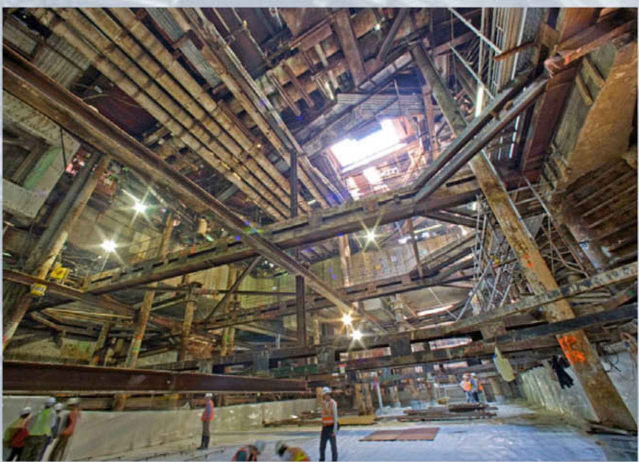
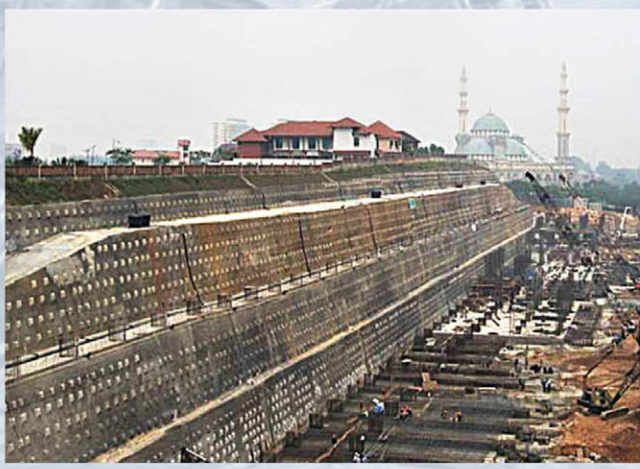
APEC Seminar on the State-of-the-Practice of Deep Excavation Works in Malaysia, Taiwan and Hong Kong

21st May, 2011





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HKGES
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Since 2003

APEC Seminar on

The State-of-the-Practice of Deep Excavation Works in Malaysia, Taiwan and Hong Kong

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FOREWORD

The Hong Kong Institution of Engineers (HKIE), in particular the Geotechnical Division (GD) and the Hong Kong Geotechnical Society (HKGES) promote close collaboration with other Professional Engineers' Institutions within the region to enhance sharing of engineering knowledge and practical experiences. Over the past 5 years, there have been extensive technical activities involving Chinese Taipei, Malaysia and Hong Kong as evident from the many seminars, conferences and technical visits jointly organised or undertaken in these places.

This seminar is a continuation of the Chinese Institute of Engineers (CIE) – Institution of Engineers, Malaysia (IEM) Geotechnical Engineering Joint-Seminar held in Yilan, Taiwan in late August, 2009 in which the HKIE – GD and HKGES sponsored and organised a delegation to attend the Joint Seminar as well as the Chinese Taipei Geotechnical Society (CTGS) Conference. The delegation was generously received by CTGS and CIE. Our delegation gained first hand appreciation of the geo-hazards highlighted by the extreme weather conditions of Typhoon Morakot, that hit Taiwan in August 2009 as well as the Malaysia's design and construction practices which are much less regulated allowing room for the geotechnical engineers to maneuver or innovate. It is appreciated that in strengthening our collaboration, better solutions can be found for our geotechnical challenges and problems. It is on this basis that the CIE-IEM Joint Seminar has become CIE-IEM-HKIE cum APEC seminar in which our Taiwan and Malaysian engineers can come to Hong Kong to share their experience and views with Hong Kong engineers. This seminar comprises nine presentations on topics of start-of-the-art design and construction practices of deep excavation works in Hong Kong, Taiwan and Malaysia, with three presentations from each of the cities/country.

On behalf of the Division's Organising Committee and HKGES, we would like to express our gratitude to Ir Yu Ter Chyuan of the Chinese Institute of Engineers and Ir Tan Yean Chin of the Institution of Engineers, Malaysia for their help in organising their delegations to participate in the Seminar. We would also like to thank the delegates for their efforts in preparing the technical papers and presentations.

Finally, we would like to thank the work of the Organising Committee of the Seminar, in particular the dedicated efforts of Ir James Sze and Ir Raymond Koo.

Ir Ringo YU
Chairman
HKIE Geotechnical Division 2010/11 Session

Ir Albert HO
Chairman of Organising Committee
President of the Hong Kong Geotechnical Society

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The Underground Circle MRT Station under the Intersection at the Busiest Transportational and Commercial Spots – Application of 140M Diameters Circular Diaphragm Wall for Kaohsiung Mass Rapid Transit System O5R10 Station

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ABSTRACT

The Formosa O5/R10 Transit crossing station of CR4, where the KMRT (Kaohsiung Mass Rapid Transit System) routes (The Red and Orange Lines) meet, and is one of the busiest transportations and commercial spots at the intersection of Chung Shan and Chung Chen Roads. This Formosa Boulevard station (O5/R10) of CR4 is built by 140m diameter circular diaphragm wall, which has been applied for LNG underground Tank Project, but such a large scale diameter diaphragm wall under such severe conditions, in the center of the metropolitan city with maintaining heavy traffic flow and keeping safety and environment surrounding commercial spots, has not been used in the past all over the world. This paper introduce the Planning, Design and Construction Records regarding the above circular diaphragm wall project.

1 INTRODUCTION

Kaohsiung mass rapid transit project for rectification, the expansion of existing transport infrastructure, divided into the following two major road construction. A red line, starting at the Kaohsiung airport along the North-South artery-Chung Shan Road through Kaohsiung station reach the northern suburb of city of Gangshan (full length 28.3km). The other is the orange line, in the East of city of Fengshan, along the East-West artery- Chung Cheng Road straight reach Sun Yat-sen University where may look at the Xizi Bay (full length 14.4km).

This project works by turnkey contract (Design and Construction merger contract), and BOT mode (Built, Operate, Transfer: by private enterprises construction and operation of a facility, after a certain period and then transferred to the central or local government) (Figure 1.1 Kaohsiung rapid transit system of the road map).

CR4 Lot work area located just below the circle at the crossroads of Chung Shan Road and Chung Cheng Road, containing thereafter of the red line and orange line interchange "Formosa Boulevard Station" (following "O5/R10 station"), is the largest work area of Kaohsiung mass rapid transit project (Figure 1.2 CR4 work area as a whole).

2 BASIC PLANNING AND THE PROPOSAL OF DESIGN CHANGE (ALTERNATIVE)

2.1 Basic planning

In Kaohsiung MRT project, the RC diaphragm wall is used as the retaining system for the underground stations as well as open-cut tunnel sections, due to the mining depth of 20~30m and very close to nearby buildings, and there are many large depth digging performance in Taiwan and has a height of wall rigid, less on peripheral sites .

The basic planning of the O5/R10 station of diaphragm wall plan is shown in figure 2.1.The diaphragm plan is not round and formed complex polygon in the original design, due to the complex shapes of the station in plane, and excavation of 100m wide range; if the General method of construction is used, construction area must be divided into several parts for diaphragm wall construction, needed to erection of additional lateral bracing system for excavation, and the construction of entire station structure should be split. But if in this way at O5/R10 station construction, is bound to face various issues such as the transportation maintain planning at the crossroads, construction, and duration etc.

Figure 1.1: Road map for the Kaohsiung MRT system

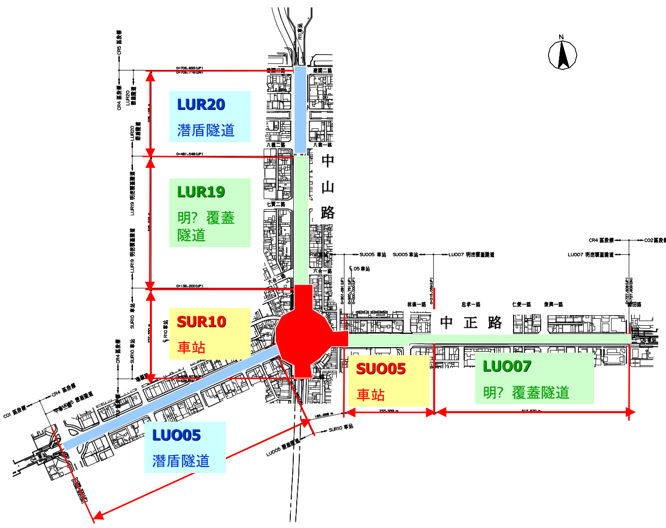

Figure 1.2: CR4 work area as a whole

2.2 Change design (basic design)

Therefore, in order to solve the above problems, the construction team (CR4 RSEA-KAJIMA JV) aimed at the time the Japan has many LNG underground storage tank project performance, the non-supporting construction method – using of diameter up to 100m level main circular diaphragm wall construction of the station was suggested to the owners.. This method was later adopted as the basic design of the original programme.

However, as the basic design of jobs, There were many rooms for additional equipment such as ventilation and cooling tower increasing around the outside of the circular diaphragm wall. The last basic design, in internal diameter 105m circle of perimeter, were surrounded by ordinary continuous wall and become double diaphragm wall system (see Figure 2.2 Basic design stage (2002.2)) programme).

