

# Design Practice On Tunnels & Tunnelling in Singapore — Current State & Future Trends

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**ABSTRACT:** Reinforced concrete segments have been used from 1980s till today for the Singapore Mass Rapid Transit (MRT) bored tunnels constructed by tunnel boring machines (TBM) as permanent linings. This paper describes very briefly the typical general arrangement of the segmental linings used for the MRT bored tunnels including those used in the 1980s to the current design for the MRT lines under construction for both steel reinforced segments and the steel fibre reinforced concrete (SFRC) segments. The topics include the dimensions and segment arrangement, joints in segmental linings and bolt connections. Details of SFRC segments that are used in the recent MRT lines are given in the paper. These include the benefits of adopting the SFRC segments and the key design parameters for the SFRC segments. Ground movement predictions and its impact on buildings are discussed in detail. Key parameters for predicting ground movement due to tunneling are presented. A framework for performing the building assessment of the impact of tunnelling induced ground movements on existing structures is given in the paper with detailed description of the effects of building stiffness on the ground movements and how these effects can be taken into account in the assessment. Recent experiences in the approach in considering the conditions of the existing structures in the building assessments are shared in the paper. Finally the application of data analytics in tunnelling and its current status are presented.

## 1 INTRODUCTION

The Singapore MRT construction has widely used bored tunnels by tunnel boring machines (TBM) due to their minimal impact on the surface structures and utility services. Where possible the use of bored tunnels has been maximized except for the construction of stations, cross-overs or cripple-sidings where TBM bored tunnels were not practical without substantial ground treatment. Both approaches of design-and-build and client's own design (either designed by the LTA's in-house design team or by the LTA's consultants) have been adopted for the various stages of the MRT construction for the precast reinforced concrete (RC) segmental linings for the bored tunnels. The general arrangement and structural design have been described by Wen 2016.

## 2 GERRAL ARRANGEMENT OF PRECAST REINFORCED CONCRETE SEGMENTAL LININGS

### *2.1 Dimensions and segment arrangement*

For the existing East-West and North-South Lines (Compass Lines) the nominal diameter of the bored tunnel was set to be 5.2m (including a minimum construction tolerance of 100mm on radius) based on the structural gauge of the trains and no walkways were built in the tunnels. The internal diameters of the tunnels were determined by the design-and-build contractors to be in the range of 5.23m to 5.4m with additional construction tolerance on top of the already allowed for in the minimum diameter set by the client (Copsey & Doran, 1987). Since the North East Line (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. To allow for a standard construction tolerance of 100mm all round the tunnel the MRT bored tunnels have been built with an internal diameter of 5.8m since the NEL projects.

The thickness of the tunnel lining in the Compass Lines 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 tunnels under the Fort Canning Park. For the Circle Line (CCL) and the recent projects the thickness of the linings for the typical MRT tunnels of 6m in diameter appears to be set at a standard 275mm. Segment width has increased over the years from 1m in the 1980s to typically 1.2m for the NEL and now at 1.4m for CCL and the recent MRT lines. Potentially the segment width can be further increased based on the trend seen in projects in Europe, in particular for bored tunnels that are larger than 6m in internal diameter.

The RC segmental linings in the Compass Line have generally adopted the arrangement of five segments plus one key, except for two contractors. 40mm taper was typically used to negotiate the tunnel alignment curves. A typical section is shown in Figure 1 for Contract 301 tunnel lining with one segment replaced by a steel one at the tunnel sump location. The subsequent MRT projects have generally followed the same principles, i.e. for the steel reinforced concrete segments the ring is arranged with 5 segments plus one key. The key is tapered on plan, with the radial joint faces also tapered towards the centre of the tunnel, which is a preferred arrangement as it would provide stability to the ring once all segments and the key are installed in place. Figure 2 shows a typical segment ring used for the CCL projects. The same arrangements have been followed by the subsequent MRT construction till today. Waterproofing gaskets for the segmental linings have evolved together with the advance in technology and these have been described by Wen, 2011.

Steel Fibre Reinforced Concrete (SFRC) segment lining was used for permanent lining in one of the Downtown Line 3 (DTL3) contracts. Since its adoption in DTL3, SFRC segmental lining has been adopted for more projects in the subsequent MRT

construction. To mitigate the risk of segment cracking during handling and erection, the general arrangement of the SFRC segmental lining adopted for DTL3 is 7 segments plus the key. The details are shown in Figure 3.

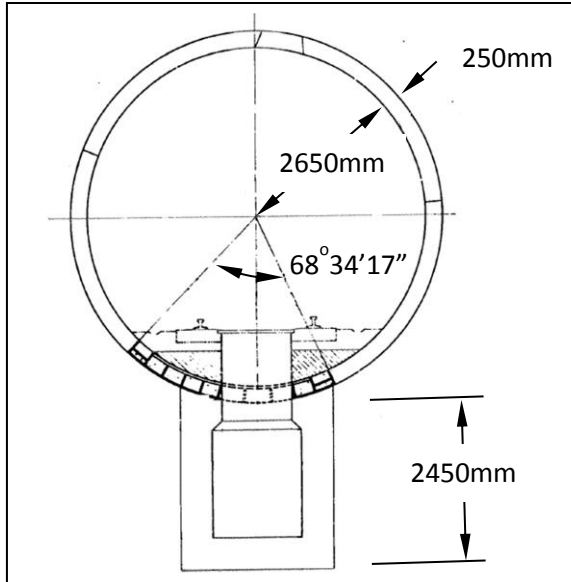


Figure 1. Typical segment general arrangement for Contract 310

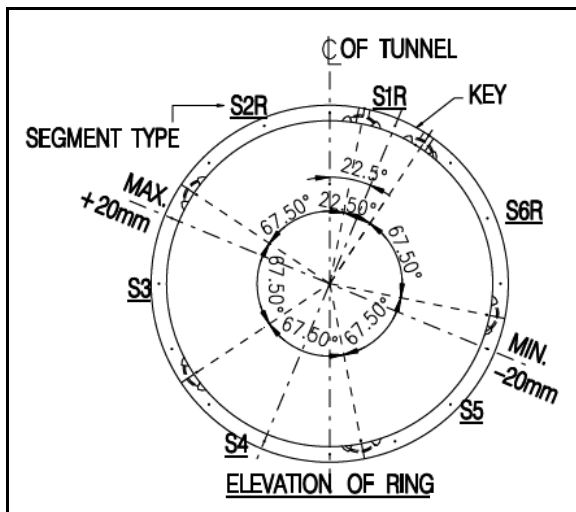


Figure 2. Typical general arrangement of segmental lining for CCL bored tunnels

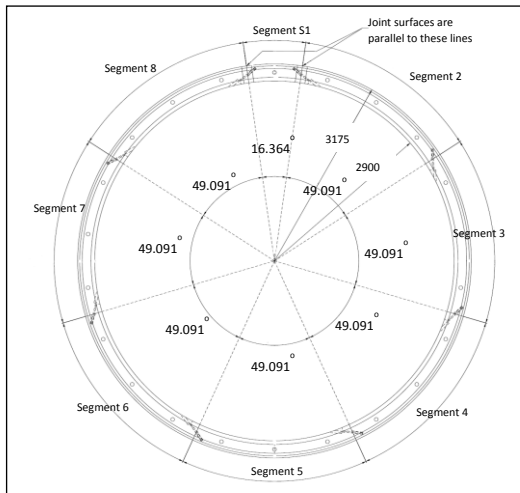


Figure 3. General arrangement of DTL3 SFRC segmental lining

## 2.2 Joints of segmental linings

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 is well defined; and the joint allows rotation and distribution of the contact stress. 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. A double convex joint however attracts higher bursting stress due to the smaller contact area. The standard solution for Hertz contact stress (an example is Roark's Formulas for Stress & Strain) can be used to evaluate the contact stress (and therefore bursting stress) for the double convex joints.

