# MRT NEL C706, Design of Bored Tunnels along Race Course Road.

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SYNOPSIS: The northern section of the bored tunnels of Contract C706 of the Singapore MRT North East Line is located in varying soil conditions ranging from cemented Old Alluvium to very soft marine clay. The bored tunnels will run along Race Course Road, underneath a residential area underpassing several religious temples. The tubes will be constructed by earth pressure balanced shield machines. The paper is presenting the key approaches adopted for the design of the tunnel lining, the assessment of ground movements under the particular circumstances as well as the basic design considerations for the intersection of MRT tunnels with the future tunnels of the Singapore Underground Road System (SURS). In this context, the effect of water pressures in terms of sectional lining forces was studied in detail. A design chart has been developed to determine the sectional forces due to the water pressure load and in relation to the subgrade reaction modulus. An approach to consider the influence of the second tunnel has been developed and is presented in this paper.

## 1. INTRODUCTION

Contract C706 of the Singapore MRT North East Line (NEL) has been awarded to Hyundai–Züblin Joint Venture and comprises the tunnels and stations between Dhoby Ghaut station (DBG) in the south and Boon Keng station (BNK) in the north. The approximately 750 m long southern bored tunnel section starts from the start shaft at Kandang Kerbau station (KDK) and reaches DBG after underpassing hills of sandstones and mudstones of the Jurong Formation with varying degrees of weathering. In the middle section, from KDK to Farrer Park station (FRP), a cut and cover section will be constructed in Old Alluvium and in soft clay sediments of the Kallang Formation. The northern bored tunnels section starts from the start shaft at FRP station and reaches BNK after approximately 915 m. This tunnel section is located in Old Alluvium and soft clay sediments of the Kallang Formation.

The tunnel concrete lining, with 5.8m i.d., 1.5m width and 250mm thickness, consists of 5 segments and 1 key stone. For sealing purpose, each segment is fitted with one EPDM and one hydrophilic gasket, both placed close to the outside of the segments. The segments are connected with straight galvanized bolts in both joint directions. Two Herrenknecht EPB shield machines will be used.

In the Jurong Formation of the southern bored tunnel section, it is expected that the TBM can be run in open mode over some length due to good ground conditions. In the northern section, however, ground conditions and building settlement control will require that the TBM is operated in EPB mode along the entire tunnel length. In this area, sections with soft marine clay are located underneath Race Course Road, a densely overbuild residential area. This paper is describing aspects of the tunnel lining design carried out for C706 and concentrates on the northern tunnelling section.

The tunnel design in this area is distinguishing basically two different design cases. First, the regular tunnel construction and secondly the construction of tunnels of the Singapore Underground Road System (SURS) directly above the, at that time operating, MRT tunnels. Whereas the first case is considered to derive design values for sectional tunnel lining and verification of expected surface settlements, the second case is putting emphasis on the assessment of measures to limit and control the heave of NEL tunnels during SURS construction.