In some special excavation works liked LNG underground storage tank projects, almost all make in the vast hinterland of tidal land , and only excavate inside the circular diaphragm wall, so not too much of bias.pressure. But in this Basic design case, outside a circular perimeter has to be a part of excavation will make bias pressure which have a tremendous impact on circular diaphragm wall.

Therefore, in the basic design of double diaphragm wall case, for controlling effects of bias pressure voltage on the diaphragm wall, the internal built must wait until the round is finished to a considerable extent, structure ontology has been formed to resist pressure in State, party to the perimeter of excavation.

However, in the basic design case in which circles the perimeter of the construction quantities are far greater than the rounded internal construction of the quantities . Furthermore, the perimeter lateral supporting in the excavation are usually required to be of a job, make excavation and building structure efficiency decrease. This project is a cricical path of the whole Kaohsiung mass rapid transit project. Duration is the main consideration point. Therefore must seek alternative to complete in the reliable prediction of the duration of this project.

2.3 Alternative (detail design)

This project works by turnkey contract (Design and Construction merger contract). Turmkey contractor in detail design stage reviewed above basic design case, and did not change under the premise of station function, proposed to owners a construction programme - using diameter 140m round diaphragm wall will perimeter all of station facilities (a single diaphragm wall case,Figure 2.3 Detail design stage (2002.6)).

For the alternative, the contruction quantities inside of the circular diaphragm wall were about three times of the quantities outside the circular diaphragm wal, not only to preserve the original conception of basic design stage using the circular diaphragm wall to fulfill the idea of non-supporting excavation to enhance work efficiency, make the characteristics of the circular diaphragm wall work, and is expected to shorten the duration of about half a year.

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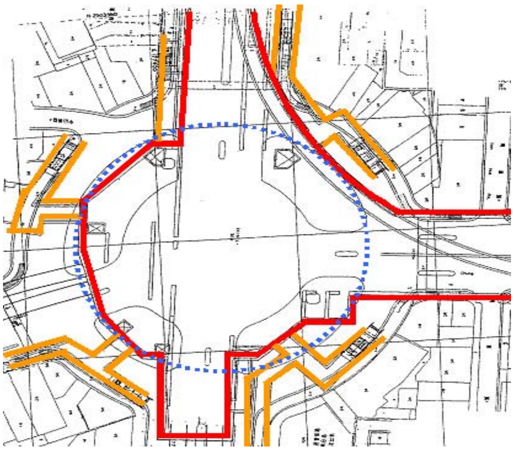


Figure 2.1: Original planning stage (2001.6)

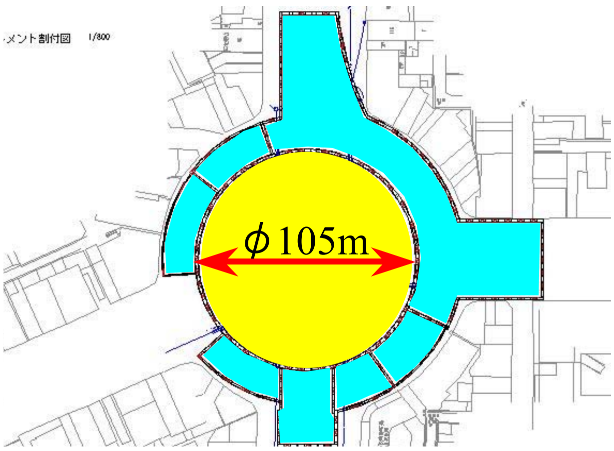


Figure 2.2: Basic design stage (2002.2)

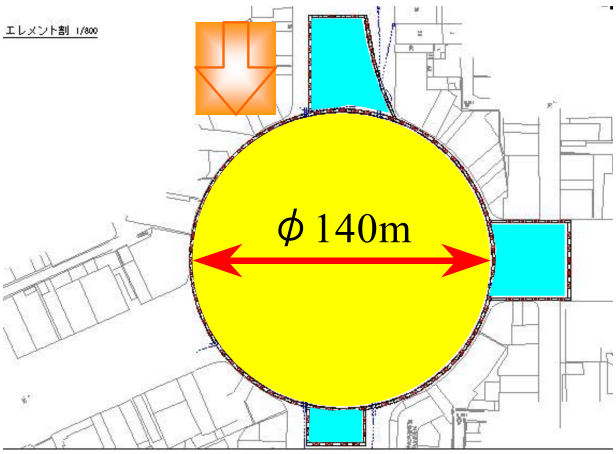


Figure 2.3: Detail design stage (2002.6)

3 CIRCULAR DIAPHRAGM WALL DESIGN

3.1 Comparison with past performances

There are many performances in Circular diaphragm wall construction method for LNG underground storage tanks. Its best feature is not supported under the condition of continuous excavation within the wall. However this works with over 140m in diameter within a large circular diaphragm wall, even in Japan are very rare.

Table 3.1 is the comparison of this work and the representative performances of circular diaphragm wall construction in the Japan. In Japan, inner diameter of more than 100m of engineering have been very rare, by exceeding the scale of the project in the table instance only Toyosu underground substation project in Tokyo (inside 144m in diameter) one more example.

Table 3.1: Previous circular diaphragm wall project performance

Project	Owner	Inner diameter (m)	Thickness of Wall (m)	Length of Wall (m)	Excavation depth (m)	Concrete Design strength fd(N/mm^2)
Toyosu underground substation	Tokyo Power	144.0	2.4	70.0	32.0	32
Trans-Tokyo Bay Road	Trans-Tokyo Bay Road	98.0	2.8	119.0	74.2	36
Futtsu LNG tank	Tokyo Power	73.47	1.1	59.7	31.7	51
Ogishima LNG tank	Tokyo Gas	48.43	1.0	70.6	37.5	60
CR4 O5/R10 station	KRTC	140.0	1.8	60.0	27.1	42

In addition, the circular diaphragm wall of the project in addition to being one of the world's largest, another special condition for construction location lies directly below the intersection of urban roads, in the similar projects being rather special, at the stage of design should be taken into account the influence.

Therefore, the setting of design criteria, analytical methods, as well as when designing the component section, must be to ensure construction safety of shapes and sizes, while nearby buildings to avoid sinking, and considering how to minimize the working area occupied, works on the crossroads of traffic and minimizes the effect of surrounding commercial area. At the same time General axisymmetric of LNG tank internal face is flat, but for our project, there is a narrow ribbon of underground excavation in EL-20~EL-27m layer and its' R10 station rail, and the switch rail of the unicom channel with red and orange line (see Figure 3.1). The bottom excavation is not uniform, thereby increasing the design analysis and construction difficulties.

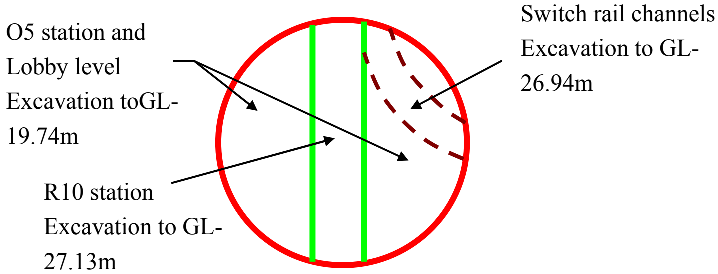


Figure 3.1: Circular diaphragm wall internal bottom excavation elevation

Therefore, in terms of design basis of strength of concrete diaphragm wall, this project uses domestic field cast of maximum strength of concrete in water-42 N/mm^2 in Taiwan. And on the design of diaphragm wall thickness calculation, experience from the past project performances is set to the minimum thickness of the demand in 1.8M.

Figure 3.2 is the comparison plot of circular diaphragm wall for this project and previous project, The vertical axis in plot represents the design basis for diaphragm wall thickness mutiplying the concrete strength of the wall, and the horizontal axis in plot represents the inner diameter of the wall. Design base strength use 42 N/mm^2 in this project is much the same with preceding Toyosu underground substation project Tokyo (inside 144m). And by the diaphragm wall thickness/diameter ratio (this project: 1.8/140=0.0129; Toyosu underground substation project in Tokyo: 2.4/144=0.0167) came to see this project diaphragm wall thickness quite thin. In conclusion, this project used circular diaphragm walls not only to one of the world's largest, and to challenge the limit of economical and reasonable design

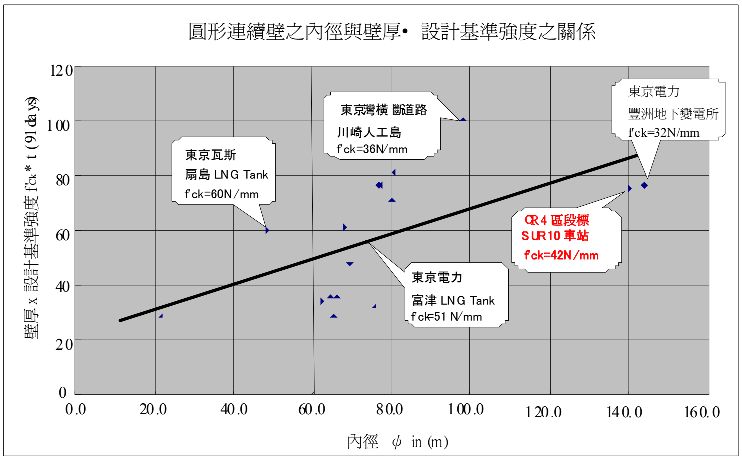


Figure 3.2: Circular diaphragm wall inside diameter and wall thicknessxdesign strength of relationship

3.2 Design concept

The design concept of the circular diaphragm wall, as simple in principle as shown in the Photo 3.1 of a cypress wood barrels. Cypress wood barrels are after the rectangular cypress wood piceses chips into doughnut-shaped, in its upper and middle to wooden boxes and wires tightly bundled like a scarf. Cypress wood swelling with water, but wooden frames and wire binding the axial compression force for a unit circle direction caused, that is, pre-force involvement, internal are not filled with water to form even to loose solid structure.

