# Design and Construction of Large Diameter Circular Shafts

T.T. Aye, M.S.Y. Tong, K.H. Yi, E. Arunasoruban
*AMBERG & TTI Engineering Pte Ltd, Singapore*

ABSTRACT: Three numbers of 30m diameter circular shafts are designed to facilitate the launching of tunnel boring machines for the construction of tunnels in Downtown Line 3 Project. The sites are located at the Eastern side of Singapore, at which the ground is mainly comprised of Old Alluvium (OA) formation overlain by backfill material, except at one of the shafts where a localized soft layer of Kallang formation (recent alluvium deposit) was encountered above the Old Alluvium. Shafts are generally 37-44m deep, of which the upper shaft of 12 to 17.5m is supported by interlocking sheet pile wall and ring beams; and the lower shaft is supported by 1.5-1.9m depth of sprayed shotcrete or cast in-situ caisson rings. This paper presents the selection and design of retaining wall system and addresses the ground and site constraints, and construction difficulties. Due to the large shaft diameter, the shafts are excavated in part and the paper will discuss on how the hoop force is distributed and how the unbalanced loading is addressed. In addition, problems associated with the soft ground at the upper shaft are discussed.

## 1 PROJECT OVERVIEW

Downtown Line Stage 3 (DTL3), a part of the fifth mass rapid transit line connecting the North-Western and Eastern regions of Singapore to the Central Business District and Marina Bay, is 23km long and involves 16 Stations.

This project involves the construction of twin bored tunnels of 5.8m internal diameter. To facilitate the launching of tunnel boring machines (TBMs) at some part of the alignments, three on-line large circular shafts of 30m diameter with varying depth of 37m to 44m, were designed and constructed. In this paper, for discussion purpose, the shafts are named as P-shaft, S-shaft and PIE shaft.

## 2 GEOLOGICAL PROFILE AND SITE CONDITION

### *2.1 Geological profile*

All three launch shafts are situated on the eastern part of Singapore Island whereby the main geological formation is Old Alluvium (OA). The OA was deposited during the time of Pleistocene period. The top formation of OA has been eroded with the corresponding retreat and rising of ground water level.

Chu et al. (1993) mentioned that OA is slightly over-consolidated with OCR ranging from 1.1 to 1.4 due to the erosion process. As the internal angle of friction '$\phi$' of OA in this area ranges from 30 to 35 degree, the $K_o$ is calculated in the range of 0.6 to 0.7. The interpreted data from the pressuremeter tests indicates a wide range of $K_o$ from 0.3 to 1.0, with the average and mean value of 0.59 to 0.61. Finally, a $K_o$ of 0.7 was selected for the design of temporary structures.

In general, the uppermost soil layer consists of 1.5m to 5.5m of man-made fill material overlaying 3m to 10m thick OA layer with SPT N-value ranging from 8 to 30 followed by a deeper competent OA formation with higher SPT N-value of 30 to 100. At P- shaft, a localized 4m thick soft peaty clay layer of Kallang formation (recent alluvium deposit) is encountered. The subsurface geological profile at the respective shafts is summarized in Table 1.

Table 1. Subsurface geological profile at the three shafts

| S-Shaft | | | P-Shaft | | | PIE Shaft | | |
|---|---|---|---|---|---|---|---|---|
| Depth (m-bgl) | | Material & SPT-N | Depth (m-bgl) | | Material & SPT-N | Depth (m-bgl) | | Material & SPT-N |
| From | To | | From | To | | From | To | |
| 0 | 1.5 | Fill | 0 | 3-5.5 | Fill | 0 | 4.3 | Fill |
| 1.5 | 4.8 | OA(D) $N = 25$ | 5.5 | 9.5 | Estuarine $N = 2$ | 4.3 | 15 | OA(E)&(D) $8 \leq N \leq 14$ |
| 4.8 | 14 | OA(C) $31 \leq N \leq 47$ | 9.5 | 12.5 | OA(D) $10 \leq N \leq 25$ | 13 | 15 | OA(C) $N = 37$ |
| 14 | 31 | OA(B) $51 \leq N \leq 100$ | 12.5 | 24 | OA(B) $60 \leq N \leq 100$ | 15 | 38 | OA(B) $80 \leq N \leq 100$ |
| 31 | 36.5 | OA(A) $N \geq 100$ | 24 | 36 | OA(B) $80 \leq N \leq 100$ | 38 | 43.5 | OA(B) $90 \leq N \leq 100$ |

The OA at the three sites comprises predominantly Sandy OA cemented together with slight amount of fine materials such as silt and clay. However; the top 4m to 13m at PIE shaft comprises mainly Clayey OA with highly cemented silty/sandy clay. The water content of Sandy and Clayey OA varies from 12% to 24% and from 17% to 27% respectively. The coefficient of permeability ranges mainly between $2 \times 10^{-7}$m/s and $1 \times 10^{-8}$m/s but a few data suggest that higher permeability of up to $1 \times 10^{-6}$m/s is expected to be encountered at localized sand lenses within Sandy OA. The undrained shear strength increases with depth and varies from 50kPa to 400kPa. The engineering properties of the various types of soil at the three shafts are summarised in Table 2.

Table 2. Engineering properties of various types of soil

| Material | | Unit Weight (kN/m$^3$) | Strength Parameters | | |
|---|---|---|---|---|---|
| | | | Total Stress | Effective Stress | |
| | | | $S_u$ | c' | $\phi$' |
| | | | (kN/m$^2$) | (kN/m$^2$) | (°) |
| | Fill | 20 | 30 | 0 | 28 |
| | Kallang Formation - Estuarine Layer , E | 15 | $0.75z + 16.25$ $(20 \leq S_u \leq 35)$ | 0 | 15 |
| Old Alluvium | (a) OA(E) (N<10) | 20 | 5N | 0 | 30 |
| | (b) OA(D) (10≤N<30) | 20 | 5N | 5 | 32 |
| | (c) OA(C) (30≤N<50) | 21 | 5N | 10 | 32 |
| | (d) OA(B) (50≤N<100) | 21 | 3N+100 | 10 | 35 |
| | (e) OA(A) (N≥100) | 21 | 400 | 20 | 35 |

N= SPT N-value; z= depth below existing ground level

### *2.2 Site condition*

P-shaft is situated in green field; with some structures such as single storey buildings and tennis court located at about 30m away from the shaft. S-shaft is located in a soccer field with nearby road and the closest residential buildings are located about 50m away from the shaft. PIE shaft is located on a small island at slip road of expressway flyover with no nearby buildings.

