# Geology and Its Impact on the Construction of Singapore MRT Circle Line

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**ABSTRACT:** The Circle Line (CCL) is a fully underground railway line in Singapore connecting the inner suburban areas of the city. It is 39.5 km long with 34 stations and built in six separate packages. Site investigations comprising boreholes, CPTs and geophysical surveys for the project were carried out in various phases to reveal ground conditions along the route and decide on the construction methods. Extensive field and laboratory testing were also carried out to establish geotechnical design parameters. This paper summarises geological conditions encountered along the CCL route and highlights the effect of geology on selected construction methods.

**KEYWORDS:** Site investigation, Construction methods

## 1. INTRODUCTION

The Circle Line (CCL) is Singapore's fourth Mass Rapid Transit (MRT) line (Figure 1). This fully underground line is 39.5 km long with 34 stations and is fully automatically operated. Station abbreviations are given underneath the station names in Figure 2. The Circle Line is the first medium capacity line in Singapore. As a medium capacity line, each Circle Line train has only three cars instead of the six-car configuration as seen on other currently operating MRT lines excluding the Downtown Line. As the name implies, the line is an orbital circle route linking all radial routes leading to the city. However, the line has a branch from Promenade station to Dhoby Ghaut station.


Figure 1.The Singapore MRT Circle Line (CCL)

The CCL project was planned in six stages. Stage one (CCL1) of the project included 5.4km long and six stations, from Dhoby Ghaut to Stadium. The second stage (CCL2) which is 5.6km long included five stations from Stadium to Bartley. The stage three (CCL3) was a five station segment stretching 5.7km between Bartley and Marymount. Stages four and five (CCL4 & CCL5) involved 17km length stretch from Marymount to HarbourFront, with 13 stations including a future station at Bukit Brown.

Construction first started in April 2002 and the line (Stages 1 to 5) commenced operation on 8$^{th}$ October 2011 at a cost of nearly S$10 billion.An extension of CCL from Promenade to Marina Bay had been constructed recently with another two new stations, Bayfront, and Marina Bay. The 4km long Stage six (CCL6) which is currently at design stage will "close the circle" by connecting Marina Bay to HarbourFront when it is completed in 2025.

Since the CCL6 has not been constructed, the geology and its impact on the construction methods for only the CCL1-5 are presented in this paper.

## 2. GEOLOGY ALONG CIRCLE LINE

### 2.1 Investigations

The CCL1-5 route and stations are overlaid on the surface geology of Singapore map in Figure 2. As can be noted from the figure, the CCL route is located within all four major geological formations in Singapore: (a) Kallang formation (marine clay, fluvial clay, fluvial sand, estuarine), (b) Old Alluvium, (c) Residual soils and completely weathered to fresh rock of Bukit Timah Granite, and (d) Residual soils and completely weathered to fresh sedimentary rock of Jurong formation. Part of the route in the south is located within the reclamation area which was reclaimed in various phases from 1970s to 1990s. The soft marine clay underneath the reclamation fill, typically sandy soils, is still undergoing consolidation due to additional loads from the weight of reclaimed fill.

Site investigations comprising boreholes, CPTs, geophysical surveys and field and laboratory testings to reveal ground conditions and their geotechnical design parameters were carried out along the route in early 2000. These investigations were carried out in several phases including various types of investigation methods as described by Jeyatharan et al. (2003).

Before the civil construction contracts were awarded, an over 22 km total length of borehole drilling was carried out in about 700 boreholes. Furthermore, over 5 km of geophysical surveys were also carried out. After civil construction contracts were awarded, further site investigation works were carried out by each of the civil contractors to verify the ground conditions.