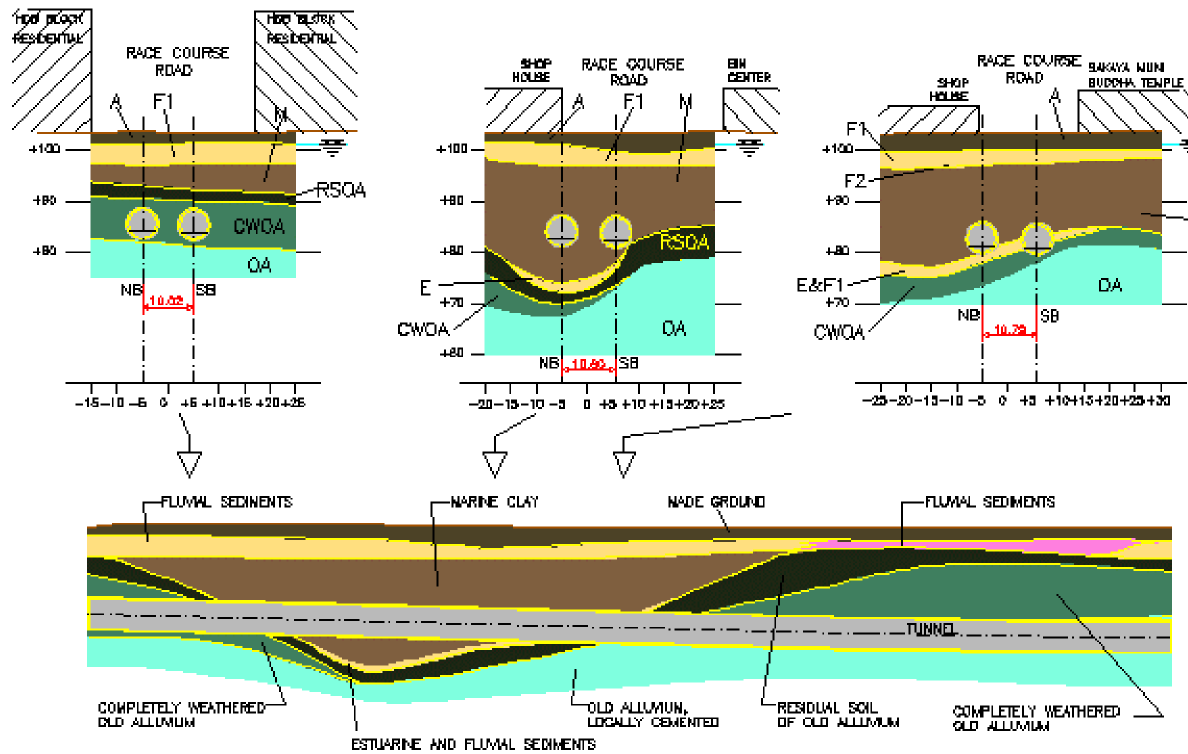


**Figure 1 Typical Geotechnical Sections in Race Course Road**

## 2. GROUND CONDITIONS

In the northern section, between FRP and BNK, the tunnels are located in the alluvial-coastal plain of the Rochor and the Kallang River, where the young deposits of the Kallang Formation overlie the sediments of the Old Alluvium. The upper layers of the Old Alluvium are completely weathered (CWOA) to residual soil (RSOA). The stratigraphic units of the Old Alluvium and the Kallang Formation are subdivided into geotechnical units based on the unified soil classification system (USC) and SPT N300 values. The natural water level is generally located approximately 1m - 3m below ground level. Hence, ground water head above the tunnel

**Table 1 Basic Soil Design Parameters**

| Formation | Fill | Kallang | | | Old Alluvium | | |
|---|---|---|---|---|---|---|---|
| Geotechnical Unit | **A** | **F1** | **F2** | **M** | **RSOA** | **CWOA** | **OA** |
| Unified Soil Classification | | SC, SM | CH, CL | CH | SC, SM, CL, ML | | |
| SPT N300 – values | | 5 - 30 | 0 – 5 | 0 - 5 | 5 - 25 | 25 - 100 | > 100 |
| Bulk unit weight, kN/m$^3$ | 18 | 20 | 20 | 16.50 | 21.5 | 21.5 | 21.5 |
| Undrained cohesion, kPa | | | 50 | 30 | 50 | 167.5 | 333 |
| Undrained Young's modulus, MPa | | | 10 | 6 | 20 | 67 | 133 |
| Poisson's ratio | 0.5 undrained, 0.3 drained | | | | | | |
| Drained angle of friction, ° | 30.0 | 30.0 | 22.0 | 22 | 30.0 | 35.0 | 35 |
| Drained cohesion, kPa | 10.0 | 0.0 | 0.0 | 0 | 0.0 | 5.0 | 20 |
| Drained Young's modulus, MPa | 12.5 | 11.0 | 8 | 5 | 15.0 | 50.0 | 100.0 |
| UCS, measured, kPa | | | | 0-70 | 0-600 | 17-800 | 76-2800 |
| Permeability, m/s | | $10^{-4}$ to $10^{-7}$ | $10^{-7}$ to $10^{-10}$ | $10^{-8}$ to $10^{-11}$ | $10^{-5}$ to $10^{-8}$ | | |

crown ranges between 12 to 19m and the soil overburden above tunnel crown is between 15 to 20m. The distance between the tunnel axes varies between 10.5 and 22m.