Although the adjacent buildings are located out of the influence zone of the shafts, excessive water table drawdown and/or its induced settlement are to be minimized during the shaft excavation and the retaining wall system has to be designed to achieve this requirement. Also, there is another requirement to remove the upper 10m of retaining wall structure for future development.

## 3 SHAFT DESIGN AND ITS CHALLENGES

### *3.1 Selection of shaft geometry*

The shafts are required to facilitate the launching of TBMs with 4 tunnel drives (2 TBMs at each shaft), and will also function as temporary shafts. A minimum area of $900m^2$ or internal diameter of 28m is found to be necessary to facilitate the launching of two TBMs and supporting the logistics for two tunnel drives. A circular structure is preferred as it is structurally stable and can be constructed with no struts spanning across the excavation, hence providing a relatively obstruction free area for excavation works. Also, by taking the ground loading through hoop forces (circular shape), a circular shaft can minimize the ground displacement during excavation.

### *3.2 Selection of earth retaining and support system*

The ground condition as indicated in Section 2.1 has introduced some challenges to the planning and design stage. As it is anticipated that localized high permeable sand lenses in Sandy OA may be encountered, there is a potential risk of excessive water ingress that will be encountered as well when the ground is exposed during excavation. Moreover, the fines may be washed out together with the high water flow and subsequently this may result in face instability if the supporting elements are not provided promptly.

At P-shaft, the soft peaty clay layer is located at localized area of the upper shaft perimeter and hence there is some unbalanced lateral pressure acting on the shaft due to the different type of soil around the shaft. The supporting system would have to be designed to withstand such forces.

In order to support the loose soil and soft clay pocket in temporary condition and to extract the top 10m after construction without facing difficulties, sheet pile wall supported with ring beams was chosen as the retaining system for the upper shaft.

The OA at depth of about 12m to 15m below ground level is mainly dominated by cemented to semi-cemented, dense and generally low permeable soil although localized sand lenses are expected. Based on the engineering soil properties of this material, it is expected that the soil can have sufficient stand-up time during the excavation and may not induce significant ground movement if the face is left unsupported for short period of time prior to installation of supporting elements. Such a ground condition is suitable for adopting the caisson method.

The total diameter of shaft is determined to be 30m after taking into consideration for the construction tolerance and shaft utilization for the TBM lowering, pipes, utilities and other tunnel related installations and services.

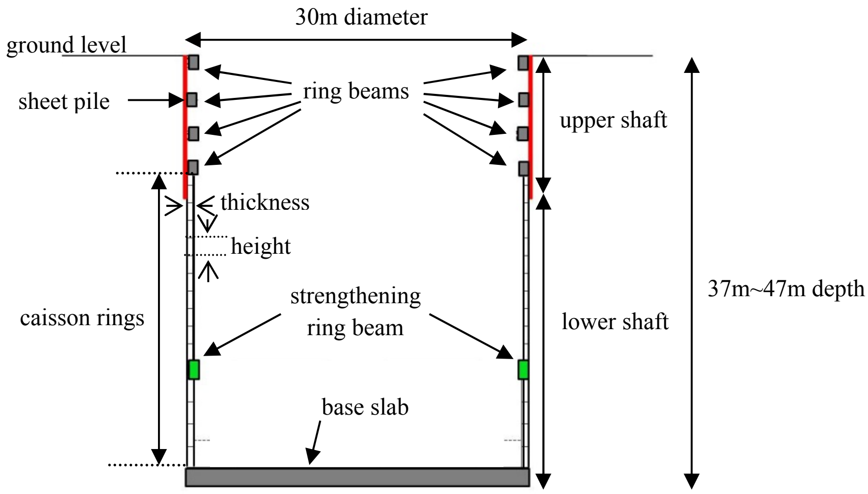


Figure 1. Typical support system of circular shaft

### *3.3 Design of upper shaft*

Finite element (FE) analyses were performed to simulate the soil-structure interaction for the shaft design. Multiple layers of sub-soil profile, construction sequences and time-dependent behavior of OA were also simulated in the modelling. As the diameter of shaft is 30m, the pressure distribution over a meter length of sheet pile wall in the longitudinal (circumferential) direction represents a plane strain situation. In addition, excavation of the entire area has to be carried out portion by portion, thereby resulting in some unbalanced load on the sheet pile wall. Therefore, it is more suitable to adopt plane strain model instead of axisymmetric model for design of sheet pile wall.

U-type sheet pile wall was designed to retain the top soil and piling joints were designed to be interlocking joints for cutting off of the ground water seepage. To achieve the required geotechnical capacity and also to support the self-weight of ring beams, a minimum design length of sheet pile wall of 12m to 17.5m long is implemented. A verticality of 1:75 was required to be attained for the pile driving.

The circular sheet pile wall was supported by cast in-situ reinforced concrete (RC) ring beams (RB) spaced at 3m to 3.5m vertically. As the shaft was subject to unbalanced ground condition, different soil lateral pressure and applied surcharge due to construction loadings, the RC ring beams were designed to resist these unbalanced loadings to prevent excessive distortion on the shaft. An eccentricity of 20mm was also considered for construction tolerance. The design size of ring beams varies from 0.5mx0.8m to 1.0mx0.8m (thickness 'T' x height 'H').

### *3.4 Additional unbalanced loads due to site activities on upper shaft*

Further analyses on the influence of site activities in close proximity to the shaft have been performed before construction as it will induce additional distortion to the shaft. Figure 2 presents the typical site utilization in the vicinity of shaft. There are several installations such as muck pits, gantry cranes foundation, and large crane foundation such as that for 700ton crane for lowering of TBMs etc. In general, these installations result in a maximum depth of 4m excavation which would have unloading effect on the shaft design. On the foundation loading, according to the stress distribution from elastic theory, this loading acting over a certain area distributes the additional unbalanced load until a depth of approximately 3B (B=width of foundation) below ground level. Thus, the upper shaft has to design for all these loadings and also the unloading effect, which will be resisted by the provisions of ring beams.

Based on the site utilization plans and construction sequence of shafts, the possible additional critical unbalanced load combinations were determined and presented in Table 3. Due to the complexity of additional unbalanced loads, FE analyses were carried out to simulate effect of unbalanced loads on ring beams. The localized excavation adjacent to the sheet pile wall was simulated in the structural model by removing the localized soil spring supports on it. Figure 3 presents a typical distortion of ring beam due to 'Case 2' unbalanced load combination in Table 3.

