19th Australasian Tunnelling Conference – Acosta-Martínez, Stewart and Hudson-Smith (Eds.)
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The mined tunnels of the Sydney Metro West – Western Tunnelling Package: Narrow pillars and heritage sewers

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ABSTRACT: The scope for the Gamuda Laing O'Rourke Consortium (GLC) on the Sydney Metro West – Western Tunnelling Package (WTP) consisted of more than twin TBM tunnels. GLC delivered a significant amount of mined tunnel scope with multiple challenges. The mined tunnel scope was split between Clyde and Westmead. The scope at Clyde included twin 750m long spur tunnels from the surface of the Clyde stabling facility, joining twin junction caverns intersecting the TBM tunnels. This paper explores some of the associated technical challenges, the solutions developed and the results of those efforts through delivery. Some of the notable challenges included: excavating below the heritage-listed Lidcombe Auburn Granville (LAG) sewer tunnel including temporary support, instrumentation and monitoring in live sewer conditions; creating a less than 1 m thick shale pillar between tunnels; intersection of adverse geological features, including dykes; and reduced rock cover at the tunnel portal.

1 INTRODUCTION

The Sydney Metro West project involves a 24 km metro rail line between Westmead and the Sydney CBD.

The Clyde caverns were accessed from the Clyde Dive site, located beside James Ruse Drive and Rosehill Gardens Racecourse as part of the WTP project. This site links the Maintenance and Service Facility (MSF) to the mainline running tunnels via a cut and cover dive structure and two spur tunnels. The horseshoe-shaped spur tunnels are 7 m wide and 8 m high, converging upon each other to a minimum distance of 900mm at the tunnel portal. They are vertically offset by 1.5m to maximise cover to tunnel under the LAG sewer.

This paper discusses the engineering challenges of excavating the tunnels in close proximity to each other as well as below the active LAG sewer. Figure 1 shows the arrangement of Spur tunnels and location of Heritage Sewer and narrow pillar zone.

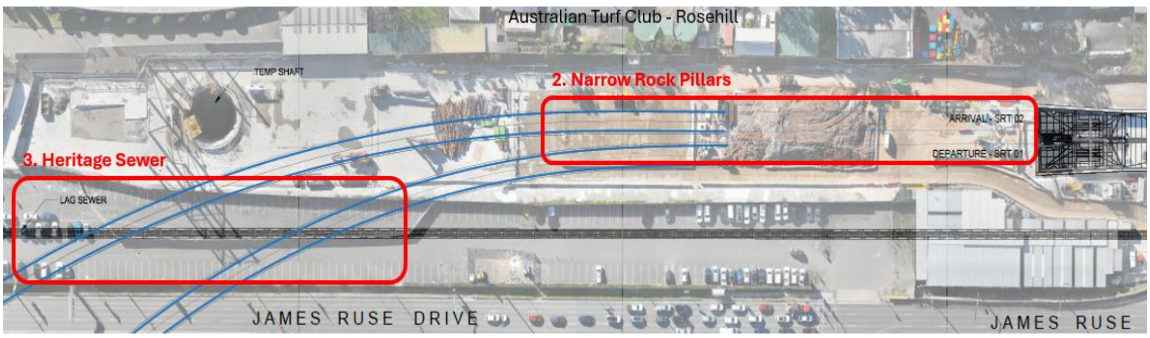

Figure 1 - Project Plan

2 NARROW ROCK PILLARS

As the spur tunnels connected to the dive structure, GLC and Sydney Metro (SM) revised the design by extending the tunnels 40 metres and shortening the dive structure. A cost-benefit analysis showed this approach was more economical, sustainable, and quicker to construct. However, it introduced new challenges. The converging tunnel alignments became more complex, and ground conditions near the portals worsened. While majority of the tunnel alignment was excavated through stable Class 2 Sandstone (SS-II) with over 20 m cover, the new alignment reduced cover to 5 m, transitioning into weaker alluvium and weathered shale posing increased risks for stability and requiring more robust support and excavation techniques. Figure 2 demonstrates the ground conditions at the narrow pillar zone including the dyke encountered during excavation of the Spur tunnels.