The principle and design of the circular diaphragm wall is similar with the cypress wood barrel structures, every diaphragm wall unit is like a rectangle cypress chip around the barrel. But because of the diaphragm wall above the ring beam and external forces generated by the loading effect of circumferential direction axial compression force, had a tremendous effect of constraints and resistance, making it a even if no lateral bracing may also be the mining of solid earth-retaining structure.

The design concepts of the circular diaphragm wall are consist of the following two points:

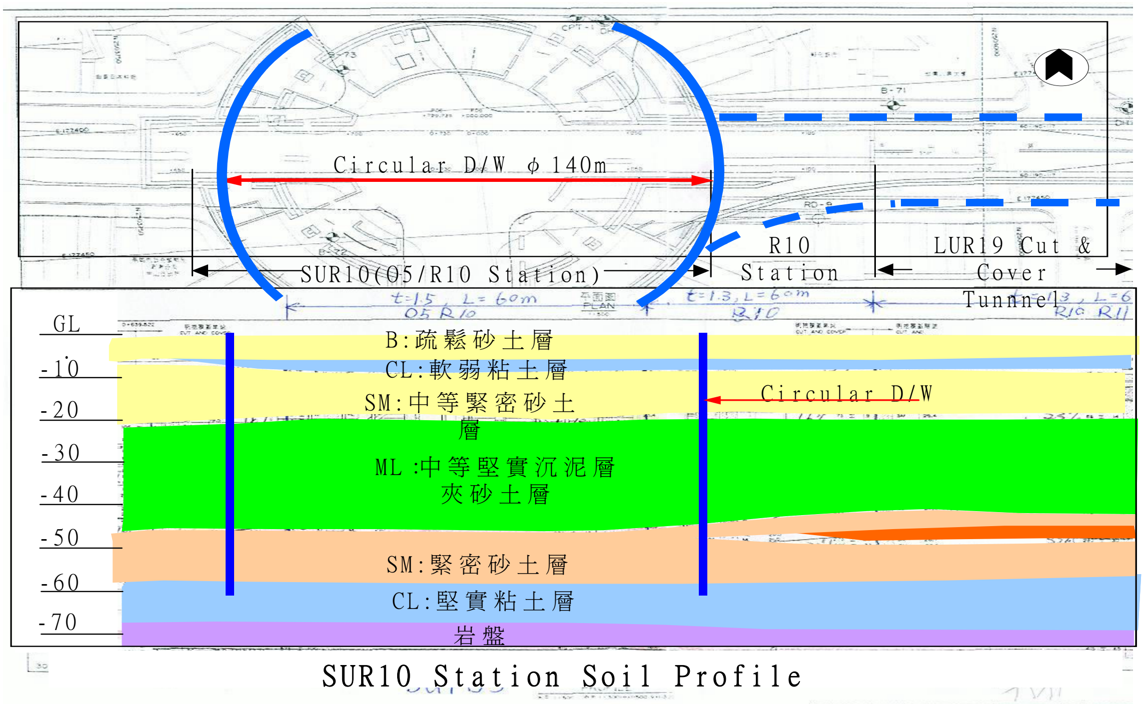
(1) The use of "Hoop Compression effect" (organized by the circumferential direction of axis pressure suppression of diaphragm wall by bending tensile stress)

(2) The use of "Ring Stiffness effect" (produced by circle shape the toughening of retaining rigid to inhibit deformation of diaphragm wall and surrounding land subsidence), making excavation work in no lateral supporting under the condition also can significantly enhance the efficiency of excavation and building construction.

Photo 3.1: A cypress wood barrels

3.3 Geological conditions

Figure 3.3 is the geological section of the O5/R10 station nearby. The stratum of the site generally can be distinguished as follows: From the ground to the soil near the GL-20m,the silty sand (SM). Below near its in GL-20~-45m, the main layer is the sandy silt (ML) with SM/ML/CL interbeds. The silty sand (SM) near the GL-45~-58m, GL-58m the following as a clay layer (CL).



According to the geological survey of general physical test results (Table 3.2) was informed that the silty sand layer (SM) and sandy silt layer (ML) of fine particle containing high rates, and standard penetration N value since surface down to GL-40m near about 10 more or less. This phenomenon forms a unique, special, and dangerous formations of Kaohsiung area, in mining excavation of the trench of the diaphragm wall will cause caving or easily occur internal water leakage occurred when accompanied by high sand flowout.

The circular diaphragm wall, in the above geological conditions, because of uplift countermeasures of excavation the bottom and the effects on peripheral sites by pumping for excavation inside, is decided to include to impermeable layer (CL) in-depth GL-58m. The length of the diaphragm wall is decided to be 60m.

Table 3.2: Geological simplified profile and soil parameters list

NO.	Soil	Depth (m)	SPT N	$_t$ kN/m^3)	e	C' (kN/m^2)	' (degree)	S_u (kN/m^2)	E' (kN/cm^2)	k (m/day)
1	SM	6.5	7	19.00	0.75	0	31.7	-	8500	0.340
2	CL	8	5	18.80	0.90	16	34.1	51	9180	0.0086
3	SM	18	13	18.80	0.85	0	31.7	-	14000	0.2065
4	ML	27.5	9	19.50	0.80	6.7	32.3	89.4	16094	0.0111
5	ML	37	11	19.50	0.80	6.7	32.3	118.3	21287	0.0111
6	ML	46.5	14	19.50	0.80	6.7	32.3	147.1	26481	0.0111
7	SM	58.5	33	19.50	0.65	0	35.8	-	28000	0.0979
8	CL	69	85	21.00	0.65	20	34	254	44027	0.0134

3.4 Analysis model and loading combinations

When in design for a circles diaphragm wall, in order to faithfully simulation on bias soil pressure (static earth pressure of 20%), bias water pressure (water level 2m) and surround buildings load of the asymmetric loads, and varies by region of the digging depth (GL-20 m and-27m), using 3D cylinder shell model for analysis. Figure 3.4 is analytical model diagram.

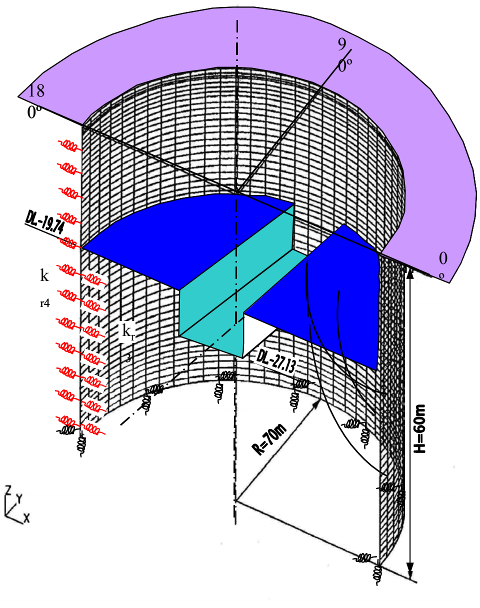


Figure 3.4: analytical model diagram

The other hand, loading combination, successive analysis each excavation stage in construction phase. When in exception (load considering bias soil pressure, bias water pressure and temperature) and earthquakes (earthquake increased soil pressure and wall of inertia force and temperature load) analysis of the excavation depth down to final mining limited in the case of the bottom line. In addition, bias soil pressure increase, seismic soil pressure and load inertia force direction of 0 °, 45 °, 90 °, 135 ° implemented in four directions.

3.5 Analytical results

We use CSI SAP 2000 v7.4 for analysis. From the analytical results of diaphragm wall, the maximum amount of displacement occurred in near about GL-27~-30m. During construction case, at the final mining shift most of approximately 26mm, maximum displacement in the exception case is 36mm, most about 50 mm during the earthquake case.

4 THE CIRCLE DIAPHRAGM WALL CONSTRUCTION

This project uses circular diaphragm wall at a stage of construction, in addition to the need to overcome the technical problems, due to construction site located just below the urban trunk road to the crossroads, traffic maintenance plan developed, and how to mitigate the impact the influence of construction to the surrounding buildings is also a very important subject. Therefore, shall, before the construction, review of various construction methods, and then select the most appropriate method.

4.1 The construction conditions

The circular diaphragm wall construction site are located in the crossroads of Chung Shan Road and Chung Cheng Road. The construction shall be subject to the following conditions of restriction:

(1) Must maintain a six-lane for Chung Shan Road and a four-lane for Chung Cheng Road of vehicular traffic and secure, restrictions on construction sites, The work areas had to be cut to four areas scattered in Chung Shan and Chung Cheng Road.

(2) The stablizer-storage tank shall be fixed and cannot be moved, so the equipment must be taken in any traffic maintenance phase does not affect the minimum limit of vehicle maximum capacity

(3) Due to limited construction site, in a diaphragm wall construction,(placing reinforcing cages, set concrete) the lifting equipment must also be strongly miniaturizated.

