

# FOUNDATION DESIGN FOR UNDERGROUND METRO STATIONS IN DUBAI

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## ABSTRACT

This paper is based upon the investigation and design stages for two of the 30 metre deep underground stations constructed in water-bearing sands and very weak rock as part of the initial development for the Red line of the Dubai Metro, United Arab Emirates (UAE). The paper firstly outlines the geological and hydro-geological conditions encountered at the City Centre and Burjuman underground stations designed by Hyder, with specific details of the geotechnical design parameters and groundwater data. The design criteria and constraints for the structures are subsequently discussed from a geotechnical perspective, in particular the issues of dewatering during construction and the impact of the long term uplift pressure after completion on the design solution. Two critical geotechnical design issues are the use of the tension piles/barrettes against uplift for the station box and load bearing barrettes for the viaducts immediately above the underground station structures. This paper also describes the progression from concept to detail designs and how the uplift issues were resolved and the lessons learnt.

Both empirical design methods and numerical modelling for the design of tension piles/barrettes are presented in the paper to emphasize the complexity, of what at face value appears straightforward. The load transfer mechanism of the viaduct load through the barrette panels and their displacement compatibility with the station box slabs were analysed using both 2D analysis and 3 dimensional methods.

At this time the permanent box structures have been completed and the design assumptions validated through the construction stage using an observational approach/monitoring, reflecting the effective and successful application of the design and decision making process.

## 1 INTRODUCTION

With the increasing development in major industrial centres of the world, underground rail metros are being adopted more readily as public transport solutions. The design of underground stations is often required to satisfy not only the functionality of the railway operation but also the commercial aspects of the underground complex. One of our recent projects is the Dubai Light Rail Transit Project (DLRT) in Dubai, United Arab Emirates (UAE). The entire DLRT stages 1 and 2 (Red Line and Green Line respectively) were awarded to Mitsubishi Heavy Industries in July 2005 under a Design & Construct Contract. The proposed light rail transit provides a rail link between the city and the airport along the coast in Dubai. Comprising some 70km of rail, and 45 stations, 10 of which are underground, the entire project has an estimated US\$5 Billion construction cost.

Design of an underground station needs to consider a range of aspects including structural strength, serviceability, long term durability and geotechnical interaction. Geotechnical design is often related to the characteristics of the subsurface materials and the groundwater nature. This paper presents a case study of design and construction processes for both Burjuman and City Centre Stations, with a focus on the Burjuman station.

## 2 PROJECT DESCRIPTION

The stations are generally some 125m long, 25m wide and 30m deep over 3 levels comprised of a concourse level, intermediate level and the lower platforms level. Each station was constructed in reinforced concrete formed by the use of external diaphragm walls, used in both the temporary and permanent conditions, and reinforced concrete floor slabs placed at three levels by a top down construction methodology. The station box reached a depth of approximately 30m at the bottom of the base slab. These stations were unique in that the internal structure spanned clearly between external diaphragm walls. This spatial requirement heavily influenced the extent and complexity of geotechnical analysis in forming a solution.

The base slabs were ultimately held down by two rows of tension barrettes across the transverse structural section. This element was a result of the design development and not an original consideration during tender design.

For both stations the entrances and ventilation shafts were designed to project from the station box, surfacing in the pedestrian areas clear of the main road traffic areas. Stations were based on a central twin track arrangement, with platforms located against the external walls of the structure, to suit the single bore rail tunnel

Burjuman Station (BUR), as shown in Figure 1 and Figure 2, is an interchange facility between the Red & Green Lines. Its plan arrangement forms a cruciform shape due to the interfacing rail alignments and is located directly under the intersection of Trade Centre Road & Khaleed Bin Al Waleed Road. The future R777 Highway will have two dual lane elevated viaducts directly over the station along Trade Centre Road. Five piers from the viaduct will be founded directly on the station.

There are a number of existing buildings with various basement levels in the vicinity of the proposed Burjuman station as shown in Figure 1. Therefore the design of the underground station needed to consider the potential impact of the station works on the adjacent building and underground services.

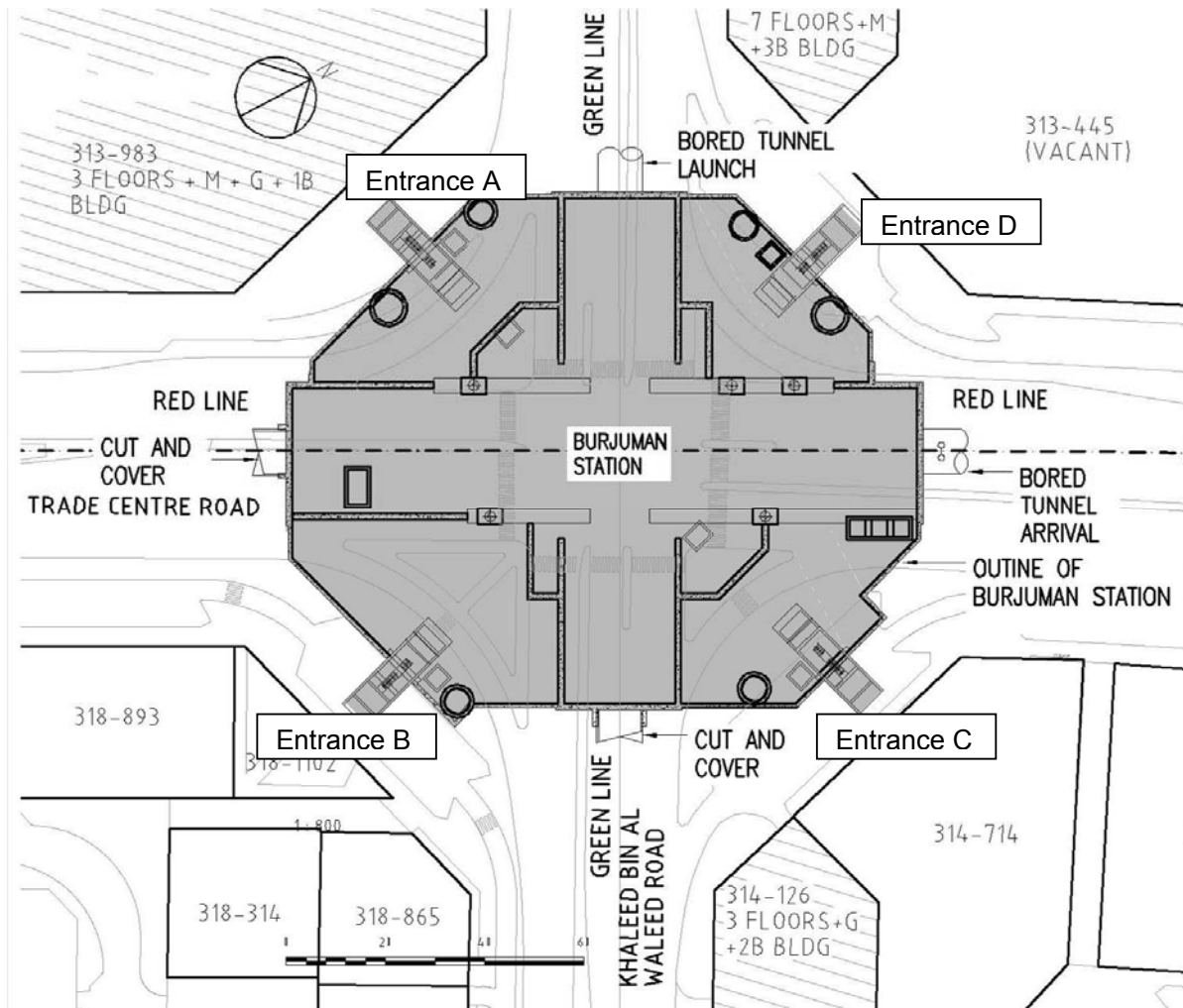


Figure 1: Layout Plan of Burjuman Station

City Centre Station (CIC) is to be constructed beneath Al Ittihad Road, just south of the intersection with Al Garhoud Road. It too, is a 3-storey reinforced concrete basement structure of rectangular shape serving as one of the four underground stations for the Red Line. The station is designed as a column free structure, with the floor slabs acting as props to the diaphragm walls to resist horizontal earth and groundwater pressure spanning some 24 metres between the diaphragm walls. The station reaches approximately 29 m depth at the lowest point beneath the base slab, and is approximately 125 m in length along the Red Line.

This paper will primarily discuss about the Burjuman station in the following sections.

