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ABSTRACT

The mainline running tunnels for Legacy Way comprise of two-lane tunnel boring machines (TBM) driven tunnels. The length of the TBM tunnel section is approximately 4.2 km. The TBMs employed are double shield TBMs. Typical support to the segment ring while advancing a double shield TBM comprises of filling the excavated annulus between the ground and the segmental lining by injecting pea gravel, followed with primary grouting to fill between the pea gravel some distance behind the excavated face. The methodology adopted for the Legacy Way project, however, did not follow this typical approach, instead, two component grouting of the annulus was carried out.

Where the annulus is typically filled with pea gravel, support is provided to the segment ring very close to the end of the tail shield, whereas in the case of two component grout this is not practical as the risk of grouting in the tail shield is high. The staging of the two component grout injection, the grout early age strength properties become key issues in ensuring that the segment ring leaving the TBM tail shield is stable and can support the necessary external loads.

Two-component grout is a grout with two components, cementitious grout (component A) and accelerator (component B), commonly adopted for earth pressure balance (EPB) TBM. The grout reaches its thixotropic state in a matter of seconds providing initial support for the segment rings. Manual injection of two component grout to fill the bore annulus was carried out in stages leaving a portion of the segment rings unsupported. Ring stability during TBM advance was provided by a combination of ram thrust and the segment circumferential joint shear connectors.

This paper outlines the design considerations such as rock load derivation by probabilistic analysis carried out utilising the software JBlock, 3D finite element analyses using the software Strand7 for ring stability and the testing and back analysis of the shear connector load bearing behaviour. In particular, the actual available capacity of the shear connectors against their published capacity by supplier was discussed.

PREAMBLE

Project background

The Legacy Way (formerly Northern Link Tunnel) is the fourth of five components of Brisbane City Council’s TransApex Project. The project involves two bored tunnels with an 11.3 m internal diameter carrying dual lane motorway traffic in each direction from the Western Freeway in Toowong (Mt Coot-tha) to the Inner City Bypass at Kelvin Grove. The total length of the tolled motorway is approximately 4.6 km long. The bored tunnel section excavated by tunnel boring machines (TBM) is approximately 4.2 km long.

The tunnels are lined with precast concrete segmental lining, typical steel fibre reinforced only. The segment lining thickness is 350 mm with an 8 + 1 configuration. The ring type is universal ring with trapezoidal shape segments with skewed longitudinal joint.

The project geology along the tunnel alignment is dominated by fresh Bunya Phylite with approximately 500 m of Neronleigh Fernvale Beds at the eastern end.

Double shield tunnel boring machine excavation

The tunnels were excavated using two Herrenknect double shield TBMs. Subject to the overcut requirement for TBM steering and cutter wear allowance, tail shield thickness,

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shield body configuration and ring build clearance/tolerance inside the tail shield there is an annulus of 200 mm between the excavated bore and the segment extrados. Typical double shield TBM segment ring installation and stabilisation comprises a three-stage process, cement mortar is injected into the annulus for the lower portion of the ring while pea gravel injection through segment adjacent to the tail shield, where pea gravel is prevented from entering the gap between the shield and the excavated bore by spring plates. Following the first stage process of pea gravel injection through segments, the second stage is to further fill the annulus with grout. Therefore, voids within the pea gravel infill will be filled by grout. The last stage is to carry out secondary grouting ensuring the full annulus is filled. This is commonly carried out some distance behind the shield and completes the backfilling of the bored annulus.

The injection of pea gravel provides immediate support to the segmental lining at haunch and sidewall locations. The final crown section of the annulus is typically filled with primary grout at the same time that the pea gravel is grouted. In the Legacy Way project, mortar/pea gravel was replaced by injection of two-component grout. Two-component grout is common for earth pressure balance (EPB) TBM. During EPB tunnelling, the annular gap is filled entirely up against the tail shield by injection of two-component grout through the tail shield. Given that the entire annulus is filled supporting all segments, the segments are generally stable.

