DIAPHRAGM WALL EMBEDMENT DEPTH DESIGN IN ROAD UNDERPASS TUNNEL PROJECT IN SINGAPORE

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ABSTRACT

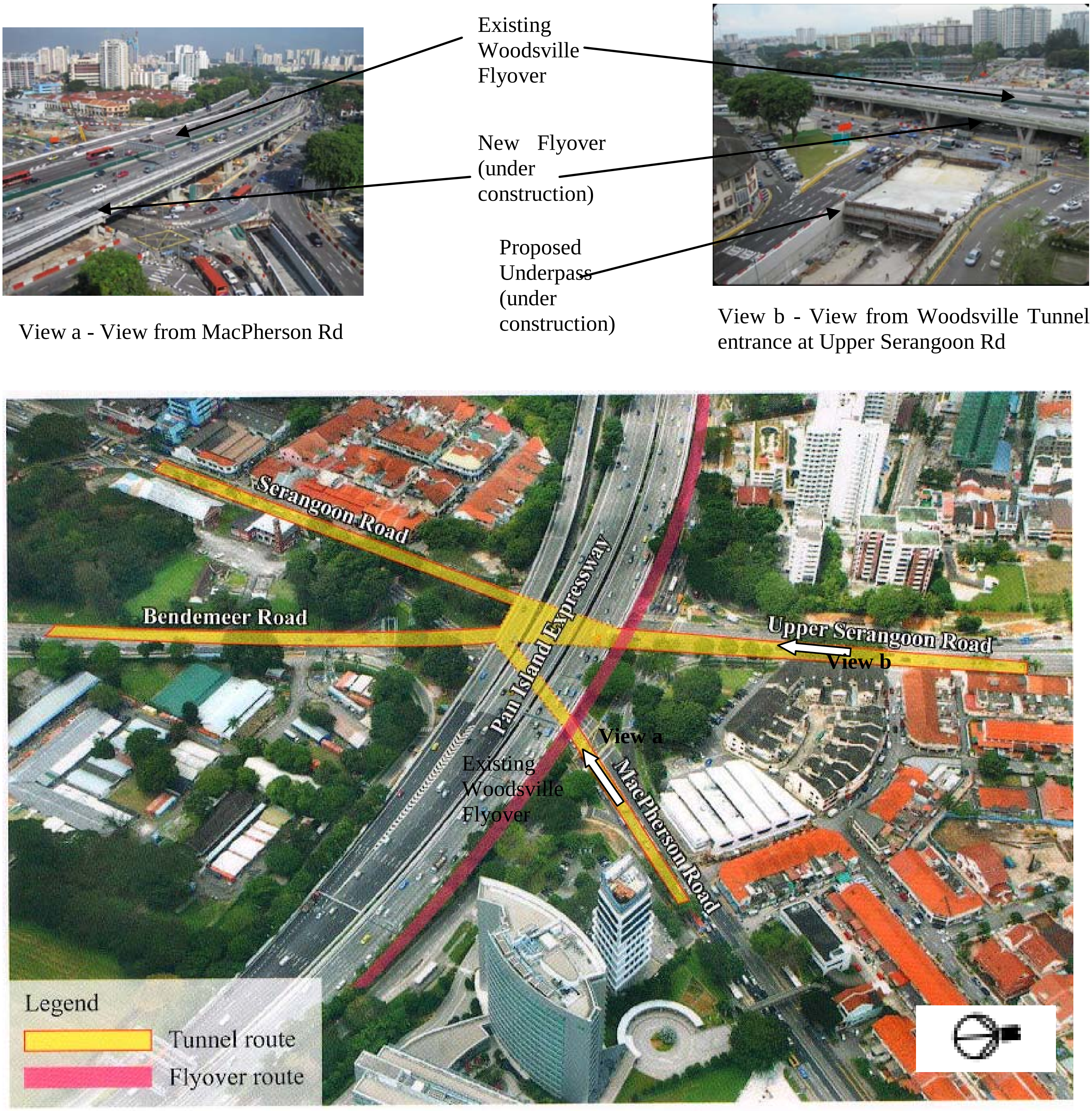
This paper describes the process for diaphragm wall embedment depth design in a single level underpass project in Singapore. The proposed underpass is located at an extremely heavy traffic junction/interchange. The existing Northeast Line MRT (NEL) tunnels run directly beneath the underpass for much of its length. The existing Woodsville Flyover is in close proximity to the proposed underpass structure. Other major constraints include an adjacent existing deep sewerage tunnel and residential/commercial buildings.