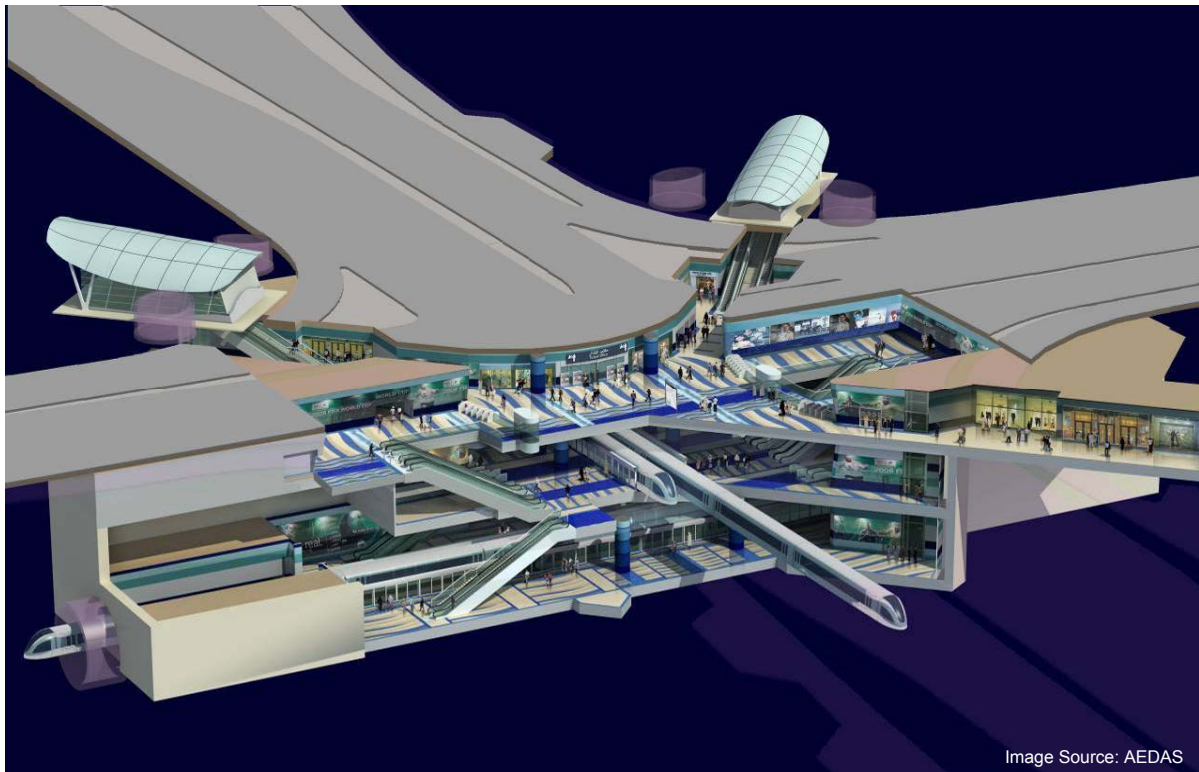


Figure 2: Cutaway Schematic of Burjuman Station

### 3 GEOTECHNICAL AND HYDROGEOLOGICAL CONDITIONS

The geology of the UAE has been substantially influenced by the deposition of marine sediments associated with sea level changes during recent geological time. With the exception of the mountainous regions shared with Oman in the North-East area, the country is relatively low-lying. The solid geology of the study area is shown to comprise alternating beds of calcarenite, calcareous sandstone and cemented sands, underlain by gypsiferous sandstone, calcisiltite, conglomerate and siltstone.

Around the footprint of Burjuman Station nine boreholes were drilled to varying depths of 55 to 61 metres and nine pumping wells drilled to varying depths of 10 to 52 metres. Standard penetration tests (SPT) and in-situ pressuremeter tests were carried out during borehole drilling. A pumping test was also carried out at the site but we did not have the field data.

The generalised geological profile consists of medium dense marine sands to approximately 15m depth with a 3 metre thick very loose to loose sand at depth of around 15m, then a 3 metre thick very low strength calcarenite overlying calcareous sandstone to approximately 35m depth, before becoming weak sandstone with siltstone layers.

The ground surface level varies approximately between +4.3 and +5.8 metre Dubai Municipal Datum (DMD). The measured groundwater level ranges between 4 to 5 metres below the ground surface. The groundwater level was taken to be at approximately 4.0m below the ground surface for the serviceability condition. For the ultimate limit state case the groundwater level was assumed to be the ground surface in a prolonged heavy storm event. The sand and calcareous sandstone was assessed to be highly permeable and friable, implying the strata could be susceptible to piping during excavation.

#### 4 GEOTECHNICAL UNITS AND DESIGN PARAMETERS

The geotechnical information was interpreted by others and the resultant design parameters provided to us for design. The geotechnical Units for the Burjuman station design is reproduced in Table 1 below:

Table 1: Geotechnical Units at the Burjuman Station

Stratum	Material Code	Description	Average Approx. Depth (m BGL*)	Average Approx. Level (m DMD)
Marine Deposits	Unit 1a	Medium dense to dense, light brown, silty fine to medium SAND with shell fragments, with some very loose to loose sands at varying depths.	0 to 16	4.7 to -11.3
	Unit 1b	Medium dense to dense, pinkish brown to greyish brown, silty fine to coarse SAND with shell fragments and occasional cemented lumps	2 to 16	2.7 to -11.3
Sandstone (Calcarenite)	Unit 2a	Slightly to moderately weathered, light brown, very weak to weak, fine to medium grained, CALCARENITE/calcareous SANDSTONE interbedded with layers of dense to very dense to silty cemented carbonate SAND	16 to 35	-11.3 to -30.3
Sandstone	Unit 2b	Slightly to moderately weathered, reddish brown, very weak to weak, fine to medium grained, gypsiferous calcareous SANDSTONE	35 to 45	-30.3 to -40.3
Siltstone (Calcsiltite)	Unit 3	Slightly to moderately weathered, off white, very weak to weak, CALCISILTITE/calcareous SILTSTONE	45+ (thickness unproven)	-40.3+ (thickness unproven)
Conglomerate	Unit 4	Slightly weathered, light reddish brown, weak to locally moderately weak, CONGLOMERATE, consisting of fine to medium gravels cemented in a sand matrix	51+ (thickness unproven)	-46.3+ (thickness unproven)

\*Note: BGL = Below ground surface level

The geotechnical design parameters used for the design of the station structures were taken from the Geotechnical Interpretive Report (GIR). These parameters are reproduced in Table 2. It was noted that the proposed stiffness parameters are lower than those from in-situ pressuremeter tests, in particular during the unloading and reloading. We note that the proposed geotechnical design parameters are of the similar order of those adopted for Hyder's other projects in the Dubai. In our discussion with the wider design team the lower values were assumed due to the fact that the calcareous sandstone could quickly reduce its strength and in turn, shows relatively lower stiffness.

Given the potential for the upper layers of marine sands to liquefy during a seismic event, it was conservatively assumed for stability considerations to ignore any positive influence of this layer. The resultant analysis considered the station box as free standing in the ultimate condition of an earthquake and ignored any skin friction acting against the diaphragm walls, supported only from the sandstone layers below Unit 2a.

We generally adopted the geotechnical parameters in Table 2 for the design and analysis of the Burjuman underground station structures.

Table 2: Geotechnical parameters adopted for the Burjuman Station structures

		Depth (m bgl)	$\nu$	$c'$ (kPa)	$\Phi'$ (°)	$E'$ (MPa)	$\gamma_b$ (Mg/m <sup>3</sup> )	$K_0$	$k$ (m/s)	$E_{sv}'$ (MPa)	$E_{sh}'$ (MPa)	$G_{max}$ (MPa)	$f_a$ (kPa)	$f_b$ (kPa)
Soil	Units 1a & 1b Marine Deposits (medium dense to dense sand)	0 to 16	0.3	0	33	35	1.8	0.5	-	35	24	100	30	-
Rock	Unit 2a Slightly to moderately weathered sandstone	16 to 35	0.2	50	37	140	2.0	0.5	4E-6	210	150	400	300	3000
	Unit 2b Slightly to moderately weathered sandstone	35 to 45	0.2	80	38	160	2.0	0.5	5E-6	240	170	460	310	3100
	Unit 3 Moderately weathered calcisiltite	45+ (thickness unproven)	0.2	50	33	180	2.0	0.5	2.5E-6	270	190	520	350	3500
	Unit 4 Slightly weathered conglomerate	51 to 55	0.2	100	38	200	2.2	0.5	-	300	210	575	300	3000

### 5 DESIGN BRIEF AND CRITERIA

A design brief was prepared initially by the Principal designer (Initially Capita Symonds (UK) and subsequently Atkins (ME)). Hyder provided substantial input into this document with significant modification made to the documentation prepared at the tender stage to normalise the criteria and methodology with the conditions observed at the sites.

