Design and construction of a very large span and shallow ramp tunnel using pre-activated steel sets

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ABSTRACT

The increased demand to future-proof tunnel projects with respect to long-term traffic predictions has led to the proposal of some exceptionally large span tunnels in recent road projects in Australia. For example, some of the current projects are excavating main line tunnels that allow up to four or five lanes of traffic with excavated spans up to 22 m, besides their Y-junction caverns approaching 34 m in span. This is mostly possible due to the generally favourable tunnelling conditions in Sydney within the Hawkesbury Sandstone formation. However, the ramps and portals of such large span tunnels are subjected to more significant challenges due to their shallow cover and often less favourable ground conditions. These ramps and portals have increased tunnelling risks particularly in urban settings where settlement induced damage could be a major concern. This paper will present a study case of a challenging 23 m shallow tunnel span with approximately 8 m of total ground cover, ie with ground cover to tunnel span ratio as low as C/D = 0.35, and how some of the risks associated with potential damage to properties at the surface were mitigated via tunnel support design. The development of the design and how it changed from an initial tender design concept using more specialised pre-reinforcement techniques, to a simpler and more favourable option using pre-activated steel sets will also be discussed. This modification achieved an effective balance between cost and construction program to meet tight timescales, thus providing the contractor with significant savings. In addition, the paper will also present how the design was subsequently validated through monitoring both in-tunnel and surface during construction works.

INTRODUCTION

With rapid development of cities, it is crucial that the use of the underground space is made efficiently with projects that can cater for the needs of the population for several decades or more to come. This has led to an increased demand for future-proofing of tunnel projects with respect to traffic. This has resulted in the proposal of some exceptionally large span tunnels in recent road projects currently under construction in Australia, all with the requirement for a minimum 100-year design life.

For example, several kilometres of four and five lane tunnels are currently under construction in Sydney with mined spans of up to 22 m. Until recently, such spans had only been experienced in localised excavations in widened sections such as breakdown bays and Y-junction caverns but not for long lengths of main line tunnels.

Such large span excavations have likely been considered in Sydney mostly because of the favourable tunnelling conditions generally experienced within the Hawkesbury Sandstone formation. However, the ramps and portals of such large span tunnels are subjected to more significant challenges due to their shallow ground cover and often less favourable ground conditions. Some of the current projects have excavated ramps and portals with ground cover to tunnel span ratio as low as C/D = 0.35 and in rocks of poorer quality. These factors result in increased tunnelling risks particularly in urban settings where risks associated with settlement induced damage is always a major concern.

As these excavation spans are somewhat unprecedented in Australian civil tunnelling, a direct comparison with local experience and precedent design is not always straightforward. One study case that could be compared to the one discussed in this paper is the Boggo Road Tunnel in Brisbane (Nye and Kitson, 2009) which had 15 m span and crossed under properties with a cover between 5 m and 8 m, thus with C/D = 0.3 to 0.5. It should be noted, however, that the effects of either an increase in excavation span or a reduction in ground cover does not follow a linear relationship. Deflections and surface settlements could be expected to increase by a power of two with an increase in span for the same C/D whereas the effect of a reduction in ground cover could be expected to narrow the settlement trough, thus, increase the magnitude of settlement non-linearly, for the same C/D. As a result, although simple 'extrapolation' of precedent designs could potentially provide some initial indicative solution, the induced effects of increase in tunnel spans combined with low cover are highly non-linear. As a result, substantial analysis to justify the design is required particularly to satisfy that the proposed solution is either unsafe or overly conservative and uneconomical.

This paper presents a study case of a challenging 23 m span tunnel with approximately 8 m of total ground cover, ie C/D = 0.35, and how some of the risks associated with potential damage to properties at the surface were mitigated via tunnel support design.

The significant three-dimensional complexity associated with the interaction between excavation sequence, tunnel support and ground conditions were addressed via robust 2D and 3D numerical analyses. These analyses also allowed for a better understanding of the ground response which eventually led to the modification of the design from an initial tender design concept using more specialised pre-reinforcement techniques, to a simpler and more favourable option using pre-activated steel sets. This modification achieved an effective balance between cost and construction program to meet tight timescales, thus providing the contractor with significant savings.

The paper also presents how the design was subsequently validated through monitoring both in-tunnel and surface during construction works and how the pre-activated steel sets successfully met the design targets.