In contrast, a block joint is more susceptible to cracking and with risk of cracks or chips at corners if there are workmanship issues during segment erection. The advantage is that the larger contact area of a block joint results in lower bursting stress at the joints. Consequently, with the improvement in tunnel erection technique in modern day TBMs, block joints are being used for tunnel linings where bursting capacity are lower than the bursting stresses, such as in the steel fibre reinforced concrete segments. Figure 4 shows the schematic of both joints.

## 2.3 Bolt connections

The design of tunnel segments have to make allowance for bolt connections between segments at circumferential and radial joints, so that each precast segment is bolted to the adjacent segments (radially) and rings (circumferentially) to form the tunnel lining. In Singapore, the bolts are usually permanent and tightened to the fullest

extent necessary to ensure competent building of the lining and proper contact between faces of gaskets. A good practice would be to re-tighten the bolts on a ring after erection when the lining passes through the tail seal brushes.

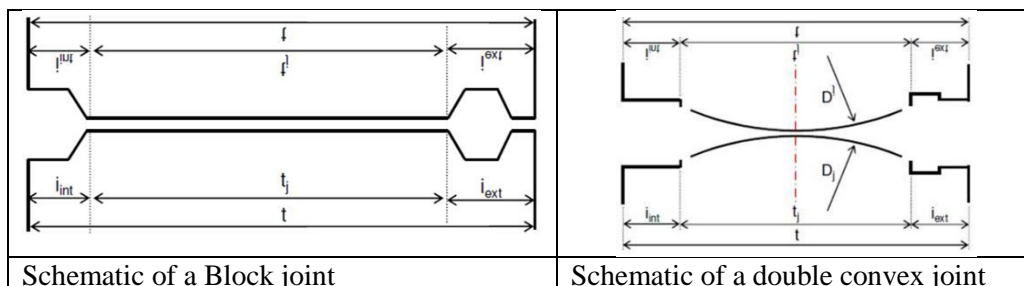


Figure 4. Schematic of block joint and double convex joint

During the earlier phases of MRT development in Singapore, such as some of the North-East Line projects and Circle Line Stages 1 to 5 projects, curved bolts were typically used for erecting the tunnel segments. Curve bolts give more flexibility during segment erection as there is some clearance between the bolt and bolt pocket and therefore have greater tolerance on the alignment of segments and bolt pockets. Typically a M24 bolt (galvanized or sherardized) is pushed through a cast-in plastic sheath ranging from 28mm to 36mm diameter across various projects in NEL, CCL and DTL, and bolted at both ends. The offset between the bolt and the plastic sheath is necessary to allow practical erection of segments – a tighter offset would make ring erection more difficult whilst a bigger offset would affect quality of the ring build in terms of greater steps and lips.

In recent years, there is a trend of moving towards straight or spear bolts as observed in some DTL projects and all the TEL projects. In contrast to curved bolts which require fastening at both ends, spear bolt requires finishing only on one end of the bolt as the end socket is already cast into the other segment. It is faster and easier to install provided that the segments are properly aligned and the tighter control on bolt pocket alignment arguably encourages better workmanship during segment erection. However, the tighter tolerance against misalignment of bolt pockets may be problematic if the segments are not set properly during ring build. Figure 5 shows the schematic of a curved bolt and of a spear bolt connection.

Other than steel bolt connections, there is a new development in the use of dowel connections at circumferential joints and the use of guiding rods in radial joints. These are plastic dowels placed into the circumferential dowel holes just before the installation of each segment, and offer advantages in terms of faster ring build, tighter control on ring build and improved durability due to greater clearance to reinforcement and elimination of steel bolts. The major concern in using such push-

fit plastic dowels is that repairs would tend to be more extensive as adjacent rings would be affected if there ever is a need to remove erected rings that are defective.

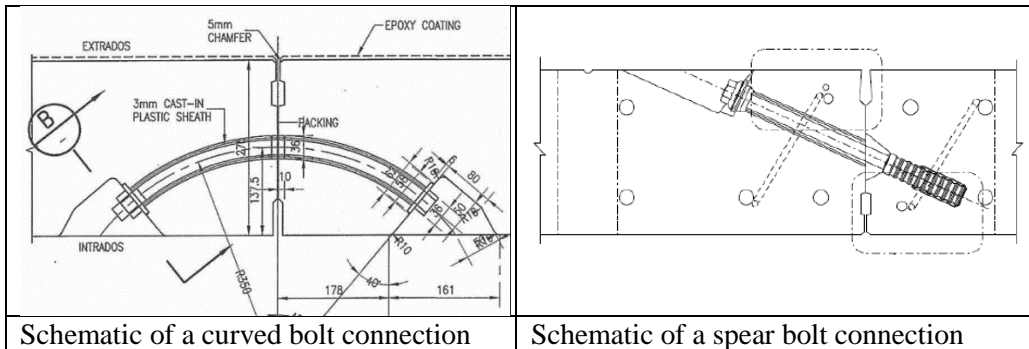


Figure 5. Schematic of a curved bolt connection and a spear bolt connection

### 3 STEEL FIBRE REINFORCED CONCRETE SEGMENTS

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<sup>3</sup> and 70 kg/m<sup>3</sup> and tested to failure (Doran et al., 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. Based on the

empirical data from various projects around the world, ITA (2016) suggests that an aspect ratio of less than 10 would be suitable for segments made of SFRC alone.

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 key benefit is in enhancing durability of concrete segments. Potential corrosion propagation that is seen in reinforced concrete is minimized for SFRC as steel fibres are discontinuous and discrete and the corrosion of fibres in the concrete mass will not generate the spalling of concrete as observed in steel bar reinforced segments. Fibres are also effective in preventing micro-cracking and mitigating related cracks-induced deterioration such as corrosion and chemical attack that is observed in reinforced concrete. In terms of minimizing damage, steel fibres are uniformly distributed in the concrete matrix resulting in improved resistance to potential minor damages on edges and corners during handling of 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 6) 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 7) verified the bending capacity of the segments to resist the self-weight during handling.

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<sup>3</sup>.



Figure 6. Full scale tests of segment joints under concentrated compression loading to simulate the hoop thrust at segment joints



Figure 7. Full scale segment test to verify moment capacity of SFRC segments

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. 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.

Since then, more projects have started using SFRC. Table 1 below summarises the design parameters and target performance of the SFRC in various LTA tunnel projects for the Downtown Line, Thomson-East Coast Line, Circle Line Stage 6 extension and North East Line extension. As SFRC tunnel linings have lower

bending capacity than RC tunnel linings, hybrid segments have been employed in various projects to cope with the higher bending moments at localised stretches within the SFRC tunnel sectors to take benefit from the enhanced durability nonetheless, such as tunnels with shallow overburden or tunnels that are designed to resist the additional construction loads and distortion from future developments.