In the area of Race Course Road, the bored tunnels intersect two buried valleys, which were eroded into the Old Alluvium and subsequently filled with soft marine clay reaching a maximum thickness of up to 30m.

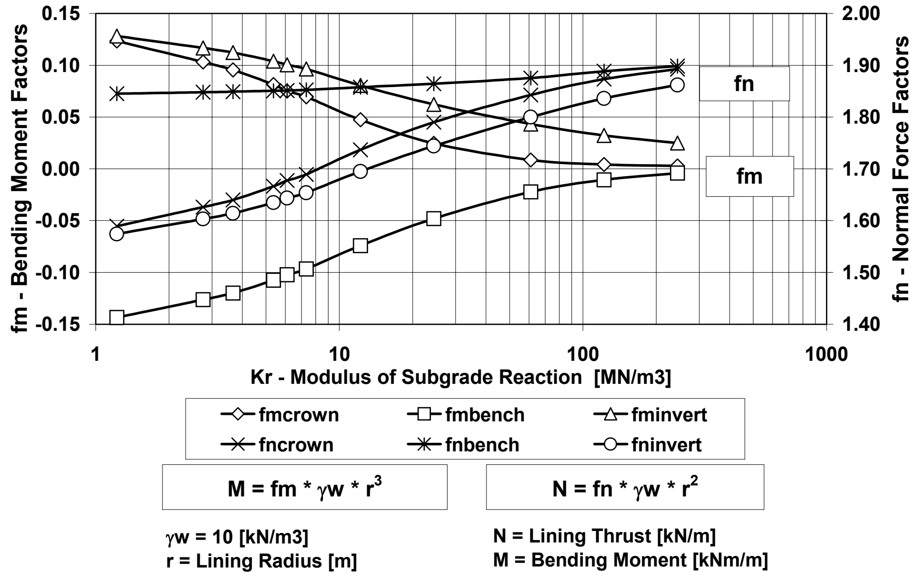


**Figure 2 Factors for Sectional Forces due to Water Pressure Loads**

The marine clay may locally overlie a layer of estuarine peat and fluvial sands, which were laid down directly on the weathered materials of the Old Alluvium. These buried valleys of marine clay represent the major tunnelling challenge in this area both, in terms of lining design as well as settlement control and building protection. Table 1 is presenting the basic geotechnical design parameters. Figure 1 is showing typical geotechnical sections in this area.

## 3. Tunnel Design

Lining design calculations have been carried out according to the bedded beam model after Duddeck and Erdmann (1982). The approach was applied considering tangential slip between the lining and the subsoil. This yields slightly larger bending moments, which was considered more appropriate to deal with the varying ground conditions of marine clay and Old Alluvium. According to the design criteria a surcharge, q, had to be applied to the ground