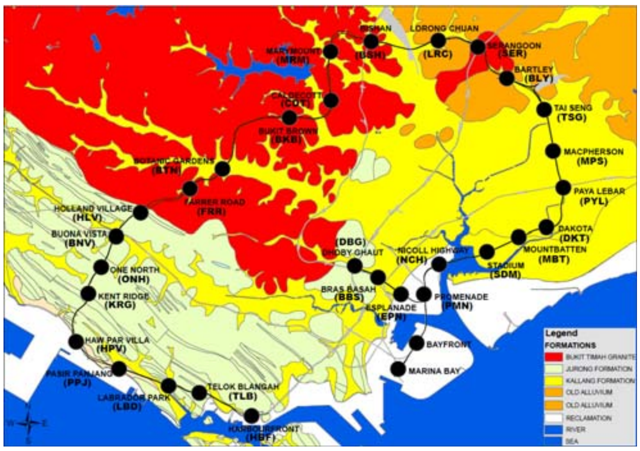

Figure 2  Station abbreviations and Surface Geology of Singapore

### 2.2 Brief Geology

A cross section showing the various geologies encountered along the CCL tunnels and stations is shown in Figure 3. Geology along CCL1 route is predominately within the Jurong from DBG to EPN; and Old Alluvium and Kallang formation from EPN to PRN; and Kallang formation from PRN to MBT. Part of the Kallang formation in this stretch is undergoing consolidation due to recent reclamation (from 1970s to 1990s). Localised Fort Canning Boulder Bed (FCBB) is also observed at Bras Bash Station.

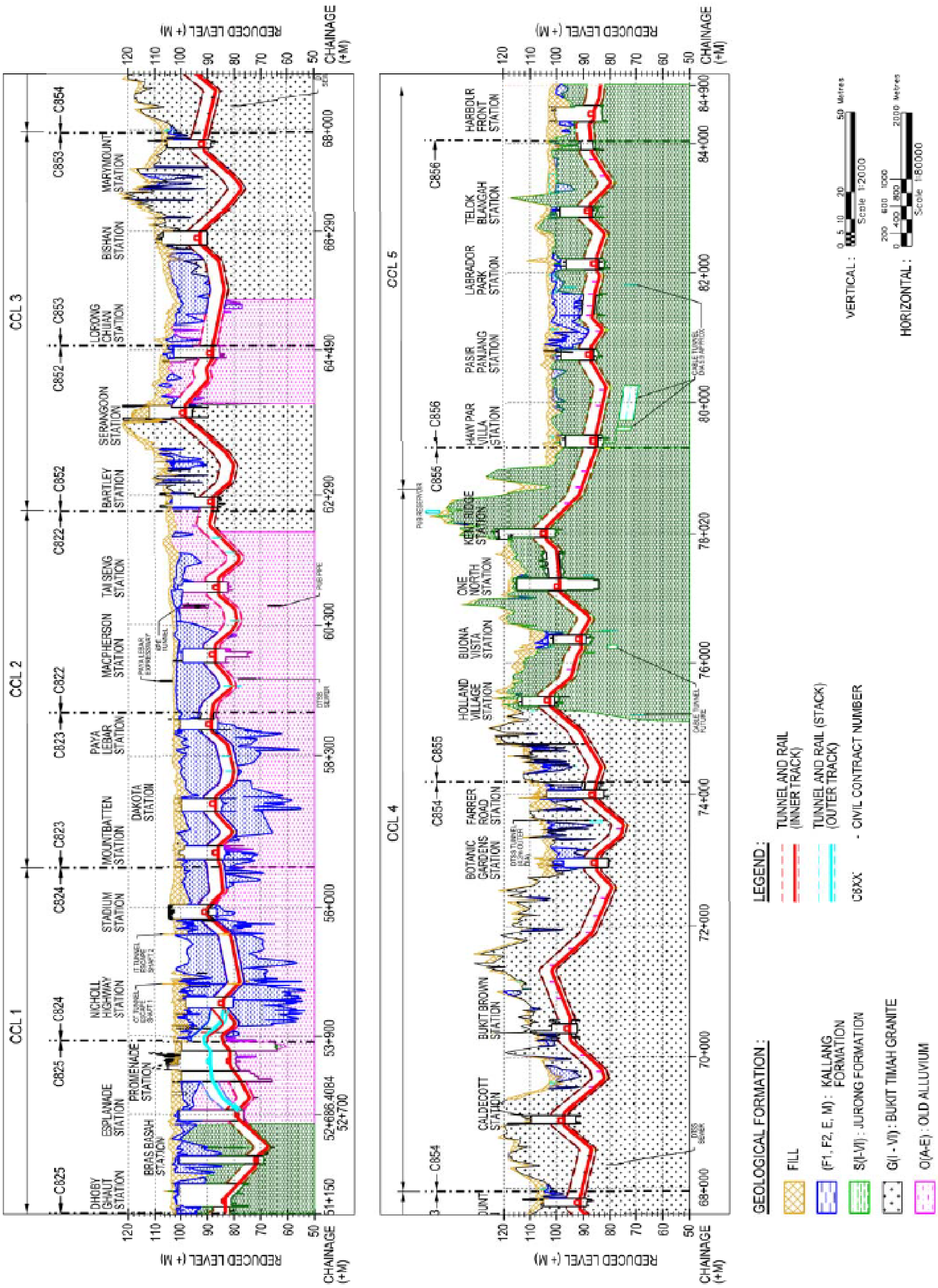


Figure 3 Geological Profile along the circle line

Geology along CCL2 route is predominantly within Kallang formation from MBT to MPS; and predominantly within Old Alluvium from MPS to BLY with Kallang formation in few localised valleys.