The initial risk mitigation strategy for the converging pillar comprising of alluvium and weathered shale between the two spur tunnels involved the excavation of a pilot drift, which was subsequently backfilled with mass concrete, to provide temporary structural integrity and control unravelling of any weathered material as shown in the light green colour in Figure 3. However, this method proved to be neither environmentally sustainable nor efficient in terms of construction time. It required the removal of a significant volume of rock, only to replace it with concrete that would eventually be re-mined once the two spur tunnels were fully excavated.

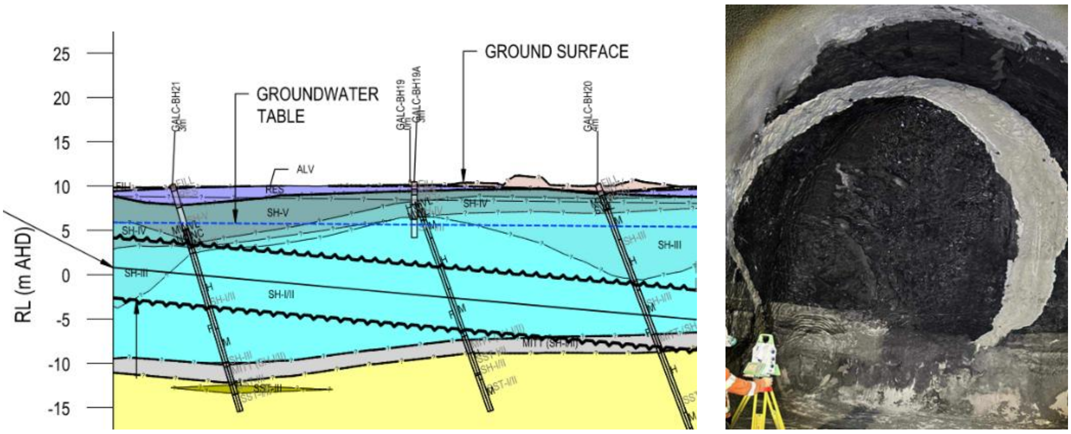

Figure 2 – Left: Geological long section of tunnel at the narrow rock pillar, Right: Geological feature (Dyke)

As excavation of the dive portal progressed and greater confidence was gained in the geotechnical data and rock mass quality, GLC revisited the design with the aim of optimizing the approach. Rather than removing and replacing the reducing pillar, efforts shifted toward retaining and reinforcing the in-situ rock. The Spur tunnels were subsequently divided into five distinct support categories, ST7A, ST7B, ST7C, ST7D, and a specialized portal support system tailored to varying ground conditions. Figure 4 shows the distribution of the varying support systems utilised to reinforce the rock pillar.

To assess the feasibility of preserving the rock pillar, stability analyses were conducted along the 160 m tunnel extent using both empirical methods and numerical modelling. Preliminary findings indicated that in zones where the rock mass was classified as Class 3 Shale (SH-III) or poorer, the in-situ material lacked the necessary strength to ensure stability. In these critical sections particularly where Class 3 Shale (SH-III) was anticipated the initial strategy of removal and replacement with reinforced concrete remained necessary to maintain structural integrity.

The conventional pillar stability checks showed that the Class 2 Shale (SH-II) rockmass had adequate capacity to withstand the ground loads as a rock pillar. These approaches considered the rockmass as a continuous material and the mode of failure is the intact rock yielding assuming that the discontinuities are supported with additional measures such as temporary stitching bolts. Conventional stitching methods were used where the rock pillar was greater than 2.5 m in width. This type of stitching bolt could be installed post excavation of both tunnels after the excavation face had passed.

Where the remaining rock pillar between the tunnels was less than 2.5 m, the assumption that the remaining rock is self-supported and stable may not be feasible. Additionally, the consequences of a local rock wedge failure may end up in losing a major portion of resisting rock material which will significantly increase the risk of pillar collapse. Considering these elements, an innovative inclined temporary stitching bolt (sidewall forepoling) has been designed for the narrow pillar zone within the Spur tunnels. The inclined stitching bolt was installed after excavation and support of the first tunnel, SPT-ST7B, Figure 4. All stitching bolts were temporary until the secondary lining was installed.

