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Design of bored tunnel linings installed within partially excavated C&C boxes

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ABSTRACT: According to the construction programme of the MRTA ISP Underground Structures North, in Bangkok, Thailand, four cut & cover station boxes will be tunnelled through by earth-pressure-balance shield machines. The passage of the shield machines and the tunnel lining installation will occur when the excavation works inside each station is in a different stage. The tunnel section along the stations will be used for construction operational works, such as the transport of muck from the excavation face and the lining will be removed when the excavation within the station is finalised. For the design of the temporary precast concrete tunnel lining, a detailed finite difference analysis was carried out to assess the interaction between the lining, the soil and the station diaphragm walls as the excavation within the station proceeds. The paper outlines the criteria utilised in the analysis and summarises typical results obtained from the assessment.

1 INTRODUCTION

The Initial System Project (ISP) Underground Structures, North Contract, is the northern section of the Blue Line, which is the first stage of the underground mass rapid transit system being implemented by the Metropolitan Rapid Transit Authority (MRTA) in Bangkok, Thailand.

In this contract, nine cut & cover station boxes will be connected by twin single track bored tunnels in a total length of about 16 km.

Owing to the construction schedule, the shield machines will drive through four cut & cover station boxes after the installation of the diaphragm walls and at different phases of station excavation. A temporary tunnel lining will be installed and removed when the excavation inside the station is finalised.

Figure 1 shows a section through a typical station box, after the installation of the tunnel lining.

The excavation inside the station will cause a continuous change in the ground stresses around the tunnels. Vertical loads will decrease whilst the horizontal loads will increase with the passive earth pressure.

A detailed modelling of the ground-tunnel-diaphragm wall interaction with respect to the tunnel lining section forces and overall lining stability was carried out for each station.

The paper describes the typical structures and soil conditions involved in the modelling, the general criteria applied and the typical results obtained from the analysis of a selected station.

2 DESCRIPTION OF THE STRUCTURES

2.1 *Bored tunnel precast concrete lining*

The temporary tunnel lining consists of precast concrete rings of six segments plus one keystone. The nominal inner diameter is 5700 mm and the segment thickness is 300 mm. The design strength of the segments is 40MPa.

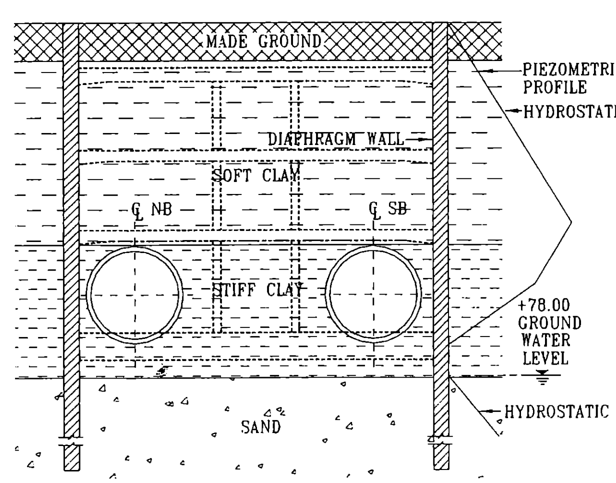

Figure 1. Typical section through a station box

Segments will be installed in a staggered arrangement and will be connected in longitudinal and radial direction by curved steel bolts.

The top of the bored tunnel lining will be generally located at approximately 14.5m below the ground surface.

2.2 *Other structural elements*

The cut & cover station boxes will be typically 200 m long and 23 m wide. Retaining walls consist of 1.0m thick diaphragm walls. The toe level of the walls will be slightly different for each station, ranging from 32.5m to a maximum depth of 37.5m.

The construction will follow a top-down excavation method. The bracing support is provided by the installation of temporary steel struts and the definitive concrete slabs. The stations will have a roof, a retail, a concourse and a base slab.

Steel arch supports, consisting of H joists and longitudinal stiffeners, were also considered in a separate analysis for the assessment of their potential strengthening effect on the precast concrete lining at critical excavation stages.