(4) Similarly, the Assembly of reinforcement cage are limited to one site only, to save Assembly time, also of reinforcement cage of light, must as far as possible be brief

(5) For considering traffic around, play established time of concrete (concrete passage of time) must also strongly reduced

(6) Around the buildings and diaphragm wall some distance 3~4m only, it must be used to reduce the trench wall caving risk of safety construction method

(7) In commercial and residential areas around the site, consider reducing the dissatisfaction of residents on the night-time noise and vibration, must use low noise, low vibration of the machine.

Based on considering the above construction, the project using the following method.

4.2 Concrete cutting joint method

In the domestic diaphragm wall project of Taiwan, "MASAGO" are used to be as representative of the scoop bucket excavators, and the connect method of unit is “Overlapping connections reinforced diaphragm construction methods". The other hand, Japan’s domestic circular diaphragm wall excavators are with the horizontal axis of Rotary excavator. the concrete to construction of mother unit directly after cutting, then applied for the provision of public housing units "construction method of concrete cutting joint". This method is based on the axial stress in the circumference direction of the circular diaphragm wall of the reasonable construction method, from pass construction performance among its various units were able to achieve good waterproof effects.

To sum up, this circular diaphragm wall project also uses the horizontal axis of Rotary excavator digging, unit of joint between the mining "cutting joint method" (Figure 4.1 cell segmentation map and Figure 4.2 the excavation map of public rental units). The advantages of jointing methods are described below:

(1) Cell does not need to use a spacer plate and horizontal reinforcing bar jointing, miniaturization of reinforcement cage for the light and brief, crane.

(2) Separated plates when engaged, the mother unit is not located on both sides after construction concrete part of the trench wall caving or, as occurs in the construction of public rental units for unit occupies a leak caused by poor construction all these situations, but this method without the risk, reducing the threat to the surrounding buildings

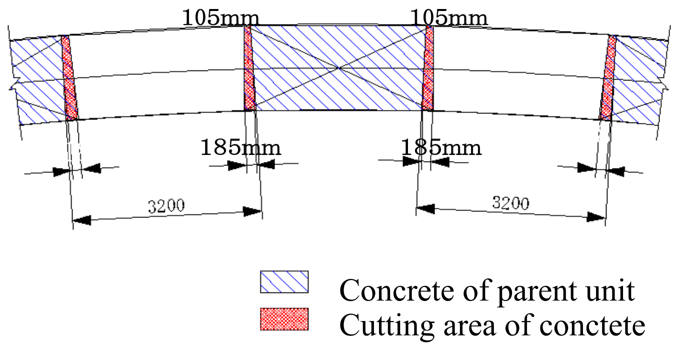


Figure 4.1: Units segmentation map

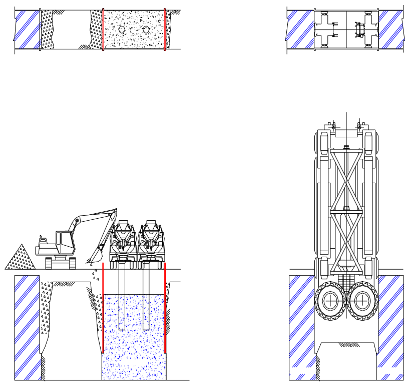


Figure 4.2: Diagram of the excavation of public units

4.3 The selection of the excavator

As noted above, this works by the use of a "construction method of concrete cutting joint", specifically called from Japan the horizontal axis of Rotary excavator for the construction of the circular diaphragm wall. In line with the circular shape of diaphragm wall (140m of inner diameter and wall thickness 1.8m) in the machine selection original consider EMX (ERECTRO-MILL EXTRA-150) (standard thickness 650~1,500mm construction, excavation wide 3,200mmx146 unit) and EMX-240 (1,200~2,400mm wall thickness of standard construction, excavation wide 2,400mmx196 unit) of two models. But consider cutting concrete may cause mechanical failures, even affecting the duration, so decided on a smaller unit number programmes, Two excavators will be modified into the corresponding 1,800mm of wall thickness, construction of 146 units (Photos 4.1 EMX-150 excavator).

Photo 4.1: EMX-150 excavator

Photo 4.2: Stabilizer storage slot

4.4 Construction of 1 unit 1 knife method

The cell segmentation of circular diaphragm wall as mining and construction law is in two ways. (1) the parent unit by 1 unit 3 knife, and public unit by 1 unit 1 knife, or (2) parent and public unit by 1 unit 1 knife If the parent unit with 1 unit 3 knife, notwithstanding the advantages of reducing the duration, but when the parent unit in the construction of excavation, the open length for almost 10m, excavation and concrete setting time too long, increased risk of trench wall caving. In addition, the following will mention of sludge water treatment, good stabilizer required to replace fluid bath installations increases, causing problems on the construction site. This work area of construction except strongly evaded trench wall caved of risk, and reduce on around building of effect outside, traffic security of maintained also for important topics., Therefore, selecting the most suitable solution for this work area construction conditions, including unit configuration and the construction method by using excavation wide 3.2m for 146 unit segmentation; parent unit, and public unit are mining 1 unit 1 knife of construction method (Figure 4.1 units segmentation map)

4.5 The stabilizer storage tank and soil-water separator

Stabilizer storage slot device due to the limited room of work site must strongly narrow, but due to this work used 1 unit 1 knife of way, each unit of construction by required capacity for minimum limits (3.2x1.8x60=350 m^3), so setting cycle slot (110m^3), and recovery slot (450m^3), and benign liquid slot (450m^3) each the 2 unit of volume (two EMX excavator volume), Total 2,000 m^3 of liquid slot device (Photos 4.2 stabilizer storage slot).

In addition, in the soil-water separator selection, with low noise, low vibration as elements. Processor parts of rotary drum type of rotary screen type 4x2 sets; secondary processor using spiral sedimentation separator MW550 x2 sets. In particular, the construction of the diaphragm wall must be for fine sand, silt, precipitation fractionation have to rely on the work of spiral sedimentation separator, so specially selected large and high stability of the secondary processor, reaching a good construction effects (Photo 4.3 soil-water separators)

4.6 Benign liquid replacement

To ensure the quality of concrete of the diaphragm wall, stabilizer quality management is important. Using larger density in a diaphragm wall for the trench excavation stability. Before placing reinforcement cage to trench, should replace all stabilizer with a low density, rare sand composition of stable liquid (benign liquid), implementation of the so-called "benign liquid replacement". In other words, the stability of liquid quality management into “the excavation of diaphragm wall" and "before the concrete setting" two-stage, in order to implement diaphragm wall construction safety management and quality management.

Photo 4.3: Soil-water separators

4.7 The construction performance and construction precision

The circular diaphragm wall construction since October 14, 2002 to June 27, 2003 a total of 257 days (about 8 months), the total design quantities completed of about 26,700m^2 (48,000m^3), construction of 146 units (Photo 4.4 Diaphragm wall construction photos)

Photo 4.4: Diaphragm wall construction photos

From the Performance by construction, excavation of pure time (does not include preparing jobs, EMX excavator lifting and lowering of the body, ultrasound examination, amendment excavation required time) in the excavation of the average speed, the parent unit for 17.8m^3/HR (3.1M/HR); public unit as 10.4m^3/HR (1.8M/HR), a Japan domestic performance value of 20~22m^3/HR is slightly lower. The reason for this is the clay layers of Gl-58m the following (CL) for the more-than-expected hard clay, increases the time needed for excavation. Furthermore, due to job site is separated into four areas, excavator and soil-water separation equipment of transmission distance of up to 150m, result in reduced efficiency of excavation. Essentially the average construction time for the parent unit: 1.5 days /units, public unit: about 2.5 days/units

On the other hand, in the construction of the circular diaphragm wall, construction management of excavation on precision is set to 1/1,000 (=6cm). By excavator mounted inclinometer in implementation during excavation of ditch and outer shape of ultrasound measurement to confirm, and on the use of cutting drum device of hydraulic jack on one side to the trench wall trimming force on manipulating excavator, to ensure a certain degree of accuracy

In terms of actual construction management, engineering management practices will be implemented for use by ultrasonic measurement of trench wall location away from the normal trench wall of the maximum amount of motion control in -60mm (-: inside of the ditch), while 146 units within all excavation of accuracy control in-50~+100mm.

5. INTERNAL EXCAVATION

5.1 Excavation engineering

Approximately up to 340,000m^3 of the opened excavation inside circular diaphragm wall of excavation work began at the end of February 2004, and at the end of August 2003 smooth excavation to end bottom of the excavation of GL-27m. Daily maximum excavation soil moved out of 5,000m^3, average daily volume of more than 2,000m^3.

Excavation operations can be divided into the following two stages:

Phase 1: the internal overall mining to lower tier II (GL-20m) phase. For the control on the bias soil pressure of the circular diaphragm wall, each mining steps to 3M as a unit, and the soil removed method in the excavation, from the surface down to the GL-13m, is to dig accumulated soil by the telescope-type excavator to mounted soil directly on the dump truck. While in the GL-13~-20m phase, a 100t crawler crane is set on a cover decking,and lift the soil by a 5m^3 container and loaded in dump trucks.

Phase 2: for excavation of the third floors underground red line and switch track (GL-20~-27m) phase. First up to the GL-23.5m to be tilted surface excavation, went on to play set steel sheet pile (type IV, L=9m), to self retaining excavation to GL-27m.