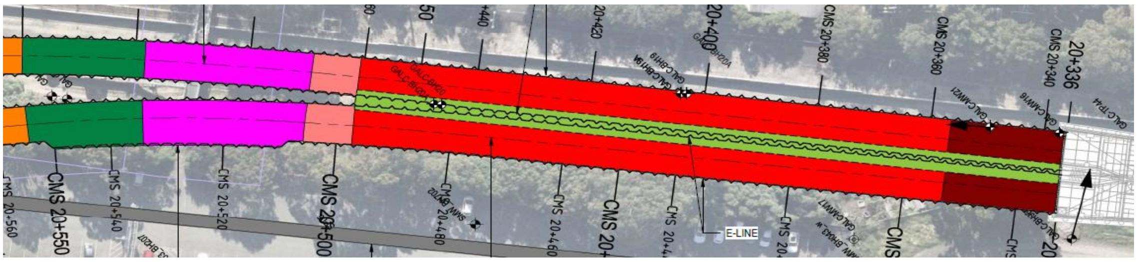

Figure 3 – Initial design

The intention of the inclined stitching bolt was to ensure installation before the excavation of the second tunnel i.e. before the rock pillar is formed. In this approach the newly formed rock pillar in each excavation advance of the second tunnel, is pre-supported by GRP bolts which is bonded into rock mass at the face of the excavation. The following parameters were considered in geometrical design of inclined stitching bolts:

- Excavation advances in second tunnel: An excavation advance of 1.4 m was adopted for the second tunnel. Inclined stitching bolts were designed in order to ensure sufficient bond length was present within every 1.4 m of excavation advance.
- For a bolt length of 6 m, three different inclinations were designed to optimise the utilisation of the inclined stitching bolts.
- Arrangement of discontinuities: Typical Class 2 Shale (SH-II) discontinuities including systematic beddings and subvertical defects were considered in the design. The presence of fault zones was not allowed within the pillar zone.

The inclined stitching bolts were designed for the narrow pillar zone considering that the first tunnel will be completely excavated and supported and the encountered ground condition at the rock pillar zone can be verified before excavation of the second tunnel.

The revised design, which allowed for the retention and reinforcement of a significant portion of the rock pillar, marked a major success in optimizing both construction methodology and structural stability. Due to design constraints requiring a 20 m longitudinal offset between the excavation faces of the adjacent tunnels, only a single roadheader was needed for this section of the works.

Excavation began with the departure spur tunnel in accordance with the design requirement to prioritize the excavation of the lower tunnel first. This sequence was essential for enabling the effective stitching of the pillar between the two tunnels. The use of a temporary active support system, consisting of CT bolts and shotcrete, allowed the initial drive to progress quickly through to breakthrough. Following breakthrough, the installation of the passive support (pillar stitching bolts with additional mesh and shotcrete), were carried out off the critical path ahead of the adjacent tunnel drive.

The success of this staged approach relied on ensuring that the passive support in the Departure tunnel reached its full design strength at least 20 m in advance of the advancing face of the Arrival tunnel. This was crucial to maintaining the stability of the shared pillar during the sequential excavation process. Due to the deterioration of the rock class to Class 3/4 Shale (SHIII/IV) within the final 20 m of the pillar zone, removal and replacement became necessary. A combination of pillar replacement and canopy tubes at tunnel crown were adopted at the tunnel portal zone. Steel set arches embedded in shotcrete were constructed under the canopy tubes at tunnel portal before the tunnel breakthrough, SPT-ST7C/D.

