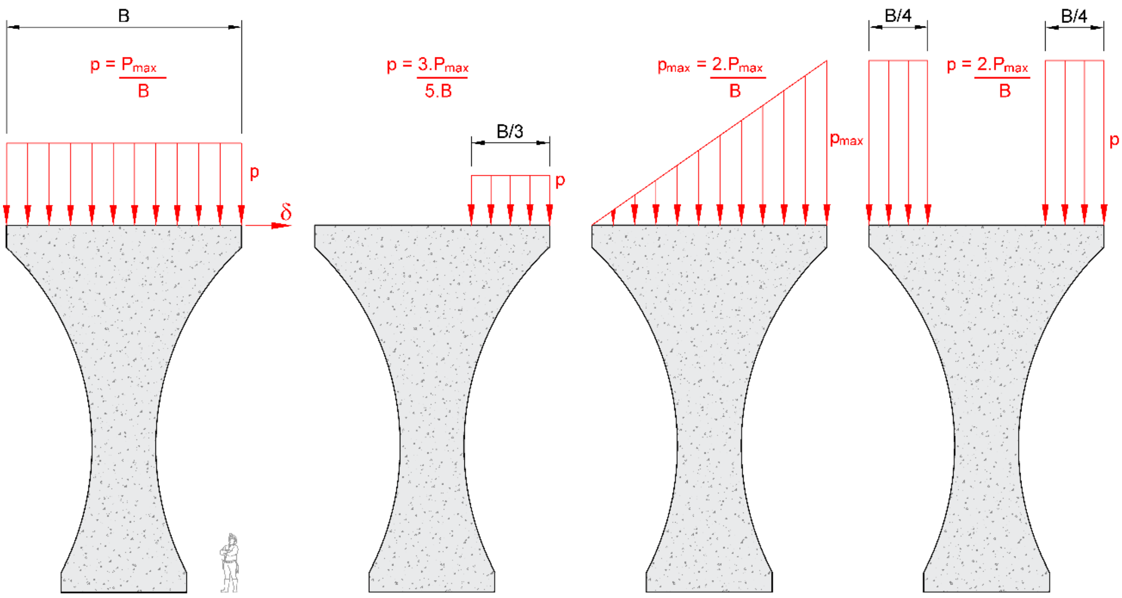
Based on the ground conditions at the pillar locations, the maximum acceptable vertical stress for retaining rock pillars (including potential treatment / reinforcement) was set at 3 MPa. This meant that virtually all the rock pillars needed to be replaced by reinforced concrete.

The use of plain concrete was not a viable option due to the pillar geometry and stress-relief deformation experienced after excavation of adjacent tunnels.

At one location the pillar width was up to 5.4 m, and upon inspection of the rock conditions during construction it was considered acceptable for this to remain as a rock pillar.

5.2 Geometry

Due to the curved sidewalls of the tunnels located either side of the pillars, each pillar comprised an hourglass shape, with a narrow neck at around mid-height, and a wider base and top (Figure 5).



(a) Uniform (b) Low axial force (c) Eccentric (d) Concentrated

Figure 5. Design loading scenarios developed for the permanent reinforced concrete pillars.