Photos 5.1~5.3 show the internal excavation phases. The excavation of the final outsole period at the end (2004 ~ August) due to Typhoon rainfall and other factors, caused drainage treatment is difficult, but luckily at the end of August, the successful completion of excavation and PC set.

Figure 5.1: Status of the internal circular excavation

Figure 5.2: Internal excavation (GL-10m)

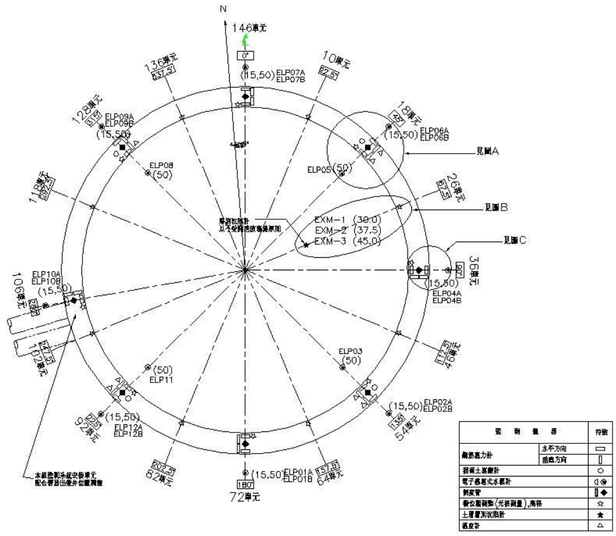
5.2 The results of monitoring system

At the time of O5/R10 station construction, to reach a excavation safety management,of circular diaphragm wall, import an automatic monitoring system. Following is a summary of the monitoring plan and carry out internal monitoring results of the excavation.

5.2.1 The monitoring plan

According to the design and analysis results of the circular diaphragm wall, in circle direction at 45° intervals is set in the 8 direction of monitoring instruments configure it as shown in Figure 5.4. In addition, the design and analysis of results, in the circular diaphragm wall of internal monitoring instrument of buried depth is set to maximum deflection, the maximum axial force, and the maximum depth of vertical direction bending moment.

Figure 5.3: Internal excavation (GL-23.5m)



Figrue 5.4: Configuration of automated measuring systems monitoring instrument

5.2.2 The monitoring results during internal excavation

Figure 5.5 is set within a circular diaphragm wall inclination pipe (SID) angle position 45 ° (SID-02) and 180° (SID-05) by observing that displacement of wall. Displacement of diaphragm wall of about 15~25mm, and the analysis values of detail design shown in much the same. The maximum amount of displacement and analysis values same result occurs in part of the switch rail of the digging depth GL-27m (SID-02), about 35mm.

Approximately 10mm displacement increases during the excavation to GL-20m~GL-27m, its analysis values of detail design also roughly the same. Displacements in the direction of the Circular more fragmented, barely visible inclination of bias pressure effect, and with "exceptions case" of detail design about the same, compared to the maximum amount of displacement of earthquake 50mm is very small, but also proof the appropriate design of this circular diaphragm wall.

In addition, Figure 5.6 is the time-historical change of steelbar meter in the direction of horizontal. As the chart show, the the horizontal reinforcement barmeters set in ring beam above diaphragm wall were observated to the increases after excavation job, to about maximum stress of 300kg/cm^2 in early May, but is subsequently stabilised. In addition, depth GL-15m,-27m,-32M of numerical values along with horizontal steel bar gauge monitoring increasing after excavation job too. In GL-15m about increased to 600kg/cm^2 about increased to 200kg/cm^2 in GL-27m. In GL-32m about increased to 500kg/cm^2. This situation can be considered as the excavation proceeds, diaphragm wall deformation and circumferential direction of location of the axial compression force gradually move results.

Furthermore, set in the depths of GL-15m concrete stress meter observation value also increased to around 80~140 kg/cm^2, speculated that this value should be about design analysis value (maximum value of about 1700t/m in the vicinity of GL-30m) of the same of functional circle direction axial compression for circular diaphragm wall.

3 CONCLUSIONS

The project has effectively applied the Design-Built concept, and the design and construction technology of the circular diaphragm wall-related experience in LNG project from Japan to reproduce in the urban center of the Kaohsiung MRT underground station works. This can be said to be very bold and innovative and a high degree of achievement in civil engineering technology.

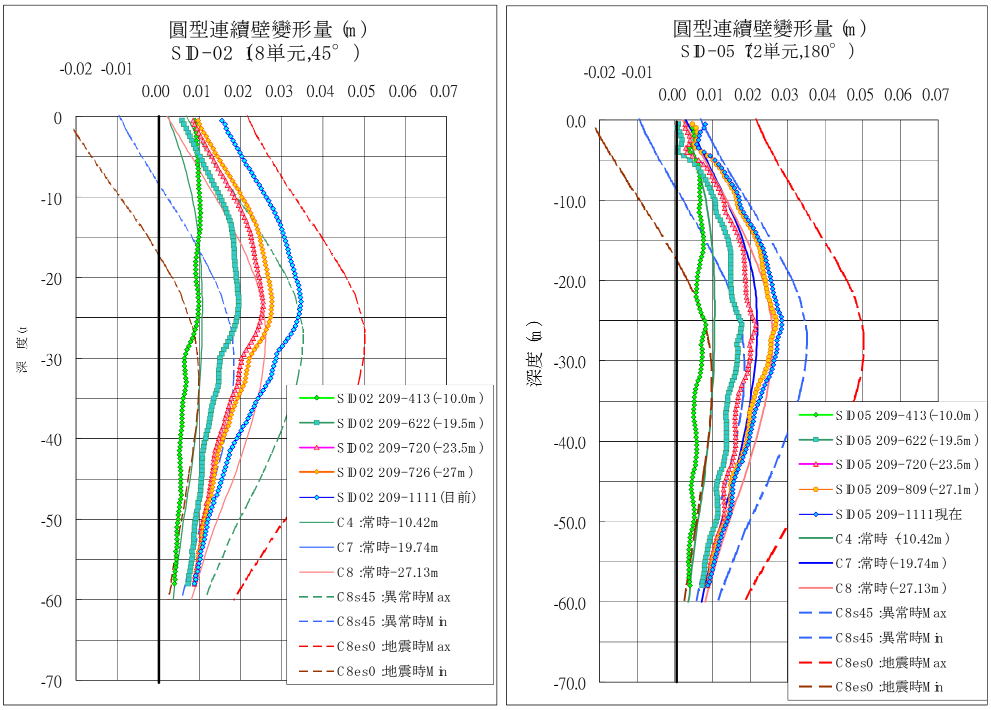


Figure 5.5: Circular diaphragm wall deformation monitoring results

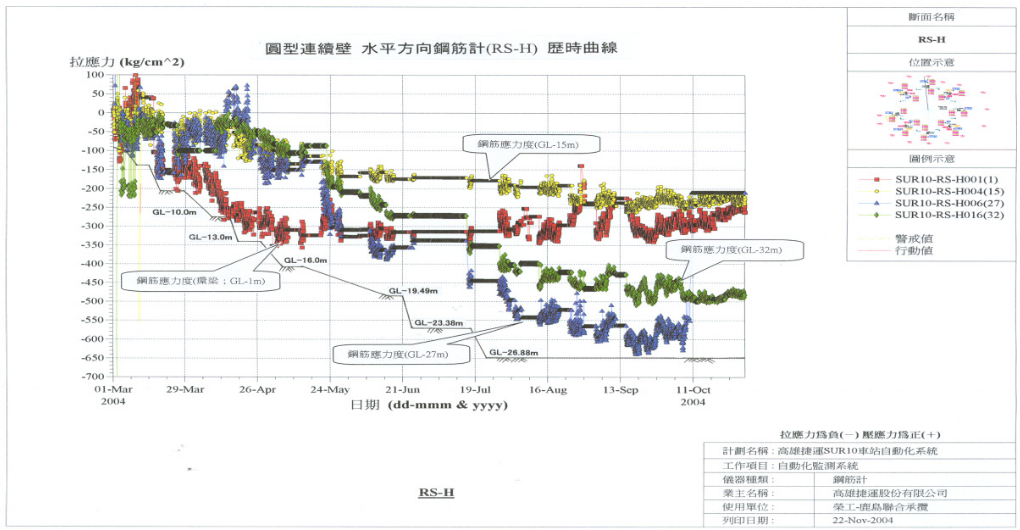


Figure 5.6: Monitoring results of horizontal reinforcement

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Thanks for Mr. Tada Yukio (多田幸夫) - the manager of CR4 RSEA-KAJIMA JV affort many construction data and photos.

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Excavation Effects on Adjacent Construction of Rapid Transit Facilities – Case Studies

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ABSTRACT

As the network of Taipei Rapid Transit System (TRTS) spreads in response to the development and expansion of the Taipei metropolitan area, limited space has made the facilities of the TRTS set up nearby existing buildings or those to be constructed in the future. The soil-substructure interaction becomes more complicated if the construction effects influence each other. This paper investigates excavation effects of buildings on adjacent construction of TRTS facilities in two cases; one is an underground station and the other is shield tunnels. In both cases, a series of numerical simulations is conducted following construction steps. The computed sequential results and the corresponding measurements are investigated for excavation effects. Through comparison of computed and measured wall deflections with the reference envelopes proposed based on the Taipei's excavation experience, the efficiency of the implemented reinforcement or mitigation plans are validated. These analysis results could thus serve as references for the study of similar cases.