DESIGN CHALLENGES

The study-case tunnel under discussion is an exit ramp for one of the major road tunnel projects in Sydney with up to four lanes of traffic. These four lanes of traffic combined with other space-proofing requirements, including the allowance for installation of both primary and secondary lining, meant that the excavated span of this tunnel was approximately 23 m in very shallow conditions with approximately 8 m of total ground cover, ie C/D ≈ 0.35.

As one could expect in such shallow conditions, the ground cover was primarily comprised of fill and residual soils with a minor rock cover of less than 2 m of highly weathered siltstone of the Ashfield Shale formation (Class IV/V). As the tunnel descends, the quality of rock slightly improves but still comprising predominantly rock masses classified as Shales Class III and Shale Class IV though with increasing rock cover. These rock masses had, for design purposes, compressive strength varying from 2 to 6 MPa and Geological Strength Index (GSI) varying from 35 to 50.

Figure 1 presents a high-level overview of the geology in the tunnel with the focus area of this paper highlighted.

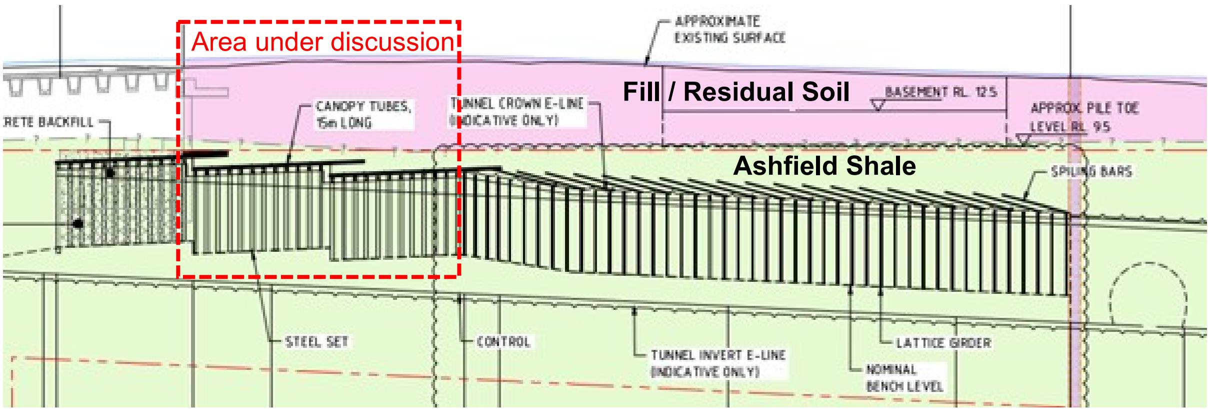

FIG 1 – Longitudinal geological section of ramp under discussion.

One of the main challenges associated with this very large span and shallow tunnel was that, only approximately 15 m away from the portal, there were several small buildings located at the surface, mostly terrace houses. Closer to the mined tunnel portal, there was a residential three storey building with a single level basement carpark with its corner sitting right above the tunnel crown. The footings of this building were located only some 2 to 3 m away from the excavation profile of the tunnel as depicted in Figure 2. As a result, considering the shallow conditions and the size of the excavations, surface settlements and its associated potential damage risks were a major concern for this length of tunnel.

FIG 2 – Indicative properties on top of the ramp tunnel.

TUNNEL SUPPORT DESIGN

Concept design (tender)

Given the situation described above, heavy duty canopy tubes using micro tunnelling methods were originally proposed for the ramp tunnel during tender design stage. The proposed tubes had proposed diameter of 406.4 mm to be installed from the portal for a length of approximately 40 m. These tubes would then be fully grouted both inside the pipe and around the annulus.

Primary support would be installed immediately after each excavation round using 310 UC 96.8 steel sets embedded within a 500 mm thick lining with steel fibre reinforced shotcrete (SFRS) every 1000 mm advance. The steel sets were to be blocked with steel shims and/or grout pads. Conventional grout bags were planned to be installed between the canopy tubes and the steel set extrados flange to ensure full contact between the excavated tunnel profile and the steel sets. Fibreglass face dowels were also planned to be used to stabilise the tunnel excavated as a full-face advance. This original support is illustrated in Figure 2.

The main advantages of these large diameter canopy tubes were that they would likely provide robust pre-reinforcement ahead of the excavation face and help better distribute the loads from the face to previously installed and hardened tunnel support. For the first few rounds of excavation, these large diameter tubes would mostly act as cantilever beams reducing the loads on newly installed support while after a few rounds of excavation, they would then act as temporary but stiff structural beams that could span ground loads at between their embedded depth into the face and a zone of higher support stiffness behind the face.