Different design conditions were considered in the diaphragm wall embedment depth design, which include soil retaining walls in bottom up construction and load bearing elements in top down construction and permanent condition. Finite element models (FEM) using geotechnical software Plaxis were implemented for earth retaining structure design and damage assessment for the impacts on the adjacent existing tunnel/flyover/building. Adequate wall embedment depths were to be verified by hand calculations in terms of geotechnical capacity of the diaphragm wall, floatation resistance, toe kick-out stability, basal heave stability and blowout failure issues. The proposed embedment depths in design drawings were reviewed by using predrilled holes results during the construction stage.

1 INTRODUCTION

The proposed development is to construct a single level underpass in a double ‘Y’ shape and a 400 m long single lane flyover along the existing Woodsville flyover to cross the junction. The construction site is located on one of Singapore's most congested interchanges linking four major roads, namely Bendemeer Road, Macpherson Road, Serangoon Road and Upper Serangoon Road (Figure 1). The complexity of the project is increased further due to the existing infrastructure in the vicinity, for example, the existing Northeast Line (NEL) MRT tunnel running directly beneath the underpass for much of its length along Serangoon Road and Upper Serangoon Road, the existing deep sewerage tunnel (DTSS) running across the tunnel section from south to north direction and the existing Woodsville flyover located in close proximity to the proposed underpass structure.

Geological conditions for the proposed underpass is interpreted and based on the factual geotechnical report for this project. Generally, the proposed underpass traverses through mixed soil profiles of softer soils underlain by the residual soils and weathered rocks of the sedimentary old alluvium. The softer soil deposits consist of peaty clay (Estuarine Deposit), marine clay and fluvial deposit. The peaty clay deposit is generally described as dark brown to grey peaty clay with organic matter or decomposed vegetation. It overlies the loose peaty sand or marine clay and is generally located within 10 m from the ground surface. Generally, the peaty deposit (E) exhibits medium to high plasticity, low strength, high compressibility and low permeability. The marine clay (M) is a soft bluish grey clay with shell fragments. The marine clay has high plasticity, medium to high sensitivity, low permeability and high compressibility in general. The fluvial deposits (F) generally consist of two major soil groups. They are the granular soil group F1 (sandy silt, silty sand and clayey sand) and the cohesive soil group F2 (silty clay and clayey silt). F1 sand is generally very loose to medium dense and is highly to moderately permeable. The cohesive soil group (F2) usually has a low to high plasticity and is lightly overconsolidated. Old alluvium, which consists of semi-consolidated/lithofied sand and fine gravel with silt and clay lenses, is formed by the sediments brought down by the rivers in the region during the Pleistocene time. Old alluvium was divided into five sub-formations, based on SPT ‘N’ value (O(A), O(B), O(C), O(D) and O(E)). Typical geotechnical design parameters adopted in the design are presented in Table 1 below.



View a - View from MacPherson Rd

View b - View from Woodsville Tunnel entrance at Upper Serangoon Rd

Figure 1 Site location

Table 1 Geotechnical Design Parameter

Soil	Bulk Density (kN/m^3)	K_o (min)	C_u (Max) (kPa)	C' (Max) (kPa)	Ø' (Max) (°)	k (m/s)	E_u (max) (kPa)
F	19	0.5	-	0	30	$1x10^{-7}$	10000
E	15	1.0	#1	0	20	$1x10^{-9}$	300C_u
F1	20	0.7	-	0	30	$1x10^{-6}$	10000
F2	19	1.0	#2	5	25	$1x10^{-9}$	7500
M	16	1.0	#3	0	22	$1x10^{-9}$	250C_u
O(E) N<10	20	1.0	5N	0	32	$1x10^{-8}$	1600N
O(D) 10<N<30	21	1.0	5N	8	34	$1x10^{-8}$	1600N
O(C) 30<N<50	21	1.0	5N	12	34	$1x10^{-8}$	2000N
O(B) 50<N<100	21	1.0	5N$^{\#4}$	15	34	$1x10^{-8}$	2000N
O(A) N>100	21	1.0	5N$^{\#4}$	20	35	$1x10^{-8}$	2000N