Table 1. Performance requirements of Steel Fibre Reinforced Concrete for various MRT tunnel segments in Singapore

	DTL C933	TEL T206	TEL T207	CCL C882 / C885	NEL Ext. C715
Material requirements	Grade C50/60 concrete with 370-400 kg/m <sup>3</sup> cement content; steel fibre dosage of 40kg/m <sup>3</sup> and polypropylene fibre dosage of 1kg/m <sup>3</sup>				
Characteristic tensile splitting strength	4.5MPa	4.5MPa	4.5MPa	4.5MPa	4.5MPa
Characteristic flexural strength (LOP)	4.2MPa	7.0MPa	6.0MPa	6.0MPa	7.0MPa
Residual flexural tensile strength (fr1)	2.8MPa	5.0MPa	5.2MPa	5.2MPa	5.2MPa
Residual flexural tensile strength (fr4)	1.4MPa	2.0MPa	1.7MPa	1.7MPa	2.0MPa

#### 4 ESTIMATE OF GROUND MOVEMENTS INDUCED BY TUNNELLING

Bored tunnelling work will generally produce a settlement trough which is related to a Gaussian distribution curve. The Gaussian settlement trough for bored tunnelling works by O'Reilly and New (1982) is suitable for surface settlement prediction, provided there is no major loss of ground at the face and where there is little or no consolidation settlement. Two parameters are used to define the Gaussian trough – a trough width parameter K which varies between 0.25 and 0.5 around the world, and the volume loss V expressed as a percentage of the excavated tunnel face area. These are largely empirical and based on local experience of tunnelling method and ground conditions. Specifically, the LTA Civil Design Criteria (2010) presents volume loss values for tunnels up to 6.6m excavated diameter that occurred during tunnelling for Phase I & II MRT construction in Singapore. Many of the large volume losses were attributed to the use of open shield tunnelling machines in the early days.

Since then, earth pressure balance machines have been used for tunnelling in Singapore, where sufficient pressure was maintained to minimise movement at the face and tail void grouting was carried out simultaneously to control volume loss. Consequently, separate studies that were conducted to back-analyse the volume loss from settlements observed during the NEL, CCL and DTL tunnel construction – Shirlaw et al (2003) for NEL, Yugami et al. (2009), Zhang et al. (2011) and Thiri (2015) – had also shown that the volume loss has reduced significantly. From these

studies, an update on the design volume loss for estimating tunnelling-induced settlement in various geological conditions in Singapore can be developed, and is as suggested in Table 2.

The horizontal displacements induced by bored tunnelling may be approximated by assuming that the displacement vectors of the ground are directed towards the tunnel axis, and estimated using the surface settlement prediction (Mair and Taylor, 1997). Sub-surface settlements may be approximated using the Gaussian distribution curve and the method proposed by Mair et al (1993).

Table 2. Values of volume loss for bored tunnelling in Singapore

<b>Ground Type</b>	<b>Tunnelling Method</b>	<b>K</b>	<b>V</b>
Jurong Formation (Soil)	Pressure Balance Machine	0.45	1.0
Jurong Formation (Rock)	Pressure Balance Machine	0.45	1.0
	SCL mining method	0.45	1.5
Mixed-face of Jurong Soil & Rock	Pressure Balance Machine	0.45	1.5
Fort Canning Boulder Bed	Pressure Balance Machine	0.45	1.0
	SCL mining method	0.45	1.5
Bukit Timah Granite (Soil)	Pressure Balance Machine	0.45	1.0
Bukit Timah Granite (Rock)	Pressure Balance Machine	0.45	1.0
Mixed-face of Granite Soil & Rock	Pressure Balance Machine	0.45	3.0
Old Alluvium	Pressure Balance Machine	0.45	1.0
Kallang Formation	Pressure Balance Machine	0.5	3.0
Mixed-face of Kallang & Other Formation	Pressure Balance Machine	0.45	3.0

## 5 IMPACT OF TUNNELLING ON ADJACENT BUILDINGS

Tunnelling can cause ground movements that affect buildings within their zone of influence. In Singapore, the approach to assessing the risk of building damage due to underground construction broadly follows the staged approach suggested by Mair et al. (1996) – this is described in local guides such as the LTA Civil Design Criteria (LTA, 2010) and TR26:2010 (Spring, 2010) – and consists of three stages. Stage 1 is the preliminary assessment, where a building within a settlement zone of less than 10 mm and experiencing a slope of less than 1:500 can be considered to have negligible risk of damage. Stage 2 assessment is done using the tensile strain method, and is based on idealising the building as a beam and calculating the maximum tensile strain by using the deflection ratio from the settlement profile and the horizontal strain from the horizontal displacement profile. The induced maximum tensile strain is then checked against the limiting tensile strain corresponding to various building damage categories described by Burland et al.

(1977) and Boscardin and Cording (1989). Buildings assessed to have ‘negligible’, ‘very slight’ and ‘slight’ damage categories are considered to be at low risk of damage and can be eliminated from the assessment at this stage. Stage 3 is the detailed evaluation stage. For buildings assessed to be in a damage category of ‘moderate’ or higher, detailed evaluation is to be undertaken. This would require, although not necessarily be limited to, evaluating the structural details of the building, giving full consideration to the sequence and method of excavation and considering soil–structure interaction effects. The influence of building stiffness could be considered at this stage. Using the modification factor approach, Mair (2013) showed how a new simplified design approach can be used to take account of relative building stiffness, and validated the design approach using centrifuge model tests, finite element analyses and field data of building performance. Figure 8 shows that how field data of buildings influenced by tunnelling fitted into the design guidance developed from numerical analysis. Appendix 1 shows how the relative building stiffness can be derived using one of the buildings in the Pasir Panjang shophouses as an illustration. Building response to tunnelling-induced ground movements can now be predicted with greater certainty. Alternatively, a detailed structural analysis can also be used to review the additional stresses induced in key structural elements under the influence of externally induced distortions and movements.

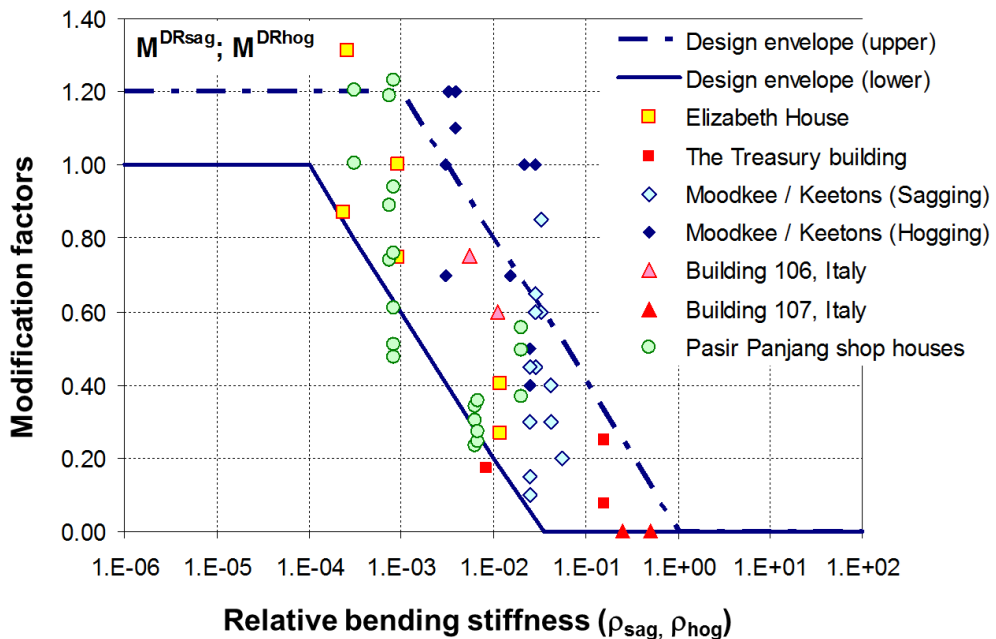


Figure 8. Field data of building response to tunnelling using new definition of relative building stiffness (Mair, 2013)

