Development of Reinforced Concrete Segmental Lining Design for MRT Bored Tunnels in Singapore

D. Wen
Tritech Consultants Pte Ltd, Singapore
E-mail: dazhi.wen.tpy@gmail.com

ABSTRACT: Reinforced concrete segments are commonly used as tunnel linings for bored tunnels constructed by tunnel boring machines (TBM). They have been used from 1980s till today for the majority of the Singapore Mass Rapid Transit (MRT) bored tunnels constructed by TBM as permanent supports. This paper describes the development and evolution of the segmental lining design from the Phases I/II of the MRT construction in the 1980s to the current design for the MRT lines under construction. The topics include the general arrangement of the segmental linings, structural design requirements, durability requirements, fire resistance and selection of waterproofing materials of the linings. The design and construction of bored tunnels in close proximity is presented with the experience gained in the past projects. Fire tests conducted by the Land Transport Authority are also presented. The rational, experience and challenges of adopting steel fibre reinforced concrete segments in recent MRT projects are discussed in the paper. The paper also presents in detail the experience gained in Singapore MRT projects in selecting the gaskets for waterproofing of the joints between segments to achieve the durability requirements for the bored tunnels.

KEYWORDS: Bored tunnels, Segmental linings, Waterproofing gaskets

1. INTRODUCTION

Bored tunnels constructed by tunnel boring machines (TBM) have been widely used for tunnel construction for the Singapore Mass Rapid Transit (MRT) system due to their minimal impact on the surface structures and utility services. The design of the precast reinforced concrete (RC) segmental linings for the bored tunnels has gone through three stages: the Phase I/II MRT projects in the 1980s; the North East Line (NEL) MRT projects which was completed in 2003 and the subsequent MRT projects for the Circle Line (CCL), Downtown Line (DTL) and the Thomson-East Coast Line (TEL) projects. The general arrangement, structural design and detailing and waterproofing details have been evolved over these stages of design development for the MRT tunnels. New technologies and materials have been adopted to enhance the durability of the tunnel structures at the time when the design was developed for the various stages of the MRT projects.

2. GERRAL ARRANGEMENT OF PRECAST REINFORCED CONCRETE SEGMENTAL LININGS

In the Phase I/II MRT projects, bored tunnels were constructed between stations within the Central Business District in 9 civil contracts. The nominal diameter of the bored tunnel is 5.2m (including a minimum construction tolerance of 100mm on radius) without a walkway in the tunnel. All design and build contractors for the bored tunnel contracts opted for more construction tolerance, resulting in the internal diameter of the bored tunnels in a range of 5.23m to 5.4m (Copsey & Doran, 1987). Since the NEL projects a tunnel walkway has been specified in the bored tunnels with a minimum clear walkway width of 800mm. The nominal diameter of the tunnels has thus been increased to 5.6m. With the specified construction tolerance of 100mm on radius the internal diameter of all MRT bored tunnels has now been standardized to 5.8m.

The thickness of the tunnel lining in the Phase I/II MRT projects varied from 225 mm to 250 mm (Hulme & Burchell, 1992). For the NEL Projects most of the design and build contractors adopted lining thickness of 250mm except one who adopted 275mm thick segment. For the subsequent projects the thickness of the lining has been “standardized” to 275mm. Segment width was designed to be 1m for the Phase I/II MRT works. In NEL projects segment width was typically 1.2m, except 2 contracts where 1.5m wide segments were used. For CCL projects and the subsequent MRT lines, the segment width of 1.4m has been adopted based on feedback from different tunnel contractors and TBM manufacturers during the design for the CCL tunnel linings.

The RC segmental linings in Phase I/II projects have generally adopted the arrangement of five segments plus one key, except for two contractors. The segmental rings were tapered with 40mm taper. The key segments were rectangular for most of the contracts, except for two contracts where the key segments were also tapered on plan. Relative large bolt pockets were used in some of the contracts in the Phase I/II MRT projects to accommodate the bolts in the segments. This resulted in large reduction of segment cross section area at the bolt pocket locations. Both fabricated steel bolt pockets (e.g. in C108) and the recessed concrete bolt pockets (e.g. C109 and C301) were used in a few contracts. Where the prefabricated steel bolt pockets were used along the circumferential joints the reduction of the cross section area was much less than some of the recessed concrete bolt pockets. For example the reduction in cross section area of the lining was in the order of 20% in the design of C109 segments with recessed bolt pockets. However the segments with the prefabricated steel bolt pockets had a relatively weak lateral bending capacity due to the inability to form an edge beam along the circumferential joints and were susceptible to cracking under TBM jacking force (Copsey & Doran 1987). Figure 1 shows three typical segments for the Phase I/II projects.

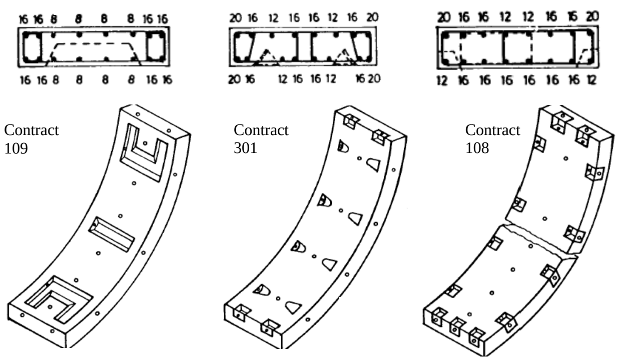
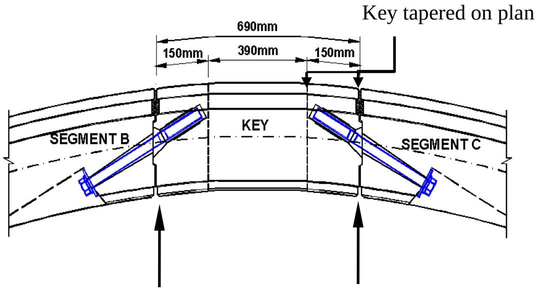


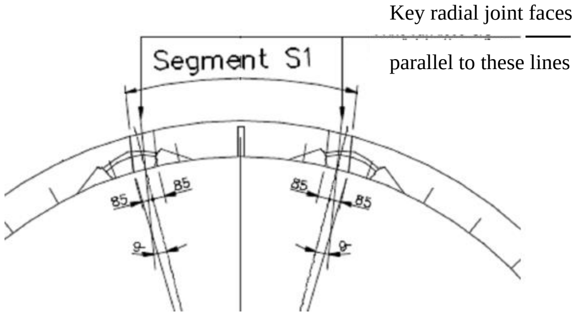
Figure 1 Typical segment used in Phase I/II MRT projects (Copsey & Doran, 1987)

These general arrangement principles have been continued for the subsequent MRT projects until today for steel bar reinforced concrete segments, with 5 segments plus one key segment.