As distortion due to unbalanced loading effect can be quite significant in some of these load combinations, it is important to develop the site utilization plan together with the evaluation of its effect on the shaft prior to construction.

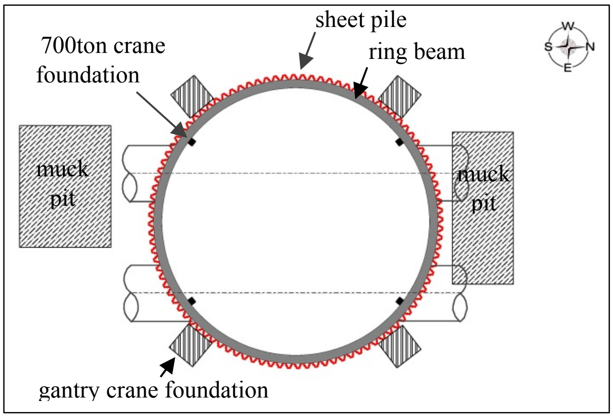



Figure 2. Typical site utilization plan

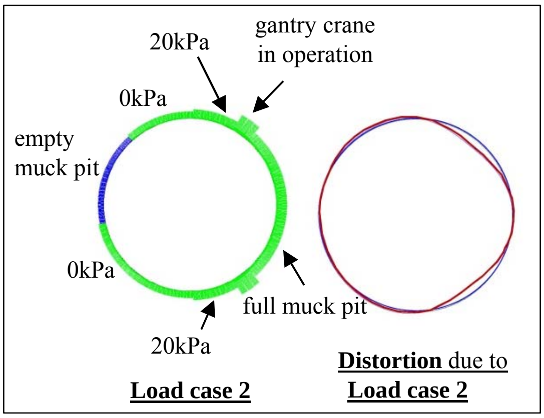



Figure 3. Additional distortion due to Load case 2 in Table 3

Table 3. Possible unbalanced load combination

| | Load due to 20kPa Surcharge | Load due to 0kPa Surcharge | Load due to Gantry Crane | Load due to Muck Pits | Load due to 700ton crane |
|---|---|---|---|---|---|
| Case 1 (Shaft excavation) | Southern half of the shaft | Northern half of the shaft | -na- | -na- | -na- |
| Case 2 (Gantry crane in operation) | Northern half of the shaft except at muck pit | Southern half of the shaft except at muck pit | Northern part of the shaft | Northern part muck pit full and southern part muck pit empty | -na- |
| Case 3 (TBM lowering operation) | Southern half of the shaft | Northern half of the shaft | -na- | -na- | Western part of the shaft |
| Case 4 (Empty muck pits) | All around the shaft except at muckpits | -na- | Northern part of the shaft | Both muckpits empty | -na- |

# In-situ ground load is not shown for clarity.

### *3.5 Design of lower shaft*

Caisson rings at lower shaft are typically designed to support the in-situ soil stresses from ground and hydrostatic pressure only. However, due to the large size of excavation, sequential excavation will be implemented and this will result in unbalanced loading on the completed caissons above the excavation. This will also depend on the depth of excavation, soil strength, caisson strength and arching effect of the soil. Due to these behaviours, several key design considerations have to be addressed in the design as follows:
- Excavation and wall installation sequence
- Standup time for unsupported vertical face
- Depth and extent of unsupported vertical face
- Unbalanced loading on already installed caisson due to incomplete ring during subsequent excavation.
- Minimum early strength of caisson rings to be achieved prior to next excavation

According to the stability of unsupported axisymmetric excavation in an undrained condition by Britto and Kusakabe (1984), the arching effect in the circumferential direction decreases as the diameter of shaft becomes larger. Hence, in order to control the face instability and to minimize the potential unbalanced load acting on the above already installed ring, a 1.5m height caisson segments was determined and the maximum unsupported circumferential length (L) was limited to 11.8m (or 1/8 of shaft perimeter) for excavation. This is generally applicable for shaft depth of up to 28m. As the undrained shear strength of OA increases with depth, for depth beyond 28m, a 1.9m height caisson segments were adopted instead of 1.5m. During the construction stage, subject to the ground condition, the maximum excavation circumferential length is allowed to be adjusted according to the observed face stability of unsupported length.

Similar to the upper shaft, the unbalanced load acting on the completed caisson ring due to the next level partial excavation was simulated in finite element analyses. It is noted from the analyses results that the smaller area of excavation (i.e. 1.5m(H) x 6m(L)) will induce more additional distortion and moment on the completed caisson ring above due to the concentrated loading acting on the smaller area. As a result, the completed ring needs to achieve the full strength before the next excavation level starts. Construction period will take longer as the strength gain may take about 7days. The optimum design was estimated when the 18m$^2$ excavation area (i.e. 1.5m(H) x 12m(L)) and the minimum early strength of 16MPa are required to be achieved before the next excavation level started.

Similar to Section 3.3, a construction tolerance of 20mm was considered in design. In addition, the maximum distortion of 20mm on radius under any load combinations was also considered in the design. The details of caisson rings is presented in the following Table 4.

For the base slab, as it was designed as a 1500mm thick slab, it was cast after the final excavation of 1500mm depth was completed. There is no intermediate step of casting the caisson wall, as in this case, both the slab and wall were cast together as one single element. This scheme provides the ease of construction and save some construction time.