The risk of damage does not refer solely to structural damage and defects, but also include defects associated with serviceability degradation such as distorted door and window frames, out-of-vertical lifts, building service disruptions, drainage relying

upon slope gradients and the like. If the risk of damage is still unacceptable even after the detailed evaluation (i.e. moderate risk of damage or higher), mitigation and/or protective measures would be provided for the buildings. This would include (1) internal structural measures which refer to actions taken within the excavation during its construction to reduce the ground movements generated at source, such as increasing wall thickness, stiffening strutting supports and modifying the construction sequencing or, in the case of tunnelling, taking measures to reduce volume loss, (2) external structural measures which reduce the impact of ground movements by increasing the capacity of the adjacent structure to resist, modify or accommodate those movements, such as underpinning and structural strengthening, and (3) ground treatment measures which include all methods of reducing or modifying the induced ground movements through improving or changing the engineering response of the ground.

Fok et al (2012) summarized the framework of building impact assessment and provided consideration for sensitive buildings such as national monuments, buildings on mixed foundations and buildings where current condition is poor, see Figure 9. To incorporate the existing heritage and structural condition of the buildings into the impact assessment framework, Goh and Mair (2014) developed a guidance to objectively classify the existing condition of buildings from visual inspections, into three broad categories, see Appendix 2. Buildings in the 'Good condition' category would not have any visible signs of structural defects / distress on the exposed structural elements, and buildings in the 'Satisfactory condition' category would have more significant cracks and material deterioration on the exposed structural elements that would affect the serviceability functions of the building. Buildings in the 'Poor condition' category would exhibit stability and severe serviceability issues, including structural defects that affect the structural integrity and stability of localized areas within the property or even the entire building. Taking best practice from recent CrossRail and Thames Tideway projects in London and considering the experience of visual inspections in Singapore over the past decade, they suggested a 'Risk-based approach' to account for the current building condition including any sensitive structural form or heritage features. Table 3 shows the various sensitivity indices to characterize the existing condition, structural form and heritage impact of such sensitive buildings.

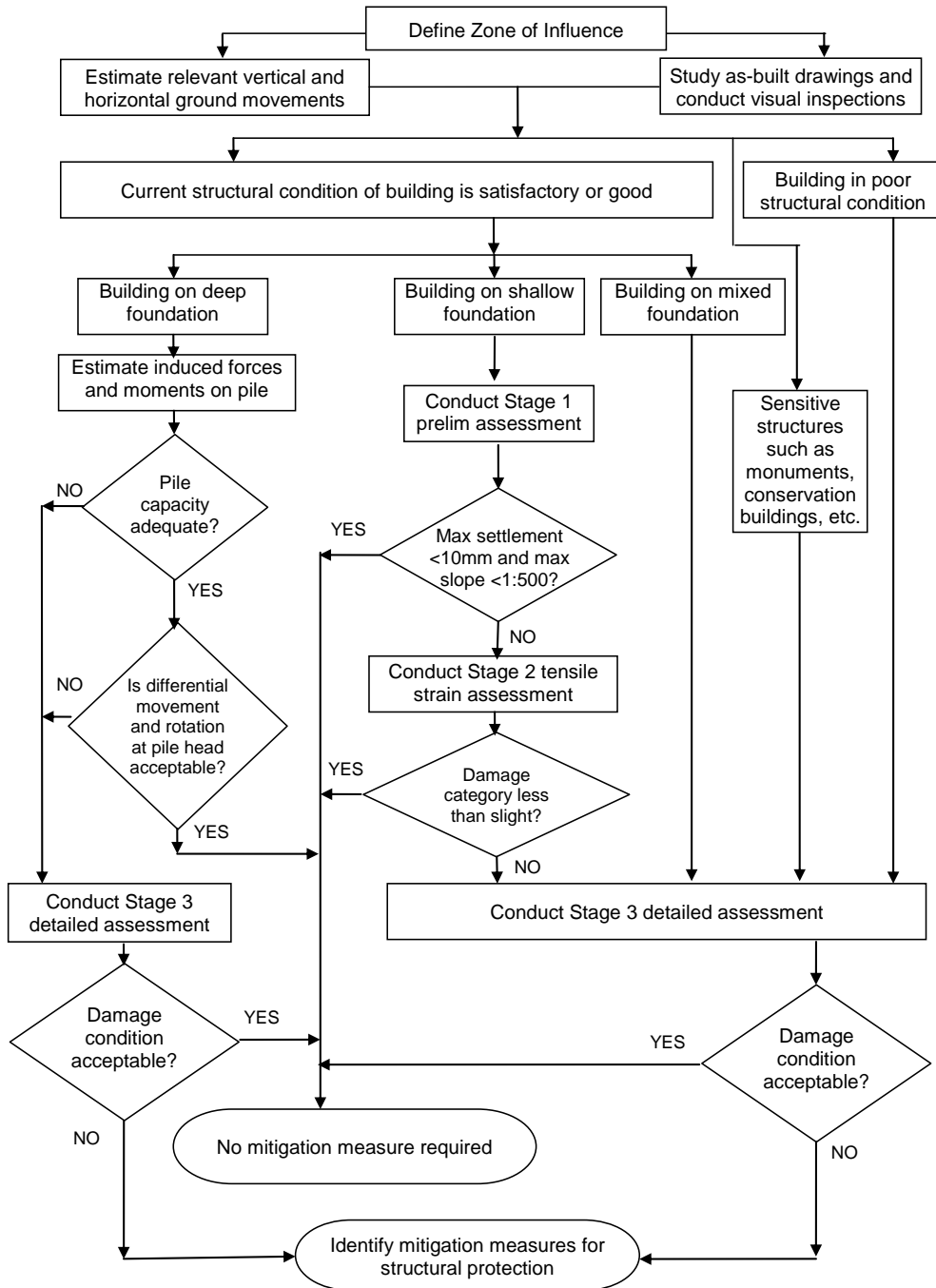


Figure 9. Framework for assessing the risk of building damage (reproduced from Fok et al 2012)

Table 3. Proposed categories of sensitivity index for Singapore context

Categories of sensitivity indices	Structural form	Heritage Features	Existing Condition
	Sensitivity of the structure to ground movements and interaction with adjacent buildings	Sensitivity to calculated movement of particular features within the building	Factors which may affect structure sensitivity to further movements
0	Similar to buildings for which the Burland et al assessment was formulated	No particular sensitive features	Good or Satisfactory (based on the visual inspection guidance given in the Appendix)
1	Buildings with some sensitive structural features in the movement zone (e.g. existing cracks where further movements are likely to concentrate)	Brittle finishes, eg. faience or tight-jointed stonework, which are susceptible to small structural movements and difficult to repair invisibly	Poor condition – may change the structural behaviour in cases of additional movements. Evidence of on-going movements that caused existing damage, but this relates only to serviceability concerns and unlikely to affect the structural stability and integrity to any part of the building
2	Buildings which by their structural form will tend to concentrate all their movements in one location (eg. buildings on mixed foundations, shophouses on shallow foundations abutting each other, long wall without movement joints)	Finishes which if damaged will have a significant effect on the heritage value of the building,	Poor condition – especially if existing damage are likely to affect the structural stability and integrity to any part of the building, and also if very small movements could lead to significant damage.