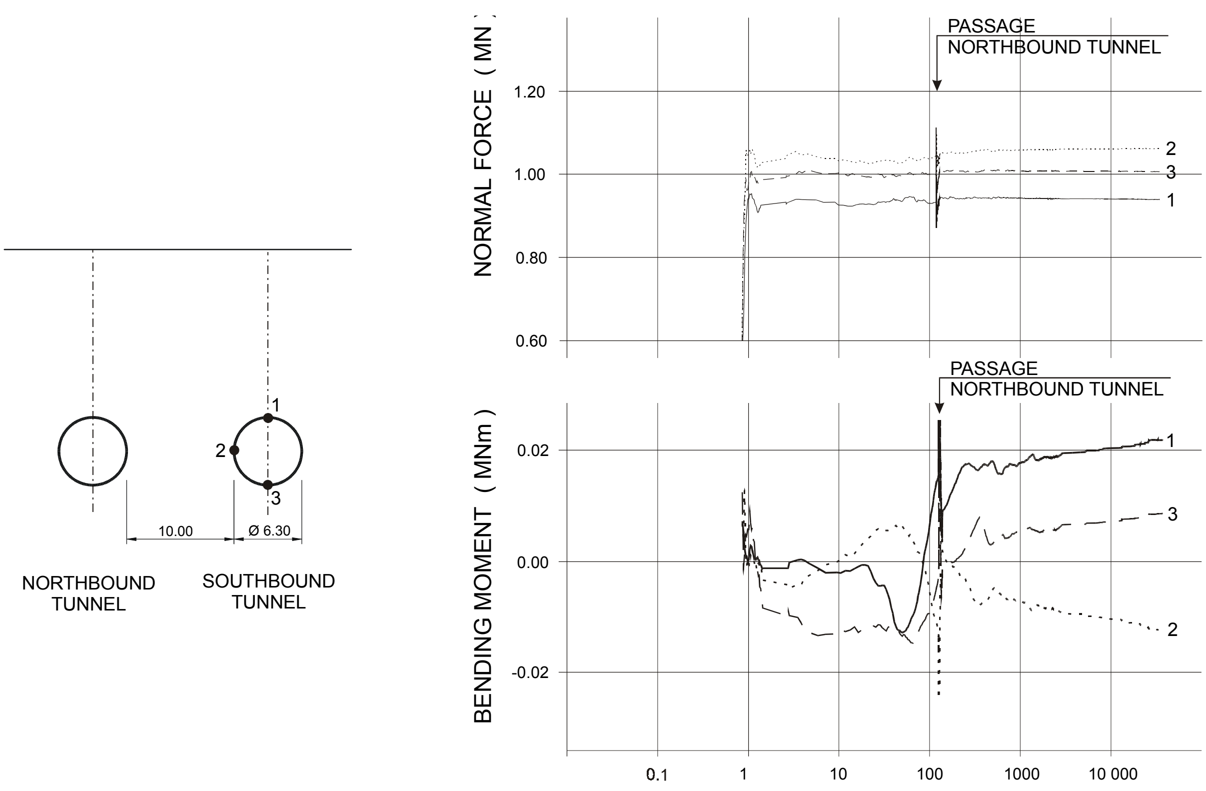




**Figure 3 Typical calculated History of Normal Forces and Bending Moments in the**

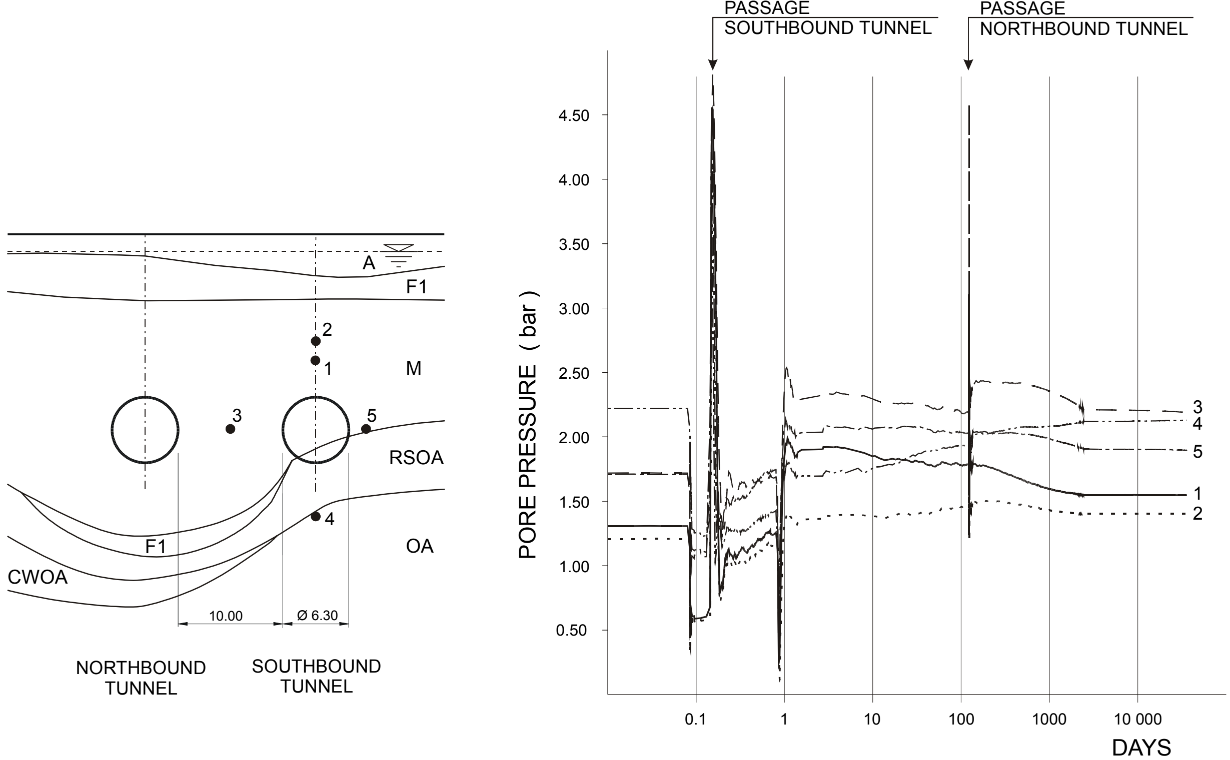


**Figure 4 Typical Calculated History of Pore Water Pressures**

surface. The surcharge has been introduced in the in the Erdmann approach by an average "effective" soil weight according to formula below. Calculations are carried out considering an average "effective" unit weight ($\gamma'_{av}$):

$\gamma'_{av} = (\sigma'_z(\text{at tunnel axis})+q)/H$

$\sigma'_z$ … effective stress at tunnel axis

H … Depth of tunnel axis

All Erdmann calculations have been carried out based on effective stresses. Groundwater pressures have been considered entirely separate. Water pressures have been split into two parts, a uniform pressure and a pressure gradient. Calculation factors for sectional forces have been derived based on the results of a detailed plain frame analyses. Results have been compiled in to a design chart, which is presented in Figure 2.

In addition to the analytical calculations according to Erdmann, fully coupled effective stress consolidation analyses calculations using the FLAC code have been carried out in real time mode. The numerical results have been compared to the results of analytical approaches to cross-check and verify the analytically calculated sectional lining forces. In the case of settlement calculations, the numerical results served the purpose to calibrate input parameters such as ground losses for analytical settlement calculations.

In the numerical calculations, the soil is modelled as an elasto-plastic material with a constitutive law according to the Mohr-Coloumb yield criterion using drained parameters. To consider the decrease of the shear modulus due to shear strains, zones with different soil stiffness have been modelled around the tunnel. The calculation results show that in Old Alluvium, 250 mm of plain concrete (Grade C60) are sufficient. In the marine clay, the close proximity of the two tunnels to each other and the relatively shallow depth required main reinforcement of approximately 12cm$^2$ per metre and face.