The main design criteria of the station box structure related to geotechnical aspects may be summarised as follows:

- To resist flotation under both serviceability and ultimate groundwater pressure conditions;
- To prevent uplift of the base of the excavation within the boxes;
- To avoid potential piping failure of silty sands or friable weak sandstone at the base of the excavation;
- To provide sufficient lateral restraint against toe kick-out stability during excavation to the wall in order to resist lateral loads from the soil or weak rock and groundwater;
- To achieve semi-watertight during excavation in order to reduce water inflow into the base of the excavation;
- To control the potential drawdown of the groundwater level behind the wall during construction in order to minimise the settlement of the adjacent buildings and infrastructure;

The ultimate limit state design method was used for the calculation of the required wall embedment depth and the depth of tension barrettes. The partial load factors used for the flotation design are presented in Table 3.

Table 3: Partial factors of safety adopted for flotation stability

Condition	Downward Forces (D)		Upward Forces (U)	
	Partial factor of safety on weights ( $Y_m$ )	Partial factor of safety on friction ( $Y_m$ )	Partial factor of safety on water density ( $Y_f$ )	
In Service	Steel Concrete Backfill	1.03 1.05 Excluded	3.0	1.05
Extreme Event	Steel Concrete Backfill	1.01 1.04 1.00	2.5	1.03
Criterion (for each condition)	$\Sigma (D / Y_m) > \Sigma (U \times Y_f)$			

## 6 DESIGN CONCEPT DEVELOPMENT

After awarding of the D&C Contract, the project continued to develop the initial design at early post tender stage. Both the contractor and architect jointly considered modifications to the concept designs to allow unrestricted excavation and a permanent clear internal space at each level to provide a more spacious architectural impression of the underground structures. These decisions however greatly impacted upon the structure and soil/structure behaviour, resulting in greater challenges for the design of the wall and base slab designs of the station box structure.

Initially the Burjuman station layout plan was to be formed by two rectangular boxes, one being for Red Line and the other for Green Line over all three levels. During the functional development, four quadrants of entrance and underground shopping complex were added to the concourse level, as shown in Figure 3. This change made the design and analysis of the station structures more complicated by virtue of the geometry and interaction.

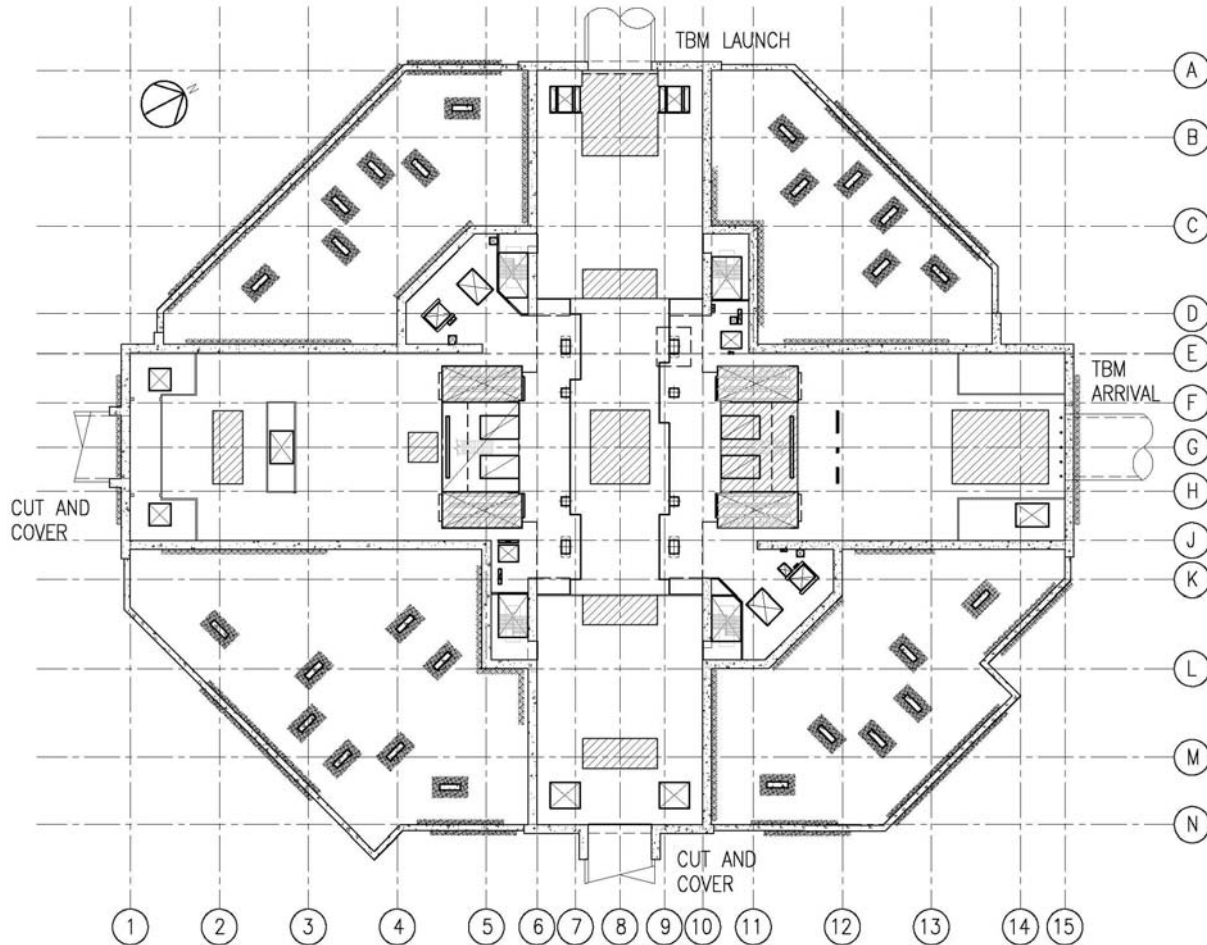


Figure 3: Detailed layout plan of Burjuman station

It appeared that the station vertical structural elements would be formed by diaphragm walls of 1.2 metre thickness due to constraints in acquiring the plant of greater width. To resist the large uplift pressure resulting from the water head difference of about 30 metres, a deep base slab of 2.8 metre thickness was initially proposed. Initially soil treatment was specified at the toe of the diaphragm walls to produce a grout plug, in order to reduce the water inflow into the excavation. The thick base slab has much greater stiffness than the diaphragm wall at their connection point, resulting in large bending moment in the diaphragm wall. During design it was rapidly concluded that the large uplift forces combined with the clear spanning structural form and limited diaphragm wall thickness proved to be competing constraints. Thus a number of options were further investigated.

These included;

- Use of one row of tension piles/barrettes to the centre of base slab

- Use of two rows of tension piles/barrettes generally at one third of the base slab span
- Use of an arched invert slab to minimise the bending moment at the diaphragm wall and slab connection
- Use of the thicker diaphragm wall to achieve compatible stiffness between the diaphragm wall and base slab

We adopted two rows of tension barrettes and 2 metre thick base slab for detailed design development. A 3D sectional view along the Red Line train direction showing the floor slabs and barrettes is shown on Figure 4. This concept enabled removal of the grout plug beneath the excavation.

Once the basic geometry and structure had been defined from the above we used a 2D *PLAXIS* analysis, to confirm and refine the design attained from empirical stability analysis through to finite element methods.