3 GEOTECHNICAL CONDITIONS

Soil conditions correspond to the typical Bangkok subsoil. It is constituted of a first layer of man made ground of about 2.5m, followed by the soft clay layer 11 to 15m thick and the stiff clay layer with thickness ranging from 6 to 14m. Below these clay layers is the topmost Bangkok aquifer, which is constituted of dense to very dense sand interlayered by thin layers and lenses of stiff to hard clays.

Regarding ground water conditions, deep well pumping has caused a drop of the water pressure of the various aquifers and an under drainage of the clay layers. The ground water level of the Bangkok aquifer, which was originally at the ground surface, has dropped by more than 20m. The present reference water table along the alignment is about 23.0m below the surface. Long-term monitoring has shown that this water table is constant for already several years.

Bored tunnels will be mainly located in the stiff clay layer with the top of the tunnel at the boundary with the soft clay layer. Diaphragm walls of the station will penetrate the stiff clay and the underlying sand layer.

A section through a typical station box, including the soil profile and the piezometric profile, is shown in Figure 1. Soil Parameters are shown in Table 1.

4 GROUND-STRUCTURE MODELLING

The two-dimensional explicit finite difference analysis (FLAC code, Cundall et al 1993) was used for the modelling of the interaction between soil and structure. With this method, a complete modelling of the structure and ground, including stress and strain distribution in the ground, deformation and section forces of the lining was possible.

4.1 *Ground modelling*

The constitutive model was based on an ideal elasto-plastic (Mohr-Coloumb) failure criterion.

Regarding stress conditions for the design of the tunnel lining, full ground stresses were used. Stress-release (ground relaxation) due to tunnel excavation was neglected as preliminary investigation had shown that it has minor influence due to the particular load conditions.

As the interaction of the tunnels with the walls will occur in short-term conditions, undrained parameters were assumed for the cohesive soil layers. The effect of ground water flow and consolidation was not considered in the model.

Strength and deformation soil parameters were assumed isotropic and defined as a function of the depth, as shown in the Table 1 and Equations (1) to (3).

Table 1. Soil parameters

PARAMETER		SOIL LAYERS			
		MADE GROUND	SOFT CLAY	STIFF CLAY	SAND
Cu	[kPa]	-	(1)	(2)	-
E'c (1.0%)*	[kPa]	20000	200Cu	350Cu	$2000N_{60}$
E'c (0.1%)*	[kPa]	-	430Cu	850Cu	$2500N_{60}$
E'r (1.0%)*	[kPa]	-	3E'c	2E'c	3E'c
Eu,c (1.0%)*	[kPa]	-	225Cu	400Cu	-
Eu,c (0.1%)*	[kPa]	-	500Cu	1000Cu	-
Eu,r (1.0%)*	[kPa]	-	3Eu.c	2Eu.c	
u'	[-]	0.30	0.30	0.30	0.25
u	[-]	0.50	0.50	0.50	0.50
k'_0	[-]	0.75	0.75	0.65	0.50
k_0	[-]	1.00	1.00	1.00	-

* Correspondent shear strains

$$Cu = 2 \cdot Dp + 5 \quad (15 \le Cu \le 50) \tag{1}$$

$$Cu = 50 - 7.8(15 - Dp) \tag{2}$$

$$N_{60} = 20 - 0.78(16 - Dp) \tag{3}$$

Where:

Cu...undrained shear strength (kPa)

N_{60}...Standard Penetration Test Value

E'c / Eu,c...drained / undrained Young's modulus for loading-reloading (MPa)

E'r / Eu,r...drained / undrained Young's modulus for unloading (MPa)

v' / v...drained / undrained Poisson's ratio

k'o / ko...drained / undrained lateral earth pressure coefficient

Dp...depth with relation to the ground surface level (m).

The standard ground model used for the design of station retaining walls and tunnels made use of different stiffness parameters. For these analyses, stiffness correspondent to 0.1% and 1.0% shear strains were applied for the retaining wall design and tunnel design, respectively. The assumption is in accordance to the typical range of strains involved in the construction of the respective structures, as shown in Figure 2.