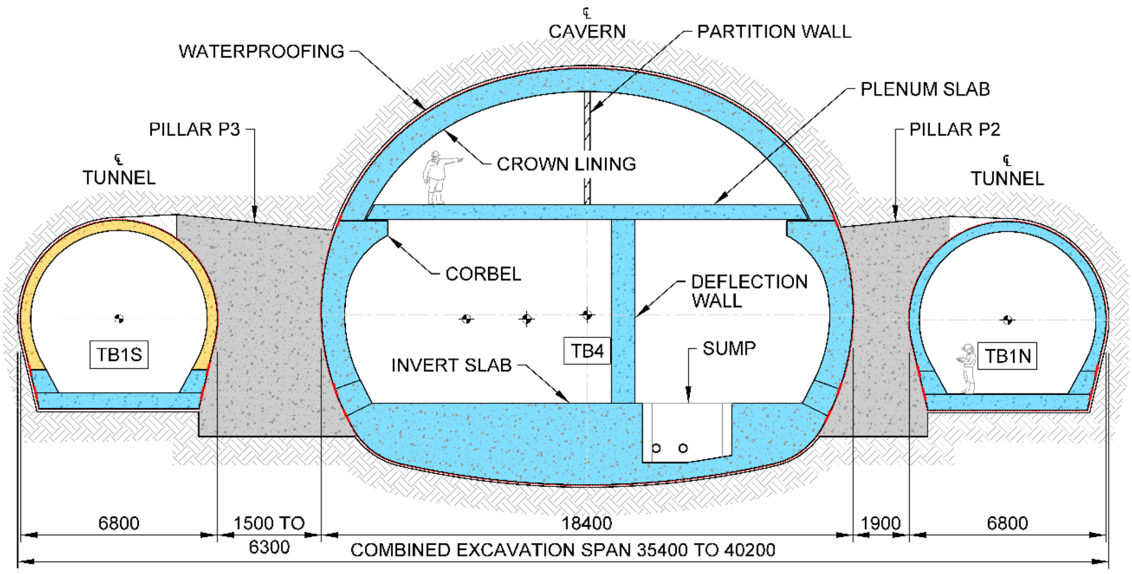


Figure 6. Section through permanent tunnel lining of TB1N running tunnel, TB4 cavern, TB1S running tunnel, and P3 and P2 pillars.

Six pillars required replacement within the turnback tunnels, with a total combined length of 175 m, and heights ranging from 6.5 m to 11.1 m (Figure 2).

The top width of the pillars ranged from 2.9 m to 7.1 m and was governed by the allowable bearing pressures applied to the poorer quality rock present in the crown. The bottom width was narrower than the top because rock quality was typically better at lower levels, although in some locations deepening of the pillar footings was required due to localised poor rock quality or to account for the lower position of the adjacent tunnel profiles.

5.3 *Design loads*

Numerical modelling of the pillars was undertaken to explore the variable loading conditions caused by the construction sequence, which initially comprised excavation of a pillar chamber, followed by construction of the pillar, then progressive enlargement of the tunnel and cavern excavations on either side (Figure 4).

The interaction between the ground and pillar structure resulted in a range of behaviours associated with the variable ground conditions, pillar and tunnel geometries, and stress-relief movements. Based on these results a series of simplified design load cases were prepared (Figure 5) to capture the different ground behaviours noting that these also included an imposed lateral deformation of up to 7 mm. The design load magnitudes, P_{max}, is described in Section 5.1.

5.4 *Design approach*

Several approaches were applied to the design of the pillars to avoid over-reliance on a single methodology and provide a basis for checking different methods. Due to the irregular and unusual shape of the pillars there was no directly applicable structural design approach, albeit there were similarities with shallow footing design approaches and reinforced concrete column design. These analogous approaches were applied to provide an initial indication of reinforcement requirements, as well as a check on the more detailed numerical modelling approach ultimately adopted.

The finite difference numerical analysis software package FLAC3D was adopted to simulate the structural response of the pillars up to and beyond the working loads. The FLAC3D models included each steel reinforcement bar with constitutive models calibrated to provide realistic material behaviours for both concrete and steel.

Validation of this analysis methodology is not given here but is documented in Tran *et al* (2025). Confirmation of the ultimate load capacity followed the requirements in Clause 2.3.6 of

AS 5100.5-2017. This clause requires the pillars to accommodate 300% of the design working load without collapse, noting that this criterion also considers the earthquake load case.

Deflection and crack width checks were also based on the FLAC3D analysis with crack widths based on correlations with the calculated tensile stresses in reinforcing bars.

The above approach resulted in reinforcement densities of 130 kg/m^3 to 185 kg/m^3, with irregular bar geometries required in some instances due to the geometric constraints imposed by the adjacent tunnel profiles.

6 PERMANENT TUNNEL LINING

6.1 *Transition structures*

A significant challenge for the turnback tunnels was the permanent lining details where the configuration changes from two adjacent caverns (i.e. TB6 and TB7) to a single cavern (i.e. TB4) flanked by running tunnels (Figure 6).

The final design involved headwalls at the ends of each cavern, with two 5 m long adits (i.e. TB3S, TB3N) conveying the track between the three caverns. The adit cross-sectional geometry was dictated by the ventilation area requirements above the continuous plenum slabs between the caverns.

The two stub running tunnels also intersect the headwalls of the two caverns. Due to the proximity of the four tunnels at the transition, permanent concrete pillars were required to be constructed prior to opening up the caverns on either side.