1 INTRODUCTION

Taipei City, the capital of Taiwan, has been developing its metro system for the last 25 years. With the first line set up for operation in March 1996, the Taipei Rapid Transit System (TRTS) currently has a total of over 90km routes and 80 stations in service that provides over 1.5M people daily with rapid and safe access to/around the urban area. It is expected that, by the end of 2015, the system would build up a network of over 150km routes and 130 stations serving over 2.3M people everyday (data summarized from TRTS 2011). Figure 1 shows the long term route map for the TRTS.

As the TRTS network spreads following the development and expansion of the Taipei Metropolis, the facilities of the TRTS is often located, as a result of limited space, at a place that is in close proximity to the existing buildings or those to be constructed in the future. The soil-substructure interaction becomes more complicated if the construction effects influence each other. This paper presents two case studies in the TRTS projects, as shown in Figure 1, where the excavation effects of a nearby building on the construction of an underground station (Case I) and shield tunnels (Case II) are investigated. Two-dimensional (2-D) numerical models are introduced to simulate the construction steps on the sites. The sequential computations and the corresponding measurements are investigated for excavation effects. The results for wall deflection are then compared to the reference envelopes proposed based on the Taipei's excavation experience; thus the efficiency of reinforcement or mitigation measures against diaphragm wall and shield tunnel deformations can be validated. These results are hoped to provide references for the study of similar cases.

2 CASE I – EFFECTS ON ADJACENT CONSTRUCTION OF AN UNDERGROUND STATION

2.1 Background

Figure 1: Long term route map of TRTS (after TRTS 2011)

Case I is about an underground station on the Songshan Line of TRTS constructed nearby a deep excavation. As depicted in Figure 2, the station (Site A) is situated under a road of over 30m wide. The nearby excavation (Site B) is located right on the south edge of Site A with a maximum spacing of about 6m. The Site B is set for a building of 18 stories above ground and 6 stories below.

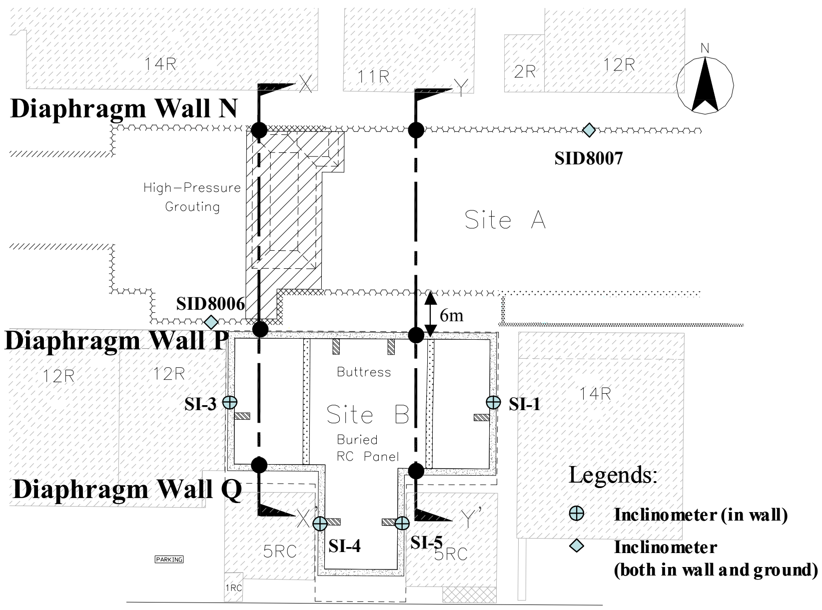


Figure 3 illustrates the retaining system for the sites. On both sites, diaphragm walls are used as the retaining structure; one is 35m deep (Site A) and the other is 43m deep (Site B). On Site A, the station is constructed using cut-and-cover, bottom-up method to a maximum depth of 23.2m with 6 levels of struts and 2 levels of reshoring as the bracing system. The building on Site B is excavated using top-down method to a maximum depth of 23.7m. Table 1 summarizes information of the excavation for the sites. It is noted that, due to close proximity of the sites to each other, high-pressure grouting (or ground improvement) and buried reinforced concrete (RC) panels and buttresses are installed to Sites A and B, respectively, to reduce excavation-induced wall deflection. In addition, the soil-substructure interaction becomes more complicated as the excavation schedule is almost overlapped for the sites. The arrangement of the reinforcement measures is shown in Figure 2 whereas the excavation sequence of both sites is illustrated in Figure 4.

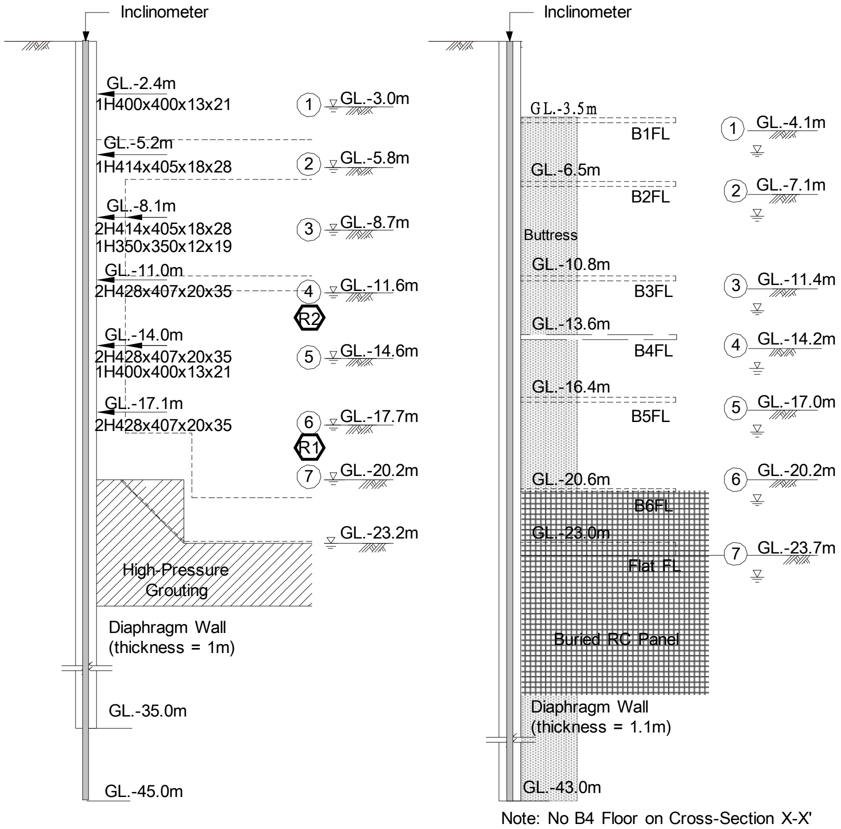


Figure 3: Profile of excavation and bracing system for Case I

Table 1: Information of excavations for Case I

Excavation Site	Site A	Site B
Depth (m)	20.2/23.2*	23.7
Method	Cut-and-Cover, Bottom-Up	Top-Down
Width (m)	26.4	21.6
Retaining Structure	Diaphragm Wall	Diaphragm Wall
Thickness (m)	1.0	1.1
Depth of Wall (m)	35	43
Bracing System	6 Levels of Struts 2 Levels of Reshoring	6 Levels of Slabs
Reinforcement Measure	High-Pressure Grouting (GL.-20.2m~-26.2m)*	Buried RC Panel (GL.-20.6m~-30.0m) & Buttress (GL.-3.5m~-43.0m)

* Sump for the station

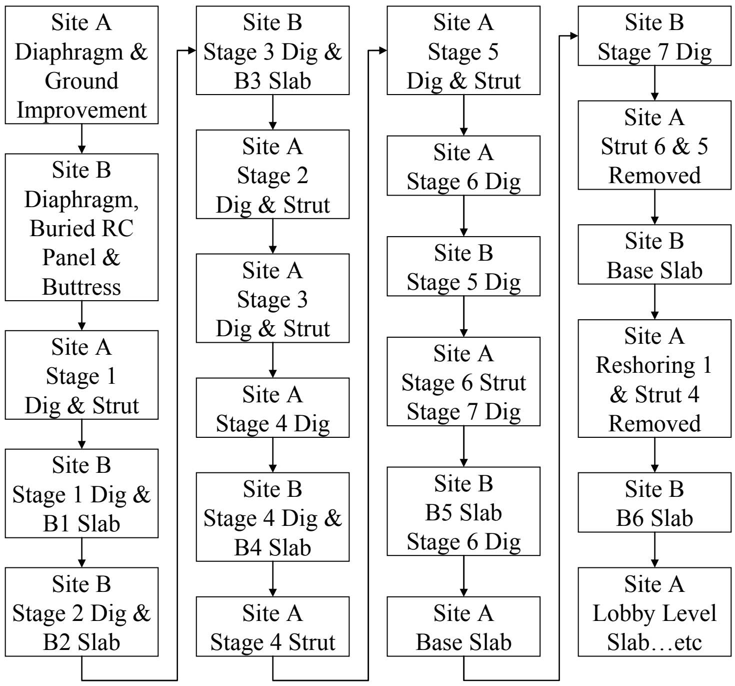


Figure 4: Excavation sequence of Sites A and B for Case I

2.2 Numerical simulation

To investigate the interaction of diaphragm wall deflections between these two excavations, the 2-D commercial software PLAXIS (PLAXIS 2002) is applied to perform a series of numerical simulations. Table 2 presents the soil profile and parameter for the analysis. From the table, it is noted that the diaphragm walls are penetrated to the layer of relatively high strength for the two sites and an impermeable layer would be encountered at the final level of the excavation. In addition, the groundwater table is set at a depth of 2.2m below the ground surface where a hydrostatic pressure distribution over depths is applied to simulate the groundwater condition.