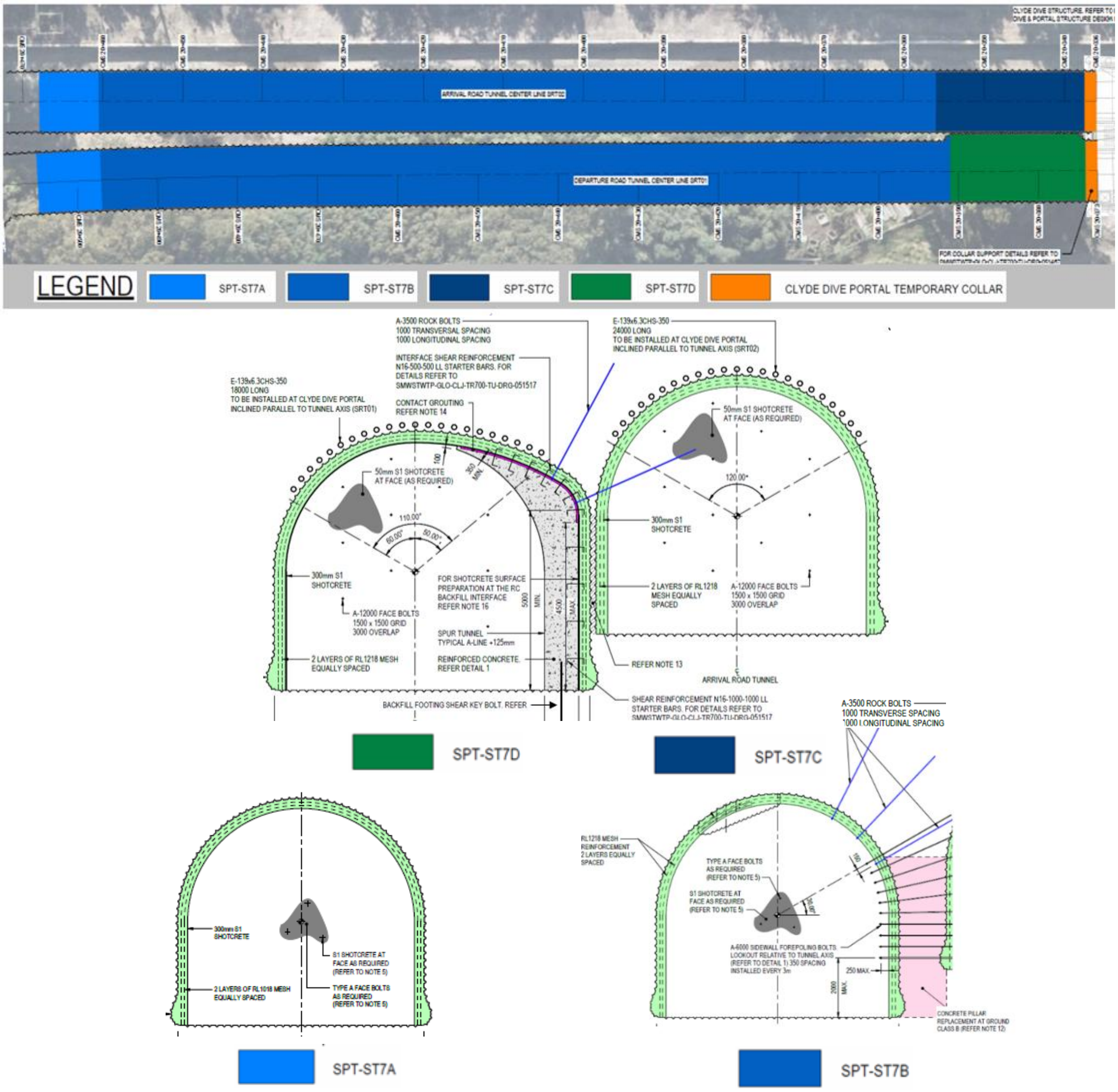

Figure 4 – (Top) Revised support categories to reinforce pillar, (Middle) ST7C/D Primary Support, (Bottom) ST7A and ST7B Primary Support.