The main disadvantage of this solution was that the size and length of the tubes required specialised techniques with guided boring systems and therefore methods of work additional to those commonly used in the project. Furthermore, additional tolerances needed for installation of these tubes combined with a curved road alignment required additional free space between the tunnel crown and the cut and cover structure at the portal and within the tunnel which would drive the tunnel profile to a flatter shape, thus, affecting its structural performance and stiffness. In addition, despite some settlement reduction achieved as a result of a more controlled stress relaxation near the excavation face, the outcome and benefits of the large canopy tube solution would always be limited by the overall stiffness of the primary support working as a passive system. This meant that surface settlements were still predicted in the order of 30 mm due to very shallow and mostly soil cover and the large volume of the excavation.

Optimisation during detailed design

During the detailed design stage, changes were proposed to address some of the disadvantages discussed above. The first change was to adopt canopy tube of a more conventional size used in tunnelling activities such that the equipment and experienced crew available for the project could be used. Three rounds of 15 m long of canopy tubes of smaller diameter (139 mm O.D) were proposed to replace the big diameter canopy tubes from tender concept. The canopy tubes could be installed with reduced spacing thus allowing for a better arched profile. This in turn allowed for a better structural performance for the lining which could then be reduced to 450 mm.

However, the changes above did not change the main fact that the outcome with respect to reducing surface settlement would still be driven by the overall stiffness of the primary support working as a passive system. In fact, the changes made the lining slightly thinner, therefore less stiff, and reduced the pre-reinforcement effect of the large canopy tubes, thus likely increasing settlement. To compensate for these changes, the design focused on changing the tunnel support system from a passive to an active support system similar to preloading a structure with a jacking system before final concreting. This was achieved in the tunnel support by pre-activating the steel sets at the excavation face. To meet this requirement the conventional grouting bags originally intended as a simple packing system were changed to a system that could be fully pressurised during installation, thus pushing the ground against the steel sets at the face.

The ability to apply a pressure to the steel sets at the face and 'push' the ground up also meant that the final ground loads acting on the lining could be induced to larger values than those obtained in a passive system where loads would be inherently taken by stress arching in the ground causing settlement. It should be noted however, that such pressures need to be controlled not only to avoid significant ground heave but also to avoid the buckling of the steel sets or failure of their footings before the full thickness of shotcrete lining was sprayed. To improve the efficiency of the pre-activation and avoid the risk of a footing failure, the footings were fully shotcreted in place prior to pressurisation of the grouting bags.

The DSI Bullflex system was adopted with bags that could take in excess of 250 kPa of grouting pressure. This provided the construction team with a fast support solution that used similar methods to other support types in the project. The Bullflex system was found to be quick and easy to install during tunnel excavation and of easy handling on-site due to its light-weight components.

The final tunnel support system adopted in the design is illustrated in Figures 3 and 4. The DSI Bullflex bags were locked to the steel sets located immediately below the canopy tubes at approximately 30° to 150° along the profile. Figure 5 show some photos of the installation during construction including the grouting bags after pressurisation.