Notes:
#1 C_u = 5kPa where Z<5 m & Cu = 5+2.25(z-5) where z>5 m, limited to a maximum of 50 kPa;
#2 C_u = 20kPa where Z<10 m & Cu = 20+2(z-10) where z>10 m, limited to a maximum of 50 kPa;
#3 C_u = 10kPa where Z<10 m & Cu = 10+1.5(z-10) where z>10 m, limited to a maximum of 45 kPa;
#4 N is limited to 50;

Typical excavation depth is 12 m below existing ground level. The excavation width between the diaphragm walls varies from 7.3 m to 22 m. Rigid retaining wall system is desirable in this project due to stringent movement requirements as shown in Table 2. Groundwater table is assumed to be 1 m below the existing ground level. The groundwater drawdown is to be limited to 1 m to prevent ground loss, in particular, consolidation settlement issues in soft clay conditions. An impermeable cut- off wall is required for groundwater control.

Table 2 Allowable Movement of Existing Structures

Foundation Type	Maximum Allowable Settlement				Maximum Allowable Horizontal Movement of the Top of the closed Box
	Total		Differential		Total
	Short Term	Long Term	Short Term	Long Term	
Tunnel and depressed road (Without piles foundation)	20mm	20mm	1:1000	1:1000	15mm

2 DIAPHRAGM WALL TECHNOLOGY

The first diaphragm wall was used in the 1940's. Generally a diaphragm wall is able to provide a reinforced concrete wall below ground in collapsing soils with a high groundwater table. Wong et al. (1997) pointed out the maximum lateral wall deflections using rigid diaphragm walls may be less than 0.2% of H (Figure 5), where H is the excavation depth. Considering the high capacity in both vertical load and lateral load capacity and acceptable water tight performance for most of service requirements, a diaphragm wall is likely not only a retaining wall but also a vertical supporting wall. In such, it is reasonable to design diaphragm walls not only as temporary walls but also as permanent retaining walls for an economic design.

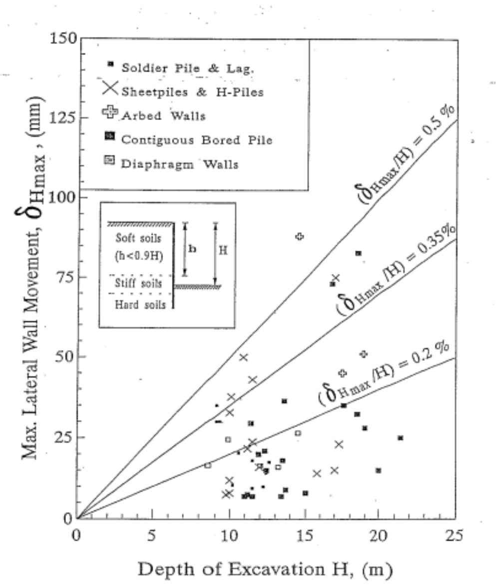


Figure 2: Maximum Lateral Wall Deflection vs. Excavation Depth (Wong *et al.*, 1997)

Typical diaphragm wall panel width is 6 m. In order to minimize ground movement in the vicinity of existing tunnels and flyover foundations during the installation stage, the width of diaphragm wall panels will be limited to 3 m as a precaution aimed to both minimizing stress change in the ground and also reducing the trench open time during excavation. From both diaphragm wall construction sequence (Figure 3) and typical wall reinforcement arrangement (Figure 4), the soil and water pressure applied on the diaphragm wall panel is unlikely to distribute to adjacent panels. It is necessary to check the embedment depth of individual wall panels to ensure the stability of the structural system.