Two-component grout is a grout comprises of component A (cementitious grout) and component B (accelerator – usually sodium silicate). The grout reaches its thixotropic state in a matter of seconds dependent on the dosage rate of the accelerator. Although the grout provides support for the lining, the grout strength is relatively weak since the containment by grout infill of the entire annulus provides significant support for the lining instead. The use of high water-cement ratio, bentonite and fly ash to reduce grout costs mean the final strength of the grout is usually just adequate to support the segmental lining.

The replacement of mortar/pea gravel by two-component grout thus poses a number of concerns:

- The need for material handling and bunkering of pea gravel is eliminated.
- Sourcing of pea gravel is not required. Grout mix components are common construction materials readily available.
- The four-stage process is reduced down to a two-stage process, primary and secondary grouting, eliminating a construction activity; this thus leads to construction cost reduction and simpler logistics.

**ROCK LOAD ESTIMATION**

Geology along the tunnel alignment within the tunnelling zone is of fresh rock, either Bunya Phylite and Neranleigh Fernvale beds. During TBM excavation, rock wedges, blocks or fractured material may fall onto the section of segment ring not completely backfilled-supported segments. This may over stress the segment ring leading to failure. The volume and contact area of the wedge is the main form of loading other than self-weight of the segment that will cause instability during the unsupported ring phase.

The rock opening created by the TBM excavation is generally stable. The role of support such as the segmental lining was to retain potential falling wedges, which in conventional tunnelling are retained by the use of rock bolts. The assignment of rock bolts relies on field mapping of the exposed rock mass. Given that TBM excavation is a closed tunnelling operation, support assignment, thus rock wedge size/rock load estimation cannot be facilitated through field mapping. In order to estimate the potential loads on the segments during TBM excavation, the available geological data was reviewed. A number of structural subdomains were identified along the tunnel alignment. These subdomains may form potentially unstable rock wedges, which if not restrained/supported, would cause substantial hazard to the ungrouted segment ring.

A probabilistic approach was used to determine the potential dimensions and interaction with support (Esterhuizen, 1996; Esterhuizen and Streuders, 1998). The probabilistic analysis was undertaken using the software JBlock. JBlock was developed to evaluate the potential for gravity driven rockfalls. Using information such as the spacing, orientation and length of discontinuities, it is possible to simulate blocks in the roof of excavations (Esterhuizen and Streuders, 1998). The results from the probabilistic (JBlock) analysis were presented as block volumes versus frequency for up to 5000 simulated blocks (Figure 1).

Key block analysis methods (Goodman and Shi, 1985) are used to evaluate whether the blocks derived from the joint data and excavation size and alignment would impact on the ungrouted segment ring. The software Unwedge from Rocscience, which applies Goodman and Shi (1985) block theory, was adopted. Unwedge calculates the maximum sized blocks which can form around an excavation. The user can then scale the size of the blocks based on experience and field observations. This is a deterministic approach and does not consider the range of block size that could occur or the likelihood of a particular block size. Adopting the characteristic geotechnical block volume (95th percentile block volume) from the probabilistic (JBlock) analysis is used to scale the block sizes in Unwedge. The corresponding loads calculated by Unwedge for support of the wedges were adopted for derivation of the patch loads on the segments during TBM excavation and the ungrouted segment ring phase.
THE CHALLENGE

The introduction of the greater length of unsupported segments during TBM excavation introduces a concern over ring stability. Rock wedges may fall onto the unsupported section of the segment rings promoting ring movement or fall onto the tail shield and be dragged onto the unsupported segment ring. Measures other than grouting that provide support to the segment ring are:

- load transfer between rings via segment to segment friction and segment ring connections
- support to the leading ring from tail shield guides and auxiliary thrust rams
- integral ring stiffness due to staggered/skew longitudinal joints.

In the case of the Legacy Way segments the connection between segment rings was to be by dowel only without spear bolts or cam and socket connections which provide an established degree of load transfer. There was also to be an allowance for packers in the circumferential joints potentially reducing the segment to segment friction.

STRUCTURAL ANALYSIS

Analysis method

There is no established method for analysis in this instance. Every aspect of the segment ring characteristics from joints and accessories to material modelling were looked. The final adopted form of analysis was by finite element method (FEM). The software Strand7 was utilised. Strand7 is a structural finite element code capable to incorporate the various aspects of the design structure. Illustrated in Figure 2 is the adopted numerical model.