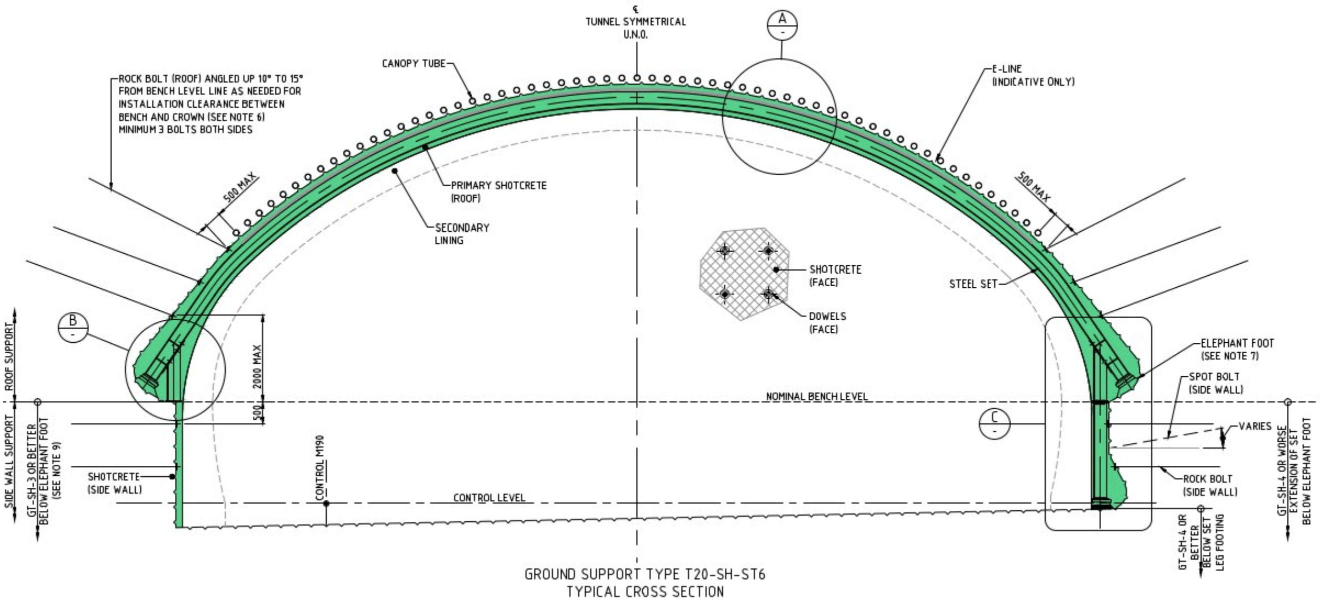

FIG 3 – Ground support developed during detailed design.

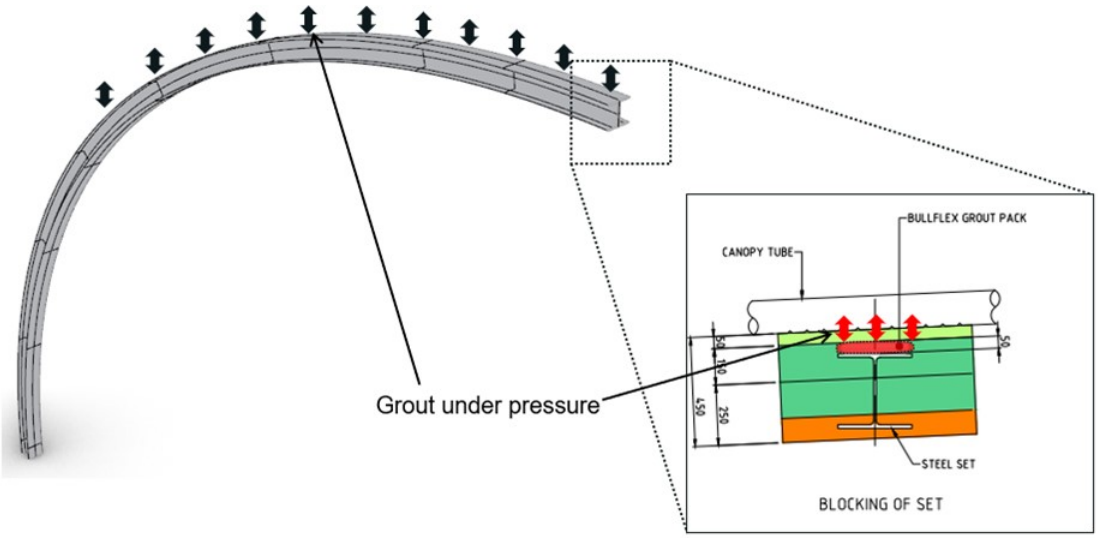

FIG 4 – Detail A of tunnel support in Figure 3.

FIG 5 – (A) Installation of steel sets with grouting bags. (B) Shotcreting of steel set footing prior to pre-activation. (C) Grouting bag/steel set after pressurisation (photo looking back towards portal before shotcreting).

DESIGN ANALYSES

Ground support design

The tunnel support system of the ramps and portal were designed as an arched profile comprising both permanent primary and secondary linings.

Due to the limited rock cover above the tunnel crown and portal effects and stress relief due to weathering, the shallow rock mass has lower confinement than similar rock masses at greater depth. As a result, and due to the potential to impact structures at the surface, the primary lining was primarily designed to control tunnel deformation as much as reasonably practicable to reduce the surface settlement.

The exit ramp tunnel was designed as a full-face excavation using steel sets with the Bullflex grout pack system as discussed above. The steel sets with Bullflex system were designed to allow for pressurisations of up to 250 kPa pressure which would both improve the tunnel global stability and reduce the settlement impacts at the surface. Mandatory face support with a pattern of GRP bolts and shotcrete were used to maintain the face stability for Class III or worse due to the large excavation face and its associated risks in a fractured ground condition. The tunnel primary support system was designed to be installed with a maximum 1 m maximum advance length during heading excavation and with a set period of time for the Bullflex bags to be pre-stressed and shotcrete lining gained enough strength before the next advance. The Bullflex system could not be applied for the first 12 m due to interaction with the existing cut and cover structures. However, double row of canopy tubes was designed to be installed within this section to better control the settlement at this initial phase. This allowed for some comparison between two sections of tunnel support with and without the effects of the Bullflex bags.