Geology along CCL3 route is predominantly within the residual soils and completely weathered to fresh Granite rocks except for a stretch after SER to half way between LRC and BSN where the route is within the Old Alluvium. Localised valleys where Kallang formation is noted especially between LRC and BSN.

Geology of CCL4 until just before HLD station is predominantly within the residual soils and completely weathered to fresh Granite rocks with some localised valleys where Kallang formation is observed.

Geology of the remaining CCL4 from HLD and CCL5 is predominantly within the residual soils and completely weathered to fresh sedimentary Jurong rocks with some localised valleys where Kallang formation is observed. Two major faults, Peppy's fault and Henderson fault, are identified along the CCL5 route (DSTA 2009 and Moe et al,. 2003). During the site investigation, localised limestone is also identified near KRD and HPV stations.

### 2.3 Major Formations in Singapore

#### 2.3.1 Bukit Timah Granite (BTG)

The Bukit Timah Granite (BTG), which is about 220 million years old, is found at the centre of the Singapore Island and at Pulau Ubin (DSTA 2009). The rocks in this formation include acid rocks such as granite, adamellite and granodiorite, and also hybrids of acid and intermediate rocks (granodioritic and dioritic). This formation is intruded by dykes.

In many places, the BTG has been deeply weathered, 30 to 40m deep, and often boulders are encountered within the residual soils and completely weathered granite. The interface between the soils and rock is highly variable within a short stretch and the change in strength of soil to rock is very drastic.

#### 2.3.2 Jurong Formation

The Jurong formation, which is about 190 million years old, is exposed mostly at the western and southern parts of Singapore. The formation consists of a variety of sedimentary rocks of which six facies (units) have been identified: Queenstown, Jong, Ayer Chawan, Rimau, St John and Tengah.

The structure of the Jurong formation mainly comprises a series of open folds, but also includes isoclinal folds and overfolds. The general strike is NE-SE, but the dips may vary over short distances from a few degrees to vertical or even overturned. There are three major thrust faults in the Jurong Formation (DSTA 2009). Apart from these, there are numerous small scale faults which are mostly wrench faults with offsets of up to 5 to 6m.

#### 2.3.3 Fort Canning Boulder Bed

It comprises sandstone boulders in a matrix of stiff over-consolidated silty clay. The deposit was found in the central business district, around Raffles Place and City Hall MRT stations. The boulders are usually fresh and about 2 to 3 $m^3$. At shallow depths, the proportion of clay to boulders is about 40 – 50%, while at depths, the proportion increases to 90%. The thickness of this deposit is at least 135m at the mouth of the Singapore River (Shirlaw et al 2003a).

#### 2.3.4 Old Alluvium

The Old Alluvium, which is probably about 2 to 7 million years old, extends from Southern Johor, Malaysia to east of Singapore. It was found to lie at a depth of 150m below mean sea level, and its thickness can reach up to 195m (DSTA, 2009). The formation consists of clayey sand with pebbles. The sand is coarse, angular and mainly quartzo-feldspathic. The pebbles are predominantly quartz and generally angular. However, ryholite, chert and argillite pebbles are also found, but they are sub-rounded or rounded.

#### 2.3.5 Kallang Formation

The Kallang formation is the most recent formation and was deposited in river courses, coastlines and some inland low-lying areas. The Kallang formation deposited within the deeply eroded valleys into the older formations. Deposition is believed to have started about 18,000 years ago. There are five members recognized within the formation namely marine, alluvial, littoral, transitional and reef. The marine member is locally referred as soft marine clay. The base of the member is usually characterized by a peaty clay and sand layer. Thin sand and peat layers may occur within the sequence. The marine member is sometimes separated by an intermediate layer of reddish brown clay. This intermediate layer divides the marine member into upper and lower marine members.