The analysis is undertaken to investigate stability of the lining with the proposed staged annulus grouting methodology. The segment rings are connected through the use of shear dowels at the circumferential joints. Hydraulic rams at the back of the TBM shield always exert thrust forces onto the precast segmental ring in the longitudinal plane both during excavation and as TBM tailshield advances. While the thrust remains in place acting against the lining maintaining minimum contact forces, the annulus is filled with grout in stages up each side of the ring. The hardened grout, including strength increase with time, within the annulus provides bearing support as well as preventing torsional deformations of the partly grouted rings. All these aspects are captured in the adopted analysis.

FIG 1 – Mainline tunnels block volume versus frequency.

FIG 2 – Adopted finite element model.
The numerical model comprises of the following key elements:

- Precast segments are modelled as shell elements with elastic properties.
- Each segment in a ring is modelled discrete interconnected to adjacent segments by ‘zero length’ connectors.
- Infinitely stiff connectors are adopted to simulate the circumferential shear dowels.
- Compression only connectors with friction attributes on circumferential joint between segment rings to simulate the frictional effects on the joint when in contact. This also applies to the ram element to segment interface.
- Stiff ram elements to simulate the ram thrust action onto the lining.
- Compression only springs to simulate the grouted annulus support. Different properties are adopted to represent the grout strength increment with time and the corresponding grout stage. The respective grout strength estimation is by assuming a tunnel advance rate of 2.0 m/h.

**Segment rings**

It is common to simulate the jointed segment ring as a homogeneous/continuous circular cylinder. The segment rings are usually accounted for by established closed-form estimation of the effective ring stiffness. Alternatively, it is also common to simulate the segment joint as ‘zero-moment’ hinge. It was noted through development of the analytical model that neither of the aforementioned provide reasonable estimate of the unsupported segment ring behaviour.

The homogeneity of a continuous ring suggests that there is negligible differential rotation at segment joints. Given that the segments are unsupported, the differential rotation of the unsupported segments at joints is significant. When the joint rotation exceeds certain magnitude, the segment ring will become unstable leading to snap through failure. Simulation of the unsupported segment ring as continuous cylinder is therefore inappropriate.

The use of ‘zero-moment’ hinge to simulate segment joints yielded maximum flexibility for the unsupported segment rings. Known to continuum analysis of underground lining (ie segment rings with full grout annulus), the more flexible the segment ring is the less stresses there are in the segment ring; however, this associates with higher segment ring deformation. Contrary to this the more flexible the segment ring is the more stress there are in the segment ring. Though it is opposite to conventional segment ring analysis, this is common in structural mechanics. In common above ground structural engineering such as a portal frame structure, the more movement there is the higher the structural stresses owing to the ‘P-Δ’ effect or second order effect. The approach to maximise the unsupported ring stiffness in the analysis is hence not appropriate.

The unsuitability of the common method of segment analysis means that the use of discrete modelling is required. The segments were assigned as plate elements with elastic concrete properties interconnected with ‘zero-length’ connectors.

**Segment longitudinal / radial joint**

Strand7 offers the use of ‘zero-length’ connectors with allowance for moment-rotation and force-displacement characteristics/stiffness assignment. The assignment with effectively infinite displacement stiffness but zero rotation stiffness on the segment radial joint is equivalent to the use of ‘zero-moment’ hinge. Adding the moment-rotation stiffness defined by the closed form solution developed by Jančen (1983) (Figures 3 and 4), the ‘zero-moment hinge’ becomes a moment hinge. This added stiffness to the unsupported segment rings and closely approximated the flexural behaviour of the rings.

The closed form solution developed by Jančen (1983) is axial force dependence. The joint energy (area under moment-rotation curve) increases with the increasing axial force on the joint (assuming no joint failure). Strand7 only permits the use of a single moment-rotation curve at one time. Stepwise iterations was thus required to determine the appropriate moment-rotation curve for each segment radial joint.

**Segment ring joint**

There are three primary aspects of the segment ring joint model:

1. compression stiffness
2. shear/joint friction
3. rotational stiffness.
The appropriate implementation of these parameters governs the accuracy in the unsupported segment ring behaviour.