The primary shotcrete lining was designed as a 40 MPa steel fibre reinforced shotcrete (SFRS) shell with steel sets as reinforcement. The lining thicknesses varied from 400 mm to 450 mm but mainly the latter for the section with Bullflex bags. The structural response of the lining for the imposed combinations of loadings at different stages (during excavation and after excavation completed) was checked using the moment-thrust interaction diagram based on the different age design strength of the shotcrete in accordance with AS5100.5.

As discussed above the steel sets were implemented as means for allowing the pre-activation of the lining but also to retain the tunnel profile and provide temporary support whilst the shotcrete was brought to full thickness and gained strength. The steel sets were not provisioned to be corrosion protected and were therefore not considered in the permanent design due to potential issues related to encapsulation as highlighted by Nye and Alt (2010). A full secondary lining was designed to compensate for this loss of capacity.

The temporary canopy tubes were designed as pre-support for tunnel roof during construction to not only reinforce ahead the face, thus minimising risks associated with face instability but mostly to act as ‘beams’ temporarily transferring the ground loads between the excavation face and the shotcrete lining further back from face with full thickness and higher strength/stiffness.

Steel sets and the primary shotcrete were designed to transfer load into a ledge abutment (elephant feet). To promote retainment of the rock mass within these footings the bench excavation height was designed to be from the abutment height to the invert and excavated in stages. On excavation of the bench, two rows of fully grouted rock bolts were designed to support the abutment, with provisional extension of the steel sets allocated based on the geological conditions of the invert.

A secondary sprayed shotcrete lining was assigned through this section of tunnel, designed to accommodate a portion of the long-term ground loads, fire effects, and loads from any internal anchored equipment (mechanical and electrical).

2D numerical analysis

Two-dimensional finite element analyses were carried out using the commercial program RS2 from Rocscience to simulate the deformations and the forces generated during tunnel excavation and after support installation. Figure 6 presents details of a typical RS2 model.

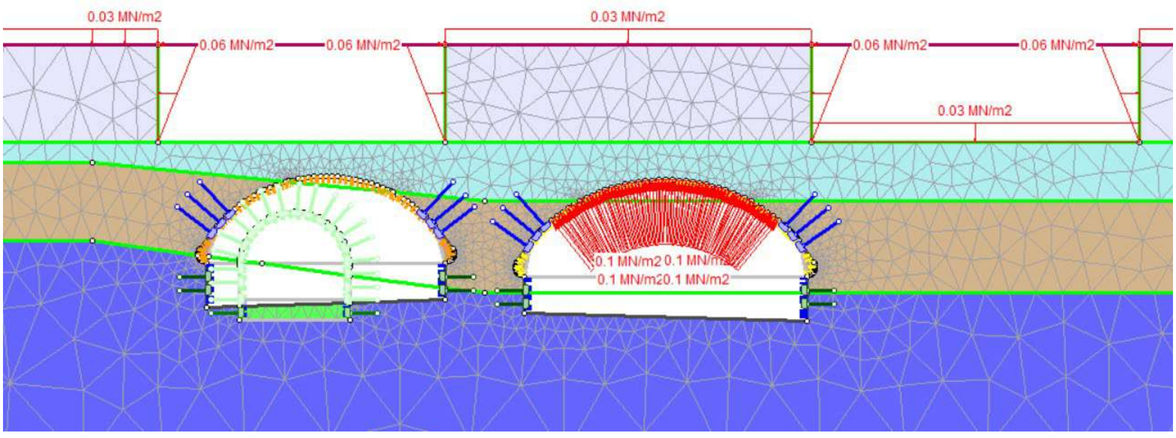

FIG 6 – Typical RS2 model.

The analyses considered the sequence of support installation and ground-structure interaction using a continuum rock modelling approach. The materials were modelled as an elastic-plastic continuum using the generalised Hoek–Brown failure criterion and tunnel lining shotcrete was modelled as a composite lining (primary shotcrete and joint interface of rock and shotcrete). The *in situ* stress condition was assumed approximated to an isotropic stress state (horizontal equals to vertical stress) considering the proximity of the shallow excavation depth and the presence of the fault zone.