Typical results of the numerical lining calculations are presented in Figure 3 and in Figure 4.

## 4. GROUND MOVEMENTS

For soft cohesive soils, such as marine clay, long-term settlements due to consolidation are of considerable relevance in the assessment of ground movements. As reported by Shirlaw and Copsy (1987), the assumption of a settlement trough approximated by a Gaussian normal distribution based on the ribbon sink analysis according to O'Reilly and New (1982) is also valid for ground conditions in Singapore including Old Alluvium and marine clay.

Therefore, a combination of numerical and analytical methods is chosen for settlement analysis. Numerical methods allow more accurate simulation of geometrical features and the use of more complex constitutive relationships. Analytical approaches, on the other hand, are offering the advantage of quick calculation time and easy handling but are rougher with regard to modelling details and various boundary conditions such as soil layering. Therefore, the numerical consolidation settlement calculations using the FLAC code have been carried out using realistic input parameters to derive and calibrate input parameters to be used for analytical calculations, which are then used as regular tool for settlement analysis. Analytical settlement calculations have been carried out based on the ribbon-sink volume loss approach according to O'Reilly and New (1982).

For the analytical calculations, volume losses between 0.6% and 3.3% and trough width parameters between 0.4 and 0.5 have been used.

In tunnel sections where the shield has to negotiate curves, additional ground losses occur owing to the forcing of the rigid shield body into the curve. The proposed alignment includes curves with radii ranging from 2000 m to a minimum of 500 m. An approach has been developed to derive estimates for the value of additional volume loss due to negotiating the the shield into curves.