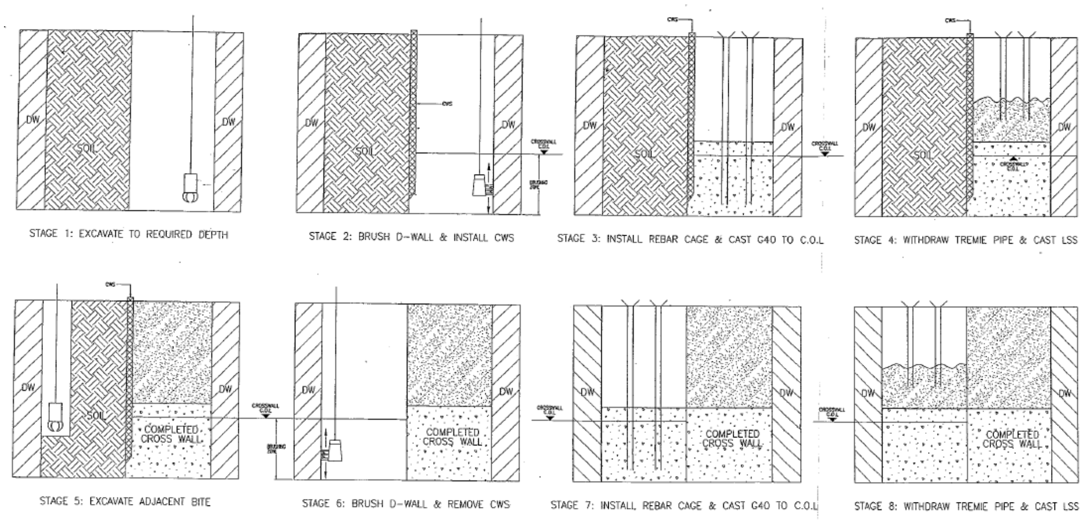
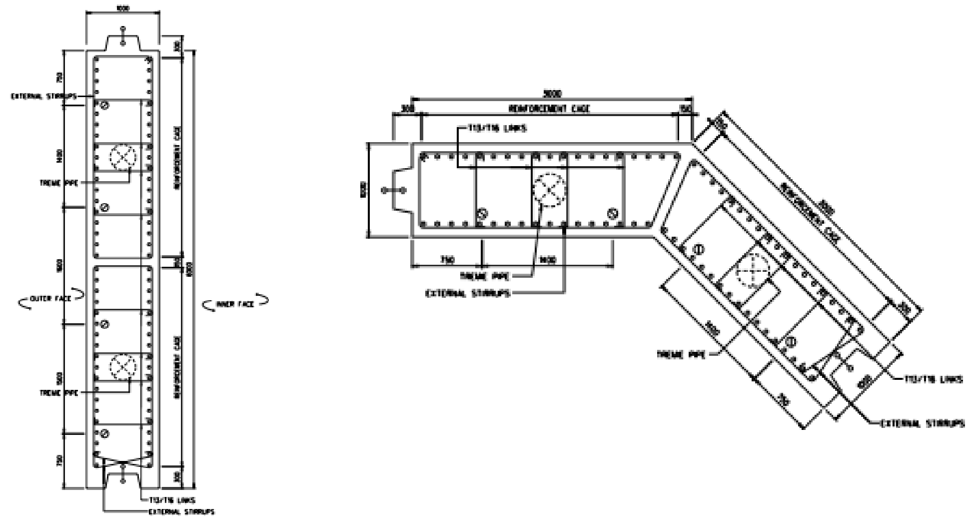


Figure 3: Construction Sequence of Diaphragm Wall.



Typical Panel Corner wall Panel

Figure 4: Typical Wall Reinforcement Arrangements

3 DESIGN METHOLOGY

3.1 TEMPORARY WORK STAGE

The proposed underpass consists of vehicular tunnel and depressed road structures, which will be generally constructed by bottom up method using 1 m thick permanent diaphragm walls as retaining walls during the construction stage. Adequate diaphragm wall embedment depths for overall lateral stability are to be designed by limit equilibrium method according to CIRIA C580. Earth coefficient k_a and k_p will be derived by using Caquot & Kerisel (1966) Chart. Seepage pressure is computed based on the simplified formula as presented in CIRIA 104. The methods for toe stability checks are illustrated in Figure 5.