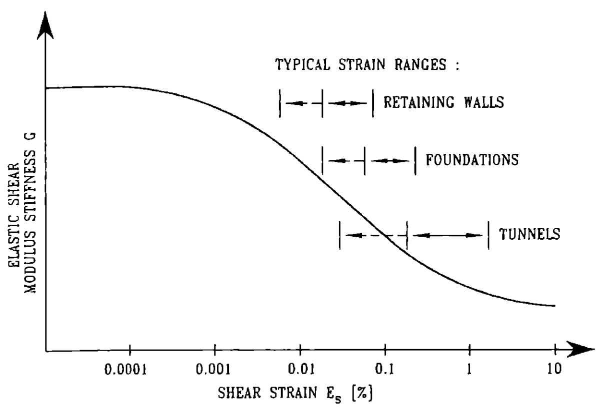

Figure 2. Stiffness-shear strain relationship – by Menzies (1997).

For the assessment of the interaction between the tunnels and the diaphragm walls, a combined ground model was used. Ground stiffness correspondent to 0.1% shear strain was generally applied to the model and the higher shear strain associated with the tunnel construction was considered by the definition of a zone of larger strains (1.0%) within a distance of approximately 3.0m measured radially from the periphery of the tunnel lining. At this distance, shear strains due to the tunnel construction will be less than 0.1% shear strains, as shown by deformation plots.

To avoid an unrealistic heave and to consider the elastic behaviour of the ground due to the unloading from the tunnel excavation, an unloading Young's modulus (correspondent to 1% shear strain) was used for the area below the tunnel invert.

The stiffness-shear strain model is shown in Figure 3.

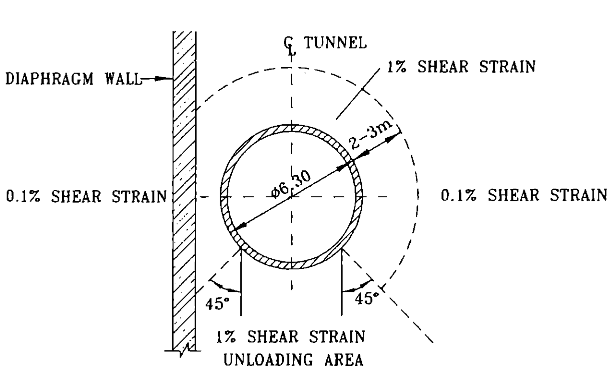

Figure 3. Stiffness-shear strain model.

4.2 *Modelling of the structural elements*

The diaphragm wall was modelled by continuum elements with an ideal linear elastic behaviour. The interaction between the walls and the adjacent soil was simulated by interface elements (springs) in normal and tangential direction.

The normal springs were defined as very stiff so that the transfer of forces between the soil and the wall would be controlled by their relative stiffness. Tangential (shear) parameters were defined according to the shear parameters of the soil layers.

Permanent concrete slabs and temporary steel props were simulated by compression-only beam elements connected to the diaphragm wall grid points.

The steel arch support used for the strengthening of the tunnel lining was simulated by beam elements with the correspondent flexural and axial stiffness per linear metre.

4.3 *Bored tunnel precast concrete lining*

Two different approaches were used for the modelling of the precast concrete tunnel lining structural behaviour. These models were applied in dependence of the purpose of the assessment, i.e., for the assessment of the section forces in the lining (Structural Design) or the analysis of the overall lining stability (Stability Analysis).

4.3.1 *Structural design*

The assessment of section forces for the design of the temporary precast concrete lining was carried out with the assumption of a continuous lining of beam elements with reduced lining flexural stiffness.

The decrease in the lining flexural stiffness is caused by the joints and their reduced lining thickness. The reduced moment of inertia can be back calculated from the number of segments per ring (excluding the key-stone) and the moments of inertia at the joint and at the standard lining section in accordance to Muir Wood (1975).

The results obtained from this assumption, which was the basic approach used for the design of the tunnel lining along the route, provided the critical section forces for the structural design of the tunnel lining at station sections.

The detailed methodology used for the structural design of the precast concrete tunnel lining is described by Prinzl and Gomes (1999).

4.3.2 *Stability analysis*

For the assessment of the overall tunnel stability, the lining was simulated by means of continuum elements with detailed consideration of the geometry and the layout of segments and joints, as shown in Figure 4.

The precast concrete tunnel lining was defined as a very dense radial grid with interface elements at the joints. A linear elastic behaviour was assumed, with exception of the areas close to the radial joints, where the Mohr-Coulumb failure criterion was applied to simulate the possible plastic behaviour of the joints.