The additional curvature loss due to curvature can be estimated based on the geometrical relationships given in Figure 5, which yield the following formula:

$$Vc\% = \frac{\sqrt{(R+rs)^2 + \left(\frac{L_S}{2}\right)^2} - (R+rs)}{2 \cdot rs}$$

| | |
|---|---|
| Vc | Volume loss due to curvature |
| Ls | Rigid shield length |
| rs | Shield radius |
| R | Radius of alignment curve |

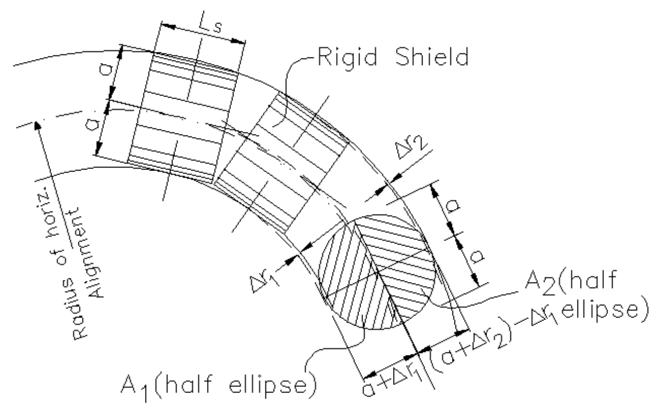


**Figure 5 Geometrical Situation for Calculation of Curvature Loss**

## 4. RISK ASSESSMENT

For tunnelling in urban residential areas, the analysis of ground settlements and the assessment of related risks is a critical design issue. Although, a highly sophisticated state-of-the-art EPB shield machine with tail void grouting system is used, tunnelling work in soft ground inevitably induces movements in the surrounding ground.

For NEL contract C706, several alignment options were studied and analyses with regards to the tunnelling risks carried out. Owing to the soft ground conditions, one would tend to increase the distance between the tunnels such that the settlements are minimised. However, it was found that an increased distance between the tunnels is of course resulting in smaller maximum settlements but the tunnels would be located directly underneath the buildings. This would eliminate any possibility for ground treatment measures or contingencies to be installied in the vicinity of the tunnels. On the other hand, if the distance between the tunnels is decreased the maximum settlements increase but access for ground treatment could be maintained. After evaluation of all aspects related to the construction and allowing for contingencies, the alignment with a distance of 10.5m between the tunnels axes was chosen due to better control over tunnelling risks. Detailed assessments of building damages have been carried out based on the limiting tensile strain approach presented by Burland and Wroth (1974).

## 5. INFLUENCE OF SECOND TUNNEL

It is proposed to consider the construction of the second tunnel reduction of the at-rest earth pressure coefficient, $K_0$, to a reduced value, $K_d$. This phenomenon has been called by some authors "increased distortional loading". The reduction is derived from the radial stress change, which occurs at the location of the first tunnel due to passage of the second tunnel.

The approach is based on a fictitious internal pressure applied in the tunnel. This pressure is chosen such that it produces radial deformations corresponding to the anticipated volume loss. Then, at a distance corresponding to the location of the first tunnel, resulting horizontal stress can be calculated using conventional elastic continuum formulae. By putting this horizontal stress value into relation to the original stress value, this approach yields following relationship:

$$K_d = K_0 * \left[1 - f_K * \left(\frac{R}{D}\right)^2\right]; \quad f_K = \frac{R}{D} * \frac{E'}{1-\nu} * \frac{1-\sqrt{1-V_s}}{K_0 \cdot \gamma \cdot H}$$

$K_d$ = Reduced earth pressure coefficient
$K_0$ = Initial at-rest earth pressure coefficient
$f_K$ = Influence factor
R = Outer tunnel radius
D = Distance between tunnel axes
E' = Drained Young's modulus of soil
$\gamma$ = Unit weight of soil
$\nu$ = Poisson's ratio of soil
$V_s$ = Volume loss
H = Height of overburden above tunnel axes

As a quick conservative estimate, the influence factor $f_K$ can be taken to 1.0. This is corresponding to the case where the fictitious internal pressure is zero and the deflections of the secondary tunnel are maximum causing maximum disturbance to the ground.

## 6. INTERACTION WITH SURS TUNNELS

At the intersection Race Course Road with Balestier Road, the proposed tunnels of the Singapore Underground Road System (SURS) will cross above the bored tunnels of NEL contact C706. In this area, the NEL tunnels are located in a 30 m deep buried valley filled with soft marine clay. The excavation of the SURS tunnel will be 14.5 m deep. The water table is considered to be 2.0 m below the ground surface.