The key segment is tapered on plan, with the radial joint faces also tapered to assist the installation. The radial joint faces typically tapered radially toward the centre of the tunnel. However there have been two projects where the taper of the radial joint faces were not towards the tunnel centre. One such design adopted the parallel radial joint faces and the other semi-parallel radial joint faces for the key segment, see Figure 2a and 2b. From the performance of the key segment it is believed that the radially tapered joint faces toward the tunnel centre would be preferred as such arrangement would provide stability to the ring once all segments plus the key segment are installed in place. Parallel radial joint faces of the key segment has the inherent risk of it being pushed out or having large movements during tail void grouting while the TBM is propelled forward during mining and during subsequent secondary or tertiary grouting when required.



(a) Parallel radial joint faces of key segment



(b) Semi-parallel radial joint faces of key segment

Figure 2 Examples of parallel and semi-parallel radial joint faces of key segment

From NEL projects onwards recessed concrete bolt pockets have been used for straight or curved bolts. Since CCL projects all segments have been cast with both right hand and left had tapers of 40mm to facilitate the negotiation of alignment curves. Figure 3 shows a typical arrangement of bored tunnel segmental lining for CCL projects.

In the detailing of the segments for the CCL bored tunnels, the reinforcement bars are also intentionally placed toward the edge of the circumferential joints so that the segment would have an enhanced lateral bending capacity to facilitate the installation, see Figure 4. This enhanced bending capacity would resist the potential formation of cracks due to bending of the segment when there is inaccuracy during shoving of the TBM as shown in Figure 5.

On the joint design both block joints and double convex joints have been used for NEL and the subsequent projects. Where double convex joints are used they are all for the radial joints. The radius of the convex joint surface is typically 2m. The advantage of double convex joints is that the contact area between the two segments are well defined; and if there is any step across the joint due to installation inaccuracy, the eccentricity due to the step is only a half of the actual measured step across the joint, see Figure 6.

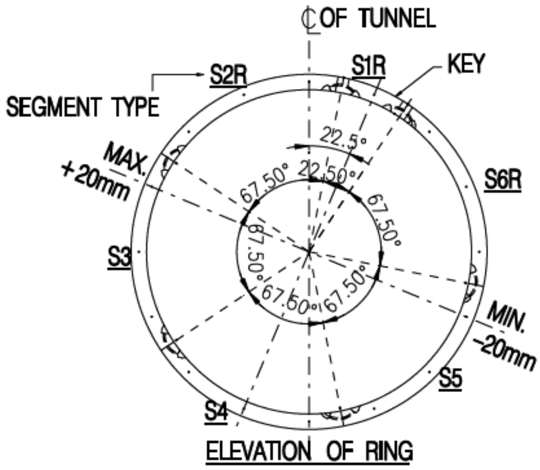


Figure 3 Typical general arrangement of segmental lining for CCL bored tunnels

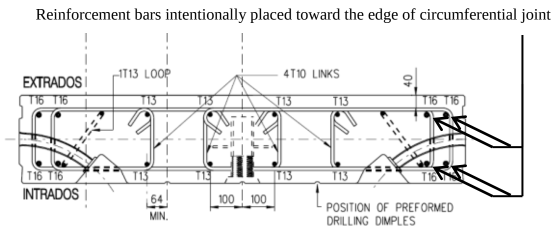


Figure 4 Detailing of steel bars at circumferential joints

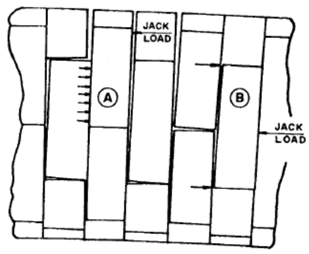


Figure 5 Segment under lateral bending due to installation inaccuracy during shoving of TBM (after Copsey & Doran, 1987)

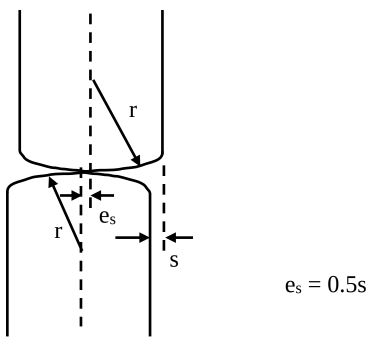


Figure 6 Eccentricity due to step across a double convex joint

3. STRUCTURAL DESIGN OF PRECAST REINFORCED CONCRETE SEGMENTS

3.1 Design Methods and Ground Loading

The current acceptable design methods in the Civil Design Criteria for Roads and Rail Transit System of the Singapore Land Transport Authority (LTA) include the continuum model by Muir Wood (1975) modified by Curtis (1976), bedded beam model by Duddeck & Erdman (1982) or the finite element or finite difference methods. These acceptable design methods have generally been the same for all bored tunnel works for the NEL projects and the subsequent MRT projects. For the Phase I/II MRT projects the recommended method by the then Singapore MRT Corporation was that proposed by Muir Wood modified by Curtis, although other similar methods would be considered by the Corporation. It turned out that most of the design and build contractors adopted the recommendation.

It has always been the requirement for the MRT bored tunnels to be designed for the full overburden in all type of ground conditions other than slightly weathered or fresh rocks. The most onerous water pressure should also be considered. It is considered that the requirement of full overburden is conservative and represents the upper bound solution of the loads in the lining that is likely to be developed over the design life of the tunnels.

The current Civil Design Criteria also require that the bored tunnel linings be designed for a surcharge of 75 kN/m^2. This surcharge is considered to be the upper bound of a typical foundation loading for 5-storey buildings. The use of a relatively high surcharge for the bored tunnel lining design is to achieve the objective of minimizing the impact of the presence of the bored tunnels on any future developments above or adjacent to them. Although the surcharge appears to be much higher than the usual surcharge of 25 kN/m^2 for cut and cover tunnels, it will not have any impact on the cost of the tunnel linings, as the forces in the linings are mainly compression. The adoption of the relatively high surcharge for bored tunnel lining design is thus good to have to make allowance for future flexibility in land use.