Table 4.Design of caisson rings with depth

| Depth (m-bgl) | | P-Shaft & S-Shaft | | Depth (m-bgl) | | PIE Shaft | |
|---|---|---|---|---|---|---|---|
| | | Caisson Ring Segment | | | | Caisson Ring Segment | |
| From | To | Height (m) | Thickness (m) | From | To | Height (m) | Thickness (m) |
| 11 | 26 | 1.5 | 0.5 | 11 | 30.5 | 1.5 | 0.5 |
| 26 | 27.5 | Strengthening Beam | | 30.5 | 33 | 1.5 | 0.6 |
| 27.5 | 35 | 1.9 | 0.6 | 33 | 34.5 | Strengthening Beam | |
| 35 | 36.5 | Base Slab | | 34.5 | 42 | 1.9 | 0.6 |
| - | - | - | - | 42 | 43.5 | Base Slab | |

### *3.6 Design of strengthening structure*

As presented in Section 3.5, the lower shaft was retained by cast in-situ caisson wall which was designed to take the forces from the in-situ stresses, hydrostatic pressure and unbalanced load due to the partial excavation. For TBM launching, 4 large openings of 7m diameter are required and there is a need to hack the caisson wall above the base slab for construction of these openings. This will interrupt the hoop force distribution and result in stress concentration around the tunnel-eye openings. Figure 4 shows the load path due to the construction of these large openings in the caisson wall. The major stress redistributions around opening are (1) compression stress concentration at top and bottom of opening (2) tension stress at the sides of opening (3) vertical bending moment due to radial displacement at the sides of opening. Therefore, a strengthening system which comprised cast in-place RC ring beam at top and column at sides of the openings was introduced to distribute the concentrated stresses around the openings. This complex system was modelled and simulated in 3D FE analysis. From the analysis, it was observed that the stresses at pillar between two openings were less due to ground relaxation and therefore, the major stresses were found to occur at the sides of single opening. The opening is modelled as rectangular opening instead of circular opening. This is to simplify the analysis and this does not affect the maximum compression stresses (at the top and bottom of opening) and tension stress (at the side of opening) since same height and width of opening is adopted. Accordingly, the reinforcements of the RC columns beside the openings were designed to take the tension force and transfer the force to the relatively stiffer ring beam on the top of the opening and base slab below the opening. This strengthening beam and columns were planned to be constructed while constructing the caisson wall (as an integrated structure) of the lower shaft.

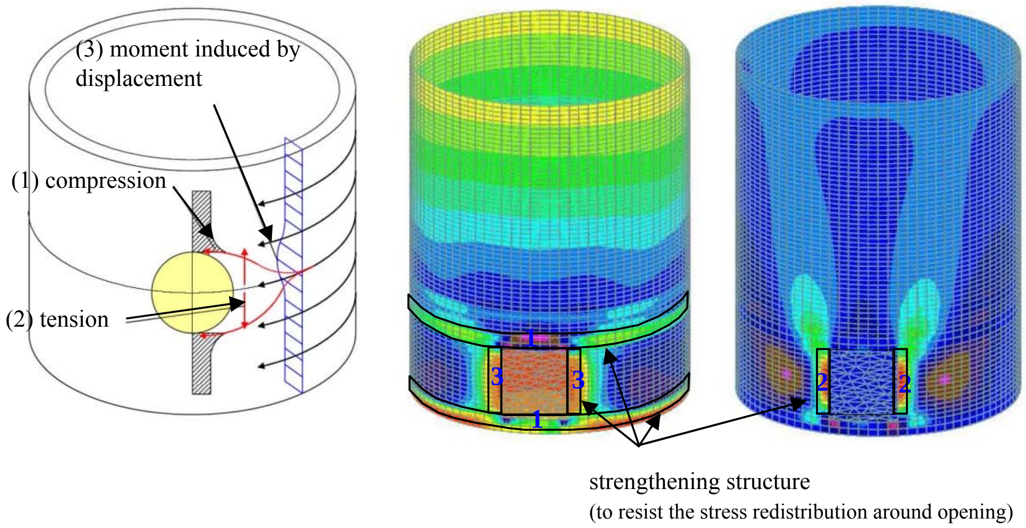



Figure 4. Stresses around opening and its strengthening system

## 4 INSTRUMENTATION AND MONITORING PLAN

To monitor the displacement of the large circular ring beams and caisson rings, optical prisms were installed. These will monitor the distortion of rings due to unbalanced loads and sequential excavation. 8 numbers of prisms were installed at each selected ring of the shaft. In addition, concrete strain gauges were almost embedded in the RC caissons and ring beams to monitor the hoop forces development within the rings.

Ground settlement markers and inclinometers were also installed to observe the behavior of ground movement and influence zone of the excavation of the shafts. A typical detail of the instrumentation plan is presented in the following Figure 5.