During segment erection, thrust forces are applied by the hydraulic cylinder at the back of the TBM shield to keep the erecting segment ring in place. These thrust forces are also to facilitate forward movement of the tail shield when required. Grouting of the annulus takes place as the thrust forces remain active against the segment. This action promotes the thrust compressive stresses to be locked within the lining over the lifetime of the structure.

Prior to segment erection a group of rams is unloaded giving space necessary for installation of a segment. As soon as the erecting segment is in the correct position the relaxed rams are loaded pushing the segments in place. This action also compresses the gasket on the circumferential joint, which provides water tightness of the tunnel segmental linings.

The act of the ram pushing against the lining at all times not only induces longitudinal stresses to be locked in the lining but also ensure active contact between adjoining segment rings inducing behaviour similar to that of a continuous shell structure.

Though it is acknowledged that given the structure is not a continuously cast structure but a jointed structure. The segment rings will not act completely the same as a continuous shell structure. The segment rings will likely to exhibit characteristics of a continuous shell structure such as load sharing/transfer between adjoining rings, hence the coupling effect.

The modelling of the segment ring joint is therefore the correct simulation of the coupling effect. The ram thrusting against the segment ring joint promotes minimum axial stresses. Together with the friction coefficient of the interface, shear resistance is provided for stability of the unsupported segment ring. However, the preference to adopt packers in between segment ring joints means that there is minimal frictional resistance to be offered. Added to the challenge in the design analysis is the by-product specification and sourcing of the suitable packer materials.

Strand7 permits the use of ‘gap’ elements to simulate the frictional effect on a joint. It applies the frictional component as a function of the normal stiffness, i.e. the shear stiffness is the product of friction coefficient and stiffness. This then permits the ram thrust forces to be transformed to joint shear resistance in the analysis. Note the ‘gap’ elements permit no tension thus simulating the joint interaction only in compression contact.

The application of the appropriate joint compression stiffness was thus critical to the segment ring joint model. The compression stiffness must be large enough such that ‘P-A’ effect in the direction of the tunnel axis does not occur and that the compressive deformation of the ring in that direction is close to that in actual practice. Concurrently the compression stiffness must be low enough to prevent overestimation of the joint shear resistance. Iteration of the joint compression stiffness was thus required.

The simulation of the shear dowels further added complication to the already complex iterative analysis. Though Strand7 permits the use of ‘zero-length’ connector, it is limited to one direction only and does not apply to intersecting interface. Therefore, a discrete gap was implemented to separate individual rings. Though theoretically the gap dimension does not affect the simulation, it is observed in this the gap dimension shall be kept as minimum. Else the connector may rotate (due to the act of the thrust forces) adding additional shear to the dowels.

From capacity specification stand point, the dowels were simulated with assignment of effectively infinite shear stiffness. The compression stiffness was kept the same as that of the segment ring joint. The yields the corresponding maximum dowel shear forces and is the corresponding dowel strength specification. This scenario is thus the dowel is retaining the majority of the load and slippage of the joint does not occur.

In practice, joint slippage is imminent. The use of the dowel tested stiffness was therefore also adopted. In this scenario, substantial unsupported ring deformation was yielded (Figure 5). It shall be noted that in this case the flexural stresses on the unsupported segment ring has substantially increased. This case therefore governs the flexural design of the segments (Figure 6). In fact, the critical load case for the design of the Legacy Way precast concrete segments is the construction load case of these unsupported segment rings. The notable deformation provided a mean of monitoring during excavation.

Zero rotation stiffness was assigned to the segment ring joint as its likelihood to ever estimate the joint stiffness. Further the axial stresses are relatively small in magnitude to mobilise large resistance for the joints. It was deemed unnecessary to incorporate another degree of freedom/complexity into the analysis given the little difference it would make.

**Boundary conditions**

Detail simulation of the ram interaction was undertaken as the transfer of the axial stresses defines the magnitude of the joint shear resistance for ring stability. The most unsupported section of the segment rings also locates next to the first ring.
immediately behind the thrusting/assembled ring. It is thus a critical aspect of the analysis to the ram-to-segment interface.