The tunnel excavation and ground support operations effectively validated the initial design assumptions and analytical predictions. Predominantly, the geological conditions encountered during the first tunnel drive comprised Class 2 Shale (SH-II), with the rock pillar zone exhibiting overall structural integrity. A single, discrete geological anomaly (identified as a dyke with a thickness of less than 1 m) was observed within the rock pillar zone, Figure 2. Excavation proceeded through this zone, and a supplementary design verification was undertaken to assess the structural adequacy of the inclined stitching bolts in accommodating this feature. Targeted core sampling of the dyke material was conducted, resulting a minimum of 8MPa uniaxial compressive strength, confirming compatibility with the support design parameters. The material within the dyke zone was identified as medium to high-strength rock. After completion of the first tunnel and prior to the advance of the second, it was concluded that the inclined stitching bolts were adequate to maintain the rock pillar through the dyke zone.

The design outputs show that the excavation of the second tunnel would have some impacts on the primary support of the first tunnel, the additional loads due to further ground relaxation was considered in the design capacity of the lining of the first tunnel. The numerical modelling results showed a potential movement of up to 5 mm in the first tunnel due to excavation of the second tunnel depending on the in-situ stress in rock mass.

The primary support of the first tunnel was monitored at every 10m length of the tunnel during the excavation of the second tunnel, Figure 5. This additional monitoring aimed to address the residual risks due to unknown ground conditions, so that if any trend of movement is observed in the first tunnel ahead or at the location of the newly formed rock pillar, the excavation advance of the first tunnel will be adjusted.