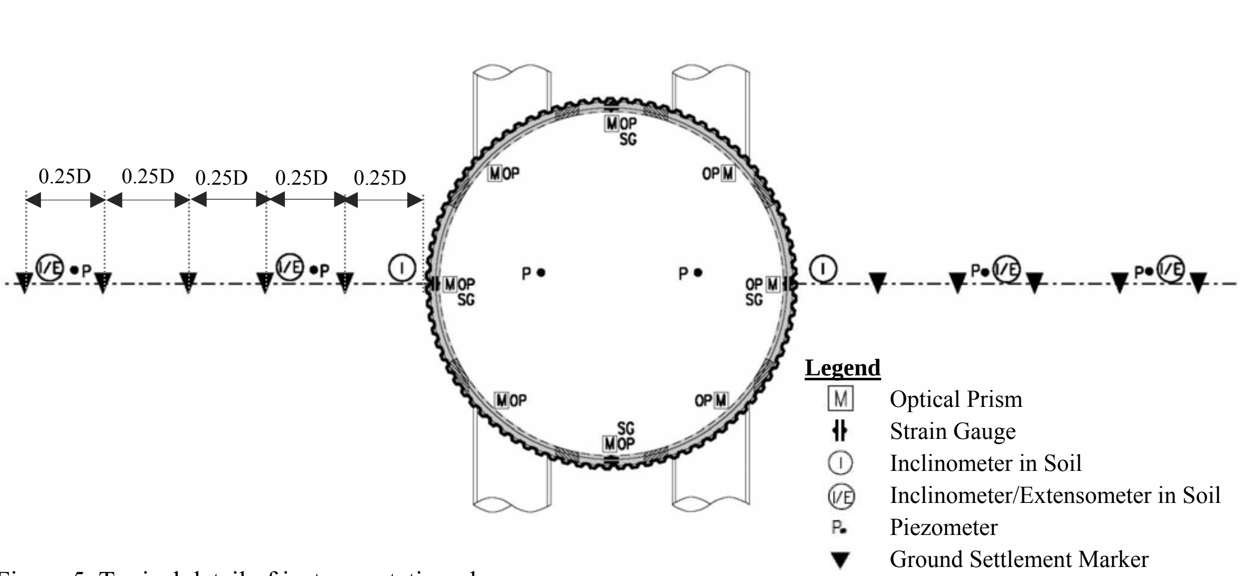



Figure 5. Typical detail of instrumentation plan

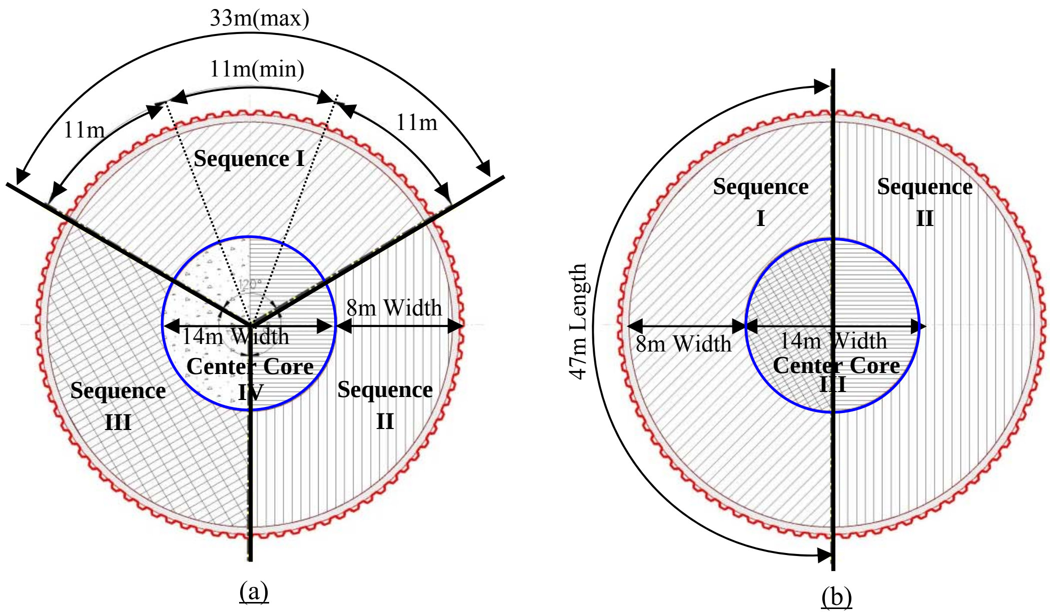
## 5 SHAFT CONSTRUCTION AND ITS CHALLENGES

### *5.1 Construction method and period*

Sheet piles were constructed using pre-drilling and driving methodology. As the soil at the level of sheet pile toe was expected to be of more than SPT N-value of 40, pre-drilling was carried out to facilitate the driving of sheet piles into the harder soil to reach the design depth and prevent damage to the sheet piles. Shafts were excavated using the mechanical excavation method assisted by the hydraulic excavator. Hydraulic buckets were adopted to excavate the soil. However, the deeper OA is generally stiffer and hydraulic breaker or claw-hook was adopted instead. This stiffer OA soil layer has SPT N-value of more than 80. Ring beams at the upper shaft and strengthening structure around the tunnel openings were constructed using conventional cast in-situ concrete method while caisson rings at the lower shaft were constructed using sprayed shotcrete method except for PIE shaft which was constructed using cast in-situ concrete method.

Sequential Excavation Method (SEM) was selected to construct the large diameter shafts. Shaft excavation area was divided into several parts around the shaft, with a center core. The casting of rings was carried out sequentially upon completion of the excavation at a particular area. Excavation of the shaft was schematically presented in Figure 6.

At the lower shaft of P-Shaft and S-Shaft, upon completion of the excavation, it was followed by the application of 50mm sealing shotcrete to the unsupported advance face within 4 hours of excavation in order to minimize the ground movement. Ring was casted at the excavated area, and at the same time excavation commenced to the adjacent side. These sequences were repeated until full caisson ring was completely cast. Subsequently, the center core excavation was carried out while waiting for the minimum structural strength of 16MPa to be achieved.




# Minimum and maximum excavation length shall be adjusted according to the stability of unsupported advance face.

Figure 6. Proposed sequential excavation method

For the PIE Shaft, the contractor preferred to construct the lower shaft using cast in-situ method to have better control over the geometry of the shaft. Steel formwork was fabricated to facilitate the casting of the concrete elements. The formwork will be assembled in the shaft prior to every casting and this will form a uniform inner diameter through the entire shaft, hence all layers of ring beams and caisson rings were redesigned to ensure that the final inner diameter was maintained. In this case, standard ring beams size of 0.8mx0.85m (TxH) was adopted in order to use the standard formwork.

In order to maintain the verticality and lay the formwork against the above completed ring, the orientation of the strengthening columns (as thickness is more than that of typical caisson ring) adjacent to the tunnel openings were adjusted so that the inner face will be to be flushed with the caisson ring above and the outer face was adjusted to be protruded into the soil side. The column width was also thickened to 1.32m from 0.8m accordingly. Consequently, the total excavation diameter of lower shaft was reduced from 30m to 29m.

Table 5 summarizes the construction period taken by the sprayed shotcreting method (at P-Shaft and S-Shaft) and cast in-situ concrete method (at PIE Shaft). During the construction stage, full area excavation instead of SEM was carried out at PIE Shaft. As the soil became harder as the excavation went deeper, longer excavation time was taken to complete the full area excavation. Even though the excavation area at PIE Shaft is slightly smaller, but the period to excavate and construct each typical caisson ring appear to be quite similar for both construction method – sprayed shotcreting method with SEM and cast in-situ concrete method with full area excavation.

For the case of caissons below the strengthening ring beams (just above the tunnel opening), the construction time is also affected by the construction of strengthening columns. In PIE shaft, the cast in-situ lining was used for lower shaft and hence casting of caisson rings and strengthening columns can be done together. This apparently made it easier to construct and also provided better workmanship compared to that of spray shotcrete method.