The distortional loading, which is the difference of the vertical and the horizontal pressures on the lining for the continuum model, is determined by the distortional load factor, often denoted by the letter "k". The distortional loading and the interaction between the ground and the lining are responsible for the amount of bending moment in the lining for design. The distortional loading factor, k should be distinguished from the coefficient of earth pressure at rest, k_o. The final loading acting on the tunnel lining is the results of ground-lining interaction, yielding of soils around the excavation and the subsequent re-consolidation of the yielded zone around the lining. Because of the yielding and re-consolidation process, the workmanship and the construction process by the TBM will have an impact on the final loading condition. The specified distortional load factors in the LTA Civil Design Criteria are shown in Table 1. These values are generally in line with the back-analysis of lining deformation for bored tunnels constructed in the marine clay and weathered granite by Copsey & Doran (1987) and Wen & Ong (2003) for bored tunnels constructed in the soils of Old Alluvium in Singapore.

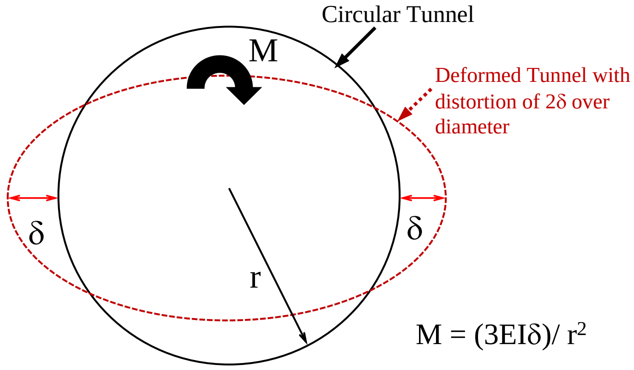
Table 1 Distortional load factor, k

Soil Type	**k**
Marine Clay, Estuarine Clay or Fluvial Clay	0.75
Fluvial Sand, Old Alluvium, Weathered Granite	0.5
Weathered Sedimentary Rocks	0.4

3.2 Design for Additional Tunnel Distortion due to Future Ground Movement

The LTA Design Criteria require that the bored tunnel lining should be designed for an additional distortion of +/- 15mm on diameter to allow for future development. This design check is necessary as the Code of Practice for Railway Protection stipulates that the future construction close to the MRT tunnels should limit the tunnel movements within 15mm in any direction, which means that the tunnel structure is deemed to be structurally safe if the movement is less than 15mm in any direction. For this design check the Design Criteria allows the use of the long term Young's modulus of the concrete considering creep effects and the reduction of the moment of inertia of the lining considering the effects of the radial joins in a ring.

One common design approach is shown in Figure 7. As suggested by the Design Criteria, it is common in the design approach that the Young's modulus, E of the concrete is taken as a half of the short term modulus when calculating the moment due to the additional distortion of 15mm on diameter. Arguably the adoption of the so called long term Young's modulus being a half of the short term Young's modulus might under-estimate the moments as the Young's modulus of concrete will actually increase over time in the same way that the concrete strength increases over time. The Young's modulus that should be used in estimating the moment should be the modulus at the time of the deformation occurring (i.e. at the time of loading), which is an undefined future time. If the modulus used in the estimation is about a half of the modulus at the time of the deformation taking place, the moment is also under-estimated by a factor of 2. What would actually happen is that the lining may crack when the additional moment generated due to the distortion as a result of ground movement and the locked-in moments already existing in the segments due to ground loading, water pressure, etc. exceed the cracking moment capacity of the segments. When cracks occur the moment of inertia of the segment section is reduced and the moment is reduced accordingly. Thus the safety of the lining will not be in any way compromised as a result of the additional distortion of a maximum of 15mm on diameter.



Where M is the additional moment due to distortion of 2δ of 15mm on diameter, or δ of 7.5mm on radius of r; E is the Young's modulus and I the moment of inertia of the segment.

Figure 7 Additional moment due to distortion of lining

3.3 Tunnels in Close Proximity

When twin tunnels are to be built parallel to each other, the rule of thumb is that the clear space between the two tunnels should be at least one tunnel diameter of the larger tunnel. Otherwise the influence of the construction of the second tunnel should be considered in the design of the tunnel lining of the first tunnel. The main consideration is that the construction of the second tunnel in close proximity to the first tunnel that has already been built would cause movement of the lining of the first tunnel. This movement will generate additional distortion and therefore moment in the lining. To ensure the structural adequacy of the lining it is necessary to cater for the additional moment in the design.

Wen et al (2004) developed a procedure to derive the additional moment on the first tunnel lining when the second tunnel is being constructed based on the expected volume loss of the second tunnel construction. In the derivation of the formulae to calculate the additional moments, it is assumed that the ground would move elastically when the second tunnel is under construction. The lateral movement under a volume loss Vs would result in a differential movement (distortion) of the first tunnel lining, δ of (u_a – u_b), see Figure 8. With the distortion δ being defined, the additional movement can be calculated by elastic formulae as shown by Wen et al (2004).

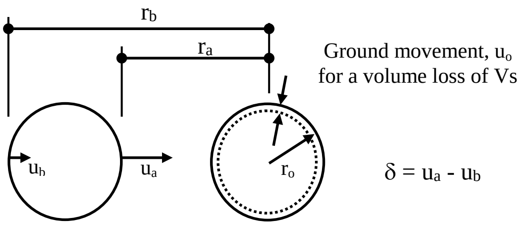


Figure 8 Distortion of first tunnel lining due to second tunnel construction in close proximity, Wen et al (2004).

During the CCL Stage 3 construction one stretch of the twin bored tunnels at Lorong Gambir were driven in weathered granite at a minimum clearance of 2.3m, see Figure 9. To cater for the impact of the second tunnel construction, the lining of the first tunnel was designed to resist the additional moment that would be generated due to the additional distortion that the lining would be subjected to when the second tunnel was being constructed. Based on the interpretation of geological data, the tunnels were expected to be constructed within the completed weathered granite, GV. Assuming the volume loss of 1% during the construction of the second tunnel, the hoop trust and the combined moment due to ground loading and the additional moment are still within the service limit state (0.2mm crack width) and the ultimate limit state, as shown in Figure 10. The figure shows that there would have been more capacity to cater for a higher volume loss than 1% as the additional moment, combined with the moment due to ground loading has not reached the capacity limit yet.