The alluvial member deposits vary from pebble beds through sand, muddy sand, clay and peat. It is found to be extensive in Kallang and Jurong river basins. The littoral member includes those sediments deposited in active coastal regions as beach deposits, immediate offshore deposits and as tidal sand banks. The member consists mainly of clean sand and pebbly sand. The transitional member is found in the river mouths and tidal swamps surrounding Singapore. It consists of black to blue-grey mud, muddy sand or sand with a high organic content. Extensive areas of this member can be found on the western end of Singapore, Pulau Ubin, Pulau Ketam and Pulau Seletar which is typified by mangrove swamps. The reef member is essentially coral sand and can be found in the south-western islands.

## 3. CHOICE OF CONSTRUCTION METHODS

Major CCL construction activities are categorized into two main types: cut-and-cover excavation and tunnelling. All stations as well as cross-over and siding tunnels were constructed using cut-and-cover method while most of the running tunnels were constructed by bored tunnelling method. Conventional mining method was adopted to construct all cross passages and a cross-over tunnel in front of ONH station but it is not discussed in this paper.

### 3.1 Cut and Cover Excavations

A brief summary of the major temporary Earth Retaining or Stabilising Structures (ERSS) adopted for the cut and cover excavations for main stations and their details such as type of retaining wall, no of struts and thickness of grout, if any, are summarised in Table 1. Where deep soft marine clay exists basal instability would be a concern. In Singapore, this has been addressed by providing grout slabs between the temporary retaining walls. Typically, the grout layer is below the final formation level. However, in some cases an additional layer of grout (sacrificial layer) is required above the final formation level which would be removed during the constriction.

Typically excavation depths for a majority of stations were about 18 to 25m except few deep stations. The table does not include the scheme adopted for the excavation to construct the entrances which were typically connected to concourse level with an excavation depth of about 10 to 12m below ground level.

#### 3.1.1 Choice of Wall Types

All CCL 1&2 stations except TSG station used diaphragm wall as temporary ERSS for excavation. All CCL1 stations except SDM station were constructed by top-down construction method and the diaphragm wall was designed as part of the permanent walls. Thickness of the diaphragm wall varied from 0.8m to 1.5m depending on the thickness of Kallang formation, depth of excavation, thickness of JGP and number of strut levels. The TSG station used CBP as the temporary ERSS for the excavation to construct the station structure.

Various types of temporary ERSS walls were adopted in the CCL3 such as diaphragm walls, secant pile walls, contiguous bored pile walls and soldier pile walls plus sheet pile walls depending on the ground conditions and contractors' preference. Generally no jet grouting was adopted as the soft Kallang formation is limited in thickness.

Six out of thirteen stations in CCL4&5 adopted diaphragm walls as temporary ERSS for excavation and the same diaphragm walls were designed as part of the permanent structure of the stations. For the remaining stations, the contractors adopted either contiguous bored pile wall or soldier/sheet pile walls as temporary ERSS walls for excavation.

Table 1 Details of cut and cover station excavations

| Station | Max exca. depth (m) | Typical Geology | ERSS wall type | No of struts | Thickness of JGP (m) |
|---|---|---|---|---|---|
| DBG | 23 | KF (10m), JF(VI - V) | DW | T/D | Nil |
| BBS | 35 | KF (22m), FCBB | DW(1.2 - 1.5m) | T/D | 2 x 2m |
| EPN | 25 | KF (10m), OA | DW | T/D | ? |
| PMN | 25 | KF (30m), OA | DW | T/D | ? |
| NCH | 20 | KF (40), OA | DW (1.5) | T/D | 7m |
| SDM | 19 | KF (20m), OA | DW | 5 | 1.5 – 2m |
| MBT | 19 | KF (30m), OA | DW (0.8m) | 5 | 1.5 - 2m |
| DKT | 18 | KF (40m), OA | DW (1.0m) | 6 | 2 - 3m |
| PYL | 18 | KF (30m), OA | DW (1.0m) | 6 | 3 - 3.75m |
| MPS | 20 | KF (12m), OA | DW (1.0m) | 6 | ? |
| TSG | 21 | KF (5m) & OA | CBP (0.9m) | 6 | Nil |
| BLY | 21 | BTG(VI - II) | SBP (1.2m) | 5 | Nil |
| SER | 22 | BTG (VI/V) | DW (1.0m) | 5 | Nil |
| LRC | 23 | OA | DW (1.0 -1.2m) | 5 | Nil |
| BSH | 20 | BTG(VI - III) | CBP (0.8m) | 5 | Nil |
| MRM | 20 | BTG (VI - II) | Sheetpile+SP | 4 | Nil |
| CDT | 24 | BTG (VI - II) | CBP(1.0 - 1.2m) | 6 | Nil |
| BKB | 18 | BTG (VI - II) | CBP (1.0m) | 5 | Nil |
| BTN | 26 | KF (18m), BTG(VI - III) | DW(0.8 - 1.0m) | 6 | Nil |
| FRR | 28 | KF (5m), BTG (VI - V) | DW (1.2m) | 8 | Nil |
| HLV | 20 | JF (VI - III) | DW (1.0m) | 6 | Nil |
| BNV | 21 | KF (5m), JF(VI - III) | DW (1.0m) | 6 | Nil |
| ONH | 23 | JF (VI - III) | CBP (1.2m) | 6 | Nil |
| KRG | 35 | JF (VI - V) | CBP (1.2m) | 7 | Nil |
| HPV | 22 | JF (VI - V) | CBP (1.0m) | 5 | Nil |
| PPJ | 20 | KF (20m), JF(VI - III) | DW (1.0m) | 6 | cross-walls |
| LBD | 21 | KF(10m), JF(VI - III) | DW (0.8 - 1.0m) | 6 | Nil |
| TTLB | 18 | KF (5m), JF(VI - III) | SP + Sheetpile | 4 | Nil |
| HBF | 21 | KF (5m) & JF(VI – III) | SP | T/D | Nil |