The 3D effect of tunnel excavation and support installation was accounted for in the 2D sections by applying the convergence confinement method with an assessed Ground Reaction Curve (GRC) and Longitudinal Displacement Profile (LDP) applicable for different cases. The estimation of ground relaxation was adopted to model the construction sequence including excavation advance length and timing of support installation. Where canopy tubes were proposed as pre-support during excavation, the ground relaxation was assumed to have the equivalent tunnel roof displacement to deformation of the canopy tube support. The effect of the 250 kPa applied to the grouting bags were modelled as a 'smeared' 100 kPa internal pressure.

The outcome of the analysis results was then used to assess primary support system performance including rock bolts, shotcrete lining, tunnel stability, tunnel deformation and settlement.

3D numerical analysis

FLAC3D analyses were carried to assess the 3D effects relevant to the design of these shallow tunnels. The 3D analyses were used to assess and confirm the design assumptions and excavation performance observed in the 2D analysis. The model has been developed including all major excavations in the ramps, ie (shallow cover support zone. Figure 7 presents an overview of the 3D model and Figure 8 presents the tunnels geometry mesh.

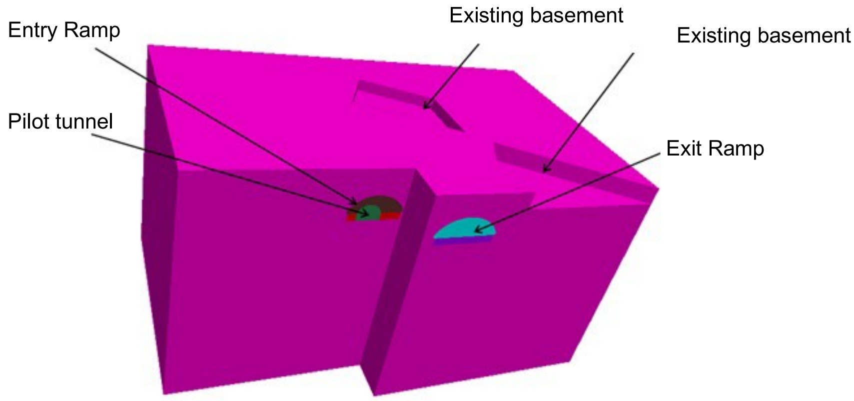

FIG 7 – Global scale model layout.

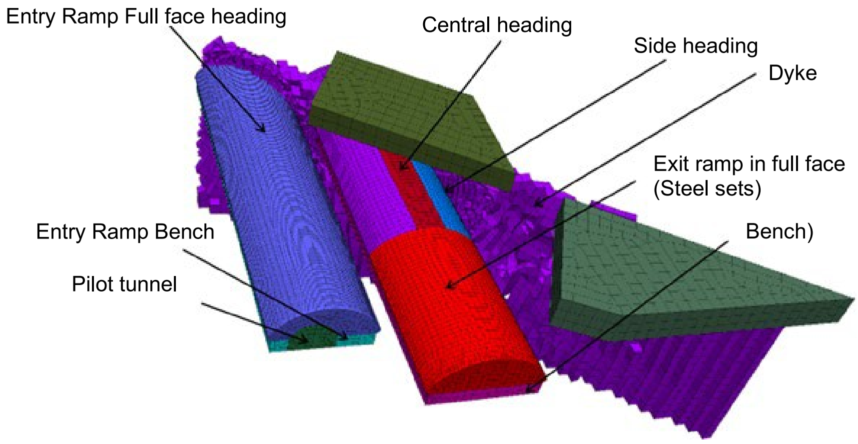

FIG 8 – Ramp tunnels cut-away.

The objective of the model was to assess the excavation and support performance during construction of the ramps near the cut and cover structure. Such information was then used to assess some of the assumptions adopted in the 2D models confirming the satisfactory performance of these analyses. Detailed excavation sequence/advance and support installation was included in the model sequence as specified on the design drawings. These included the loss of confinement due to cut and cover excavation, ie portal effects, pre-support, shotcrete layer build-up (ie thickness), early age strength and stiffness of shotcrete and pre-stressing where applicable. Steel sets and lattice girders were not explicitly modelled but have been considered for shotcrete lining structural capacity check as reinforcement. No face dowels and rock bolts were considered for the continuum approach.

Figures 9 and 10 present the design predicted surface settlement contours and in-tunnel displacement. The maximum estimated settlement values are approximately 20 mm above the entry ramp and 36 mm above exit ramp in the proximity if the cut-cover, where the Bullflex system was not used, and away from the terrace houses. Where the Bullflex system was used, the estimated settlement values were approximately between 13 and 16 mm. All these values were consistent with the ground movements estimated using the 2D numerical models.