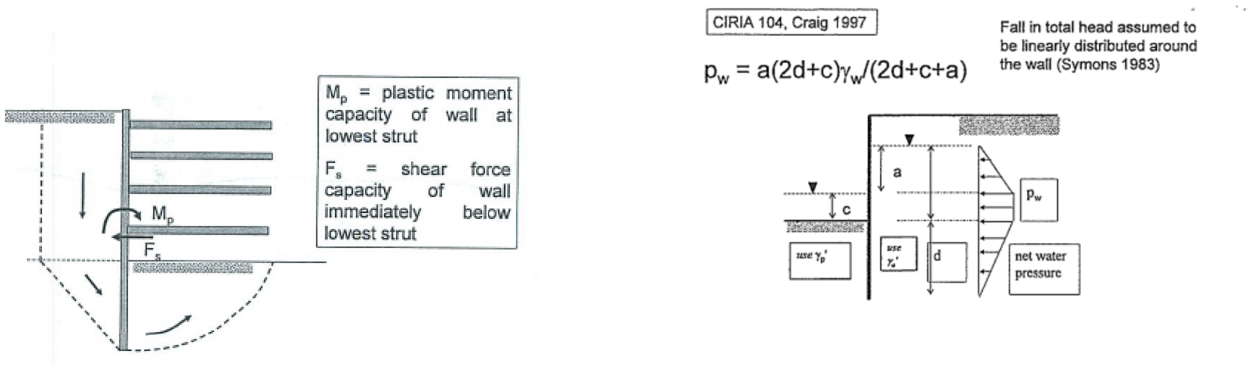
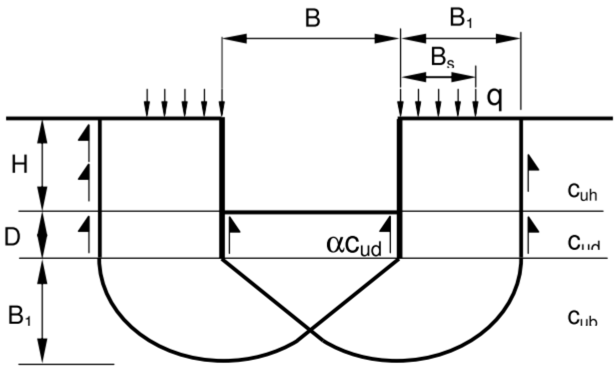


Figure 5: Toe Stability Check and Seepage Pressure

The basal heave is checked based on the modified Terzaghi method as illustrated below in Figure 6. It is assumed that the diaphragm wall is an ideal rigid wall. The skin friction of the diaphragm wall and the reduced depth to the hard stratum (B1) shall be taken into account.

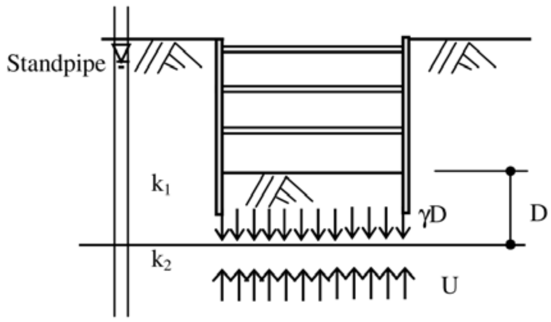


$$FS = \frac{5.7\, c_{ub}\, B_1 + c_{uh}\, H + (1 + \alpha)c_{ud}D}{\gamma H B_1 + q\, B_s} \quad \text{Equation (4)}$$

where α is the adhesion factor, α=1; B_1=0.7B or (T-D) whichever is smaller; T is clay thickness below formation level; and B_s is the width of surcharge loading where B_s< B_1, C_{uh}, C_{ud} and C_{ub} are the average shear strength (based on moderately conservative value divided by 1.5) within the zones, respectively, as shown above.

Figure 6: Basal Heave Based on Modified Terzaghi Method

If there is an impermeable layer overlaying a permeable layer at excavation stages, blowout failure shall be checked based on Figure 7.



$$\text{Factor of Safety} = \frac{\gamma D}{U}$$

Figure 7: Blowout Failure

In order to estimate the diaphragm wall deflections and the impacts on the adjacent existing infrastructure, FEM using geotechnical software Plaxis was carried out. However, it is necessary to carry out design check using limit equilibrium method as illustrated in Figure 5 to Figure 7, not only to ensure the excavation stability, but also to better understand possible failure mechanism, which is important to assess complex FEM models.