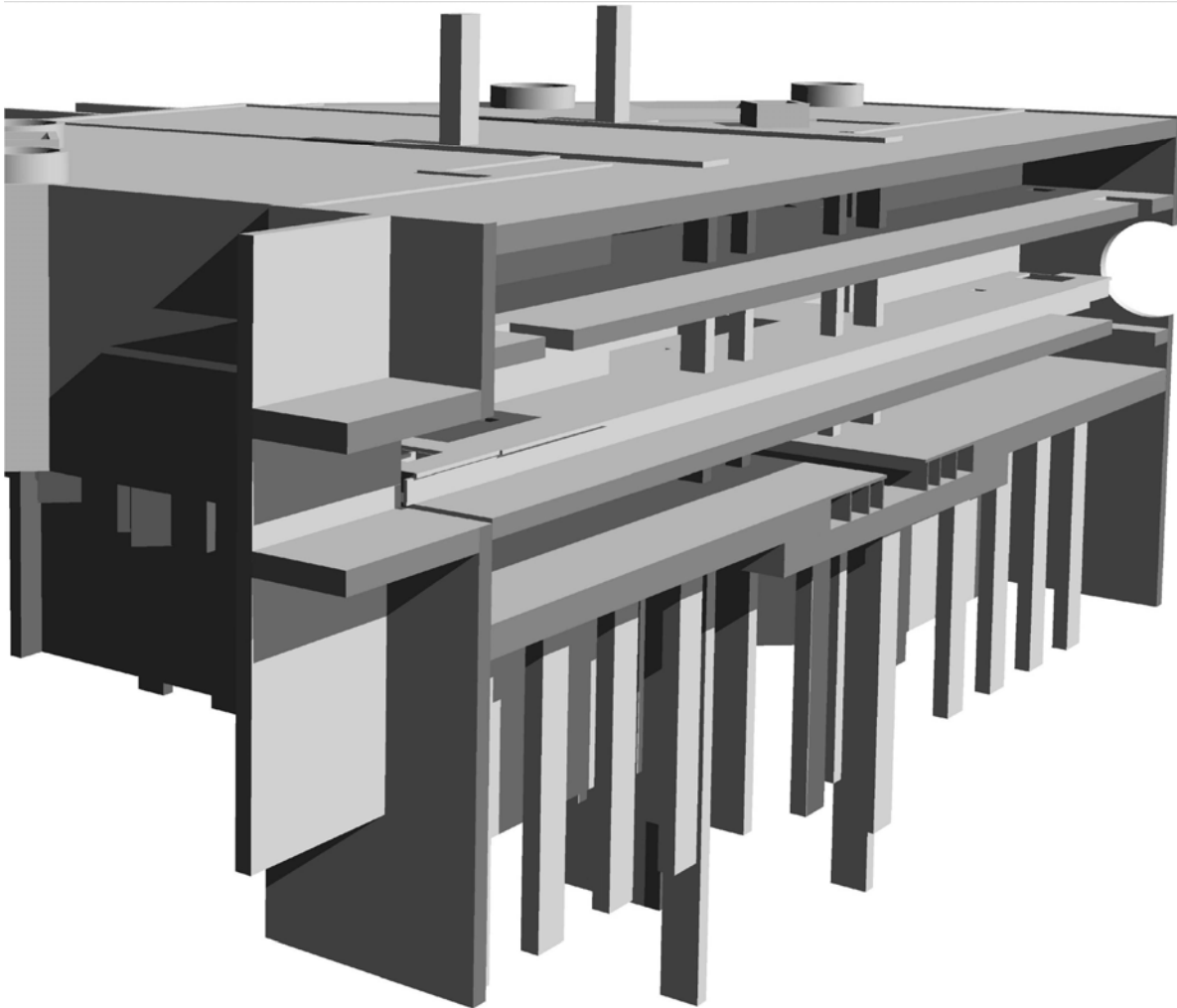


Figure 4: 3D sectional view of diaphragm walls, slabs and tension barrettes

## 7 DESIGN DEVELOPMENT AND CONSTRUCTION METHOD

The Burjuman station box utilised diaphragm walls, formed by cast in-situ construction techniques as the permanent station vertical perimeter walls, which are structurally connected by the four horizontal massive cast in-situ slabs, as shown in Figures 3, 4 and 5, to resist the soil, traffic and hydrostatic loads on the external face of the wall. The diaphragm walls were constructed using either cutter or grabs dependent upon the depth and the anticipated strength of the materials to be excavated. The top-down construction technique presented many challenges in coordination of the dewatering, temporary propping of the diaphragm walls and construction of the slabs prior to proceeding to the next level of excavation. Due to the complexity of the layout plan of the station many different sections were analysed in order to consider the most heavily loaded diaphragm wall panels of the station.

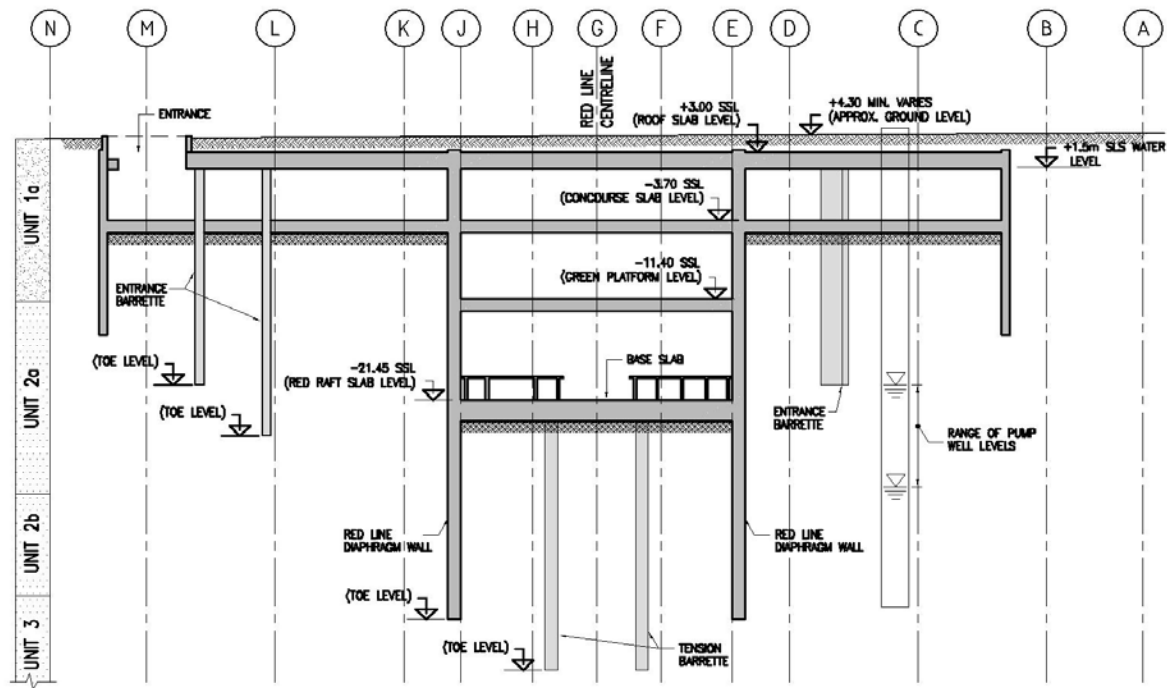


Figure 5: Typical cross-section at Grid 3

A flotation check was also carried out for the entire station box including the entrance structures in order to satisfy both serviceability and ultimate conditions.

A three dimensional structural frame analysis was concurrently carried out to assess the equivalent stiffness of the columns in the entrance shopping areas at the shallow depth for the two dimensional model. Some 11 sections were selected for the detailed design and analysis in order to consider all the diaphragm wall panel depths, reinforcement and connection details to the floor slabs. For each of the typical cross sections the following analyses were undertaken:

- Lateral toe stability of the diaphragm wall at the final excavation stage using Philip's limit equilibrium method as outlined in CIRIA C580 (2003).
- Seepage analysis to assess the potential of piping at each stage of propping, dewatering, excavation and installation of slabs.
- 2D Plaxis analyses taking account of the soil and water in front of and behind the diaphragm walls, the multiple staged dewatering, excavation propping and placement of the slabs.
- Prediction of diaphragm wall lateral movements and ground settlements.
- Assessment of the bending moment, shear force and axial forces in the diaphragm wall and slabs.
- Assessment of the tension loads in the barrettes connected to the base slab.

### 7.1 FLOTATION OF STATION BOX STRUCTURE

The flotation design was carried out for the entire underground structure to initially determine the minimum diaphragm wall depth to give vertical stability of the structure against uplift in accordance with the design criteria. Both serviceability groundwater level at +1.5 m DMD and ultimate limit state groundwater at approximately +5.0m DMD, were considered in the flotation calculations.

The restoring downward forces against uplift for the station box included:-

- Dead weight of the permanent structural elements
- Weight of permanent fill above the roof slab
- Frictional resistance to diaphragm wall based on the inner and outer surfaces of the wall below the underside of the base slab.

Frictional resistance of the ground on the diaphragm wall was only considered beneath the base slab level and taken to be the product of the effective frictional coefficient and the effective horizontal stress. In all cases, the friction coefficient on the “active” side was taken to be 0.67 whilst it was assumed to be 0.5 on the “passive” (excavation) side to take account of the interface effects.

Uplift forces were calculated for two operating conditions, in service with a design ground water table at +1.5m DMD and an extreme water table at ground surface level.