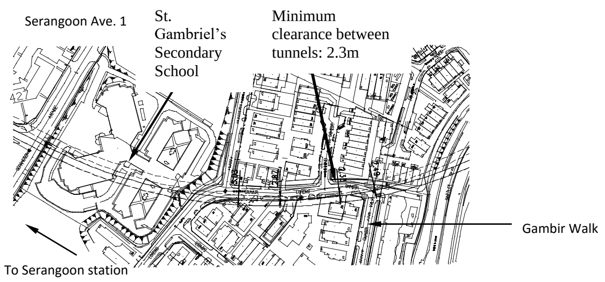
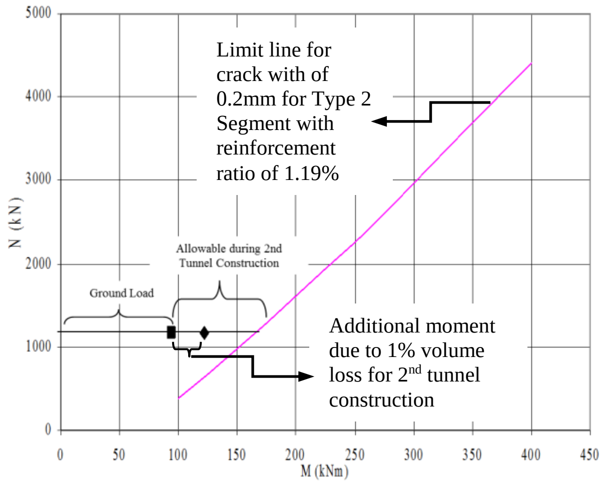


Figure 9 Tunnels at close proximity at Gambir Walk

During construction, additional measures were taken to minimise the risk of excessive volume loss when the second tunnel was driven. The Earth Pressure Balance Machine was equipped with a double screw conveyor to control soil and water flow during removal of spoils from the cutter head chamber. The addition of the second screw increased the length of the screw conveyor by 50% providing a better pressure gradient reduction along its length from the chamber to the discharge gate. The rotation speeds of the augers inside the conveyors were independent of each other in order to better control the discharge of spoil. Permanent glass fibre reinforced polymer (GFRP) dowels were installed through the segmental lining of the first tunnel to strengthen the soil pillar between the twin tunnels to ensure the stability of the pillar in the event of excessive volume loss. This measure was controversial in terms of its real need. It was recognised that the drilling of the segments for the dowel installation would potentially have long term durability implications as the drilling locations will become the weak points of water seepage even though subsequent grouting had been carried out after the passing of the second tunnel. Passive supports specified in the construction contract were also erected within the first tunnel, see Figure 11(a) and (b). These measures were to ensure that the soil pillar between the twin tunnels would still be stable and that the tunnel lining would be supported by the pillar should excessive ground loss occur in the construction of the second tunnel.



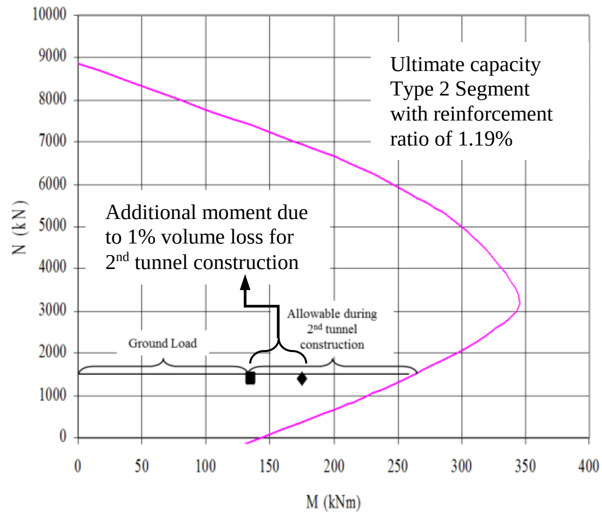


Figure 10 M-N interaction diagram for serviceability limit state (0.2mm crack width) and ultimate limit state

During construction, the volume loss for the second tunnel was controlled to be 0.6 to 0.9%. The measured diametrical distortion of the first tunnel lining was a maximum of 5mm, well within the alert level and work suspension level of 9mm and 13mm, respectively. As a result the passive supports were not loaded by the segmental lining during the second tunnel construction.

While these additional measures have generally been considered necessary when tunnels are to be driven relatively close to each other, these measures are considered as temporary to ensure that the impact of the second tunnel construction on the lining of the first tunnel is minimised and that the stability of the lining of the first tunnel is not compromised. It is however argued that the design should also check the permanent condition of the soil pillar between the two tunnels after the installation of the linings for both tunnels.

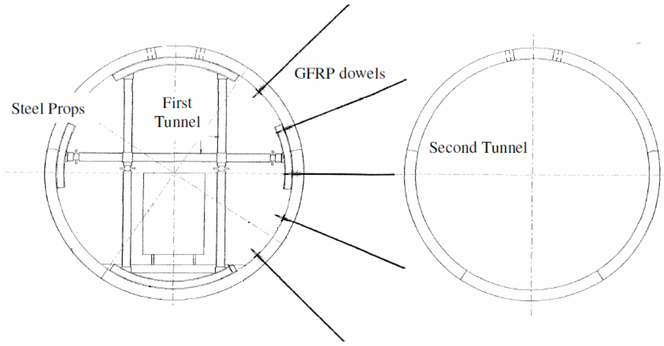


Figure 11 (a) Scheme of passive support and GFRP dowels for tunnels in close proximity (Lim, et al 2008)

Figure 11 (b) Installed passive support in the first tunnel and GFRP dowels installed through the segments of the first tunnel (Osborne, et al 2008)

Figure 11 Passive supports and GFRP dowels for tunnelling in close proximity in CCL Stage 3 projects

The soil pillar between the two tunnels will be subjected to stress concentration, as shown in Figure 12. A design check would be necessary to verify that the soil pillar is not fully yielded and that soil resistance can still be developed in the soil pillar to effectively brace the segmental lining and prevent the lining from being distorted excessively under the overburden pressure. This is believed to be extremely important as the stability of the segmental lining relies on a continuous compressive pressure all round the ring. If there is any loss of the compressive pressure locally, the segmental lining will be subjected to excessive distortion and the stability of the ring can be compromised due to the failure or yielding of the soil pillar.