Table 5. Summary of construction period at the three shafts

| Shaft | Construction Method | Location | Days per ring for completion |
|---|---|---|---|
| P and S Shaft | Spraying Concrete Lining Method | Rings (above strengthening beam) | 4-8 (typically in 5 days) |
| | | Rings (cast together with strengthening column) | 10-18 (typically in 13 days) |
| PIE Shaft | Cast in-place RC Method | Rings (above strengthening beam) | 3-6 (typically in 5 days) |
| | | Rings (cast together with strengthening column) | 6-10 (typically in 6 days) |

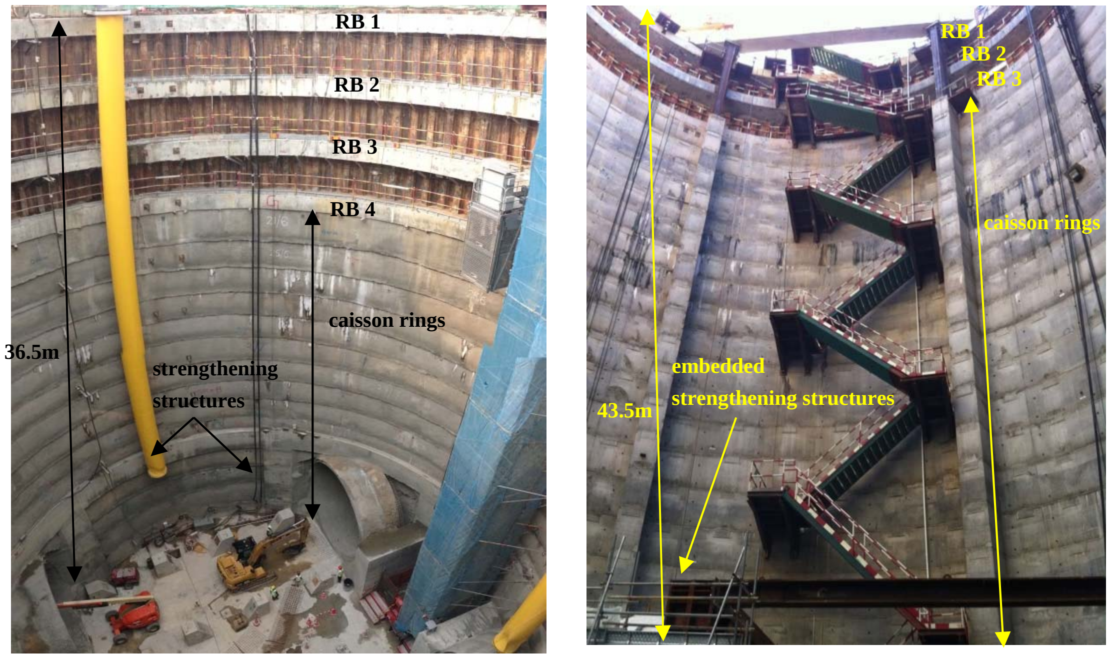



Figure 7. Completed sprayed shotcrete structure of S-Shaft and cast in-situ concrete structure of PIE Shaft

### 5.2 *Ensuring circularity of large diameter shaft*

Sheet piles are flexible and they are likely to deviate from their intended position during driving if site control is not carried out properly. In this project, the circular sheet pile wall appeared to have displaced when exposed during excavation. Therefore, the as-built position of sheet pile wall at ring beam level was verified before casting the ring beam. It was found that the total deviation of some piles from their design positions was larger than the allowable tolerance of 1:75. Consequently, it would cause non-circularity of the shaft and induce additional bending moment on the ring beams due to the additional eccentricity which was greater than the allowable limit. During construction, for areas where the sheetpile is deviating away from the theoretical line (or the excavation), the excess volume will be filled up with concrete while the rebar cage was adjusted as close to the theoretical line as possible prior to casting of the ring beam so that the eccentricity of the ring beam will be reduced to 150mm which is the maximum design limit. With this control, circularity of ring beams was maintained as much as possible to reduce the additional forces due to eccentricity.

It is generally difficult and challenging to form a circular profile when adopting the sprayed shotcrete method for construction as compared to that formed by conventionally cast in-situ concrete. During spraying, some forms of interactive system were implemented to check the profile and thickness of sprayed shotcrete.

Caisson rings at P-shaft and S-shaft were formed and cast adopting the following system. After completion of excavation, a set of 2 rebars were installed at 1.5m c/c along the circumference of shaft to aid in measuring the thickness and fixing the wire mesh. Metal plummets were spaced at every 3m c/c along the circumference of shaft and hanged down from the top survey station at the 1$^{st}$ ring beam. Profile and thickness of spray shotcrete were checked by forming a mesh system of 0.5m in vertical direction and 3m in horizontal direction based on the aforesaid fixed point of plummets. A minimum 50mm initial (sealing) coat was sprayed upto the theoretical installation line of the wiremesh. Occasionally, thicker coat was sprayed to backfill the overbreak or irregular excavation. Both layers of wiremesh were laid according to the theoretical installation line to maintain eccentricity within the allowable limit and circularity. Shotcrete was sprayed at 2-3 layers at 150-200mm thickness and each profile of shotcrete was checked as per aforesaid method. Construction record shows that construction tolerance of shotcrete surface ranges from -0mm to +100mm using this method which has provided good control over the profiling.

Caisson rings at PIE Shaft were formed and cast adopting the following system. Setting out the excavation line and verticality was controlled by using Optical plum method. Upon completion of 50mm thick sprayed sealing coat, wire meshes were positioned and the formwork was assembled. The position of formwork was checked using the plumbing system situated at every 2m along the perimeter of the last beam (at the upper shaft) and the concrete lining will be cast if the entire set-up is in order.

### 5.3 *Ensuring stability of vertical face during excavation*

At the P-shaft and S-shaft, the caisson was typically constructed by sequential excavation. The initial 1/8 length of the shaft perimeter or 11.8m of excavation for the caisson was excavated. Checking on the stability and seepage condition of exposed face was then carried out. The exposed face was observed to be quite stable and there was no trace of high permeable layer found. Thereafter, subsequence excavation length in the circumferential direction was adjusted to 24m and then raised up to 35m, depending on the observation and ground reaction. Under all cases, a 50mm sealing was sprayed to the exposed face within 4hours to control the ground movement. In some area, though the 50mm shotcrete was applied and left standing for about 12hours, no instability of exposed face was found and the movement of ground was also considered to be minimal.

At the PIE Shaft, before commencing the excavation below the sheet pile, the trench excavation was carried out to check the face stability and sand lenses. As the exposed face was stable and no trace of sand lenses was found, full face excavation was subsequently carried out. A 50mm sealing was sprayed to the exposed face. Though the face was left for about 2-3days (fixing up of reinforcement cages) before casting, no instability of exposed face was observed and the movement of ground was also considered to be minimal.