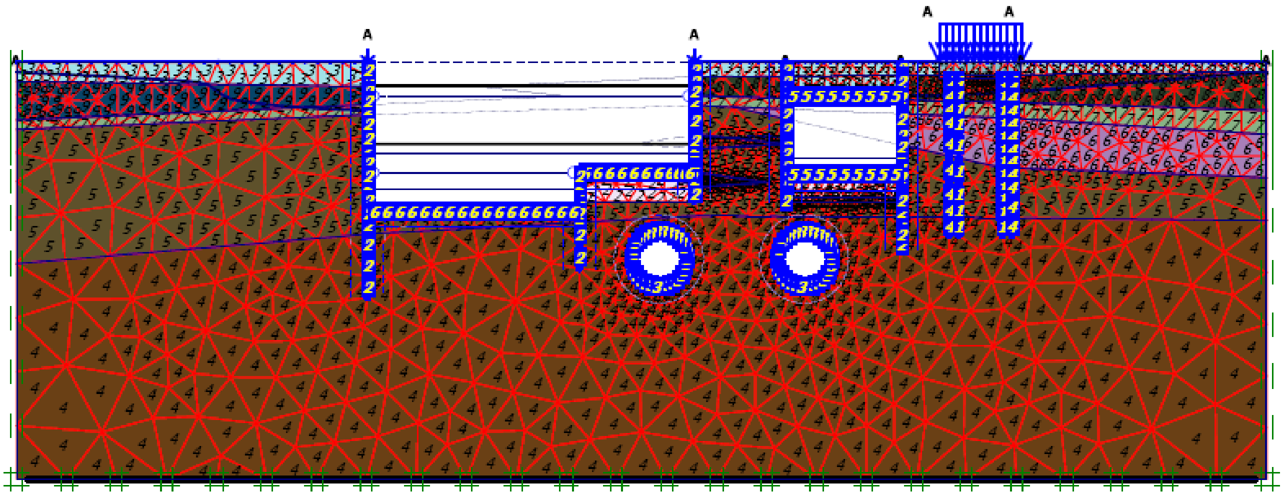


Figure 8 Typical FEM Mesh

Beneath the existing Woodsville flyover, approximate 70 m long top down construction is required due to the nearby existing flyover foundations and the presence of thick weak soil layer. For top down construction, the diaphragm walls act not only as earth retaining wall but also support the roof slab, the backfill above the roof slab and the live load from traffic and construction vehicles. The bearing capacity of diaphragm walls is a combination of wall face friction and end bearing. The wall embedment depth can be assessed based on partial factor method specified in Singapore foundation code CP4.

3.2 PERMANENT STAGE

The tunnel section is a box type structure with top and bottom slab span between diaphragm walls, while the depressed road section is a U-shape structure with bottom slab between diaphragm walls. The bottom slab of both the tunnel and the depressed road is an on grade slab which is also rigidly connected to the diaphragm walls with a full moment resisting joint. The tunnels and depressed roads need to be checked for the possibility of floatation at all stages of the construction and throughout the service life of the structure. The required soil-wall interface uplift resistance will govern the diaphragm wall embedment depth design during the permanent stage.

4 CROSS WALL FOR SHORT DIAPHRAGM WALL PANEL

The NEL tunnels underlie the proposed Woodsville interchange underpasses. At their closest distance, the NEL tunnels are located within 6 m of the final formation level for the proposed underpass structures. In order to avoid encroaching in the 3 m no dig zone of the existing tunnel, 3 m (high) x 1 m (wide) cross walls, which has similar construction method as Diaphragm wall, are constructed to provide lateral support for diaphragm wall panels below formation level. A grout curtain is installed below the diaphragm walls as an ‘equivalent’ cut-off wall to prevent groundwater drawdown. The cut off effect of the grout curtain is examined by field pump test.