To illustrate how this new framework may be applied, Goh (2016) created a case study of Downtown Line tunnelling directly below the National Museum of Singapore. The National Museum is the oldest museum in Singapore and was

gazetted as a national monument in 1992. It is a two-storey masonry building supported on shallow footing foundations, with some parts that have undergone additional and alteration works that are reinforced concrete supported on piles. Prior to tunnelling works, a visual inspection survey was conducted for the national monument in 2011. There were several cracks up to 0.9 mm wide on the building facade, whereas the internal cracks were up to 1.5 mm wide. The existing condition of the building was classified as satisfactory. During construction in 2014, earth-pressure-balanced machines were used to construct the twin bored tunnels in the thick layer of Fort Canning Boulder Bed, and directly below the National Museum. An intensive array of building settlement monitoring was conducted using automated survey prisms, where the maximum building settlement measured was 11 mm while the maximum ground settlement in the vicinity was 18 mm after both tunnels had passed successively below the building. The considerations in deriving the damage category using the risk-based, index-system approach, is shown in the following steps:-

- Induced tensile strain of 0.0074% based on instrumentation monitoring results, hence Damage Index = 0
- Typical structure form, hence structure sensitivity index = 0
- Significant heritage for national monument, hence heritage sensitivity index = 2
- Satisfactory current condition, hence condition sensitivity index = 0

Since Damage Category = Damage Index + structure sensitivity index + heritage sensitivity index + condition sensitivity index = 2, the National Museum building would be subject to slight degree of severity and no further mitigation would be necessary. Indeed, there were no additional building protection measures or mitigation measures and the tunnelling was carried out without causing undue damage or adverse impact onto the National Museum building. Goh et al (2016) summarized all the cases of bored tunnelling directly below buildings in the Downtown Line project, and observed that tunnelling directly below buildings (sometimes in close proximity to the pile foundations) do not cause a building to settle significantly, provided that appropriate tunnelling controls are applied to limit ground deformations. Influences such as building stiffness and competent ground can further help to mitigate impact to buildings so that the tunnelling works can be carried out successfully without affecting the safety of occupants of the buildings.

## 6 APPLICATION OF DATA ANALYTICS IN TUNNELLING

The advent of big data analytics opens up new possibilities to look into design of tunnelling works. Today's tunnel boring machines (TBMs) collect a lot of real-time data during tunnelling operations, such as advance rate, cutterhead torque and rotation speed, thrust force, penetration rates, face pressures, grouting volume and pressures, slurry inflow and outflow volumes, etc. These operating parameters provide continuous evidence of the geological conditions along the tunnel drive and are being interpreted by experienced tunnelling engineers to adjust the operations of

the tunnel boring machines. For example, Shirlaw et al (2016) combined cutterhead contact force, the cutterhead torque and the advance rate into new functions, the Penetration Index and Specific Energy, to derive insights on tunnelling in weathered rock.

Goh et al (2016) reported on a pilot effort to use data analytics on TBM tunnelling, by mining borehole data and TBM data to predict ground settlement during tunnelling works. Figure 10 gives an overview of the data analytics model used in the pilot effort. This comprises of the input and data treatment, the configuration of the Artificial Neural Network (ANN) used to power the mining, and the training and verification technique so that a prediction can be made using new data sets. The input data consists of tunnel and TBM geometric relationships between the bored tunnel/s and the TBM/s, ground conditions and engineering properties from borehole records, operating parameters from the TBMs, and the distance and position of the instrumentation monitoring points in relation to the TBMs. Input data was normalized into bands of [-1, +1] to avoid unit and scale effects using Min-Max Normalization method, and a logarithmic conversion was applied where necessary (for example onto soil permeability). The linear/log normalization of the data removes scale effects and achieves better convergence in training.

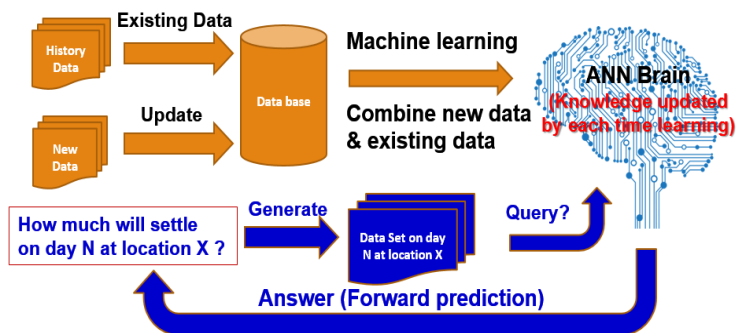


Figure 10. Overview of pilot trial of big data analytics on TBM, after Goh et al, 2016

The brain behind the data mining was constructed by Matlab using the Artificial Neural Network (ANN) technique. ANN is a machine learning approach that imitates biological neural networks and is versatile in learning & correlation generalization. The ANN model consists of nodes from an input layer, through hidden layers, to an output layer which was defined as the surface settlement in this project. Each node is connected to all nodes at adjacent layers through activation functions, which is analogous to connection strengths between the neurons. The weights for activation functions was generated from feed forward and back propagation methods during the training of the ANN model and would be used for subsequent prediction. The number of hidden layers and number of hidden nodes were chosen to optimise the mean error arising from the machine learning. As a rule-

of-thumb, the number of hidden nodes should be less than twice the input nodes. In addition, a trial and error method was used to determine the most suitable transfer functions for the ANN model by maximizing the R-correlation coefficient between the output prediction and the actual monitored data. Eventually, one hidden layer consisting of 668 nodes was used to connect the layer of 433 input nodes to the output layer related to ground surface settlement. A log-sigmoid transfer function was used to relate input nodes to hidden nodes whilst a pureline transfer function was used to relate hidden nodes to the output node. To mitigate external influence from non-tunnelling related events, training data was selected based on location of borehole, TBM and instruments that are between 100m ahead and 200m behind tunnelling face. Conventional training was done by partitioning the entire data set into 80% for training and 20% for verification. K-fold cross verification technique was also used to enhance robustness of ANN learning.

A case-study was made using the data from the completed twin bored tunnelling drives between MacPherson and Ubi stations in the Downtown Line Stage 3 project, which is about 1km long using earth-pressure balanced machines and in the Old Alluvium. The geological character and type of TBM is important as the ANN model would only be suitable for grounds in similar condition, just as how a human brain would apply heuristics to determine the kinds of operating parameters expected when tunnelling in the Old Alluvium vis-à-vis other geological conditions. Two scenarios were made, firstly based on full data set to verify learning capability of ANN model by predicting response of a specific settlement marker which was continuously monitored for 9 months; and secondly based on partial data set to verify forward prediction capability of the ANN model by only using TBM and monitoring data acquired prior to the subject settlement marker.