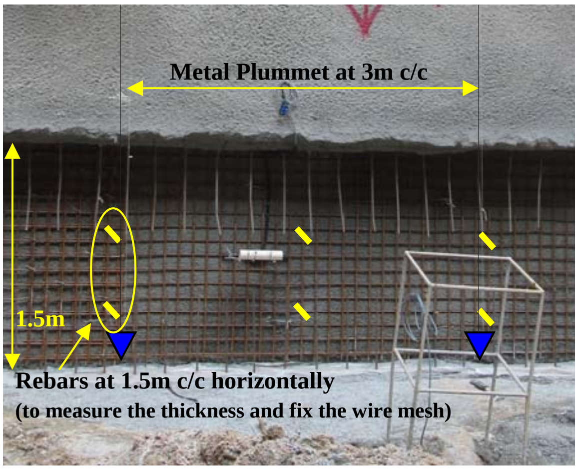



Figure 8. Measuring system of shotcrete profile


Figure 9. Picture showing exposed soil face

### *5.4 Ensuring strength of sprayed shotcrete*

In most projects, lining thickness of less than 400mm is conventionally applied for SCL tunnel in soft ground (i.e. soil and weak rock). Design thickness of 500-600mm in this project is considered to be thicker than its usual thickness; and therefore, it is important to establish the right spraying technique and control on quality especially for manual spraying in order to achieve the intended thickness and strength. Wet mix sprayed concrete was adopted in this project to permit greater control over quality. The quality of sprayed shotcrete is greatly influenced by the skill of the nozzleman and spraying technique. Before shotcreting work, test panels were conducted to verify the competency of the nozzleman and spraying technique.

Each layer of shotcrete (max up to 200mm thick) was built up by making several passes of the nozzle. The flow of shotcrete from the nozzle was maintained in a steady uninterrupted flow. Before processing the succeeding layers, the completed layer was set and all rebound and loose material at the surface and joints were removed and cleaned using combination of air and water. Distance between the nozzle man and surface was maintained within 1.5m. Shotcrete was applied from the bottom to the top as the surface is vertical.

At the beginning of spraying, it was observed that the shotcrete at the bottom 50mm of caisson ring had low strength and could easily be pulled out by hand, which is shown in Figure 10. This poor quality was expected due to the difficulty of handling the nozzle close and perpendicular to the surface of bottom. This loose end layer was cut-off and rectified. In addition, the distance and nozzle angle of spraying was adjusted to control the quality. Apart from this minor problem, the intended thickness and strength of shotcrete was able to be achieved. The recorded shotcrete compression strength development with time is presented in Table 5 with a comparison to a theoretical curve by Chang (1994). It appears that the strength development varies having almost similar mix, and most are higher than that of Chang. The actual strength development curve mainly depends on the shotcrete mix and additives, and in this case, higher early strength is generally what the contractor would like to achieve in order to commence the next level of excavation.


Figure 10. Quality of shotcreting

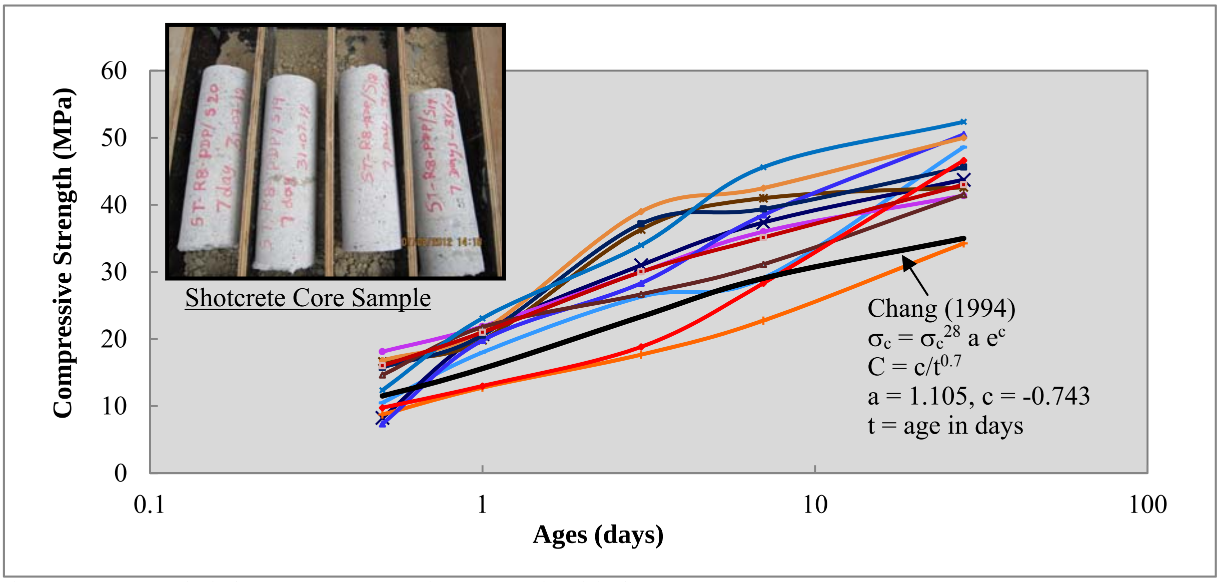



Figure 11. Recorded shotcrete compression strength with time

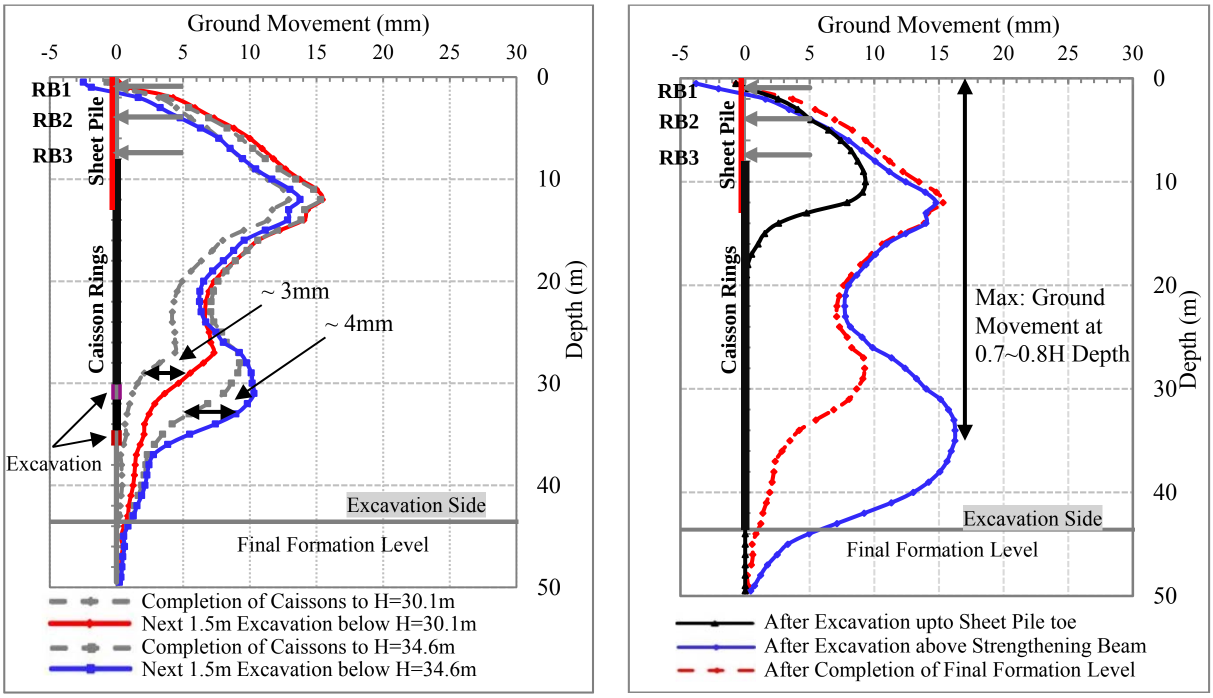
## 6 OBSERVED BEHAVIOR OF GROUND AND SHAFTS