CBP – Contiguous Bored Pile; DW – Diaphragm Wall; SP – Soldier Pile; T/D – Top-down construction method; ? – Information not available to the Author.

### 3.1.2 Choice of Grouting

Typically any excavation within the deep soft marine clay requires some form of grout slab to prevent basal heave (Page et al. 2006). Except DBG and TSG stations, all other stations in CCL1&2 are generally located within the deep Kallang formation of mostly soft marine clay and therefore Jet Grout Piles (JGP) forming grout slabs were adopted generally below the formation level to prevent basal heave. Considering the adjacent infrastructure, cross wall instead of JGP was successfully used in part of the excavations at PYL station to control the movement of adjacent MRT viaducts. Similar approach was also successfully adopted in the excavation for the PPJ station to control the movement of adjacent Pasir Panjang semi-expressway viaducts (Chua et al. 2009).

### 3.2 Issues faced During Construction

#### 3.2.1 ERSS Construction

Depth of Kallang formation varies drastically in places where there are buried valleys and therefore additional site investigations were required prior to installation of temporary ERSS walls to confirm their termination levels were adequately secured into the hard stratum. Without additional site investigations, the required depth of some type of ERSS walls could not have been verified whether they meet the design requirement to prevent potential toe kick-in.

Fluvial sand (F1) often exists within the Kalang formation either locally or extensively and it could potentially cause significant damages to any adjacent buildings/structures if wall construction is not carried out with adequate care. One such situation was where inappropriate methods and equipment adopted during ground anchor installations within the excavation led to F1 sands flowing together with water causing significant ground settlements behind the temporary ERSS walls.

Another situation was during diaphragm wall construction where the existence of F1 sands and other soft soils caused instability of diaphragm wall trench thereby causing damage to adjacent properties (Osborne et al. 2003). However, considering the extensive F1 sands and soft marine clay at the BTN station, the Kallang formation layer was grouted using deep soil mixing method prior to constructing the diaphragm walls as a proactive risk management approach to prevent any damage to adjacent sensitive buildings which are on mix foundations (Sebastian et al. 2006).

Constructing diaphragm walls is difficult when the granite rock is encountered above or close to formation level. This was evident from the selection of the ERSS wall type by the contractors who did not choose the diaphragm wall for BLY, BSN, MRM, CDC, BKB stations as the granite rocks are expected to be above the formation levels. Whereas for SER, BTG and FRR where the granite rock levels are expected below the formation levels, diaphragm wall was selected as temporary ERSS wall. However, the diaphragm wall construction progress at BTG and FRR was affected when boulders were encountered at a few diaphragm wall panels. Therefore, a detailed review of ground conditions including encountering boulders within the weathered granite shall be conducted while selecting construction methods and equipment.

#### 3.2.2 *Consolidation Settlements*

Chiang et al. (2006) and Ong and Osborne (2006) reported significant consolidation settlements outside the excavation at DBG, MSM, ESN, PRN and MPS stations caused by water drawdown in the underlying permeable layer (OA or F1) resulting from the stress relief due to the excavations. Another possible cause for the water drawdown in the permeable layers was the water flow through temporary ERSS walls either due to utility gaps or defects in the walls. It is therefore important that adequate length of temporary ERSS walls extended into the competent soils and all gaps in the walls are properly sealed. However, there could still be some water drawdown outside excavations due to construction defects and the use of recharge wells recharging into the permeable layers just outside the excavations would prevent any consolidation settlements. In the DBG and MSM station excavations, recharge wells were adopted as a contingency measure and they were very effective in controlling the consolidation settlements in the adjacent ground.