The toe levels of the diaphragm walls and tension barrettes were determined by trial and error method in order to satisfy all the design requirements discussed in the previous sections. It was noted that the toe levels of the diaphragm walls were mainly governed by the cut-off requirement in order to avoid the potential of piping when excavation is carried out down to the final excavation level. It was found that the toe levels of the diaphragm walls for the Red Line should be at RL -43m DMD while those for the Green line were at RL -40.5m DMD at the detail design stage.

## 7.2 PIPING STABILITY

After a preliminary diaphragm wall toe level is determined a piping check using the empirical method based on NAVFAC (1982) was carried out for construction stages prior to completion of the base slab. A minimum factor of safety against piping greater than 1.2 for normal section and 1.0 for the corner at which the hydraulic exit gradient is most critical was required in that the GIR presented relatively high permeability values for the material below the excavation.

This was further checked by *SEEPW* analysis or *PLAXIS* analysis to assess the exit hydraulic gradient at the base of excavation. The results indicated that the assumed permeability values for the material below the excavation and the cut-off diaphragm wall are critical in the calculated hydraulic gradient. The key issue identified was establishing the connectivity of the “flow path” through the friable and very weak rock mass.

## 7.3 WALL LATERAL STABILITY

Lateral diaphragm wall stability was assessed by hand calculations using the Phillips Method for a multi-propped statically indeterminate wall. This method was used to determine the factor of safety on the wall embedment depth. The method assumes a sliding block failure mechanism in the soil with a plastic hinge forming about the lowest support level.

For this analysis the critical stage was just before the base slab is cast. The excavation level would be at a maximum and the lowest support will be the last temporary prop. An over-excavation of 0.5m was allowed in the stability assessment. Three rows of temporary props were used for Red Line.

For working stress method a minimum factor of safety of 1.5 was required in that the consequence of structural failure is significant. For the ultimate limit state (ULS) method a strength reduction factor on effective strength,  $F_s$ , of 1.2 was required in accordance with CIRIA C580.

For a given toe level of diaphragm walls the level of the lowest temporary prop was determined not only to achieve a sufficient safety margin against lateral kick out instability but also to provide sufficient space for ease access of construction plant.

## 7.4 NUMERICAL ANALYSIS

Numerical analysis was performed using a variety of software packages, predominantly 2D *PLAXIS V8.x* Finite Element Code for soil-structure interaction. A comparison analysis was performed for a small box using *PLAXIS 3D* to establish the benchmark analysis using 2D *PLAXIS*.

The *PLAXIS* analysis was initially performed using the Mohr-Coulomb (M-C) model which was specified as part of the design requirement by the Engineers for the client. Later the analysis was refined to use the Soil Hardening (S-H) material model, in order to account for the effects of the increased stiffness of soil and weak rock during unloading and reloading at different phases of the construction. This was a pivotal conclusion as the M-C derived results produced deformations in the structure that were unlikely to occur, in particular the rebound in deformation on the passive side of the wall when recharge water pressures were established against the underside of the raft slab. The S-H mode considers this phenomenon more correctly leading to a more efficient design.

As the *PLAXIS* analysis was performed using a 2 dimensional model, load smearing and shape coefficients were derived by a standard 3 dimensional frame analysis in order to accurately represent the interactions and effects of the tension barrettes, various other concentrated loads and geometric anomalies within the station. Relative stiffness of columns and barrettes at the sections taken were determined by performing a finite element structural slab analysis and determining the effective stiffness of the barrette at the section location from the output displacement of the slab.

The 2D *PLAXIS* analysis considered so-called “coupled” stress, strain and pore water pressure analysis for each stage of dewatering, excavation, propping and construction of slabs.

A steady-state groundwater flow calculation was performed using *PLAXIS*, in between the geometry changes of the construction phases within the calculation. This provided an estimate of the pore water pressure to be expected while maintaining the water level drawdown to within 2 m outside the main station box. A separate transient groundwater flow calculation check was performed using *SeepW*, which resulted in almost identical pore water pressure profiles.

The actual pore water pressure distribution used in the modelling was based on the *PLAXIS* generated steady state groundwater flow calculation at each excavation stage.

The interface strength parameter  $R_{inter}$  in *PLAXIS* was set at 0.67 on the active side of the diaphragm wall, and at 0.5 on the passive side, in accordance with CIRIA C580.

The tension barrettes measured 1200mm x 2800mm were installed below the base slab from the ground surface, generally in between the main station box diaphragm walls, and at a spacing of 8.0m along the train travel direction. Smaller 800mmx2800mm barrettes were installed to support the entrance roof slabs for the entrance complex as shown on Figure 6. In the soil-structure interaction using the *PLAXIS* model, barrettes were modelled as a ‘beam’ element. The interface parameter between the soil and the barrette,  $R_{inter}$ , was calculated as  $P/3S$ . Note that P is the perimeter of the barrette and S is the centre to centre spacing between two barrettes along the train travel direction

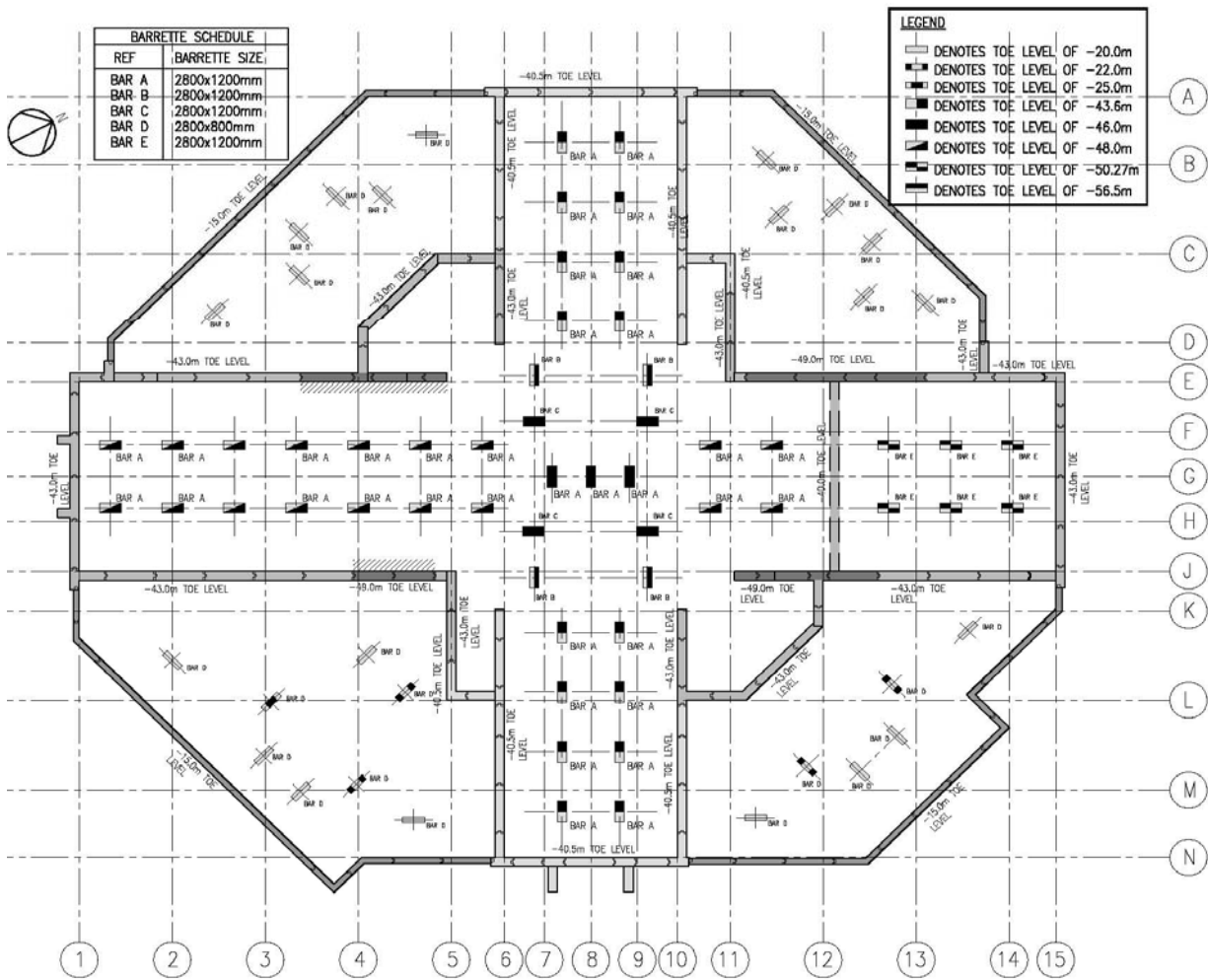


Figure 6: Layout plan and design toe levels for various types of barrettes

### 7.5 TENSION BARRETTE DESIGN

Once the toe levels of the diaphragm walls were finalised to satisfy the flotation requirements further detailed analysis was performed to determine the required length of the tension barrettes. The primary objectives of tension barrettes were to provide sufficient tie down capacity for the base slab and to reduce the bending moment at the base slab and wall connection.