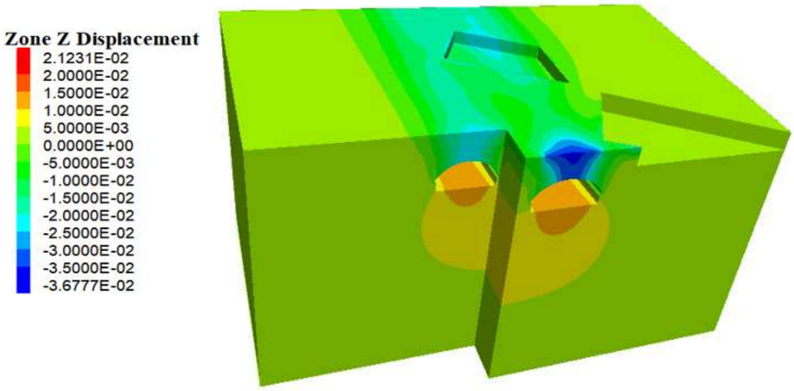

FIG 9 – Predicted surface settlement upon completion of tunnelling works.

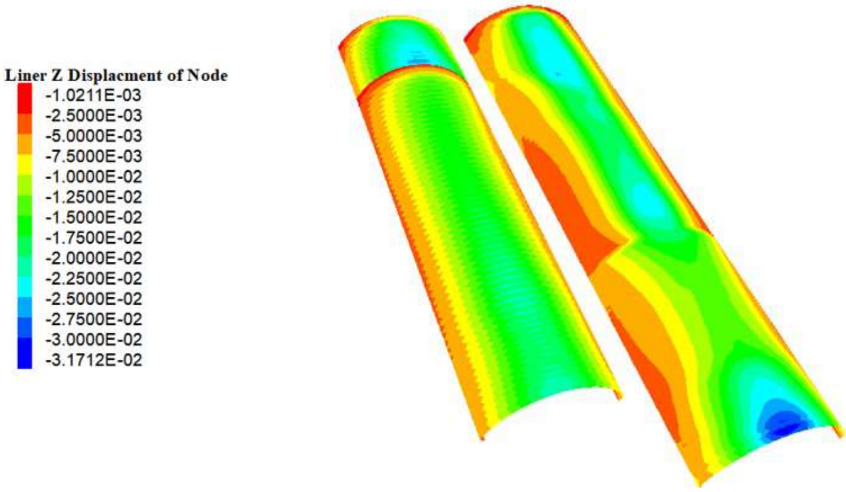

FIG 10 – Vertical displacements predicted along tunnel lining.

Table 1 compares the results between the 2D and 3D models but also between the sections of the tunnel with the same type of support, ie 450 mm lining with steel sets, but with and without the Bullflex system. The analyses result shows the outcome from both 2D and 3D analyses are somewhat consistent. As it can be observed in the table, based on design predictions, the Bullflex system was expected to reduce crown displacements by approximately 15 mm and surface settlement by approximately 20 mm providing a significant improvement on induced settlements.

TABLE 1

Exit ramp tunnel displacements and ground movements.

Analysis type	2D	3D
Crown displacement without Bullflex system (mm)	25	32
Crown displacement with Bullflex system (mm)	11	15
Surface settlement without Bullflex system (mm)	35	37
Surface settlement with Bullflex system (mm)	13	16

PERFORMANCE DURING CONSTRUCTION

Monitoring during construction confirmed the design predictions with a maximum surface settlement above the section without the Bullflex bags observed at approximately 35 mm and above the Bullflex less than 15 mm.

Some interesting behaviour was also observed within the in-tunnel monitoring targets as shown in Figure 11. There is a clear change in system stiffness with the steep downward trend stopped after a few Bullflex bags are installed and pressurised ahead of the monitoring array. The tunnel crown is also clearly pushed upwards though with some delayed response likely associated with the fact that a few rounds of pre-activation, ie pressurisation, are required to achieve this effect. Figure 11 also shows that the tunnel crown displacements were generally limited to approximately 10 mm to 15 mm consistent with the design predictions discussed above. Two other monitoring arrays show similar results to the one presented here.