The interface layer between the rock and soil of the Granite is generally very permeable and may potentially lead to water drawdown causing significant ground settlement and/or ground loss. This was one of the risk identified during the design of the temporary ERSS for the MRM station excavation where a soldier pile plus sheet pile system was adopted. Loo et al. (2006) reported that packer grouting for rocks together with TAM grouting for soils complemented by a system of recharge wells were effective to mitigate this risk successfully at MRM excavations.

#### 3.2.3 *Flow through Gaps in the Walls*

CBP wall was adopted as a temporary ERSS wall for the BLY, CDC and BKB excavations within the Granite. Typically the CBP piles have a gap of 100mm between them. There is a potential risk of the soils flowing through those gaps between the piles since the residual soils (GVI) and completely weathered Granite (GV) generally consist of sandy silt/clayey sand which are more permeable. Following the loss of ground through the gap at BKB excavation, the contractor installed JGP columns between the CBP piles to close the gaps and to prevent the risk of any future loss of ground through the gaps.

Where stations or cut and cover tunnels are located underneath busy roads, it was inevitable to encounter utility services or cables during the construction of the ERSS walls. Since not all cables were diverted, the ERSS walls were constructed with some gaps to allow the cables to be supported during excavation. When these gaps were not treated properly especially where the retained soil is more sandy nature, water and soil flew through these gaps causing ground settlements behind the walls due to water drawdown and/or ground loss. It is therefore important that all these gaps are identified early and properly sealed during the construction to prevent any such problems at site.

#### 3.2.4 Collapse of ERSS

Part of a 211m length cut and cover tunnel of the Contract 824 at Nicoll Highway between NCH and SDM stations collapsed on 20$^{th}$ April 2004.

The stratigraphy at site consists of 3 to 6m thick fill overlying 30 to 40m deep Kallang formation followed by competent Old Alluvium material. The Kallang formation predominantly comprises of soft marine clay (upper and lower) with intermediate thin layers of fluvial deposits and estuarine. The site was reclaimed about 30 years ago and the soft marine clay was still undergoing consolidation.

The temporary ERSS to construct the cut and cover tunnel used 800 to 1000mm thick and 38 to 43m deep diaphragm walls to support the of 34.5m deep excavation supported by 10 levels of struts across the excavation and a 1.5m thick sacrificial grout slab and another 2.6m grout slab, both constructed using interlocking JGPs. After excavating the sacrificial grout layer and when excavation proceeded to the 10$^{th}$ strut level, the temporary ERSS collapsed and killed 4 workers.

The Committee of Inquiry (COI) investigated the incident concluded that the failure was contributed by several factors including technical and administrative factors. It highlighted two major design deficiencies: under-design of the diaphragm wall using "Method A" in a commercial finite element program (PLAXIS) and under-design of the waler connection in the strutting system (COI 2005). Method A is when someone uses effective stress parameters in a Mohr Coulomb model to represent undrained material behaviour. These design errors resulted in the failure of the 9$^{th}$ level strut-waler connections together with the inability of the overall temporary ERSS to resist the redistributed loads as the 9$^{th}$ level strutting failed thereby causing a catastrophic collapse of the temporary ERSS.

The contractor then relocated the NCH station by 100m sideways and realigned the associated tunnel alignment to connect to the constructed SDM station causing delay in the opening of the line for operation. At the same time, a detailed review of all the CCL ERSS was also conducted following the incident and additional measures introduced to enhance the robustness of ERSS to ensure safety of the works.

Following COI investigations, several recommendations were made which led to quite a few policy changes in the design and construction of temporary ERSS works in Singapore.

### 3.3 Tunnelling Works

Majority of the running tunnels except those cross-over and siding tunnels were constructed using bored tunnelling method as shown in Table 2. The bored tunnels were typically about 15 to 20m below ground levels except some sections where they were deeper either due to deep stations or crossing underneath existing underground structures or under hill terrain.

Generally, the cross-over and siding tunnels were constructed using cut and cover method except the one between ONH and KRG which was constructed by conventional mining method. All cross passages between bored tunnels were constructed using conventional mining method. Where the ground is either too weak in strength or too permeable, ground improvement or other treatment was carried out before any mining works.