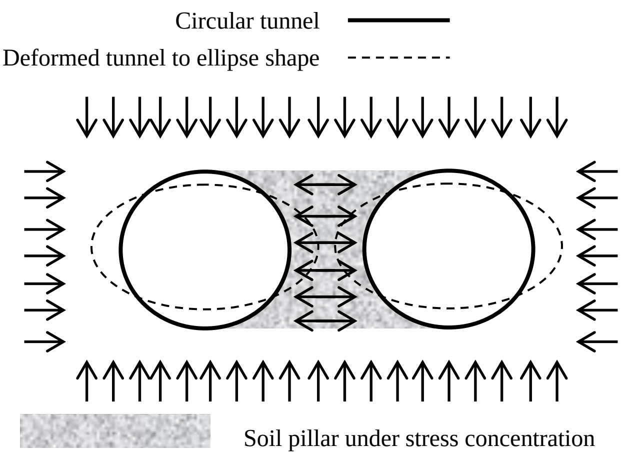


Figure 12 Stress concentrations at soil pillar between twin tunnels

4. DURABILITY AND FIRE RESISTANCE

Similar durability requirements have been applied to the segments through the MRT construction at different phases, although some of them have been further enhanced as construction and material technologies advance. In the Phase I/II MRT projects, coal-tar epoxy coating was applied to the rear and side faces of all segments prior to delivery to the construction site, Hulme & Butchell (1992). It was contemplated that the reinforcement bars should also be coated by epoxy. However for pre-coated bars there were concerns that possible damages to the coating during cage fabrication and handling or the workmanship of the coating process would create pinholes or holidays in the coating. Thus the coating of bars has not been adopted for MRT tunnel construction. The practice of coating the segment extrados has thus continued for the subsequent projects to enhance corrosion protection of the segments. The current MRT projects require the segments to be coated with solvent free or water based emulsion epoxy on all outer faces that contain steel bar reinforcement, together with all side faces, gasket recesses, caulking grooves and insides of bolt holes and grout holes. In addition, other durability measures include the use of a concrete of the grade of 60 N/mm^2 with silica fume and cement with slag or pfa in the mix design. The concrete has to satisfy the criteria of an average of 700 coulombs charge or better and not to exceed 1000 coulombs under rapid chloride diffusion test on the concrete ability to resist chloride ion penetration. Concrete cover is also required to be a minimum of 40mm. To meet this requirement and to facilitate the drilling of the segments for cable or other equipment installation pre-defined drilling locations are indicated on segments as shown in Figure 13 for all CCL and subsequent MRT bored tunnel linings. The adoption of the predefined drilling locations is a major improvement over the practice in NEL projects. Provision of possible future application of cathodic protection is also made by specifying that every steel bar in the steel cage in the segments be spot welded at least at two points along its length to an adjacent bar to achieve electrical continuity.

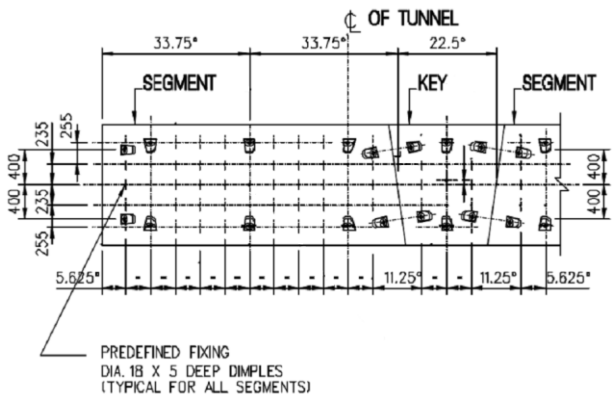


Figure 13 Pre-defined drilling locations (dimples) on the precast reinforced concrete segments since CCL MRT projects

The fire resistance of the MRT tunnels stipulated by statutory requirements is that the tunnels have to be designed to have a 4-hour fire rating. This requirement has been met by detailing the reinforcement in compliance with the rule of the Singapore Standard CP65: Code of Practice for Structural Use of Concrete for short columns. In doing so the linings are deemed to have a 4-hour fire rating. This means that the segment must be at least 240mm thick and that the reinforcement bars must satisfy the following requirements:

- When part or all of the main reinforcement is required to resist compression, links or ties at least one quarter the size of the largest compression bar or 6mm, whichever is the greater, should be provided at a maximum spacing of twelve times the size of the smallest compression bar.

- Every corner bar and each alternate bar (or pair or bundle) in an outer layer of reinforcement should be supported by a link passing around the bar and having an included angle of not more than 135°. No bar within a compression zone should be further than 150mm from a restrained bar.

While all lining thickness is generally 250mm or thicker, the key issue is the numbers of links that must be provided to satisfy the detailing rule to achieve a 4-hour fire rating.

The need for closely spaced links in a short column is for the links to provide confinement to the main steel bars to prevent them from buckling under compression. Tunnel segments are fully braced by the ground along the extrados and the main reinforcement bars would not buckle into the tunnel due to their circular shape. It is therefore believed that the link requirements can be relaxed without affecting the structural capacity of the tunnel segments even though they are designed as short columns under the hoop thrust and the bending moment. However if the rule is relaxed, the reinforcement detailing does not satisfy the CP65 requirements; and therefore the 4-hour fire rating is not deemed to have been met. As tunnel segments are cast using high-strength low-permeability concrete in view of the durability requirements, when exposed to fire, they are more likely to exhibit explosive spalling due to build-up of steam pressure inside the segment body. As concrete is a good insulator, if the spalled concrete can be held in place by closely-spaced links, it helps to protect the concrete behind it. If the link spacing is increased to more than the minimum as specified in CP65 for short columns, one recognized way of minimizing the spalling is by providing a layer of steel mesh in the cover concrete according to Clause 4.1.7 of the Singapore Standard CP65: Part2: 1996.

A fire test was conducted prior to the construction of the CCL Stage 1 bored tunnels on slab and partial segment specimens to demonstrate that the provision of a layer of mesh with reduced number of links would achieve the required 4-hour fire rating. The control specimens were cast with link spacing complying with CP65 rules for short columns. Other specimens were cast with links at double the spacing and with a layer of wire mesh of 50x50x3mm tied to the main reinforcement with a concrete cover of 40mm, see Figure 14. Figure 15 shows the partial segment specimen under test.

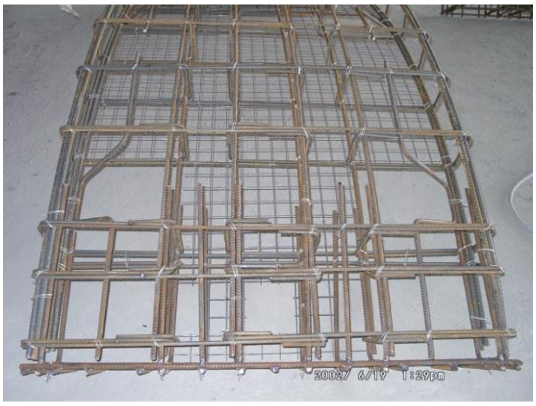


Figure 14 Steel cages with links spaced at 300mm and a wire mesh at the intrados of the segment

The test specimens were subjected to a two-hour fire based on the standard fire curve adopted from BS 476-20: 1987. The temperature rose to about 700°C within 10 minutes after the test commenced and to a maximum of 1050°C at the end of the two-hour testing period. The fire curve is shown in Figure 16.