Figure 11 shows the results from both scenarios. The results using the full data set to verify machine learning was quite close to the monitored data, with error being less than 4mm for more than 80% of the time. However, the results using the partial data set to verify forward prediction was much larger than the monitored data. It was postulated that the quality of forward prediction is affected by distribution of input data range such as TBM operating parameters which was quite skewed and narrowly defined for a single project. The significant errors in forward prediction need to be understood. Nevertheless using the full data set, a parametric study to identify significance of causal factors using a trained neural network was conducted to find that significance of causal factors varies according to position of settlement marker with respect to TBM. For example, before the TBM reaches the settlement monitoring point, factors such as earth pressure, advance rate and cutter torque was found to be more significant. After the TBM passes the settlement monitoring point, factors such as thrust force, grout filling ratio and penetration rate becomes more significant. The causal factor analysis can provide insight on controllable TBM operations that can be used to mitigate tunnelling-induced settlements.

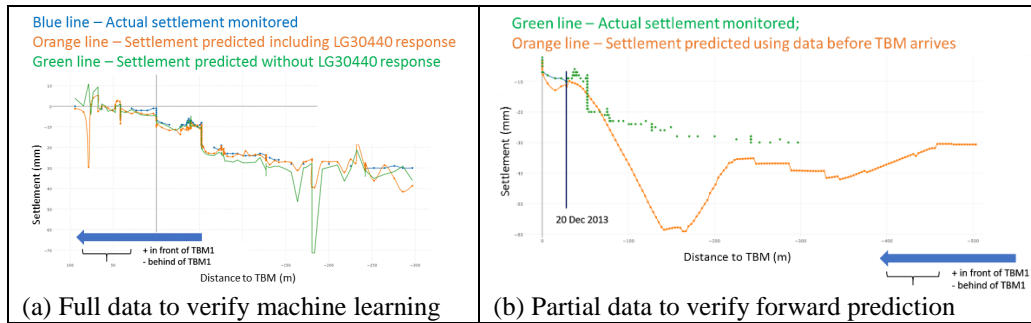


Figure 11. Prediction of surface settlement using actual data (from Goh et al, 2016)

In summary, the ANN model is a promising alternative to predict tunnelling-induced settlements. By using past experience to forward-predict settlements based on current encountered conditions, we can identify key governing parameters affecting settlements in order to implement intervention quantitatively. However, the ANN model needs to be trained using more data and different project conditions so that it can cover the full range of ground and tunnelling conditions and TBM parameters. An interesting investigation would be to look for outliers which can be related to problematic operations including large deformations, blow-outs and sinkholes. If this can be done, then it is plausible to mine relationships from these case studies and develop an early warning system that flags up risk scenarios to engineers during tunnelling works. Data analytics cannot replace the need for human judgment. Through sense-making of TBM data and identifying patterns associated with failure scenarios from historical trends, data analytics could give us more reaction time against abnormal trends so that pre-emptive mitigation measures could be implemented in a timely manner.

## 7 CONCLUSION

This paper gives a summary of various aspects of segmental tunnel lining design for the MRT bored tunnels. Recent developments in the joint design and bolt connections have been presented. The testing and design of SFRC segments in recent MRT bored tunnels are described. Recent research results and their applications in carrying out building assessment of the impact of ground movements on existing structures are given with examples in the appendices of the paper. This would assist design engineers to achieve a more comprehensive and reasonable assessment. The paper also shows the current status of the application of data analytics in tunnelling and areas that may require further researches.

## 8 REFERENCES

Boscardin, M.D. and Cording, J.C. 1989. Building response to excavation-induced settlement. *ASCE Journal of Geotechnical Engineering*, 115(1): 1–21.

- Burland, J.B., Broms, B.B. and de Mello, V.F.B. 1977. Behaviour of foundations and structures: state-of-the art report. Proc. of the 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, 2: 495–546.
- 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
- 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.
- Fok, P., Neo, B.H., Goh, K.H. and Wen, D. 2012. Assessing the impact of excavation-induced movements on adjacent buildings, The IES Journal Part A: Civil & Structural Engineering, Vol. 5(3), pp. 195-203.doi:10.1080/19373260.2012.696444
- Goh, K.H. and Mair, R.J. 2014. Incorporating the existing condition of buildings when assessing impact of underground construction, Proc. of conference Underground Singapore 2014, pp.315-323.
- Goh, K.H. 2016. Building condition and impact assessment of underground construction, Proceedings of the Institution of Civil Engineers – Forensic Engineering 170(1), pp. 14–23.
- Goh, K.H., Ng S.S.G. and Wong K.C. 2016. Case histories of bored tunnelling below buildings in Singapore Downtown Line, International Journal of Geoenvironment Case Histories 3(3), pp. 149–161.
- Goh, K.H., Zhang Y., Shi W., Cai J.G., Mao C. and Sun J. 2016. A Pilot Trial of Big Data Analytics on TBM Tunnelling, LTA-UITP Singapore International Transport Congress and Exhibition (SITCE), 19 to 21 October 2016, Singapore.
- ITA. 2016. ITATech Design Guidance for precast fibre reinforced concrete segments – Vol. 1, Design aspects.
- LTA Civil Design Criteria. 2010. Civil Design Criteria for Road and Transit Systems, Feb 2010, Land Transport Authority, Singapore.
- Mair, R.J., Taylor, R.N. and Bracegirdle, A. 1993. Subsurface settlement profiles above tunnels in clays. Géotechnique, Vol. 43(2), pp. 315–320.
- Mair, R.J., Taylor, R.N. and Burland, J.B. 1996. Prediction of ground movements and assessment of risk of building damage due to bored tunnelling, In R.J. Mair and R.N. Taylor (eds.), Proc. intern. symp. on Geotechnical Aspects of Underground Construction in Soft Ground, London, 15-17 April 1996: 713–718. Rotterdam: Balkema.
- Mair, R.J. and Taylor, R.N. 1997. Bored tunnelling in the urban environment: state-of-the-art report and theme lecture. Proc. 14th International Conference on Soil Mechanics and Foundation Engineering, Hamburg, 6-12 September 1997, 4: 2353–2385. Rotterdam: Balkema.
- Mair, R.J. 2013. Tunnelling and deep excavations: ground movements and their effects. Proceedings of the 15th European Conference on Soil Mechanics and Geotechnical Engineering – Geotechnics of Hard Soils – Weak Rocks (Part 4) A. Anagnostopoulos et al. (Eds.) IOS Press, pp. 39 - 70. DOI. 10.3233/978-1-61499-199-1-39.
- Shirlaw J.N., Ong J.C.W., Rosser H. B., Tan C. G., Osborne N. H. and Heslop P. E. 2003. Local settlements and sinkholes due to EPB tunnelling. Proc. of the

- Institution of Civil Engineers: Geotechnical Engineering. Issue GE4, pp. 193-211.
- Shirlaw J.N. 2016. The choice of EPB or slurry shields for tunnelling in mixed face conditions resulting from tropical weathering, Proc. of conference Underground Singapore 2016.
- Thiri Su. 2015. Study on ground behavior associated with tunneling in mixed-face soil condition. PhD thesis, National University of Singapore.
- TR26:2010. Technical reference for deep excavation. Spring Singapore.
- Wen D. 2011. Underground construction and tunnelling in urban environment. Key Note Lecture. *Proceedings of 14<sup>th</sup> Asian Regional Conference on Soil Mechanics and Geotechnical Engineering, Hong Kong, 23-27 May 2011.*
- Wen, D. 2016. Development of reinforced concrete segmental lining design for MRT bored tunnels in Singapore. *Proceedings of the 19<sup>th</sup> Southeast Asian Geotechnical Conference & 2<sup>nd</sup> AGSSEA Conference , Kuala Lumpur 31 May – 3 June 2016*
- Yugami S., Maw M., Liew Y. K., Wijesinghe M. Y., Kho C. M. and Nair R. 2009. Back Analysis of tunnelling settlement parameters for Circle Line Contract 854. Underground Singapore 2009, pp. 53-59.
- Zhang Y. H., Cham W. M., Kumarasamy J. 2011. Volume loss caused by tunnelling of Circle Line projects. Underground Singapore 2011, Singapore, pp. 114-121.