### *6.1 Ground Movement Around Shaft*

Subsurface ground movement due to shaft excavation is interpreted based on the results from the inclinometers and extensometers which are installed at approximately 2m and 20m away from the shaft. Figure 12 presents that the ground displacement recorded and the movement is usually observed during every level of excavation of which was initially unsupported. As the sheet pile wall retained the upper weaker soil layers, the observed ground movement after completion of excavation till the end of sheet pile wall was observed to be less than 10mm of which is quite small considering the large shaft diameter.

The maximum horizontal ground movement due to the excavation of caisson ring was found to vary from 2mm to 5mm based on the unsupported excavation before casting of ring was completion. As the construction period of installing the strengthening beam and caisson rings with strengthening column was longer as compared to that of typical caisson rings, the observed accumulative maximum ground movement was located generally at 0.7~0.8H (i.e. H=Excavation depth) below ground level and less than 20mm. The ground movement observed at the inclinometer that was installed at 20m away was almost negligible.

The maximum surface settlement was less than 30mm near to the shaft and reduced to about 1~5mm at 30m away (i.e. one time diameter, 1D). According to these monitoring records, the influence zone of circular shaft appears to be located at about 1 x diameter from the shaft perimeter.




(a) Ground movement behind completed ring above due to next 1.5m excavation below

(b) Ground movement induced at the different stages of excavation

Figure 12. Observed ground movement interpreted from record of inclinometers (Installed at 2m(approx.) behind sheet pile)

### *6.2 Shaft displacement*

The distortion and displacement of shaft is interpreted based on the record of prisms. Though prisms were installed only after casting of rings, the initial reading was taken just after the excavation and completion of casting of the immediate caisson ring below. Thus, movement shown by prisms does not capture the movement due to the excavation of the next immediate caisson ring below.

With the monitoring on construction sequence and site utilization at both the P-Shaft and S-Shaft, the observed movement on the prisms at Ring Beam 1 was found to be at the maximum during the muck pit construction. This induced movement is registered towards the muck pit side and it showed about 5mm in S-Shaft and 18mm in P-Shaft. The movement was diminished after RB2. Effect of construction of cranes foundation on the shaft was observed to be insignificant.

At PIE shaft, removal of the potential impact of the shaft due to loadings from the site utilization was adopted as additional preventive measures. The construction of muck pits was planned at the initial phase of the project prior to shaft excavation. This will minimize any unbalanced loading acting on the shaft. Also, during TBM lowering operation, the operation of gantry crane and muck pit was temporarily stopped to reduce any potential additional unbalanced loads. Based on these additional measures, no significant movement of the upper shaft was observed.

The effect of sequential excavation during the next level of excavation on the already completed caisson ring above was determined based on the monitored results as indicated in the inclinometers and prisms. As presented in Section 6.1, the maximum inclinometer movement was about 5mm at each level of excavation level and the corresponding prisms (at ring beam or caisson) moved about 1mm relatively. As the observed movement was quite small, the impact of SEM to the completed ring above the excavation was found to be insignificant. The maximum recorded movement of caisson rings is 15mm radially, and this is less than the allowable 20mm.

Some of the readings on the strain gauges embedded in the concrete or shotcrete were erratic and did not appear to be representative and the results were difficult to be interpreted, therefore the data was not presented and discussed in this paper.

## 7 CONCLUSIONS

Three numbers of large diameter circular launching shafts of 30m were constructed under this project for TBM launching. Although circular retaining system is known to be one of the most stable and economical retaining system, several challenges were encountered during the design stage as well as construction stage. This paper has addressed the important design considerations such as uneven topography and geological condition, additional loadings due to site activities and; unbalanced loading arose from the construction activities. Various construction challenges including the proper control measure to maintain the circularity of shaft and stability of exposed face are also discussed. Two of the shafts under this project had adopted shotcrete caisson wall as a retaining system for lower shafts and the successful execution of this method has proven the applicability and buildability of deep circular shaft using a shotcrete wall as retaining system. In addition, four numbers of openings were constructed at the lower shaft in order to facilitate TBM launching. Both the design and construction of strengthening structures around these openings are also mentioned. Comprehensive monitoring system was provided for these large shafts. The ground movement due to shaft excavation was found to be minimal, with the influence zone extending up to about 1 diameter away.

## ACKNOWLEDGEMENT

The authors would like to thank and to express their sincere appreciation to Land Transport Authority for permission to publish this paper.

## REFERENCE

Alun Thomas.2008. *Sprayed concrete lined tunnels. E-Library*: Taylor & Francis.

B.M.Das. 1983. *Theoretical foundation engineering. Developments in Geotechnical Engineering Vol 47*: Elsevier Science Publishing Company Inc.

Britto. A and Kusakabe. O. 1983. Stability of axisymmetric excavation in clays. J Geotech Engrg, *American Society of Civil Engineering:* 109(5) 666-681.

Chang. Y and Stille. H. 1993. Influence of early age properties of shotcrete on tunnel construction sequences. *American Society of Civil Engineers: Reston, pp*: 110-17.

Chu et al. 1993. Engineering properties of the old alluvium soil. *Underground Singapore 2003 27-28 November 2003*: 285-295.