The fire test confirmed the mechanism of the spalling of concrete cover when the specimens were subjected to fire. Traces of water appeared and cracks began to develop on all sides of the specimen about 10 minutes after the commencement of each test. Concrete spalling accompanied by noise of explosion occurred about 15 minutes after the commencement of each test. The explosive spalling lasted for about 15 minutes. No noise of explosion could be heard afterwards till the end of the test. This could be due to the fact that the escape routes for the steams had already been established through the cracks. There were no further pressure being built up inside the concrete; thus spalling had stopped. During concrete spalling water flowed at a more distinct rate and cracks widened and propagated. Water continued to flow and steam was observed until the end of each test.

Figure 15 Fire testing of a partial segment specimen with extrados exposed and intrados facing the furnace under fire

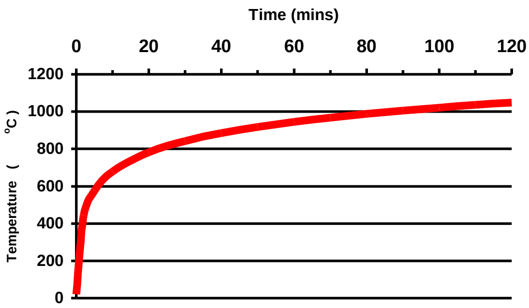


Figure 16 Fire curve specified in BS476-20:1987

The specimens were inspected after the tests and the concrete spalling depths measured. Figure 17 shows the intrados of the partial segment specimen. For all specimens with wire mesh, it is evident that the concrete spalling did not progress beyond the mesh, which proved that the use of mesh can fulfil the function of retaining spalled concrete and control the depth of spalling. The use of mesh with double link spacing from 150mm to 300mm will be able to achieve the 4-hour fire rating for the tunnel linings. As a result of the tests one contract in the CCL Stage 1 project has adopted the use of wire mesh with link spacing of 300mm instead of the usual link spacing of 150mm.

Figure 17 Concrete spalling up to the depth of wire mesh after fire test for the segment specimen

5. WATERPROOFING

The Materials and Workmanship Specification for the Phase I/II of the MRT projects specified that the completed bored tunnels should achieve an overall watertightness of 5 ml/m^2/hour or 10 ml/m^2/hour over any 10 m length. Hulme and Burchell (1992) reported that the specification had been achieved although certain contractors had found difficulty in meeting the requirements. The gaskets used for meeting the requirements in Phase I/II MRT bored tunnels varied from simple rectangular section butyl rubber to water-swelling hydrophilic rubber sealing strips fixed to the segment trailing edge and on cross joint face. Two contracts used neoprene gaskets fitted into grooves around complete outer edge of the segments. It was reported that the water-swelling hydrophilic rubber sealing strips produced significantly better results than all of the other systems. Re-grouting using cement grout and the use of polyurethane grout at specific leaking points were carried out on completion of the lining construction. It was reported by Copsey & Doran (1987) that butyl rubber gaskets behaved in a totally plastic manner and once compressed the gaskets were unable to recover their original shape when the compression was reduced when TBM shield jacks were withdrawn. Joint packings, if used to correct alignment made the situation worse. Extensive grouting programme of either back grouting of cement/water mix into the grout holes and joints, or injection of polyurethane foams into the void between the defective sealing strips and the caulking were carried out by the contractors using butyl rubber sealing strips. Figure 18 shows one leaking point of the tunnel lining with butyl rubber as sealing strips after more than 10 years of its construction. Copsey & Doran (1987) also reported numerous problems in achieving an effective seal at the corners of the segments, in particular around the keys as the protruding face of the sealing strips was susceptible to damage or misalignment during ring building as the key was pushed into place.

Figure 18 Leakages from tunnel segments with butyl rubber sealing strips after more than 10 years of its construction

From NEL projects the overall watertightness has been reduced from those required in the Phase I/II MRT projects. The Materials and Workmanship Specification required a high standard of waterproofing of bored tunnels. The ground water leakage rate is now limited to 2 ml/m^2/hour or 5 ml/m^2/hour over any 10 m length. The NEL Civil Design Criteria specified a double protection system for both hydrophilic sealing strips and EPDM (Ethylene Propylene Diene Monomer) gaskets to be employed for all joints. Typically the hydrophilic strips were installed towards the extrados followed by an EPDM gaskets fitted into a groove, Figure 19, except one contractor who opted to fit the hydrophilic strip close to the intrados of the segments.

The gasket system used in the NEL projects were more effective than those used in Phase I/II MRT works, Shirlaw et al (2006). Even so there was still some significant leakage that had to be grouted using polyurethane. The tunnels were believed to have brought within the leakage tolerance, despite the tighter criteria than the Phase I/II MRT construction. However it is believed that the grouting generally used foam type polyurethane, which is good for sealing water flow under pressure. This type of polyurethane will expand in volume and form open-cell structure when meeting with water, see Figure 20. The open-cell foam is not durable. To have long lasting effects of sealing leakage, a low viscosity grout that forms a flexible membrane, such as the low viscosity flexible polyurethane grout or acrylic gel should be used after water flow under pressure has been stopped by the foam polyurethane.

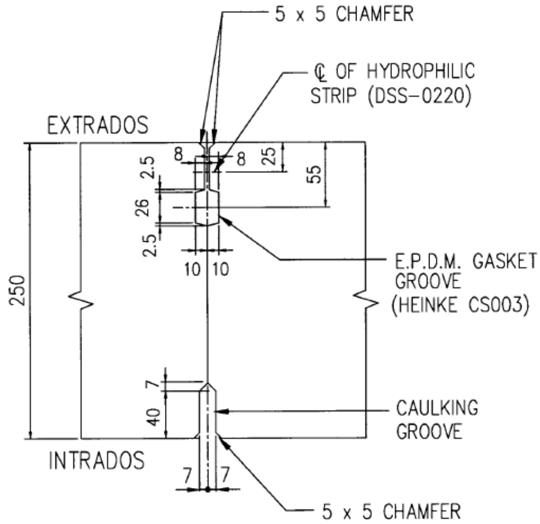


Figure 19 Typical gasket arrangements with hydrophilic strips and EPDM gaskets at extrados

Figure 20 Water-reactive polyurethane grout forming open-cell foam (left) and flexible grout forming a membrane (right)

When CCL Stage 1 was implemented LTA decided to adopt the use of composite gaskets for the bored tunnels. When used separately as in the NEL projects, EPDM and hydrophilic gaskets have their own advantages and disadvantages, Doran, et al (1999). The advantages of EPDM gaskets are that they can seal large gaps and are durable in maintaining a high compression over the design life of the tunnels, which is an important property as the performance of butyl rubber sealing strips have shown in the tunnels constructed in the late 1980s. The main disadvantage is that they have limited capacity to accommodate misalignment and the corner of segments is vulnerable in particular around tapered keys. On the other hand hydrophilic sealing strips are capable of swelling to seal off small misalignments; but they have limited ability to maintain the pre-compression forces. When both are combined in a composite gasket of an EPDM carrier with a hydrophilic face each will be able to complement the other in making a seal not only for the short term but also in the long term. The composite gasket will also ensure the pre-compression of the hydrophilic materials is achieved to enhance its sealing capacity and durability. Test data (Doran, 1997) in Figure 21 shows that the sealing capacity of hydrophilic materials can be increased substantially with pre-compression.