The rams were simulated as rigid diaphragms (Figure 7). They were connected to the segments through the use of ‘gap’ elements to simulate the compression only shear dependency of the interface. The compression stiffness was taken as the same of that for the segment ring joint. This was to yield a uniform load transfer mechanics/characteristics in the tunnel axis direction. The thrust pressure was applied onto the rigid diaphragms not on the segments directly.

The second boundary condition is the support conditions for the last rings. Compression only elements were assigned to simulate the end boundary conditions. In special scenario such as the launch portal, there was a custom design TBM launch steel ring. The steel structure was added to the analysis to provide correct simulation of the boundary conditions (Figure 8).

Though the ram thrust force was direct input parameters. Analysis runs for a set/range of ram thrust forces was required to simulate the optimal/minimum ram thrust forces required for unsupported segment ring stability. This added the next degree of complexity.

The last boundary condition is the backfill grout simulation. There is no definitive method in approximation of grout stiffness. Data for early age (in terms of hours) grout stiffness is very scarce. This was limited by also the lack of an established test method. Hence, engineering judgement was made to the grout stiffness parameters (Figure 9). It was assumed that the young grout behaves similar to that of a stiff to hard clay subjected to age. Sensitivity was also carried out yielding the final set of grout mix design parameters (Table 1).

**Loadings**

The use of the statistical approach in load estimation was the directive of a risk base design method. The loading condition
is temporary. Unless otherwise prolong period of the segment rings being unsupported, the use of worst credible/maximum possible loads will yield over specification for the works. The risk base approach also associates with a set of monitoring triggers that permit close monitoring of the unsupported segments. In the scenario which the TBM was stopped to accommodate other concurring in-tunnel activities, these triggers served to warrant the safety of the workers underground.

In Legacy Way, one of the TBM was stopped for longer than a month. Full grouting of the unsupported segment rings were not taken place in fear of grouting in the tail shield. Monitoring in accordance to the triggers set was therefore carried out. During the course of monitoring over the month long-period, additional deformation in association with progressive segment cracking was observed. This verified the risk base design given the unsupported segment ring crack development follows the prediction from the analysis.

**Time-dependency**

From the above, it is imperative to acknowledge the time-dependency (ie creep effect) of the unsupported segment ring joints. The relaxation of the ram thrust forces within the unsupported ring shall also be accounted for.

**Analysis summary**

There are multiple degrees of freedom in the analysis. It requires extensive iteration and understandings of the combined interaction and dependency of each parameter. The key parameters are summarised below:

- segment radial joint rotational stiffness
- segment ring joint compressional stiffness
- segment ring joint friction coefficient
- connector dimensions

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**TABLE 1**

<table>
<thead>
<tr>
<th>Test</th>
<th>Mix design target</th>
<th>Tolerance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mud balance (tested without B-component)</td>
<td>&gt;1.25</td>
<td></td>
</tr>
<tr>
<td>March cone (tested without B-component)</td>
<td>35 sec Increase of viscosity after 24 hours</td>
<td>±3 sec</td>
</tr>
<tr>
<td>Bleed (tested without B-component)</td>
<td>&lt;3% after 24 hours</td>
<td></td>
</tr>
<tr>
<td>Cube compressive strength</td>
<td></td>
<td></td>
</tr>
<tr>
<td>After 1 hour</td>
<td>&gt;0.1 MPa</td>
<td></td>
</tr>
<tr>
<td>After 2 hours</td>
<td>&gt;0.5 MPa</td>
<td></td>
</tr>
<tr>
<td>After 3 hours</td>
<td>&gt;0.7 MPa</td>
<td></td>
</tr>
<tr>
<td>After 1 day</td>
<td>&gt;1.5 MPa</td>
<td></td>
</tr>
<tr>
<td>After 7 days</td>
<td>&gt;3.0 MPa</td>
<td></td>
</tr>
<tr>
<td>Set-time &gt; compressive strength</td>
<td></td>
<td></td>
</tr>
<tr>
<td>After 1 hour</td>
<td>&gt;0.1 MPa</td>
<td></td>
</tr>
<tr>
<td>After 2 hours</td>
<td>&gt;0.5 MPa</td>
<td></td>
</tr>
<tr>
<td>After 28 days</td>
<td>&gt;4.0 MPa</td>
<td></td>
</tr>
<tr>
<td>Get-time</td>
<td>11 sec</td>
<td>±3 sec</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>Max 10 000 microstrain</td>
<td></td>
</tr>
<tr>
<td>Grout temperature</td>
<td>20°C</td>
<td>±10°C</td>
</tr>
</tbody>
</table>
• shear dowel stiffness
• ram-to-segment interface inputs
• end segment support conditions
• grout stiffness with time dependency
• load application.