## Appendix 1 – Estimating the relative bending stiffness of Pasir Panjang Shophouses

### References:

- Goh, K.H. 2010. Response of ground and buildings to deep excavations and tunnelling. Ph.D. thesis, Cambridge University.
- Goh, K.H. and Mair, R.J. 2014. Response of framed buildings to excavation-induced movements. Soils and Foundations, June 2014, Vol. 54(3), pp. 250–268.

### Bending stiffness of building

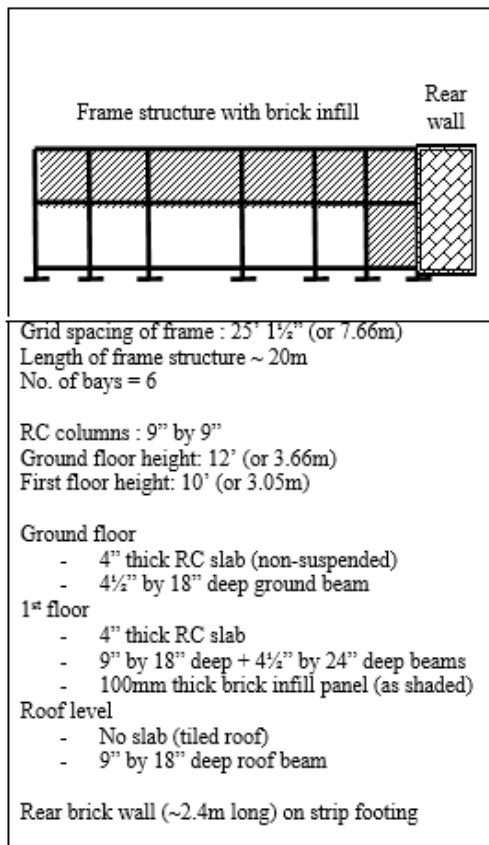


Figure A1. Bending stiffness of building EI at Pasir Panjang shophouse A29 (western end)

The bending stiffness for a frame building may be shown to be the following:

$$EI_{\text{frame}} = \sum C_i * EI_{\text{ith beam}} \text{ where } C_i = 1 + \frac{L^2}{I^2} \left[ \frac{K_{lc} + K_{uc}}{K_b + K_{lc} + K_{uc}} \right]$$

- For 9"x9" columns,  $I_c = (9*0.0254)^4/12 = 2.28 \times 10^{-4} \text{ m}^4$
- For 4 ½" by 18" deep beam,  $I_b = (4.5*0.0254)*(18*0.0254)^3/12 = 9.10 \times 10^{-4} \text{ m}^4$

- For 9” by 18” deep beam,  $I_b = (9 \times 0.0254) \times (18 \times 0.0254)^3 / 12 = 1.82 \times 10^{-3} \text{ m}^4$
- For 4½” by 24” deep beam,  $I_b = (4.5 \times 0.0254) \times (24 \times 0.0254)^3 / 12 = 2.16 \times 10^{-3} \text{ m}^4$
- Average  $I_b = 1.99 \times 10^{-3} \text{ m}^4$
- At 1<sup>st</sup> floor, bay length,  $l \sim 20/6 = 3.33\text{m}$ ;  $K_b / E = I_b / l = 1.99 \times 10^{-3} / 3.33 \text{ m}^2$ ;  $h_{lc} = 3.66\text{m}$ , hence  $K_{lc} / E = I_c / h_c = 2.28 \times 10^{-4} / 3.66 \text{ m}^2$  for lower column;  $h_{uc} = 3.05\text{m}$ , hence  $K_{uc} / E = I_c / h_c = 2.28 \times 10^{-4} / 3.05 \text{ m}^2$  for upper column; for a  $L = 10\text{m}$  long frame (in both sagging/hogging modes),  $C = 2.7$
- In a similar manner, it may be shown that  $C = 2.7$  and  $2.1$  at ground floor and roof level respectively.

Thus, the frame bending stiffness,  $EI_{\text{frame}} = E \times S(C_i \times I_{bi}) = 23 \times 10^6 \times (2.7 \times 9.10 \times 10^{-4} + 2.7 \times 1.99 \times 10^{-3} \times 2 + 2.1 \times 1.82 \times 10^{-3}) = 3.89 \times 10^5 \text{ kNm}^2$  @ 7.66m grid spacing =  $5.07 \times 10^4 \text{ kNm}^2$  per m run.

Bending stiffness of 4” thick first floor slab =  $23 \times 10^6 \times [(4 \times 0.0254)^3 / 12] = 2.01 \times 10^3 \text{ kNm}^2$  per metre run.

Total bending stiffness of building,  $EI = 2.01 \times 10^3 + 5.07 \times 10^4 = 5.27 \times 10^4 \text{ kNm}^2 / \text{m}$

### Representative soil stiffness $E_s$

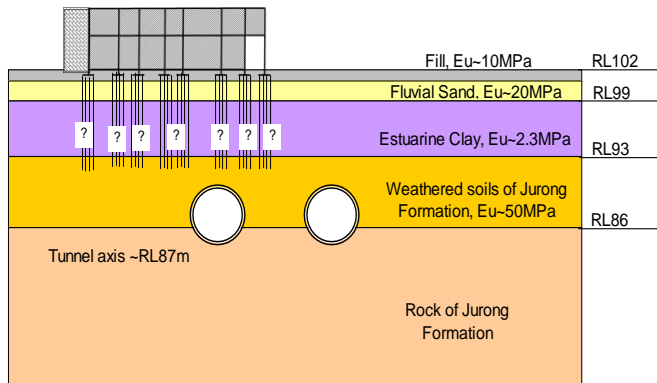


Figure A2. Typical soil section at Pasir Panjang shophouse A29 (western end)

Representative soil stiffness is defined as the weighted average of the elastic stiffness for all the soil layers above the tunnel excavation level. Soil profile above tunnel axis level, see Figure A2 consists of 1m Fill, 2m Fluvial sand, 6m Estuarine Clay, and 6m of Jurong Formation soils.