The effect of the radial connecting bolts, which connects the segments of the rings, were simulated by tension-only "cable" elements.

The boundary between segments and ground was established by interface elements, which should allow a quasi friction-free movement of the tunnel lining.

As it does not consider the staggered arrangement of the lining (effect of the longitudinal bolts and friction between the rings), this model is more flexile than the continuous lining previously described. It should provide a worst-case stability assessment and the results should be understood as indicative only.

5 ANALYSIS AND DESIGN CRITERIA

5.1 *Analysis cases and construction programme*

Tunnels will be driven through the station boxes when the excavation works are in different stages, which still can be affected by changes in the construction programme during the execution of the works.

As to cope with these conditions, five different scenarios were analysed with regard to the time of the passage of the bored tunnels and lining installation as shown in Table 2. The main excavation stages of a selected station are shown in Figure 5.

In an additional analysis, the installation of a steel arch support inside the concrete lining at the final stages of excavation was simulated.

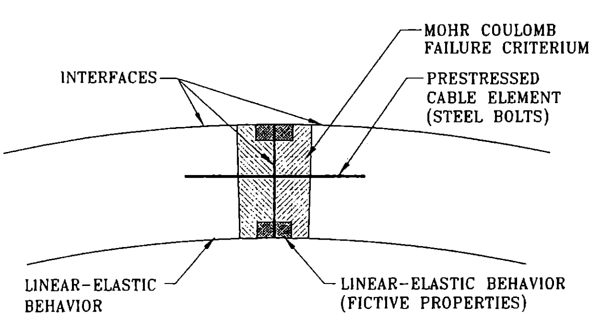

Figure 4. Detail of the longitudinal joint between two segments.

Table 2. Analysis cases - percentage of excavation completed at the moment of lining installation.

Analysis Cases	% of excavation with respect to the distance of the tunnel top from the ground surface level
a	0.0%
b	16%
c	29%
d	65%
e	96%

* The distance of tunnel top to the ground surface level is approximately 14.5m

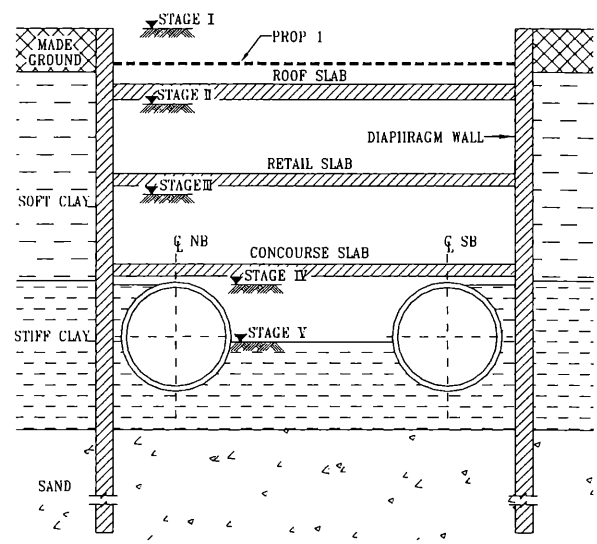

Figure 5. Main excavations stages.

5.2 *Design criteria for the temporary lining*

The following criteria was used for the assessment of the lining performance:

5.2.1 *Structural design*

Segments were designed for the ultimate limit state in accordance with the BS 8810 based on the results obtained from the beam model with continuous lining and reduced lining flexural stiffness.

Due to the temporary character of the lining, the structural capacity was checked for a reduced load factor (1.2) and reduced partial safety factors for the materials (1.3 and 1.05 for the concrete and steel respectively).

5.2.2 *Stability analysis*

Of relevance for the analysis of the tunnel lining stability, was the relative displacement between segments and the yielding of the radial connection bolts, which could indicate likely unstable joints.

As a general criteria, the deformations of the lining were considered to become critical when the maximum relative displacement between adjacent segments would be larger than 10mm, which corresponds to the gap between the bolts and the bolt holes, and the start of yielding of the radial connections bolts. The general criteria for the definition of critical lining deformations are shown in Figure 6.