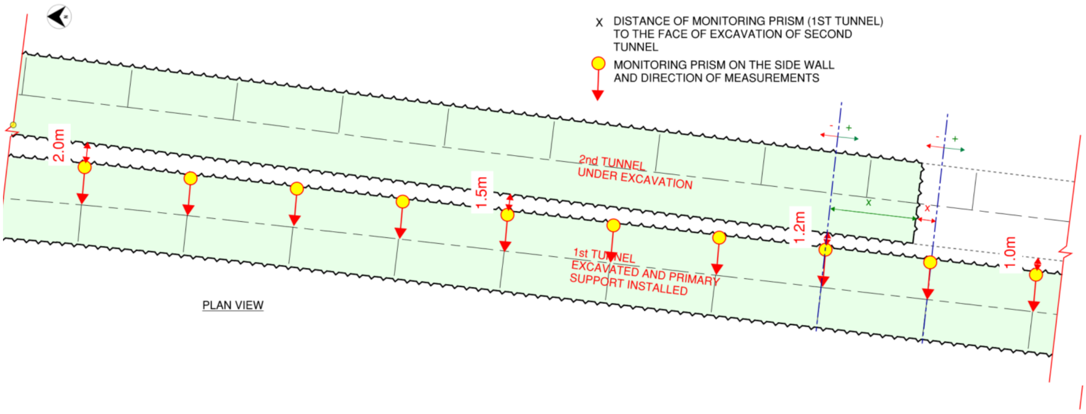

Figure 5 – Location of additional monitoring targets on the 1st tunnel sidewall

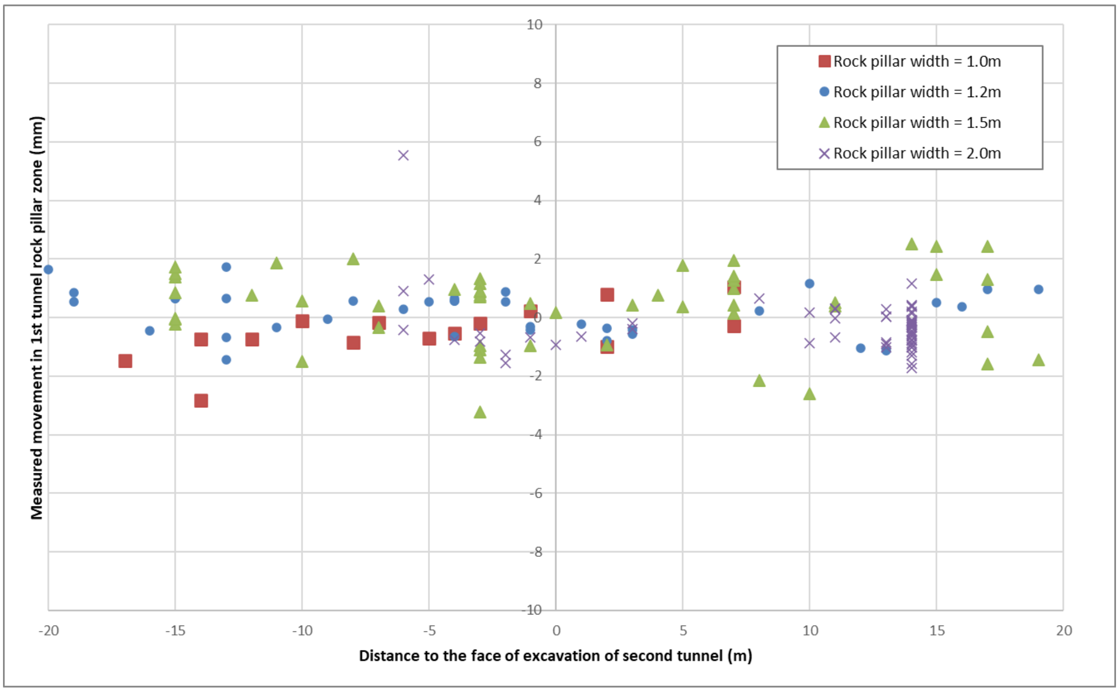

Figure 6 – Measured ground movement at rock pillar zone (different colours show different pillar width)

The measured ground movement in the first tunnel, resulting from the excavation of 2nd tunnel, was less than 3 mm in average without showing any specific trend of movement, Figure 6. As no significant overbreak was observed on the rock pillar side during the second tunnel's excavation, it was concluded that the installed stitching bolts effectively supported the rock blocks, preserving the structural integrity of the rock pillar. The monitoring data aligned more closely with the lower bound of rock mass in-situ stresses investigated in numerical modelling. This outcome was considered reasonable, given that the rock pillar was in a shallow section of the tunnel, with an overburden depth ranging from 15 m to 5 m.

3 HERITAGE SEWER

The original spur tunnel design at Clyde Dive was limited by surface service banks, restricting the ability to commence diving the rail alignment early enough for sufficient cover beneath the LAG sewer including a 4.5% maximum rail gradient. An early concept involved rerouting the LAG sewer using four shafts and micro-TBMs, but this posed tunnelling and long-term serviceability challenges due to the sewer already being at minimum grade.

SM and GLC re-evaluated the alignment, reducing proximity to the service banks and maximising cover to the sewer. Factors such as land resumption, mainline gradients, junction location, and spur tunnel length was also considered. A final alignment was achieved with 1.3 m to 2.4 m of rock cover between the sewer and primary support, Figure 7. Both tunnels crossed below the sewer line acutely therefore the length of tunnel impacting the sewer was 51m in both drives.

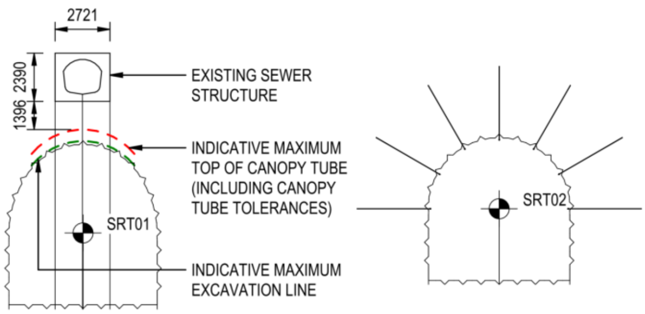

Figure 7 – Sewer minimum clearance to Spur tunnels.

The LAG Submain was constructed around 1925 with internal dimensions of 1727mm high × 1752 mm wide with a grade of 1 in 2280. Works As Executed (WAE) drawings show a timber frame (Oregon) as shoring of the tunnel excavation, Figure 8. Oregon timber is a softwood and is unlikely to have lasted for the nominally 100-year exposure period in the sewer tunnel which was evident during coring of the sewer walls. The most likely excavation technique in this ground type back in 1925 would have been drill and blast techniques fracturing the rock between the invert of sewer and tunnel crown. The excavation was then lined with an unreinforced concrete lining.