Table 2 Details of bored tunnel drives

| CCL Stage | Contract | Drive | Tunnel Length (m) | Geology | Type of TBM |
|---|---|---|---|---|---|
| CCL1 | C825 | DBG-BBS | 320 | JF | EBPM |
| | | BBS-EPN | 560 | JF | EPBM |
| | | EPN-PMN | 400 | OA | EPBM |
| | C824/C828 | PMN-NCH | 480 | KF & OA | EPBM |
| | | NCH-SDM | 1070 | KF | EPBM |
| | | SDM-MBT | 640 | KF & OA | EPBM |
| CCL2 | C823 | MBT-DKT | 540 | KF & OA | EPBM |
| | | DKT-PYL | 1000 | KF & OA | EPBM |
| | C822 | PYL-MPS | 660 | KF & OA | EPBM |
| | | MPS-TSG | 790 | KF & OA | EPBM |
| | | TSG-BLY | 700 | OA | EPBM |
| CCL3 | C852 | SER-BLY | 1200 | OA & BTG | EPBM |
| | | LRC-SER | 700 | OA & BTG | EPBM |
| | C853 | BSH-LRC | 1510 | BTG & OA | EPBM |
| | | MRM-BSH | 1250 | BTG | Slurry |
| CCL4 | C854 | BKB-CDT, CDT-MRM | 3170 | BTG | Slurry |
| | | BKB-BTN, BTN-FRR | 3360 | BTG | Slurry |
| | C855 | ONH-BNV, BNV-HLV, HLV-FRR | 2900 | BTG & JF | Slurry |
| | | ONH-KRG, KRG-HPV | 1670 | JF | EPBM |
| CCL5 | C856 | LBD-PPJ, PPJ-HPV | 1760 | JF | EPBM |
| | | LBD-TLB, TLB-HBF | 2640 | KF & JF | EPBM |

#### 3.3.1 Choice of TBMs

Twin circular tunnels of 5.8m internal diameter are required between the stations for the MRT trains to travel. These tunnels were constructed by typically 275mm thick and 1.4m long 5+1 reinforced concrete segments using a shield Tunnel Boring Machine (TBM).

Tunnels with a total length of 21 km were constructed successfully using 14 nos of Earth Pressure Balance Machines (EPBM) under the MRT North East Line (NEL) project in 1998 - 2000. However, experiences from this project in using EPBM have shown difficulties in maintaining adequate face pressure in mixed ground conditions especially at interface between the soil and rock of Granite.

Considering this concern, the contractors chose slurry TBMs for those sections of tunnels which were driven through significant soil/rock interface of Granite. EPBMs were adopted for the rest of the tunnel drives which were mainly within Kallang formation, Old Alluvium, Jurong or residual soils and completely weathered Granite.

Typically, a pair of TBMs was used to construct twin tunnels between two adjacent stations in CCL1,2&3. However, long tunnel drives were adopted in CCL4&5 where a pair of TBMs was driven through partially completed two or more stations. This required careful prior planning in the station design, construction sequence and programme.

Excessive noise and vibration on to some buildings which are located directly above the tunnelling route due to the TBM operation were reported especially when tunnelling through strong rocks such as granite. Measures to reduce the noise and vibration shall be taken prior to any tunnelling through strong rock especially when building/structures are directly above the tunnelling route.

#### 3.3.2 Issues faced during tunnelling

##### 3.3.2.1 EPB TBMs

Shirlaw et al. (2003a) reviewed the settlements measured due to tunnelling in Singapore and noted that most of the cases of large settlements or sinkholes formation were directly related to the use of an inadequate face pressure. It was also noted that it was difficult to maintain the face pressure especially when tunnelling through the mixed interfaces of Granite, Jurong or these grounds combined with the Kallang formation. Rama Venkta et al. (2008) discussed the challenges faced during tunnelling through mixed interface of Jurong rocks and Fluvial sands (F1) of Kallang formation on the tunnel face underneath few old shop houses. However, with careful planning using air bubble technique with additional grouting to support the F1, the tunnelling was completed successfully underneath the shop houses.

In the NEL project, about 25% of sinkholes were occurred during TBM launching or docking. However, this risk has been identified and eradicated in the CCL project by constructing ground treatment blocks immediately next to the launching and docking shafts such that the tunnelling at the interface was carried out with the grouted mass.