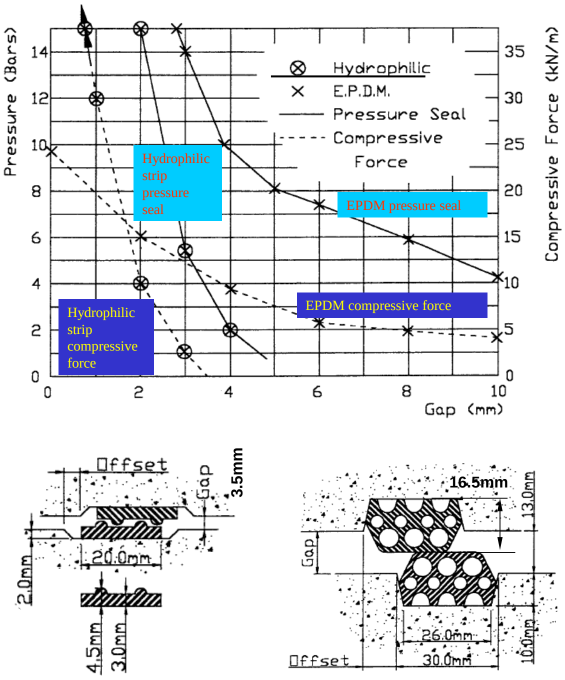


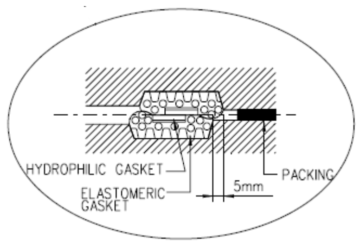
Figure 21 Sealing capacities of hydrophilic strip / EPDM vs. compression of strip, after Doran (1997)

The CCL Stage 1 contractors proposed to use a co-extruded composite gasket – a 1-mm thick hydrophilic facing material on an EPDM carrier, see Figure 22. The gasket also had vulcanized corners with internally located cavities to form a "soft corner" to prevent concrete spalling in the event of excessive volume or distortion during segment installation. To comply with the new requirement in the Materials and Workmanship Specification, the performance test for the proposed gasket was carried out in a Singapore accredited laboratory to demonstrate its sealing capacity of at least 2 times the expected water pressure for the tunnels with the expected tolerances of gaps and steps in the ring building. Since its use in CCL, composite gaskets have been specified for all subsequent MRT tunnel tunnels.

Based on the observation of the completed CCL bored tunnels the watertightness was a substantial improvement over those built in the NEL. Ong et al (2007) reported that for two contracts, i.e. C825 and C822 there was no serious leak detected in the radial and circumferential joints of the segmental tunnel linings. Only 0.28% of the joints had minor leaks that could be easily sealed.

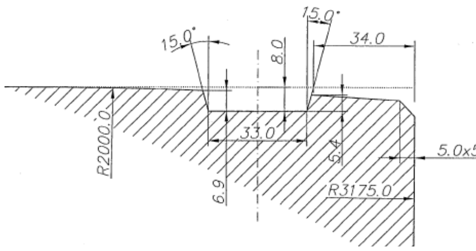
6. STEEL FIBRE REINFORCED CONCRETE SEGMNTS

There has been a trend in the tunnel construction industry to adopt the use of steel fibre reinforced concrete (SFRC) segments. Prior to the implementation of the NEL, LTA carried out a trial of the use of SFRC segments in NEL. A total of 24 segments were cast with fibre contents of 40 kg/m^3 and 70 kg/m^3 and tested to failure, Doran (1999). Although some NEL contractors explored the use of SFRC segments, none actually used them because there was no national design codes of practice for such structures and the building authorities would not accept and approve such design. During the CCL Stage 1 construction in Contract 825 a stretch of temporary bored tunnels from the launch shaft to the start of permanent bored tunnels were lined with SFRC segments. As the lining was temporary, the general arrangement for the SFRC segments made no adjustment by using the permanent segment moulds, i.e. five segments plus one key segment. Observation on site found that there were many cracks developed in the segment body, probably due to segment handling, transportation and erection process. This shows that for SFRC segmental linings smaller segments are necessary to limit the aspect ratio to prevent cracking of the segments due to handling, transportation and erection into the permanent position. Similar experience was also observed in the UK in the use of SFRC segments. The aspect ratio, i.e. ratio of length on the centre line of the segment to the segment thickness is commonly used as a measure of the susceptibility to handling damage to segments. It was reported by CRL project in UK (CRTL Technical Report, 1997) that aspect ratios of up to 7.3 for unreinforced concrete segments provide virtually no risk of segment breakage before erection and ratios of above 11.5 would provide an unacceptable amount of cracking during handling. The temporary SFRC segments used in the C825 temporary tunnel would have an aspect ratio of slightly more than 13. The site observation indeed showed that the cracking would be unacceptable for use as permanent segmental lining.



(a) Indicative gasket details on tender drawings

(b) Proposed co-extruded composite gasket in CCL



(c) Gasket groove for the composite gasket in CCL

Figure 22 Composite gaskets used in CCL projects

Since NEL and CCL projects there have been more international experiences in the use of SFRC segments. There are many advantages of using SFRC segments, including ease of casting and repair, less damage and cost saving. The most attractive benefit is the best durability that SFRC segments can offer. Due to their small size and discreet nature in the concrete mass, the corrosion of fibres in the concrete mass will not generate the spalling of concrete as observed in steel bar reinforced segments. Prior to the implementation of the DTL Stage 3 projects LTA carried out further studies jointly with the National University of Singapore (NUS) and the Nanyang Technological University (NTU) of the properties of steel fibre reinforced concrete. In addition to the conventional cube strength tests and tensile splitting tests on cylinder samples, SFRC beams were tested to verify the ductility (toughness) and post cracking behaviour. Full scale joint tests and segment tests were also carried out to verify the splitting tensile strength of joints under hoop thrust and the bending capacity of segments. The full scale joint tests (Figure 23) verified the adequacy of the tensile splitting strength of segment joints under the hoop thrust due to permanent loading from overburden and surcharging or the jacking forces on the circumferential joint faces. The full scale segment tests (Figure 24) verified the bending capacity of the segments to resist the self- weight during handling.