In June 2021, SASTTI completed a dilapidation survey of the sewer which indicated that the sewer is in reasonably good condition for its age with only minor seepage shown at isolated construction joints with no significant structural defects worth noting.

As the tunnel alignment through the LAG sewer zone of influence was situated in Class 2 Sandstone (SS-II), the typical approach would be to design an active support utilising bolts and shotcrete. With the reduced cover and uncertainty on rock completeness, GLC and the Design Joint Venture (DJV) consisting of GHD and SMEC designed a passive support of canopy tubes and shotcrete with small excavation advances. The primary focus being the implementation of support as quick as possible post excavation to reduce convergence and settlement. The final design consisted of 140 mm diameter canopy tubes at 9 m length with 3 m overlap and 250 mm shotcrete.

The DJV assessment and modelling indicated that the predicted displacements of the LAG Submain remained less than 5 mm with the proposed sequential excavation techniques as mentioned above. The assessed maximum increases in tensile strain of the LAG Submain among all of the Sensitivity Analysis cases ranged from 38µm to 90µm with the base Case predicted value of 65µm.

It was calculated that during the construction of the unreinforced concrete lining with progressive construction joints, the initial shrinkage in the concrete would equate to 44µm based on 10MPa concrete. Structural assessment of a further 65µm increase in the tensile strain would equate to cracks of 0.08 mm which can be considered as self-healing and is well less than the 150-190µm range of 10MPa concrete. Further investigations works would later prove that the concrete unconfined compressive strength (UCS) was much greater than the assumed 10MPa, increasing the actual tensile strain limit of the LAG concrete to greater than the calculated 190µm limit.

As part of the initial investigation works to confirm initial design assumptions, manned access to the sewer under controlled conditions were undertaken to measure wall thickness, check compressive strength of concrete and endoscope the annulus between rock and sewer. The compressive strength results of the concrete were tested with results ranging from 21-25MPa with a consistent wall thickness resembling the work as executed drawings. Endoscopes showed running groundwater in an annulus packed with rock spalls creating further engineering concerns.

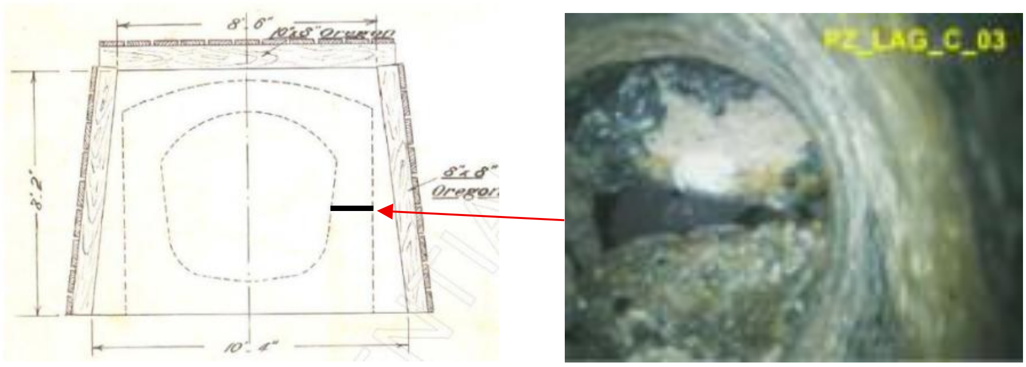

Figure 8 – Left: Work as executed (LAG Sewer 1920); Right: Endoscopes through sewer wall

The side wall endoscopes showed that gravel / rock spalls to the sides of LAG appears to be well packed providing confinement to a bottom construction joint between invert and wall against failure, Figure 8. The crown endoscope probe additionally showed gravel / rock spalls above the crown of the LAG. However, it was unable to be seen from the imaging whether the rock spalls were continuous for the full annulus. This would be consistent with any settlement that may have occurred over the past 100 years.

Due to the inability to conclusively confirm full confinement at the top of the structure, combined with localised groundwater drawdown resulting from the spur tunnel excavation below, it became evident that the sewer system could be subject to internal pressurisation. This pressurisation may exceed the available external confinement, particularly during significant rainfall events. In this event a weak point may exist between the construction joint of the walls and crown of the sewer. This joint may be exposed to a failure mechanism where the crown could lift with the walls collapsing into the sewer void with the sewer crown collapsing into the invert of the sewer main post rain event.