The SURS tunnels and the three access ramps (IX, IE and JX) give a total width of construction of approximately 80 metres.

A comprehensive study of the interaction between the NEL tunnels and the proposed SURS tunnels has been carried out to assess the conditions in the long-term case as well as during construction of the SURS tunnels. In particular, this study is addressing following issues:

(a) Stress-strain analysis of NEL tunnels related to SURS construction
(b) Recommendations for the design and construction method of the SURS tunnels in the area of interaction with NEL tunnels
(c) Design of the NEL tunnels with respect to SURS construction
(d) Recommendations for monitoring instrumentation for SURS construction

Special emphasis has been given to study the expected pore pressure development during SURS construction. Based on these results recommendations and restrictions have been defined which have to be followed during SURS construction. In the area of influence, the pore pressure will be significantly reduced due to SURS excavation. The Code of Practice for Railway Protection (CPRP) allows a maximum pore pressure change of only 10 kPa.

Within the SURS excavation area, the pore pressure reduction would have no adverse influence on the NEL tunnel lining owing to the fact that the total stresses, the effective stresses and the pore pressures are reduced due to the excavation process, all at the same time. Numerical analyses have been carried out considering the decrease of pore pressure and effective stress. It was shown that for this case the respective segmental lining design would require even less reinforcement due to decreasing effective loading and decreasing pore pressures.

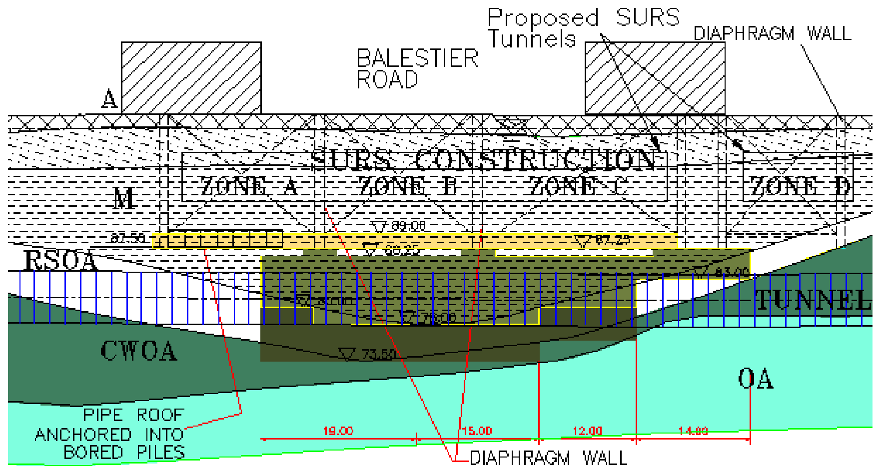


**Figure 6 Typical Arrangement at Intersection with SURS**

However, sections located outside but close to the actual excavation area may experience a pore pressure reduction without simultaneous reduction of effective stresses. Numerical analyses carried out for this case considered the decrease of pore pressure leaving the total stress at the initial level before SURS excavation. In this case the calculations have shown that the segmental lining would require more reinforcement if the water table is lowered by more than 8 m. As a contingency, re-charge wells have been proposed.

The stability of a supported excavation in clay involves a variety of design problems. These involve possible basal heave into the excavation bottom, soil wedge slippage behind a cantilever wall, and deeply seated rotational slump-type failures behind and beneath the wall toe.

Stability calculations regarding basal heave of the supported excavation of the SURS tunnel have shown that the excavation bottom is stable only when a stiff support system such as a diaphragm wall with the wall toe embedded in the underlying cemented Old Alluvium (OA) is used.

However in the vicinity of the NEL tunnels, the diaphragm wall need to be boxed out. Therefore, at the location where the diaphragm walls are crossing the NEL tunnels, a jet-grouted block needs to be constructed prior to the tunnel drive of the NEL tunnels. Later during SURS construction, the toe of the diaphragm wall can be embedded into the top of this grouted block.

Great concern was given to the heave created at the location of the NEL tunnels due to effective stress reduction imposed by SURS tunnel excavation. Both, analytical as well as numerical analyses have been caried to assess the extent and the magnitude of heave. It was found that, owing to the existance of a considerable layer of marine clay underneath the tunnels the initial ground stiffness is insufficient to keep the heave within the required limits. Jetgrouting has been proposed to enhance the confinement of the lining and to increase the soil stiffness. See Figures 6 and 7.