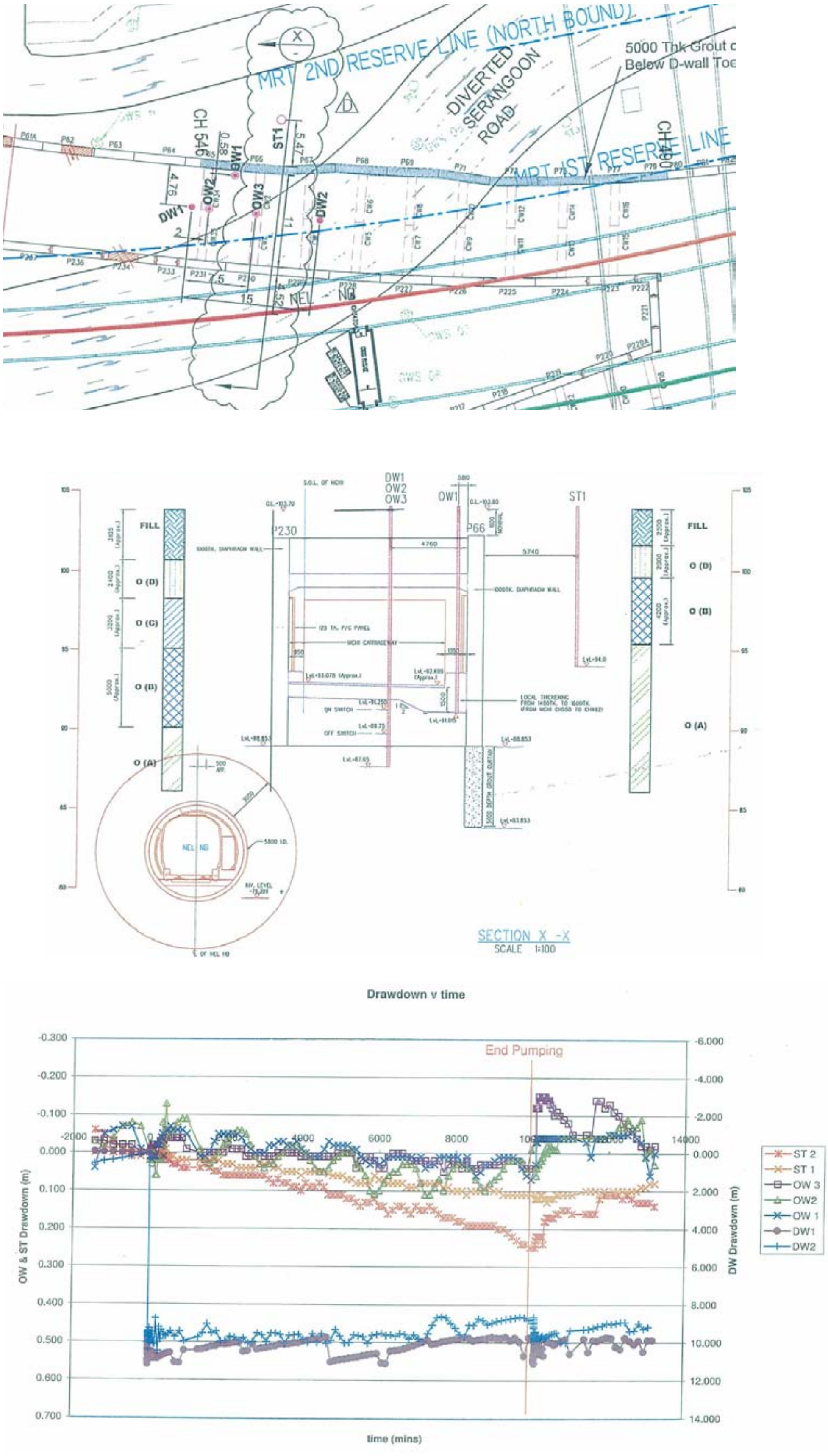


Figure 10: Field pump test at cross-wall location

5 ADEQUACY OF GEOLOGICAL AND GEOTECHNICAL DATA

The design soil profile is examined by pre-probing results, which is collected based on predrilled holes at 12 m spacing at both sides of the diaphragm wall panels. The toe level of the proposed diaphragm wall panels need to be reviewed if the pre-probing data indicate different soil conditions.