Using weighted average method, undrained modulus of soil above tunnel  $E_u = (1 \times 10 + 2 \times 20 + 6 \times 2.3 + 6 \times 50) / 15 = 24 \text{ MPa}$

Since  $E' = \frac{1 + \nu'}{1 + \nu_u} * E_u$ , the representative soil stiffness  $E_s = 1.3/1.5 * 24 = 21 \text{MPa}$ , since

$$E' = \frac{1 + \nu'}{1 + \nu_u} * E_u$$

Relative bending stiffness,  $\rho_{\text{sag}}$  and  $\rho_{\text{hog}}$

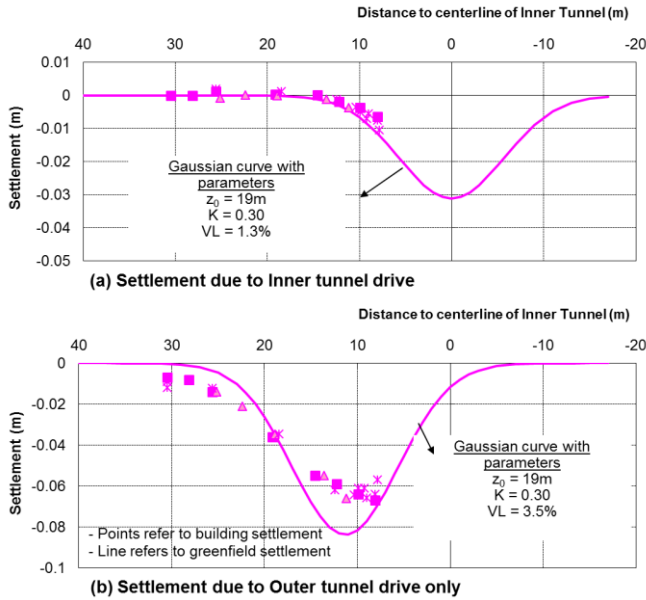


Figure A3. Relative bending stiffness of building at Pasir Panjang shophouse A29 (western end)

For inner tunnel drive, ground hogging length,  $L_{\text{hog}} = 23\text{m}$ .

Relative bending stiffness,  $\rho_{\text{hog}} = EI/E_s/L_{\text{hog}}^3 = 1.44 \times 10^5 / 21 \times 10^3 / 23^3 = 7.40 \times 10^{-4}$ .

For outer tunnel drive, ground sagging and hogging lengths,  $L_{\text{sag}} = 8\text{m}$  ;  $L_{\text{hog}} = 15\text{m}$ .

Relative bending stiffness,  $\rho_{\text{sag}} = EI/E_s/L_{\text{sag}}^3 = 5.27 \times 10^4 / 21 \times 10^3 / 8^3 = 6.43 \times 10^{-3}$  ;  
 $\rho_{\text{hog}} = EI/E_s/L_{\text{hog}}^3 = 5.27 \times 10^4 / 21 \times 10^3 / 15^3 = 9.76 \times 10^{-4}$ .

**Appendix 2: Guidance to classify overall building condition from visual inspections**

Overall building condition	Good		Satisfactory		Poor	
<b>Observations on structural elements</b>	No sign of any structural damage / distress and material deterioration on visible structural elements		No sign of structural distress on visible structural elements, but some structural cracks and material deterioration (eg concrete spalling, corrosion of steel reinforcements) are observed. Generally, the defects are unlikely to affect structural stability and integrity of any part of the building		Signs of structural distress and structural damage on visible structural elements. These defects are likely to affect the structural stability and integrity to any part of the building,	
<b>Classification of visible damage to walls<sup>(1)</sup></b>	Hairline cracks of less than about 0.1mm	Fine cracks which can be treated easily using normal decoration. Damage generally restricted to internal wall finishes; cracks rarely visible in external brickwork.	Cracks easily filled. Recurrent cracks can be masked by suitable linings. Cracks not necessarily visible externally; some external repointing may be required to ensure weather-tightness. Doors and windows may stick slightly and	Cracks which require some opening up and can be patched by a mason. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may	Extensive damage which requires breaking-out and replacing sections of walls, especially over doors and windows. Windows and door frames distorted, floor sloping noticeably <sup>(4)</sup> . Walls leaning or bulging noticeably, some loss of bearing	Structural damage which requires a major repair job, involving partial or complete rebuilding. Beams lose bearing, walls lean badly and require shoring. Windows broken with distortion. Danger of instability. Typical

		Typical crack widths <sup>(2)</sup> up to 1 mm.	require easing and adjusting. Typical crack widths <sup>(2)</sup> up to 5 mm.	fracture. Weather-tightness often impaired. Typical crack widths <sup>(2)</sup> are 5 to 15 mm, or several of 3mm cracks.	in beams. Service pipes disrupted. Typical crack widths <sup>(2)</sup> are 15 to 25 mm, but also depends on number of cracks.	crack widths <sup>(2)</sup> are greater than 25 mm, but depends on number of cracks.
<b>Classification of visible damage on ground floor slab settlement<sup>(1)</sup></b>	Hairline cracks between floor and skirtings	Settlement of the floor slab, either at a corner or along a short wall, or possibly uniformly, such that a gap opens up below skirting boards which can be masked by resetting skirting boards. No cracks in walls. No cracks in floor slab, although	Larger gaps below skirting boards, some obvious but limited local settlement leading to a slight slope of floor slab; gaps can be masked by resetting skirting boards and some local rescreeding may be necessary. Fine cracks appear in internal partition walls which need some re-decoration; slight distortion in door frames so	Significant gaps below skirting boards with areas of floor, especially at corners or ends, where local settlements may have caused slight cracking of floor slab. Sloping of floor in these areas is clearly visible; (slope approximately 1 in 150). Some disruption to drain, plumbing or heating pipes may	Large, localised gaps below skirting boards; possibly some cracks in floor slab with sharp fall to edge of slab; (slope approximately 1 in 500 or more). Inspection reveals voids exceeding 50 mm below slab and/or poor or loose fill likely to settle further. Local breaking out, part refilling and relaying of floor slab or grouting of fill may	Either very large, overall floor settlement with large movement of walls and damage at junctions extending up into 1st floor area, with possible damage to exterior walls, or large differential settlements across floor slab. Voids exceeding 75mm below slab and/or very poor or very loose fill likely to settle further. Risk

		there may be negligible cracks in floor screed and finish. Slab reasonably level. Typical gap <sup>(3)</sup> up to 6mm.	some 'jamming' may occur, necessitating adjustment of doors. No cracks in floor slab although there may be very slight cracks in floor screed and finish. Slab reasonably level. Typical crack widths <sup>(2)</sup> up to 1mm and gap <sup>(3)</sup> up to 13mm.	occur. Damage to internal walls is more widespread with some crack filling or replastering of partitions necessary. Doors may have to be refitted. Inspection reveals some voids below slab with poor or loosely compacted fill. Typical crack widths <sup>(2)</sup> up to 5mm and gap <sup>(3)</sup> up to 19mm.	be necessary; damage to internal partitions may require replacement of some bricks or blocks or relining of stud partitions. Typical crack widths <sup>(2)</sup> range between 5 to 15mm but may also depend on number of cracks, and gap <sup>(3)</sup> up to 25mm.	of instability. Most or all of floor slab requires breaking out and relaying or grouting of fill; internal partitions need replacement. Typical crack widths <sup>(2)</sup> greater than 15mm but may also depend on number of cracks, and gap <sup>(3)</sup> greater than 25mm.
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Note:

<sup>(1)</sup> Classifications of visible damage to walls and ground floor slab are adapted from BRE Digest 251 (1995), and with particular reference to ease of repair.

<sup>(2)</sup> Crackwidth is one factor in assessing category of damage and should not be used on its own as a direct measure of it.

<sup>(3)</sup> Gap refers to space usually between the skirting and finished floor, caused by settlement after making appropriate allowance for discrepancy in building, shrinkage, normal bedding and the like.

<sup>(4)</sup> For floors and walls, local deviation of slope from the horizontal or vertical of more than 1/100 will normally be clearly visible. Overall deviations in excess of 1/150 are undesirable.