The final set of tunnelling specification is summarised in Table 2. Presentation of the ring stability was by plotting the critical flexural forces from each load combination against the segment bending capacity curve (Figure 10).

### TABLE 2

<table>
<thead>
<tr>
<th>Operation parameter specification</th>
<th>Minimum value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ram thrust force (kN per ram)</td>
<td>450</td>
</tr>
<tr>
<td>Segment packers coefficient of friction</td>
<td>0.3</td>
</tr>
<tr>
<td>Shear dowel capacity (kN)</td>
<td>150 SLS</td>
</tr>
<tr>
<td></td>
<td>Minimum 5 mm offset on adjacent segment intrados to seat the dowel</td>
</tr>
<tr>
<td></td>
<td>Maximum 15 mm offset at serviceability</td>
</tr>
<tr>
<td>225 ULS</td>
<td>Maximum 25 mm at ultimate load</td>
</tr>
<tr>
<td>Grout strength</td>
<td>As per Table 1</td>
</tr>
</tbody>
</table>

**SHEAR CONNECTORS**

Given that the segment rings are not supported due to the partially filled annulus, the role of the shear connectors is the primary stabiliser interlinking adjoining rings.

Figure 11 shows the geometry of the adopted shear connector. The connector is supplied by FIP Industriale. It

![FIG 10 – Summaries of segment flexural design against staged annulus grouting.](image)

![FIG 11 – Shear connector overview.](image)
consists of three main components, aluminium centering, plastic pin encasing a deformed steel rod and plastic cast in sockets. From inspection of the dowel geometry it is clear that the connector will not undergo the exact dowel action behaviour as that from a dowel with uniform cross-section.

Figure 12 illustrates the deformation behaviour of the dowel connection subjected to shear loads. When the dowel between the concrete joints is loaded by shear in front of the joint, it is supported by a contact pressure along the part that is embedded in the concrete element. This loading condition thus dowel action normally results in considerable flexural deformation and flexural stresses in the dowel. Firstly, the dowel may suffer from flexural failure by formation of one or more plastic hinges in the dowel; simultaneously local crushing occurs in the surrounding concrete where contact pressure is high. Secondly, the dowel may fail due to reaching its yield capacity when subjected to loads. Lastly, concrete splitting failure may occur, ie the loads give rise to a highly concentrated reaction in the concrete under the dowel pin. Due to the oversizing of the aluminium centering increases the concrete crushing capacity and the failure is likely of the first mode combined dowel and concrete failure.

This type of shear connector geometry is common as it is evidenced from a number of segment accessories suppliers. These shear connectors are usually tested to establish their capacity. The designer would specify the required capacity and the suppliers propose their recommended product with testing for verification. The tests by the suppliers are often based upon testings of the connectors in a prefabricated steel mould (steel mandrel). As a result, the corresponding shear capacity is likely to be governed by the material capacity at the shear plane, ie centre length of the dowel.

Since these connectors play a key role in the stability of the unsupported rings, the verification load tests by the supplier was questioned:

- The test was carried out in a steel mould. The steel mould is considerably stronger than the concrete. The failure mode of the connector is likely the combined failure of the dowel and the concrete. Without the attribute of concrete elements, the test may overestimate the connector capacity.
- The test verifies the capacity of the connector itself but does not verify the connection system as concrete encasement was replaced by steel.
- Since plastic pin was used, the creep behaviour of the connector under load was not reflected.
- The test lacks the ability to provide the load deformation relation of the connector for input in the design analysis. Concrete is considerably weaker than steel. The test is likely to give false indication on the connector stiffness.
- Tolerance was introduced by oversizing the segment concrete pocket for retrofitting during ring build. The steel mould adopted by the supplier was a tight fit configuration of the internal dimensions of the plastic socket.