The following was considered in the assessment of the tension barrettes:

- The number of rows of tension barrette in typical cross section
- The tension load on each of the barrette
- The spacing of the tension barrette in the train travel direction
- The shaft resistance between barrette and surrounding rock mass
- The pullout capacity of cone/wedge or prism surrounding the barrette

A long and comprehensive iterative process was required to determine how many rows of tension barrettes beneath the base slab. As mentioned in Section 6, finally two rows of tension barrettes were adopted by due consideration of the thickness of the base slab and bending moment in the diaphragm wall. Three methods were used to determine the required tension load on the barrette after the layout of the tension barrettes was finalised. These are as follows:

- 2D Plaxis Analysis
- Simplified hand calculation
- 3D Strand 7 model analysis

For each type of the analyses both serviceability groundwater level at RL+1.5m DMD and ultimate groundwater level of RL+5.0m DMD were checked for the required length of the tension barrettes. We found that the design was governed by the serviceability case. Therefore the results of the serviceability analysis for groundwater level at RL+1.5m DM are presented in Table 4 below.

Table 4 : Calculated tension barrette loads under serviceability condition

Method of Analysis	Red Line		Green Line	
	Row 1 (kN)	Row 2 (kN)	Row 1 (kN)	Row 2 (kN)
2D Plaxis Analysis using M-C model	17000	17100	10200	10200
2D Plaxis Analysis using S-H model	12900	13300	9200	9200
Simplified hand calculation	15300	15300	11100	11100
3D Strand 7 Analysis	14900*	15200*	11200*	11200*

\* Note: The average of the calculated values for the row

We note that the calculated tension loads in barrettes using M-C model are higher than those from S-H model. This was due to the larger upward “heave” movement underneath the base slab, which is dependent upon the loading and unloading stiffness of the weak rock, when the groundwater level was set to restore to the original serviceability level.

The calculated tension loads in the barrettes from the 3D *STRAND7* analysis show that the loads vary from barrette to barrette. However the simplified hand calculations based on the uplift tributary area is an “average value”. By comparison of the calculated values using the three methods we concluded that it was more convenient to obtain the results from 3D *Strand7* analysis but would reasonably satisfy the design requirements. We used an average tension load over a “zone” within the station box to normalize the design the embedment depth of the tension barrettes below the base slab to take account of the ease for set-out and construction and the “group” effect in three dimensional context.

The shaft resistance of the barrette in weak rock was based on the GIR provided and a reduction of 0.7 was used on the compression shaft resistance to come up with the shaft resistance in tension. A factor of safety of 3.0 was applied to the ultimate tension shaft resistance for serviceability check while a factor of safety of 2.5 was used for the ultimate limit state check.

A wedge/prism pullout capacity check for a group of tension barrettes was also carried out. We assumed that a minimum factor of safety of 1.1 was required for the whole group of barrettes pullout, and the tip of the inverted wedge of the tension barrette was taken to be at an inclination angle of 30 degrees to the vertical and about 3 metres above the tip of the barrette. The volume of the soil and self-weight was calculated by hand and verified by three dimensional AUTOCAD software. Both methods gave us similar results.

The layout plan and the toe levels of tension barrettes beneath the base slab of Red Line and Green Line together tension barrettes for the slabs of entrances are shown on Figure 6.

There are four number of “mega” columns at the intersection of the Green Line and Red Line. These columns are needed to minimise the clear span in each direction of the train travel. These columns were designed for both compression loads during construction and tension loads in the long term. The embedment depths of these columns are shown on Figures 4 and 6.

The barrettes in the entrance and shopping complex areas, as shown on Figure 6, were designed for both compression and tension loads. For the compression capacity the end bearing was considered while for tension the base bearing capacity was ignored. It is worth noting that the embedment depths of most barrettes are governed by compression loads with only a few being controlled by tension loads.

## 8 ELEVATED VIADUCT FOUNDATIONS

The Dubai Municipality, Roads & Transport Authority (RTA) investigated the option of having a 5<sup>th</sup> crossing across the Dubai Creek which will require two dual-lane viaducts running over Burjuman station as an elevated structure. The freeway system is referred to as the R777 highway. The RTA Road Project Consultant, *Cansult*, compiled vertical and horizontal design loads for the viaduct columns at various locations on and around Burjuman Station. Five of the viaduct piers, P1 through P5 respectively as shown on Figure 7, are positioned over the diaphragm walls along the Red Line. The vertical working loads, which vary from 23 MN to 26 MN, are transferred to the station from the proposed viaduct with each column bearing on a stiffened capping beam connected to the diaphragm wall panels and roof slab.

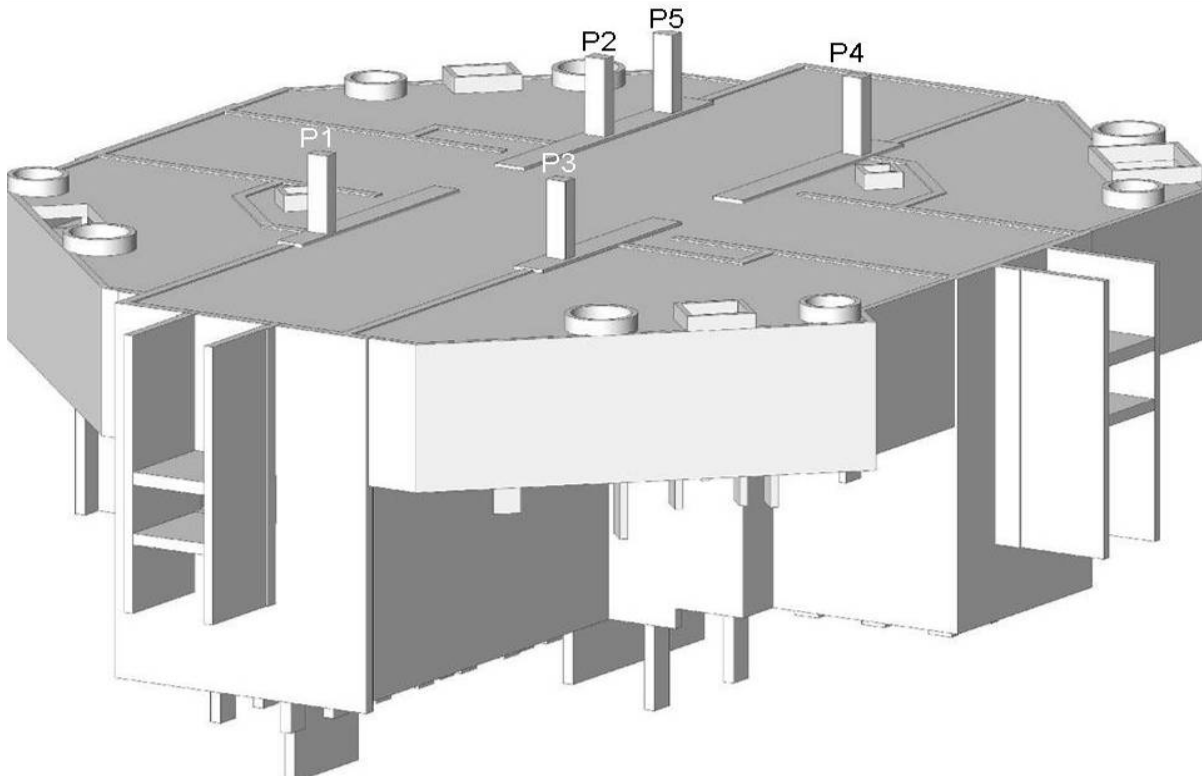


Figure 7: Layout plan of five future R777 bridge columns above Burjuman station.

Initially a design concept was developed by assessing the potential load spread due to the presence of the roof, course, Green line and Redline slabs. The analysis was carried out by 3D frame analysis. After careful consideration the vertical column loads were assumed to be transferred downward and spread laterally to a minimum of 2 diaphragm panels via the stiffened capping beam.