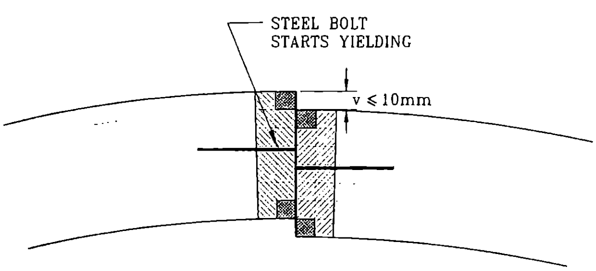

Figure 6. General criteria for the definition of critical lining deformations.

6 RESULTS AND CONCLUSIONS

6.1 *Structural design*

Based on the results obtained from the analysis of a selected station, a plot of the required reinforcement ($R_{eq}R_{einf}$) normalised with the standard lining reinforcement ($S_{tand}R_{einf}$) versus the excavation depth (H_e) inside the station normalised with the distance from the tunnel top to the ground surface (H_o) is shown in Figure 7.

According to the results, the earlier the tunnel is installed with respect to the progress of the excavation works inside the station, the larger will be the stresses on the tunnel lining. This situation will occur due to the redistribution of stresses during the station excavation, which are partially transferred to the tunnel lining. The most unfavourable section forces in the lining (required reinforcement) occur when the excavation reaches the top of the tunnel (H_e/H_0 of approximately 1.0).

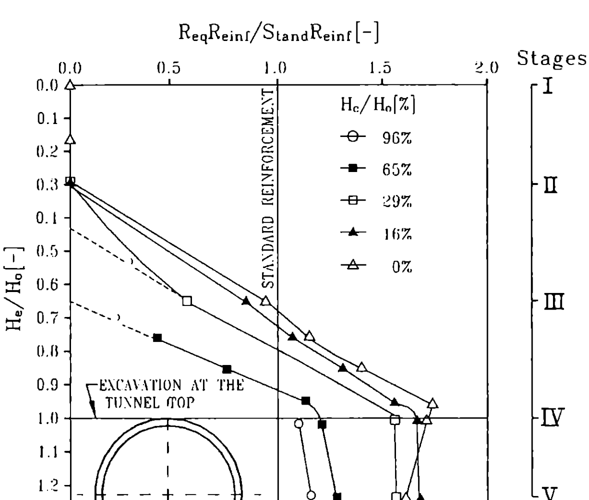

Figure 7. Normalised required reinforcement ($R_{eq}R_{einf}/S_{tand}R_{einf}$) versus normalised excavation depth (H_e/H_o).

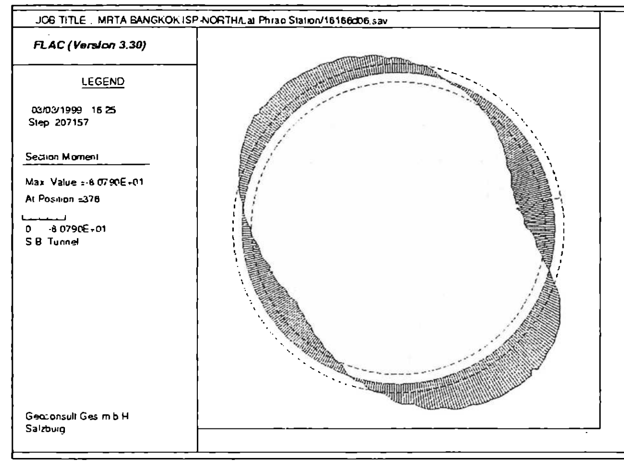

Figure 8. Typical bending moment distribution.

6.2 *Stability analysis*

Regarding lining stability, the analysis indicates that critical conditions (relative segment displacements higher than 10mm and bolt yielding) will be achieved generally for a H_e/H_0 of approximately 0.8 (80% of the tunnel cover excavated).

It is also important to notice that although section forces and lining displacements obtained from the two different structural models were of different order, they followed the same basic patterns. A typical moment distribution is shown in Figures 8.

6.3 *Additional analyses*

The installation of steel supports at the final excavation stages proved to be a satisfactory solution for the increase of the tunnel lining stability and the limitation of the required reinforcement to the standard lining reinforcement.

7 ACKNOWLEDGEMENTS

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