Table 2: Soil profile and parameters used for Case I analysis

Depth (m)	Soil Classification	SPT-N	γ (kN/m^3)	ϕ' (^{o})	s_u (kP$_a$)
3.0	SF	4	18.8	30	-
7.0	SM	6	19.5	31	-
13.0	SM	10	19.0	32	-
26.0	CL	6	18.6	-	63
34.0	SM	13	19.4	32	-
37.0	CL	13	18.9	-	110
44.0	SM	21	20.1	33	-
55.0	CL	20	19.9	-	145
>55.0	GM	>50	22.0	40	-

Figure 5 shows the meshed model for the 2-D plane-strain analysis. In the model, the diaphragm walls and slabs are simulated by plate elements with linear elastic modes whereas bracing system and buried RC panels are simulated by strut elements also with linear elastic mode. Besides, buttresses are simulated by a series of soil springs attached to the diaphragm wall with equivalent stiffness. Soil mass is comprised of a series of 15-node triangular elements with the Mohr-Coulomb soil model that takes into account the effect of high-pressure grouting.

2.3 Analysis results

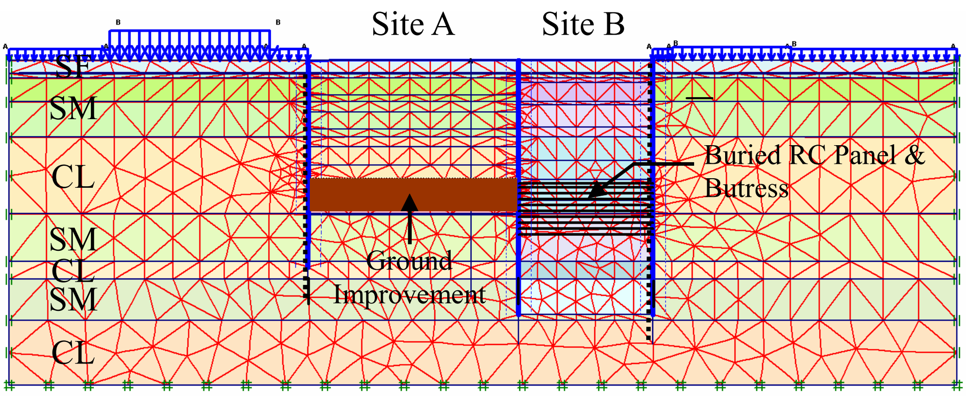
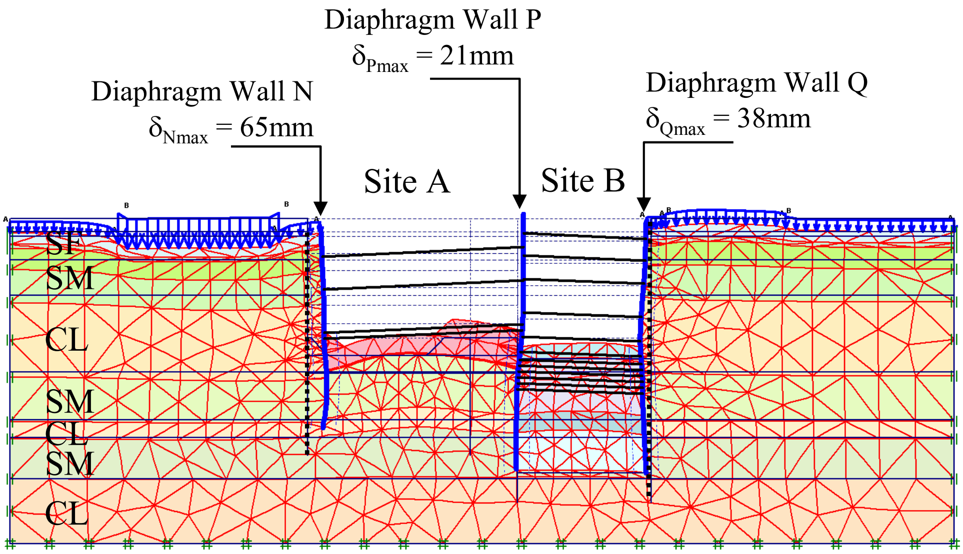
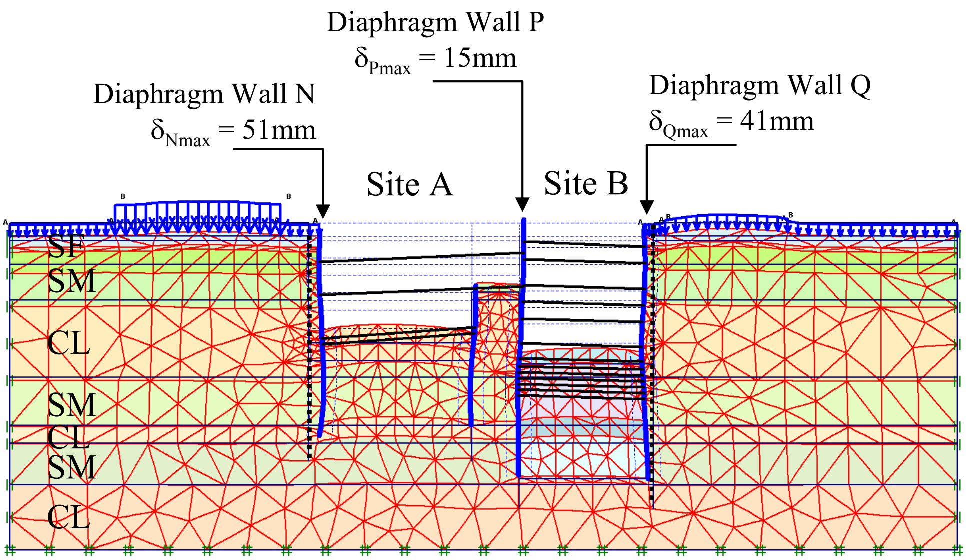


Figure 5: The 2-D meshed model for the cross-section X-X' of Case I



(a) Cross-section X-X'

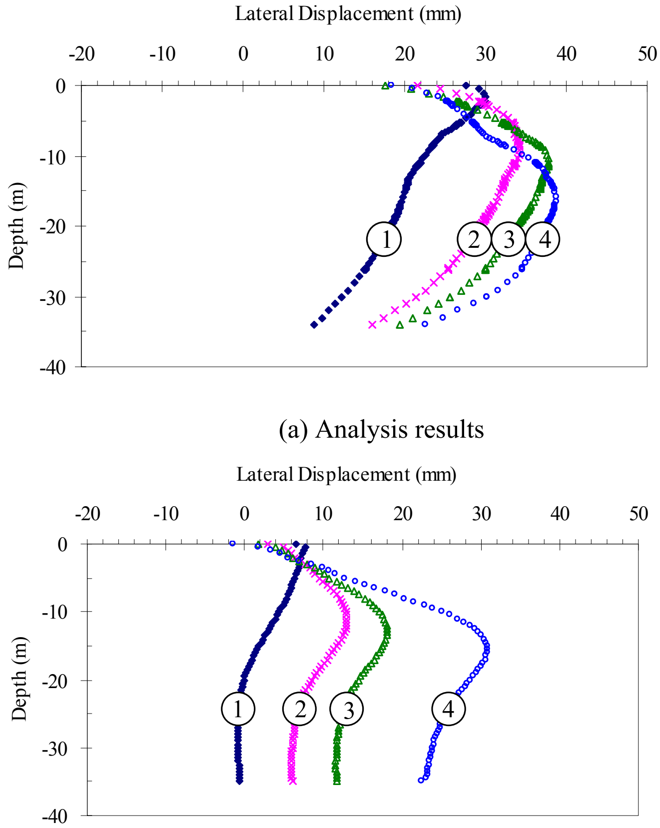


(b) Cross-section Y-Y'

Figure 6: Computed wall deflection for Sites A and B

Figure 6 presents the computed deflections of diaphragm walls for the cross-section X-X' and Y-Y' (Figure 2) at the final stage of excavation. Note that there is almost no space between the two sites in the cross-section X-X' and is about 6m apart in the cross-section Y-Y', the former exhibits higher wall deflections than the latter in the diaphragm wall N, P and Q. Considering high-pressure grouting contribute less to the stiffness of bracing system than buried RC panels and buttresses do, the diaphragm wall N in the Site A exhibits higher deflection than the diaphragm wall Q in the Site B.

Figure 7 shows the computed deflection of the diaphragm wall N in comparison with the inclinometer reading at the SID8007 (position shown in Figure 2) for the first four stage of excavation. The comparison shows favorable agreement in the trend of the deflection along the depth. The computed deflections are however larger than the corresponding measurements. Since the measurements are obtained from the inclinometer that is situated about one-excavation-width away from the Site B, such a result may be attributed to the excavation effects on the adjacent site.



(a) Analysis results

(b) SID8007 Readings

Figure 7: Comparison of computed and measured wall deflection for diaphragm wall N

Figure 8 presents comparison of numerical and measured results with the reference envelope of the diaphragm wall deflection proposed by Hwang & Moh (2007) based on the Taipei's excavation experience. As illustrated by the figure, either computed or measured maximum displacement is smaller than the proposed value for the same excavation depth, indicating the efficiency of reinforcement measures against wall deflection.