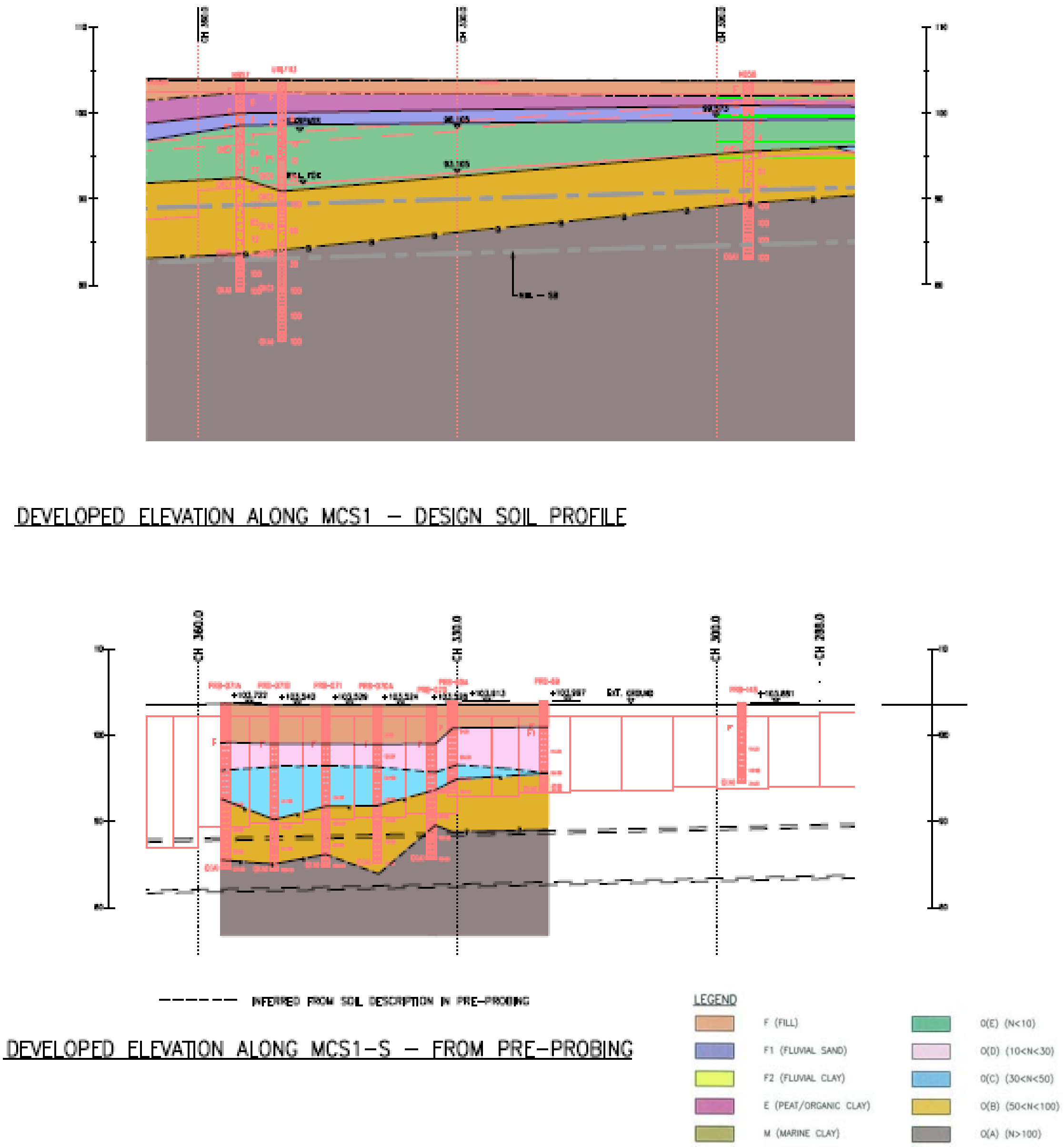


Figure 11: An example for examination of diaphragm wall toe level based on pre-probing data.

6 CONCLUSIONS AND RECOMMENDATIONS

Diaphragm walls are commonly used for deep excavation in Singapore because of its highly congested city area. This paper presents the successful application of diaphragm walls as temporary earth retaining walls during excavation stage and also as permanent external structural walls. The performance of the earth retaining structure satisfied the stringent requirements as stipulated by the local authority.

A diaphragm wall is effective in reducing wall deflections and hence ground movements for deep excavation. When there is constraint, i.e. existing tunnel, the diaphragm wall cannot be constructed to the desired toe depth. Cross wall can be installed below the formation level and provide lateral restraint to the diaphragm wall as another layer of 'strut'. However, the risks in ground water drawdown, basal heave, blow out and floatation need to be assessed also, which may not be migrated by using cross wall only.

The embedment depth of a diaphragm wall shall be designed for individual panels based on different loading conditions and design requirements at different stages. To determine the toe depth of the diaphragm wall, toe stability, basal heave, piping and hydraulic uplift shall be assessed. The toe depth of each diaphragm wall panels shall be re-confirmed based on the pre-probe holes at two wall panels spacing.

7 REFERENCES

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