The geometric complexity of this configuration resulted in additional challenges for the waterproofing details and connections between the different profiles.

6.2 *Ground and groundwater loads*

Ground loads were assessed based on consideration of several methodologies, including empirical methods and those based on detailed finite element (FE) analysis. The FE analysis approach included key construction stages to simulate the load transfer mechanism from the primary support to the permanent lining:

1. Simulation of ground conditions and existing building loads prior to commencing excavation.
2. Sequential excavation capturing heading advance and excavation stages.
3. Installation of the primary support design and proposed construction sequence.
4. Installation of the waterproofing layers and permanent concrete lining.
5. Removal of temporary primary support (i.e. allowing the rock mass to relax to transfer load to the permanent lining). The concrete modulus of concrete pillars is reduced at the same stage to shed load to the permanent lining.
6. Application of future building loads.

The adopted ground loads ranged from 65 kPa to 530 kPa, with the highest loads associated with the overlying high-rise buildings.

Different groundwater levels were adopted for the serviceability (SLS) and ultimate limit state (ULS) design checks. The groundwater design levels were constrained by the requirements of the project specification. The adopted typical groundwater pressure (i.e. SLS) closely corresponded to the basement levels of the overlying buildings, while the credible worst groundwater pressure (i.e. ULS) corresponded to the ground surface level. For the larger caverns these adopted levels resulted in uplift pressures on the underside of the invert slabs of 360 kPa (SLS) to 560 kPa (ULS). Detailed 3D hydrogeological groundwater modelling predicted long-term groundwater levels substantially lower than the adopted design levels.

6.3 *Tunnel lining and waterproofing*

The tunnel linings were designed as either bar-reinforced concrete, fibre-reinforced concrete, or plain concrete, depending on the tunnel geometry and applied ground and groundwater loads.

Bar-reinforcement was required for the sidewalls of the cavern and tunnels where a corbel was required to support the internal plenum slabs. Heavy steel reinforcement was used in the crown of the tunnels for sections subject to large ground loads associated with the overlying buildings.

The headwalls, endwalls, and invert slabs comprised bar-reinforced concrete due to their unfavourable geometry and presence of large groundwater loads. Within cavern TB4 the project specification required provision of a sump for the water treatment plant. The sump geometry was optimised from the original concept to two parallel sumps to improve structural efficiency and avoid excessive deepening of the invert slab (Figure 6).

Steel fibre-reinforced concrete was adopted for the crowns of the caverns and running tunnels where ground loads were not affected by large building loads. Plain concrete was employed for the crown of the TB5 tunnel above the corbel level, given the favourable profile and ground loads.

Waterproofing for the permanent tunnel linings comprised a 2 mm thick sheet membrane, constructed from very low density polyethylene (VLDPE).

A sacrificial sump detail was designed to allow dewatering of the ground surrounding the caverns while the waterproofing membrane was being installed and concrete lining constructed. The sump was located outside the waterproofing membrane and included details to permit effective sealing of the pipework once the permanent lining was completed and able to accommodate elevated groundwater levels.

6.4 Deflection walls and plenum slabs

The turnback tunnel design scope included deflection walls and plenum slabs which extended through the majority of the caverns and tunnels.

The deflection walls are required for train impact loads and are secured to the invert slabs.

Plenum slabs in the larger caverns and TB5 tunnel were required to create a ventilation space above the tracks, with a maximum span of 14.5 m, including post tensioned sections around openings. These were designed by structural engineering firm, TTW, working alongside PSM to integrate with the tunnel lining design.

7 CONCLUSIONS

The design of the turnback tunnels currently being constructed as part of the SMW ETP project encountered numerous technical challenges associated with onerous geometric constraints, proximity to existing tunnels and high-rise buildings, together with challenging ground conditions and the need to safeguard proposed future building developments above the tunnels.

Robust and practical design solutions were developed in conjunction with JCG JV to address these challenges. This included the extensive use of permanent reinforced concrete pillars associated with a detailed construction sequence to optimise delivery of the works whilst ensuring ground stability and minimising ground movements and impacts from the works.

At the time of writing this paper the permanent pillars and tunnel excavation works had been successfully completed with the permanent tunnel linings under construction.

ACKNOWLEDGEMENTS

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