In the geotechnical assessment it was conservatively assumed that there would be no shear contribution from the adjacent diaphragm wall panels. The additional vertical column load would be carried by the extension of the diaphragm wall below the depth for the station box structure which was required without viaduct loading.

Design analysis was undertaken using *DEFPIG* software to assess the settlement of the diaphragm walls under the R777 piers based on the working vertical loads in Table 5. The diaphragm wall was either conceptualized as a single large pile or a series of piles with equivalent perimeter of the 2 barrettes. The calculated settlements are shown in Table 6.

Table 5: Summary of working vertical column loads from future R777 bridge above Burjuman station

Column ID	Vertical Working Load (kN)
P1	24,140
P2	25,390
P3	25,850
P4	24,530
P5	23,410

A structural analysis was also performed using *RAM Concept* finite element and the plane frame *Microstran* analysis to investigate the shear transfer to adjacent diaphragm wall units.

Based on the above analysis it was concluded that the station box structure would be stiff enough to carry the additional loads from the future R777 bridge, and the predicted settlement would be of the order of 5 to 10mm, which was considered to be acceptable for the elevated bridge structure.

*PLAXIS* analysis was also undertaken to assess the response of the diaphragm wall to the additional applied load from the viaduct column. Considering the three dimensional effect the column load was either assumed to be transferred on a 12.8 metre wide barrette or 25.6 metre which is about the depth of the station box structure. The calculated vertical settlement from *PLAXIS* analysis is summarised in Table 6.

Table 6: Summary of predicted settlements of column foundations

Case	Key Assumptions	Analysis Software	Vertical Settlement (mm)
I	1) Load uniformly transferred down to 12.8m wide diaphragm wall panels. 2) Wall was simulated as a series of contiguous piles with equivalent perimeter 3) Use of Young's modulus of rock based on GIR	DEFPIG	10.5
II	Same as Case I but with 2.5 times of Young's modulus in GIR, along the sides of the diaphragm wall	DEFPIG	4.1
III	1) Load uniformly transferred down to 12.8m wide diaphragm wall panels. 2) The load in PLAXIS was assumed equal to the working vertical load divided by 12.8m.	2D PLAXIS	20.1
IV	Same as Case III but the load intensity was reduced by 50% to take account of the greater level of lateral spread	2D PLAXIS	10.0

Only the shaft adhesion on both sides of the diaphragm wall panels below toe levels which were required for station box design was considered when calculating the foundation capacity to carry the additional load. End bearing of the rock below the toe of the diaphragm wall panel was not considered due to a relatively larger settlement being required to mobilise its capacity.

It was found that to support the additional load of the bridge piers, the diaphragm wall toe depth would need to be extended to RL-49.0m DMD, in order to maintain the settlement criteria imposed by the additional load.

## 9 INSTRUMENTATION AND MONITORING

The performance of the tension barrettes for the main station and the barrettes for the entrance commercial concourse as well as the R777 bridge column foundations had to be verified during construction stage.

A comprehensive instrumentation and monitoring programme was devised by others with significant input from various components of the designers. Hyder provided input for the instrumentation requirements for the Burjuman station work. A summary of the instrument type and quantity is presented in Table 7. Two dewatering wells were installed at each of the four entrance complex areas with twelve being in the main station box.

Of all the predicted effects the following are important to the performance of the station structure:

- The diaphragm wall lateral movement
- The settlement of wall and adjacent infrastructure
- The compression loads in the slab
- The loads in the temporary props
- The groundwater levels and drawdown.

Table 7: Summary of instrumentation types and quantity installed for Burjuman station

Item No	Instrument Type	Total Quantity As Per Design	Total Quantity Installed	Percentage Installed
1	Surface Settlement Marker	155	152	100%
2	Building settlement marker	26	26	100%
3	Tilt Meter	9	9	100%
4	Inclinometer	10	9	90%
5	Piezometer	3	3	100%
6	Water Standpipe	7	6	86%
7	Load Cell*	7	7	100%
8	Strain Gauge*	10	10	100%
9	Pumping well in Entrance	8	8	100%
10	Pumping well in Station Box	12	12	100%

\* There is a pair of load cells or strain gauges at the nominated locations.

The lateral movement of the diaphragm wall was to be measured by inclinometers through the wall. In addition laser targets were installed into the diaphragm wall at various stages of excavation to measure its relative lateral movement. Load cells and strain gauges were installed in the slabs and temporary props to measure the load developed in the lateral structural elements at each stage of excavation. It was mandatory to control the drawdown of the water level outside the diaphragm wall in order to minimise the ground settlement which will in turn affect the adjacent existing infrastructure and buildings.

The original design philosophy was to maintain the drawdown of the water level behind the excavation to be no more than 2 metres by the use of injection wells or in-situ grouting if required, which was to be the responsibility of the contractor as it was heavily dependent upon the method of construction, sequencing and their control measures.

The performance of the tension barrettes cannot be directly measured in that they are beneath the base slab and will not be engaged until the groundwater level behind the station box is fully restored. However a tension pile test was carried out at another station site to make sure the assumed shaft resistance presented in the GIR is achievable. We were verbally advised that the actual tension pile performance was better than what was in the GIR although we could not attain the test report at the time of writing this paper.

During construction stages Hyder was requested to review the design with an objective to optimise the temporary prop design, in particular to eliminate one row of the temporary props. The request was from the Contractor who did no site measurements and realised the station structure behaviour was much better than originally predicted. The actual lateral

movement in the diaphragm walls based on the inclinometer readings, as shown on Figure 8, was ranging between +6mm and -6mm.

We carried out further assessment of the lateral movements using unloading and reloading Young's modulus of the weak rock below the excavation. The results of revised analyses are summarised in Table 8. The three cases analysed are described below:

- Case (a): Most probable, without eliminating the bottom temporary props but the actual unloading and reloading Young's modulus values were taken to be 2.5 times those in GIR;
- Case (b): Moderate conservative, same as Case (a) but eliminating the bottom temporary props
- Case (c): Most unfavourable, the original design case with GIR parameters

Table 8: Summary of measured and predicted maximum lateral displacement of diaphragm wall at selected sections

Section	Measured to date (mm)	Displacement (mm) "most probable"	Displacement (mm) "moderately conservative"	Displacement (mm) "most unfavourable"
Grid 2	<6	18	27	31
Grid 3	<6	16	23	27
Grid 4	<6	10	20	21
Grid 5	<6	17	29.5	31.5

It should be emphasized that the above described cases were carried out on the assumption that an observational approach would be undertaken by the Contractor on site during construction. The predicted maximum diaphragm wall lateral movements are of the order of 20 to 30mm after elimination of the bottom temporary props, which would not be worse than the expected movement for the original design.

We noted from the observed groundwater levels at the pumping wells that the water levels at the four entrances area are much lower than originally predicted in the PLAXIS modelling and subsequently recharging outside the perimeter of the station box was carried out by the contractor. As can be seen from Figure 9 the water levels at the pumping wells at all Entrances A, B, C and D have been around RL -25m to -30m DMD on 20 December 2007 except those not being measured. Note that these levels are even lower than underside of the base slab, which, in turn, would have reduced the water pressure behind the perimeter diaphragm walls of the station box. It could be inferred that the lowering of the water levels is the most likely cause of much lower measured lateral movements of the diaphragm walls instrumented and almost no leakage at the construction joints between the two diaphragm panels as shown on Figure 10.

Our original predicted maximum settlement was of the order of 50mm around the station box perimeter. The monitored ground settlement was generally of the order of 10mm to 40mm with a maximum settlement value of about 60mm which occurred around the corners of the cut and cover tunnels and the station box.

## 10 LESSONS LEARNT

Some of the lessons learnt from concept design to detail design and the construction stage advice may be summarised as follows:

- Interface between the architectural design and engineering concept should go hand in hand so that the most efficient and economic design can be achieved at early stage of the project.
- Success of such a significant project is largely dependent upon the sensible concept design.
- Soil-structure interaction can be analysed by the use of advanced three dimensional and two dimensional software packages with a thorough understanding of basic mechanics principles.
- Detail design development should always have the contractor's input so that the design would be close to the actual structure to be constructed.
- Instrumentation and monitoring should be carried out on site so that the actual performance of the structure can be evaluated during each stage of construction.
- The actual behaviours of the soil and rock mass is strain dependent which should be fully appreciated in the use of the input design parameters in the design and effect prediction.

- The designer should have close interaction with the contractor during construction stage so that any issues identified could be resolved as soon as practical.
- Observational approach is still one of the most useful and practical methods in geotechnical engineering.