Based on theoretical calculation, the concrete may reach its crushing capacity prior to the dowel yielding. From the standard dowel capacity determination procedure, both the dowel and the concrete load bearing capacity will be calculated and the least capacity calculated as the connection capacity. Often the concrete capacity is lower than the dowel capacity. As indicated from the calculation, the capacity of the connection may not reach the supplier specified capacity for the connector of up to 450 kN.

Noting the concern over the connector capacity, full-scale testing was proposed to investigate the true capacity of the connection. The test consists of two pieces of test specimen 350 thick, 1100 long × 600 deep (Figure 13). Either one or two dowels are installed connecting the two specimens. Debonding was applied between the concrete contacts to ensure loading was transferred on the connector(s).

In general, non-destructive testing in accordance to the prototype test procedure outlined in AS5100.5 was carried out. Figure 14 shows the exposed joint condition after testing. The connection was tested to its design capacity of 225 kN per connector instead of 450 kN provided by the supplier. As shown in the figure minor concrete spalling and cracking of the aluminum centering occurred upon the connector reaching the specified maximum test load. This indicates the connection system meets the design specification.

The system was also tested to failure. The capacity of the dowel was compared to the test results provided by the supplier using steel mandrel. It was observed there is significant difference in the load deformation characteristics between the two results (Figure 15). The ultimate failure load tested with concrete blocks yielded 400 kN, slightly less than the specified 450 kN. The exposed joint after the test shows completely crushed aluminum centering and yielded plastic pin associated with moderate concrete spalling around the connector. Therefore, the assumption of the combined concrete and dowel failure is verified.

The relatively large deformation of the connection under load indicates the requirement for connector capacity specification to also account for the allowable deformation. Substantial differential movement between segments is required to mobilise the connector capacity. The ability of the connector to provide its required resistance is not possible if the deformation is not controlled. Hence, a specification of the connector resistance with the corresponding deformation limit is required (Table 2).

The introduction of the deformation limit also indicates that the usable capacity of the connectors is much lower than that given by the supplier. In this case it usable capacity is less than half of that specified by the supplier.
The deformation specification is also to ensure minimum stiffness be provided by connection to avoid over loading the adjoining segments. The lower stiffness the connector is the higher flexural stresses are there in the segment due to non-linear effect.

Alternative to the use of the adopted shear connector, the cam-and-socket type (shear key) segment connector would have been suitable as would have been spear bolts. However, the stiffness of this type of connector is high. As a result monitoring of the differential movement between segments cannot be facilitated. From a tunnel construction stand point, it is often desirable to have monitoring in place to facilitate monitoring and response. When monitoring is not possible, the support system shall be designed to support the maximum load even though the likelihood of occurrence is very low.

Hence, the use of the post-installed shear connector yields the end result of the efficient support design regime.

**GROUTING**

**Grout mix**

The intention to replace pea gravel with two-component grout promotes the need to explore the possibility of adopting typical two-component grout mix design. A typical mix two-component grout mix design has a water-to-cement ratio up
to 3.0. This implies that the grout strength is likely to be low. During EPB tunnelling the role of the grout is to completely fill the annulus to provide stability of the lining by containment. For the application to double shield tunnelling a higher grout strength requirement is required given full containment will not be provided.

Table 1 shows the grout mix design requirements adopted for the TBM tunnelling. These requirements were developed based on a series of finite element analyses. These analyses consider not only the grout strength but also the various rock loads from the probabilistic analysis as well as the possible grout pattern options.

The key design requirement is the grout strength specified at varying time after injection. With the addition of component B (accelerator) it is able to control the strength gain function with time. Noting immediate support like that from pea gravel is not possible by two-component grout, it is critical to adopt a grout mix that achieves the minimum grout strength required. The grout strength requirement is designed to meet the load demands at increasing time steps.