Figure 23 Full scale tests of segment joints under concentrated compression loading to simulate the hoop trust at segment joints

Figure 24 Full scale segment test to verify moment capacity of SFRC segments

The building authorities in Singapore were consulted and it was agreed that the design of SFRC segments should be based on unreinforced concrete section in accordance with the current codes of practices and full scale tests must be carried out to verify the suitability of the design assumptions. Other requirements from the building authorities include the design consideration of the full load cycle of the structures, i.e. both temporary loading and permanent loading cases and the establishment of a rigorous quality control, inspection and maintenance for the SFRC segments. With the agreement with the building authorities, one contract in DTL3 saw the first use of permanent SFRC segmental tunnel lining for about 2350m of twin bored tunnels in Singapore. The design was carried out based on the properties established in the past joint studies with NUS and NTU. These properties were also verified as part of the quality control tests during production of the segments. The dosage of steel fibre is 40 kg/m^3.

In order to reduce the risk of cracking during handling and transportation as noted in the temporary SFRC segmental lining in C825 of the CCL, the aspect ratio of the segments was limited such that there were 7 segments plus 1 key, see Figure 25. The segment width has been kept to 1.4m with a thickness of 275mm. The aspect ratio is 9.46. No segment damage during handling has been reported with this aspect ratio.

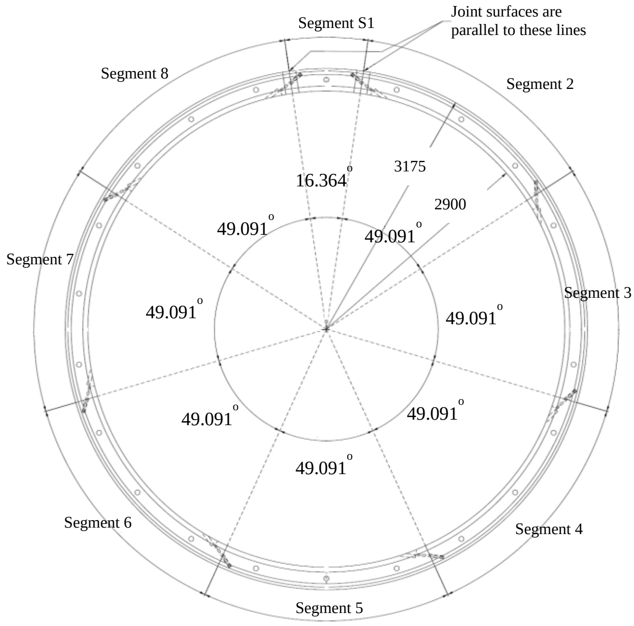


Figure 25 General arrangements of segments for DTL3 SFRC segmental lining

7. CONCLUSIONS

The design development of the reinforced concrete segmental lining for the bored tunnels in the MRT construction in Singapore has been described in this paper. The design has evolved together with the advance in material and construction technologies. The ultimate aim of the development is to have segmental linings for the bored tunnels that are structurally adequate, easy to construct, durable and cost efficient in operation and maintenance.

8. REFERENCES

British Standard BS476 – Part 20 (1987) Methods for determination of the fire resistance of elements of construction (general principles). British Standard Institutions.

Copsey, J. P. and Doran, S. R. (1987) “Singapore Mass Rapid Transit System design of the precast concrete segmental tunnel linings”. Proceedings of the Singapore Mass Rapid Transit Conference, 1987. pp225-238.

CRTL Technical Report (1997). “Steel fibre reinforced concrete for tunnel linings”. CRTL Technical Report No. 000-RUG-RLEEX-0008-AA. RLE Rail Link Engineering, Oct 1997.

Curtis, D. J. (1976) “Discussion”. Geotechnique 26, pp231 – 237.

Doran, S. R. (1997) “An introduction to the review of tunnel lining designs”. Course Note of Internal Training for Staff of Land Transport Authority, September 1997.

Doran, S. R., Poh, S. T. and Copsey, P. (1999) “Developments in precast tunnel lining design for the Singapore M.R.T.” Proceedings of International Conference on Rail Transit, 1-13 March 1999.

Duddeck, H. and Erdmann, J. (1982) “Structural design models for tunnels”. Tunnelling’82, International Symposium organized by Institution of Mining and Metallurgy.

Hulme, T. W. and Burchell, A. J. (1992) “Bored tunnelling for Singapore metro”. Journal of Construction Engineering and Management, ASCE, Vol. 118, No. 2, June 1992. pp363-384

Land Transport Authority (LTA). “Civil Design Criteria for Roads and Rail Transit System”. https://www.1ta.gov.sg/content/dam/1taweb/corp/Industry/files/DC_EGD09106A1_Overall.pdf

Land Transport Authority (LTA). “Materials and Workmanship Specification for Civil and Structural works”. https://www.lta.gov.sg/content/dam/ltaweb/corp/Industry/files/EGD09104A1-Overall.pdf

Lim. K. K., Lim, T. F., Tan, W. M. and Wong, Y. K. (2008) “Singapore Circle Line 3 – the challenges of tunneling in close proximity”. Proceedings of International Conference on Deep Excavations, Singapore, 2008.

Muir Wood, A. M. (1975) “The circular tunnel in elastic ground”. Geotechnique 25, No. 1, pp115 – 127.

Ong, J. C. W., Wen, D., Osborne, Chiang, C. C. and Shirlaw, N. J. (2007) “Waterproofing bored tunnels’. Proceedings of Underground Singapore 2007, 29-30 November 2007, Singapore.

Osborne, N., Knight Hassel, C., Tan, L. C. & Wong, R. (2008) “A review of performance of tunneling for Singapore’s circle line projects”. World Tunnelling Congress 2008, India.

Shirlaw, J. N., Wen, D. and Ong, J. C. W. (2006) “The importance of the choice of gasket type and grout mix in segmental tunnel linings”. Proceedings of the International Conference and Exhibition on Tunnelling and Trenchless Technology, 7-9 March 2006, Subang Jaya, Malaysia.

Singapore Standard CP65: Part 1 and Part 2 Code of Practice for Structural Use of Concrete, published by Singapore Productivity and Standards Board.

Wen, D. & Ong, J. (2003) “A case study of distortional load factor for segmental tunnel lining in Old Alluvium”. Proceedings of the RTS Conference, Singapore, 2003.

Wen, D., Poh, J. and Ng, Y. W. (2004) “Design considerations of bored tunnels in close proximity”. Proceedings of the 30th ITA-AITES World Tunnel Congress, Singapore, 22-27 May 2004.