## 11 CONCLUSIONS

At the time of writing this paper the construction of the base slab for the entire station has been completed. The latest monitoring results indicates that actual behaviour of the station structures are better than originally predicted. This is largely due to lowering of the groundwater levels behind the wall and the relatively stiffer responses of the soil and weak rock under much smaller strain condition.

The use of the sophisticated software packages has enabled us to investigate, analyse and design two complex underground stations, and derive a successful design with an excellent concept design using fundamental soil mechanics principles. It is reflected that sound engineering judgment is always required in geotechnical assessment and decision-making process.

## 12 ACKNOWLEDGEMENT

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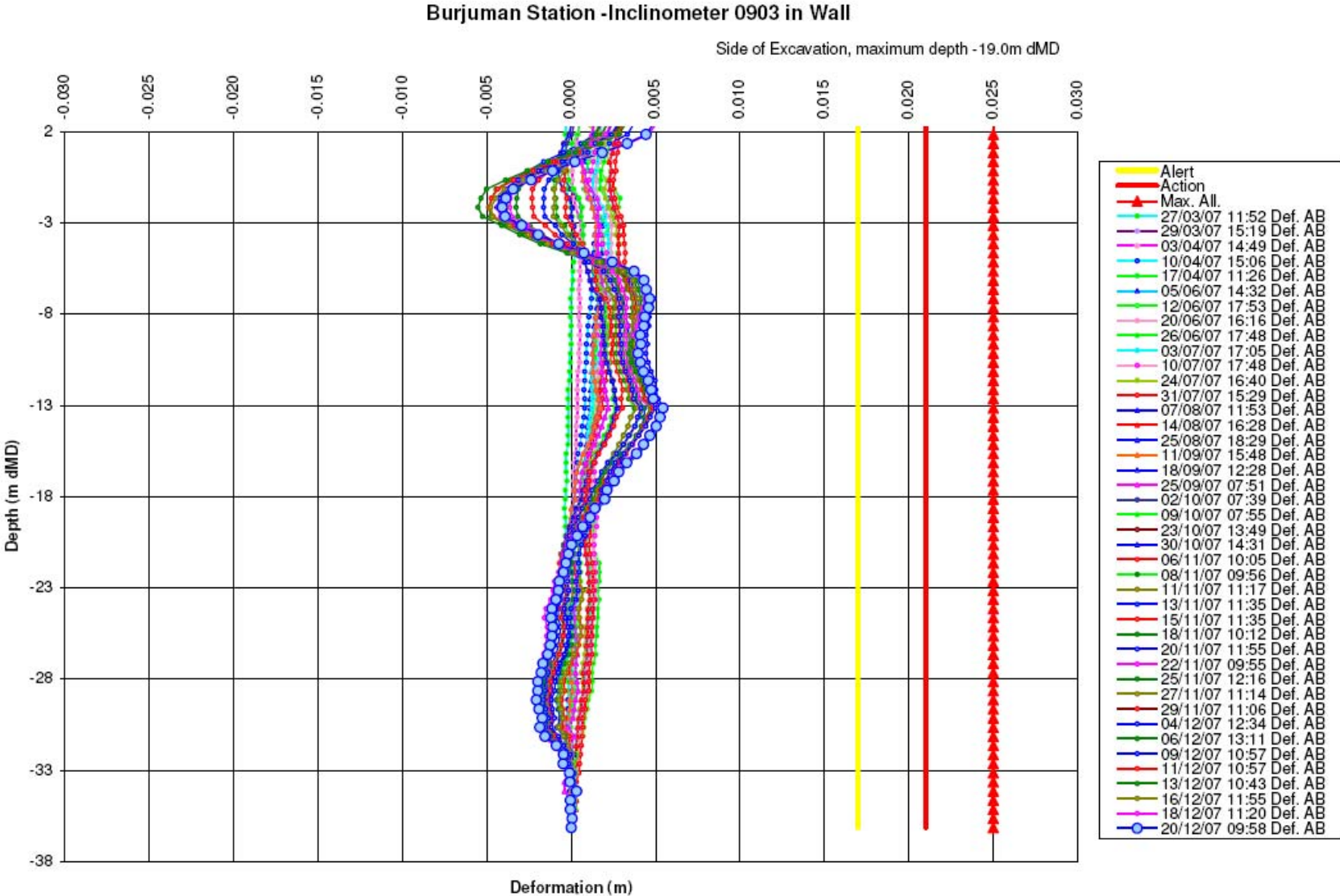


Figure 8: Diaphragm wall lateral movement measured at Inclinator 0903

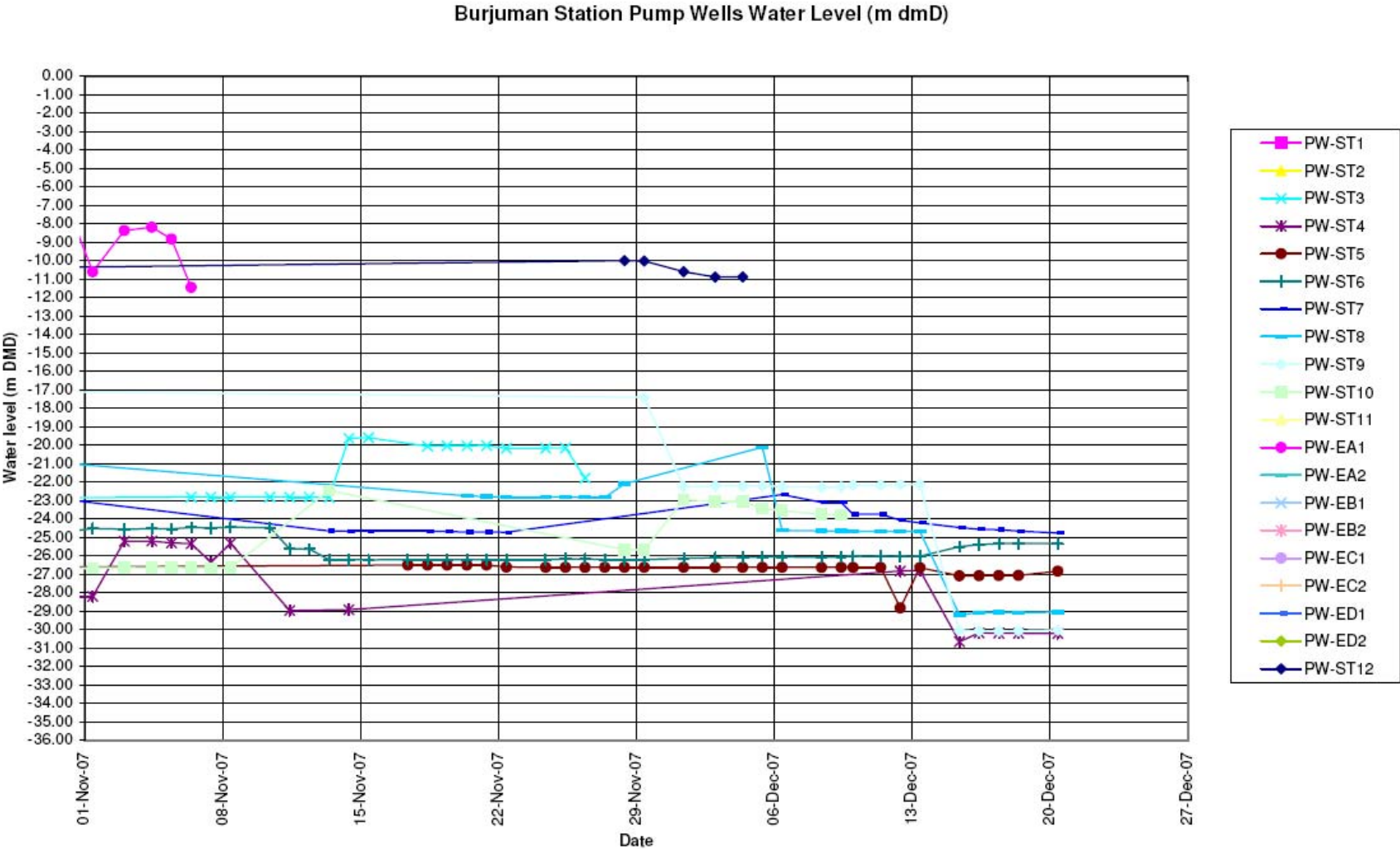


Figure 9: Measured water levels in the pumping wells at Burjuman station



Plate 1



Plate 2



Plate 3



Plate 4

Figure 10: Photos of different levels of the station