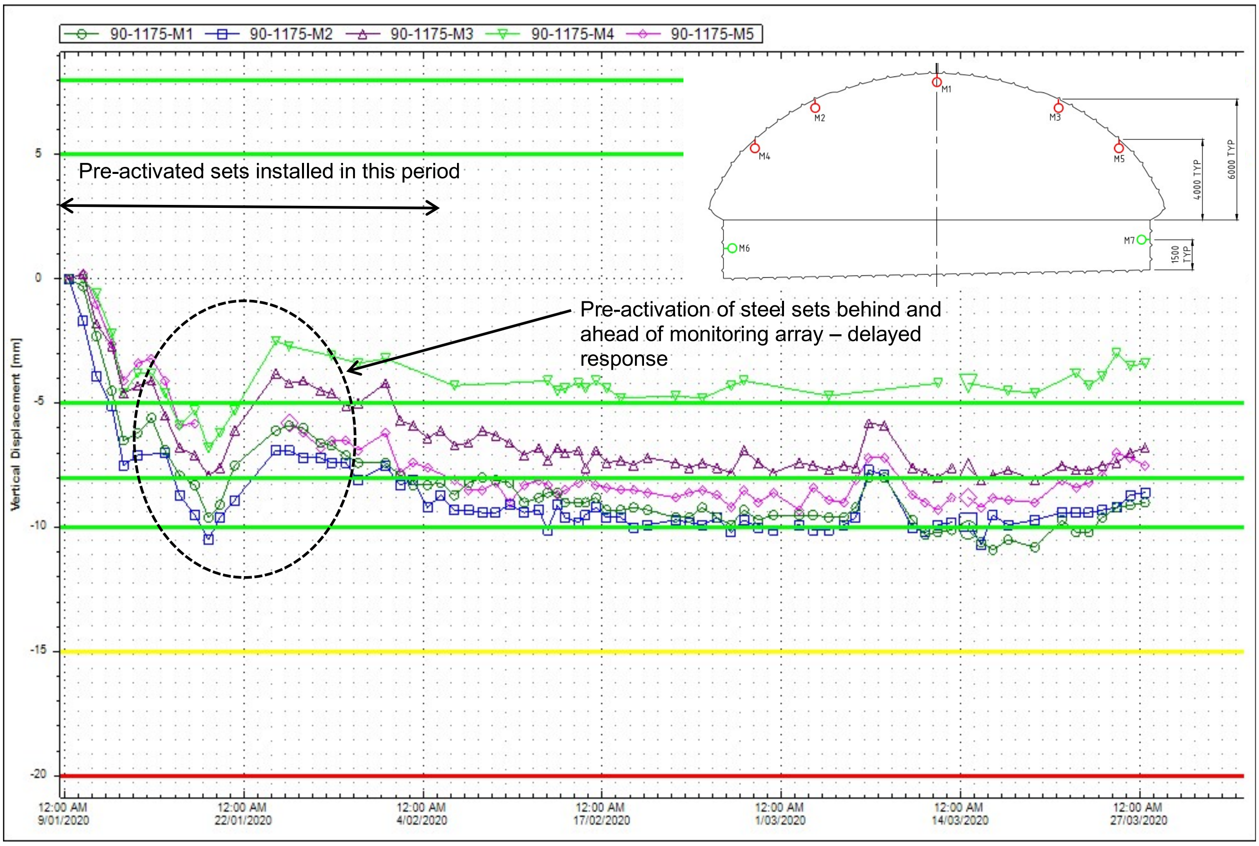

FIG 11 – In-tunnel monitoring at Bullflex support section.

CONCLUSIONS

This paper presented some of the challenges faced when excavating very large span ramp tunnels in shallow conditions. These ramps tunnels have increased tunnelling risks in urban settings where settlement induced damages could be a major concern. A study case of a challenging 23 m shallow tunnel span with approximately 8 m of total ground cover, ie with ground cover to tunnel span ratio as low as C/D = 0.35 was presented.

The paper also presented how the risks associated with potential damage to properties at the surface were mitigated via tunnel support design, including the development of the design and how it changed from an initial tender design concept using more specialised pre-reinforcement techniques, to a simpler and more favourable option using pre-activated steel sets. This modification achieved an effective balance between cost and construction program that met tight timescales, thus providing the contractor with significant savings.

The use of the pre-activated steel sets system proved very efficient with values observed during construction matching design predictions. For a 23 m span excavation, tunnel crown displacements and surface settlements were limited to approximately 15 mm which is a remarkable result for such a wide and shallow tunnel. This settlement was less than half what was predicted and observed for the original concept design without the use of the pre-activated steel sets. Figure 12 shows a photo of the completed top heading for the section of tunnel with canopy tubes, steel sets and the Bullflex system.

FIG 12 – Completed top heading for section with canopy tubes, steel sets and Bullflex system.

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