To mitigate this failure mechanism, steel strapping was designed and installed to tie the crown of the sewer to the walls prior to groundwater drawdown to overcome a pressure differential of 53kPa. The straps were installed at 1.2m centres and anchored with stainless steel chemical anchors, Figure 9.

Although the design solution was technically functional, it did not meet Sydney Water's standards in three specific criteria. Following the Specialist Engineering assessment (SEA) conducted by the DJV, a Deviation from Specification was prepared for these three items and submitted to Sydney Water for approval. The deviations were as follows:

- Approval to encroach within nominated Exclusion zone of SW asset of a new structure.
- Approval to increase limit of maximum tensile strain from 20 to 150μm and reduce the minimum incremental radius of curvature from 60km to 8km.
- Approval to adopt revised vibration limits set out in BS 7385 Part 2-1993 "Evaluation and measurement for vibration in buildings Part 2", due to current SW standard nominating a limit of 3mm/s for buried unreinforced structures in a typical soil trench.

All proposed changes were endorsed during workshops with Sydney Water and documented in the SEA report. The first two deviations were theoretical and required no further monitoring once approved. The third required vibration monitoring within the sewer to confirm compliance with revised limits. Recorded vibration stayed well below these limits. Additional sewer lining monitoring included piezometers, as well as flow and vibration sensors Figure 9.

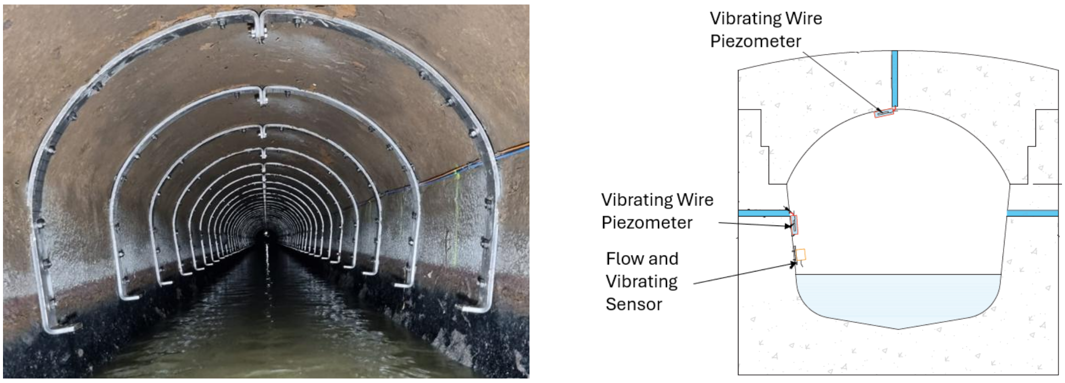

Figure 9 – Left: Sewer Strapping, Right: Sewer Lining I&M arrangement

4 CONCLUSION

At Clyde, extending the spur tunnels and shortening the dive structure improved cost, schedule, and sustainability. However, it created challenges near the portals, where ground cover was low and geology weakened to weathered shale and alluvium. A key issue was the stability of the rock pillar between tunnels. Inclined stitching bolts were used to pre-support the pillar before excavating the second tunnel, reducing risk. Monitoring confirmed the support system's effectiveness, with minimal movement observed during adjacent tunnel excavation.

In addition to the converging pillar design the spur tunnels had further design and construction challenges where the alignments crossed under the 100-year-old LAG sewer with limited rock cover and unknown structural reliability. Rerouting the sewer was not beneficial due to its shallow gradient and proximity to other infrastructure. Instead, the tunnel alignment was optimised to maintain a safe rock cover of 1.3 m to 2.4 m below the sewer. Due to uncertain rock conditions, a passive support system was adopted using canopy tubes, shotcrete, and small excavation steps to minimise ground movement. Detailed modelling showed sewer movement would stay below 5mm, expected strain levels were well within the concrete's safe limits and vibration was negligible.