The application of accelerator does not increase the final grout strength. Rather it only permits the rapid increase of early strength not long-term strength. This is illustrated in the grout strength requirement which the rate of change between one and two hours is 0.4 MPa/h. The later hour only shows a rate of change of 0.2 MPa/h which is half that of the previous hour.

A number of grout trials had to be carried out to develop a mix design that met the criteria set out in Table 1. The final adopted grout mix is given in Table 3. As shown the final grout mix has a water-to-cement ratio of approximately 2. This is lower than that from a typical EPB TBM two-component grout.

**Injection trial**

The annulus between the excavated tunnel bore and the segment extrados is filled with grout to provide a stable interface between the segment ring and the excavated rock face. The segment lining that leaves the tail shield of the TBM is approximately 400 mm smaller than the tunnel excavated diameter. This annulus (approximately 200 mm thick) is grouted in stages up each side of the mainline tunnel segmental lining.

The bottom of tail shield is capable to inject grout while the tail shield moves forward. The grout in this case is the two-component grout that is also to be used for the remainder of the annulus. This tail shield grout is able to fill up to one-third of the annulus, 3 m vertical from the bottom of the annulus.

The remainder 9 m vertical of annulus is filled by manual stage grouting process through the grout ports in the segments. The stage grouting is carried out in steps up each side of the segment ring until the crown is grouted. These grout stages are labelled as stages 2-6 in Figure 16 leaving the segment ring unsupported up to a length of approximately 8 m.

Apart from stage 6 grouting, grout is injected per stage over a spread of approximately 2 m × 2 m. Grout pressures imposed upon the lining are limited to 50 kPa. The spread of grout is controlled by varying the set time, flow rate and volume to prevent the grout injected through the segments reaching the TBM tail shield in a liquid state. Grout injection is terminated at any stage either by reaching maximum grout volume per stage pressure.

It is the first time that a double shield TBM is excavated with backfilling by two-component grout in lieu of pea gravel.

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**TABLE 3**

<table>
<thead>
<tr>
<th>Cement (kg)</th>
<th>Fly ash (kg)</th>
<th>Bentonite (kg)</th>
<th>Retarder (kg)</th>
<th>Water (L)</th>
</tr>
</thead>
<tbody>
<tr>
<td>290</td>
<td>80</td>
<td>40</td>
<td>5</td>
<td>766</td>
</tr>
</tbody>
</table>

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**FIG 16** — Grout staging for annulus backfill between the tunnel bore and the precast segmental liner.
There is a need to verify the grouting methodology and assumption, i.e. whether the grout pattern is achievable as per the design specification.

Full scale trial with the aid of purpose built formwork was undertaken. The various grout injection control parameters such as grout pump rate/injection rate, component B dosage rate and mixing rate, grout staging and pattern control were trialled. The ability to achieve the required grout infill pattern, grout infill quality (i.e. the ability to completely fill the void) and grout integrity/soundness are the key determinants for the adopted grout injection control parameters. It was also decided upon the trial that a component B nine to ten per cent of component A volume is optimal to the adopted grout pattern.

The outcome of the trial and the success of the grout procedure to achieve the required design grout infill pattern is shown in Figure 17.

THE SOLUTION
The final arrangement of the Legacy Way segments in the rear of the TBM comprised of rings joined by high shear capacity dowels, packers for the circumferential joint that had a tested friction, a higher than usual minimum auxiliary ram pressure to impart compression between the ungrouted ring at all times.

Segment geometry and gasket selection also allowed for the removal altogetether of packers in the circumferential joint.

CONCLUDING REMARK
Replacement of pea gravel with two-component grout in a double shield TBM excavation generated the need to investigate every aspect of the construction load case. From materials specification through to full-scale testing of the segment accessories was carried out to verify the design assumption. 3D FEM was undertaken to simulate the unsupported segment rings deformation characteristic to facilitate safe TBM operation. Attention to analysis parameter details is shown to be critical in assessing the load bearing resistance of the system. The end result is an efficient yet robust design that contributes to the record TBM excavation rate achieved in the Legacy Way project.

The lesson learnt outlines the important role of the appropriate analysis method and the actual segment accessories characteristics when designing segmental lining for use in double shield TBM excavation.

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REFERENCES