Osborne et al. (2008) reported that the tunnel construction in CCL was slower than planned partly due to wear of machine, cutting tools and screw. The machines experienced wear as it cut through very abrasive Granite, Jurong and/or Old Alluvium and therefore its ability to cut the rock was reduced. Furthermore, the wear led to more tool changes which slowed down the tunnelling progress.

### 3.3.2.2 Slurry TBM

The interface between the soil and rock in Granite is highly variable within short distance and the change in the strength of soil to that of rock is also very drastic. Furthermore, the interface between soils and rocks is highly permeable with high pore water pressures. These make tunnelling through the mixed ground conditions in Granite very challenging and problematic.

There were several reports of either sinkhole formed on the ground surface or slurry escaped to the ground surface during tunnelling through mixed ground conditions. With the high groundwater levels characteristic of Singapore, there was a very small margin between the minimum pressure required to maintain face stability and the maximum pressure required to control the risk of slurry escape to the ground surface.

In Contract 855, a sinkhole was developed at the ground surface while tunnelling through a mixed face of fresh, extremely strong granite and the very granular completely weathered granite at Conwell Gardens. Subsequently, localised ground water control was introduced to improve the face stability so that tunnelling could continue underneath a number of buildings (Shirlaw et al. 2009). In the same contract, slurry escaped to the ground surface near Buona Vista area due to blockage developed between excavation and plenum chambers. This blockage caused sudden spikes in the slurry pressure in the excavation chamber causing the slurry to be pushed up to the ground surface. Following modifications carried out to the TBM the loss of slurry to the surface was subsequently minimised. Another cause of slurry blow out to the ground surface was through an old borehole which was not grouted properly or a subsurface geotechnical instrument which was close to the tunnel, providing a direct path for the slurry to escape.

To mitigate sinkholes on ground surface, it is important to have a good excavation management system and appropriate face pressure application. Shirlaw et al. (2009) has discussed various methods to estimate appropriate face pressure and also proposed mitigation measures such as dewatering to enhance the stability of the face. Nakano et al. (2007) successfully adopted a method based on percentage of soil-rock interface to estimate dry soil volume in their excavation management system in CCL contract C853.

Merrie (2009) and McChesney et al. (2008) summarised various construction difficulties during the slurry TBM application in the C855 tunnel construction. Damages to cutters and bearings of cutters were due to impact forces during tunnelling through mixed ground conditions in Granite and they suggested possible mitigations measures by increasing number and size of cutters and lowering cutter head rotation and torque.

They also discussed the potential blockage of the submerged wall or slurry pipes by boulders arising from tunnelling through granite rock, or clay build up from tunnelling through Jurong formation which has considerable clay content, or long length of GFR which could be dislodged from diaphragm walls, or HDPE grout pipes used for TAM type ground improvement.

Two slurry TBMs were used for a long drive through Granite and Jurong formations to construct twin tunnels, each about 2.7km long, from the One-North to Farrer Road pulling through BNV and HLD stations. The stretch of tunnels from One-North to HLD station went through Jurong soils and rocks where blockage within cutter head chamber, clogging the cutter disc and soakage slurry pipes were reported due to the Jurong formation rocks broke down to sticky clay.

## 4. CONCLUSION

The CCL route runs through all the major geological formations in Singapore: Bukit Timah Granite, Jurong formation, Old Alluvium and Kallang formation. It also encountered localised FCBB at Bras Bash station.

This paper summarises the ERSS schemes and TBM types adopted in different ground conditions in the CCL projects and highlights some of the problems faced during construction. Though these could be used as a guide for similar future excavation and tunnelling projects, the choice of ERSS scheme and TBM types shall be selected based on a detailed review of actual ground conditions.

Though diaphragm walls as ERSS walls for excavation worked well in deep soft Kallang formation in CCL project, they were not preferred in Granite where the walls are expected to be socketed into weathering grade III or fresher. CBP walls were also not preferred in Granite due to the gaps between the piles. Proper measures during construction should be taken to minimise settlements adjacent to the excavations during wall installations and excavations. Also gaps in the ERSS should be prevented to minimise water and/or soil leakage through the walls during excavations. In addition, proactive mitigation measures such as recharge wells shall be considered especially where soft compressible soils exist.

Considering the problems faced in tunnelling through the mixed interface of soil and rock of Granite in previous projects, slurry type TBMs were first time adopted in Singapore in CCL project to deal with the mixed interface with reasonable success though some construction difficulties were still faced in execution. In all the other ground conditions in the CCL project, EPB type TBMs were used successfully.

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