At the southern side of Balestier Rd., existing buildings make it impossible to install this jet-grouting block during NEL construction. Therefore, in this area a rigid pipe roof is proposed as an additional support measure during SURS construction to limit the heave of NEL tunnels.

Although the ground treatment needs to be in place only before the start of SURS excavation, jetgrouting has been carried out already now during the construction of the NEL tunnels. This is due to the requirements of the CPRP. In particular, in the proximity of MRT tunnels, the CPRP is giving very tight limits for the allowable changes in the pore water pressure regime, explicitly interdicting any jetgrouting or similar works are allowed in the first reserve around MRT tunnels.

Seven different options have been studied on how the SURS construction can be carried out under the requirements imposed by the CPRP. Based on the considerations above, following basic construction steps are required:

- Jet-grouting of marine clay and upper part of the Old Alluvium as an annulus of 2.0 m around the NEL.
- At the intersection with future diaphragm walls, the thickness of this annulus will be increased to 3.0 m.
- Construction of NEL bored tunnels.
- Construction of bored piles and soil cement stabilisation between SURS base slab and top of jetgrouting.
- Construction of diaphragm walls as SURS excavation box support.
- Construction of pipe roof at southern side of Balestier Rd.
- Staged excavation and installation of SURS tunnel in predefined sequence and areas and controlled backfill of structure.

## 7. CONCLUSIONS

The lining design has been based on calculations according to Erdmann (1982).

Additional numerical analysis have been carried out to verify the lining sectional forces and to derive input parameters relevant for the use in analytical settlement calculations. Assessment of ground movements has been carried out based on the volume loss approach according to O'Reilly and New (1982). For the particular case of densly overbuilt residential area it was found that an alignment option featuring close distance between both tunnels, would give better control over the situation.

A detailed study with regard to the influence of the future SURS excavation revealed that the ground stiffness was insufficient to limit the heave deformations for the NEL tunnels. Jetgrouting was considered an appriopriate measure to control the heave.

In the area where jetgrouting is not possible now due to existing buildings, it was shown that the limitation of heave can also be achieved by installation of a piperoof connected to diaphragm walls and bored piles.

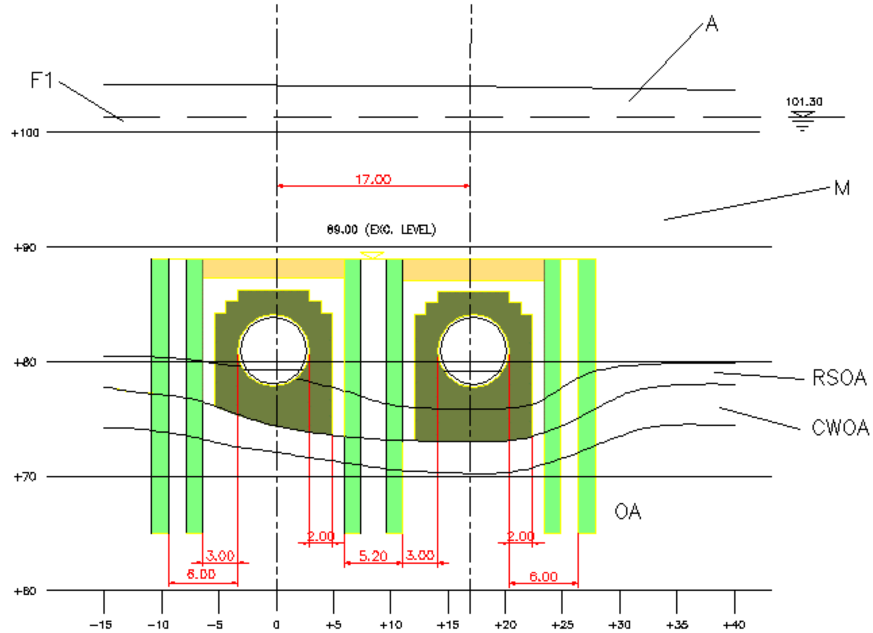


**Figure 7 Typical Arrangement at Intersection with SURS**

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