3 CASE II – EFFECTS ON ADJACENT SHIELD TUNNELS

3.1 Background

Case II is about shield tunnels on Xinyi Line of TRTS and a nearby excavation. As depicted in Figure 9, the former includes up-track, down-track, and one common duct tunnels located between Anhe and World Trade Center Station. The latter is located right on the south edge of the route that is less than one-outer-diameter distance away from the down-track tunnel at the closest point. From the figure, the excavation site is about 60m long (parallel to the route) and 30m wide (perpendicular to the route) with a depth of about 15.9m below ground surface. The retaining system includes 0.8m thick, 32.5m deep of diaphragm walls and 5 levels of

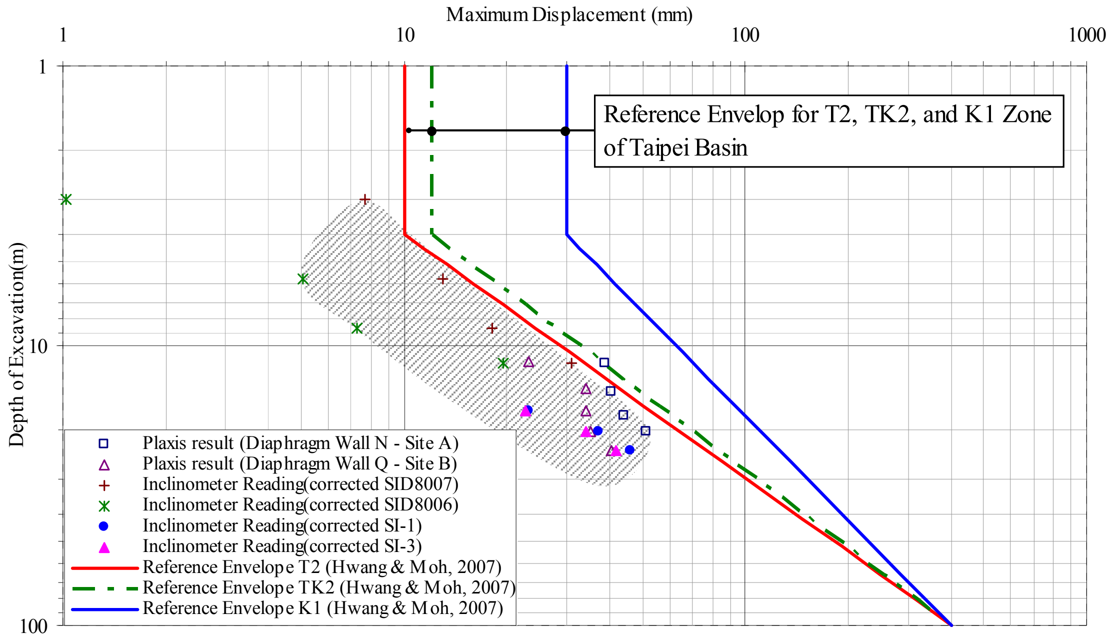
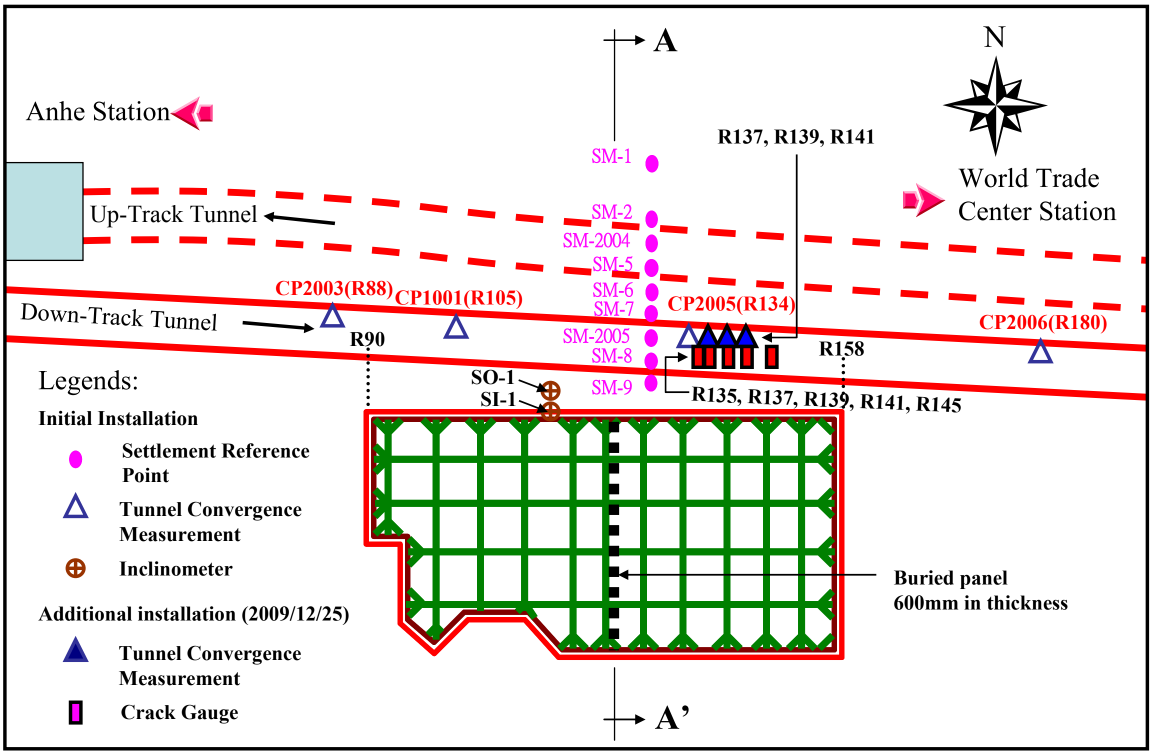


Figure 8: Comparison of numerical and measured results with reference envelope

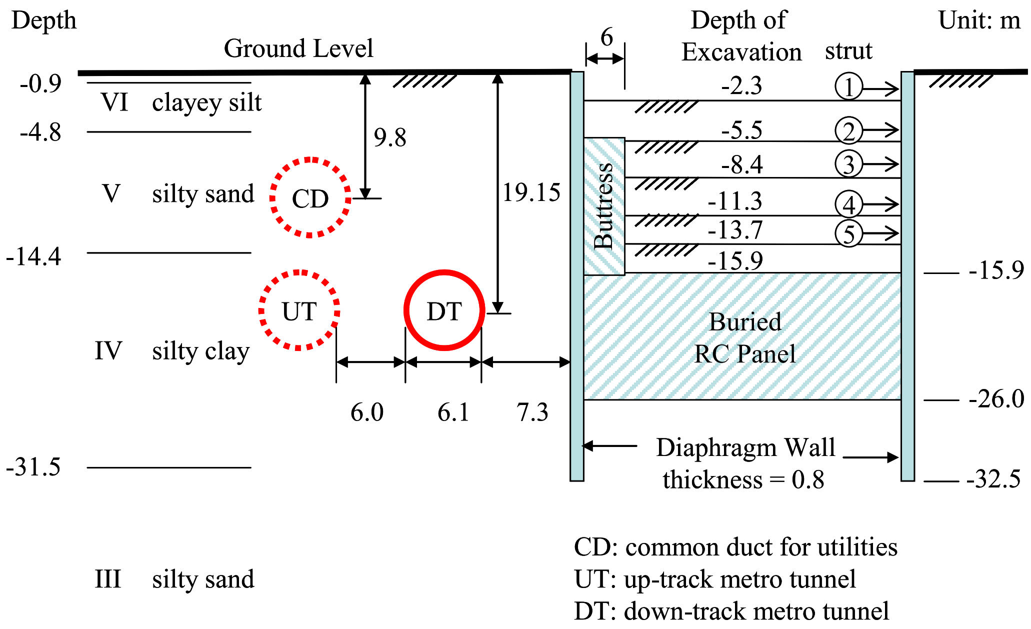
struts. The site is excavated using bottom-up method. To reduce excavation effects on the adjacent tunnels, a buried RC panel and a 6m long buttress are installed at the centerline of the excavation site on the north-south direction. The dimension of the buttress and RC panel is sketched in Figures 9(a) and 9(b).



(a) Plan view

Figure 9: Layout of shield tunnels, excavation site and instruments for Case II

According to the working plan, both the down-track tunnel and excavation site are constructed at the same time period. While the tunnel is drilled to the range of the site (about the range between Ring 90 and Ring 158), the first stage of excavation is about to launch. As the excavation proceeds, the horizontal convergence at Ring 134 (ie. convergence measurement CP2005 shown in Figure 9(a)) is observed to increase outwardly, exceeding alert level (15mm) at the fifth stage of excavation and action level (25mm) at the construction of



(b) Ground condition and excavation scheme for cross-section A-A'

Figure 9: Layout of shield tunnels, excavation site and instruments for Case II (Continued)

B4 slab. At the same time, its vertical counterpart is observed to increase inwardly. These observations imply that Ring 134 or thereabouts are subject to significantly unbalanced ground force, leading to an outward deformation in the transverse direction and an inward deformation in the vertical direction. Figure 10 illustrates the variation of convergence at Ring 134 along with the progress of tunnel boring and site excavation.

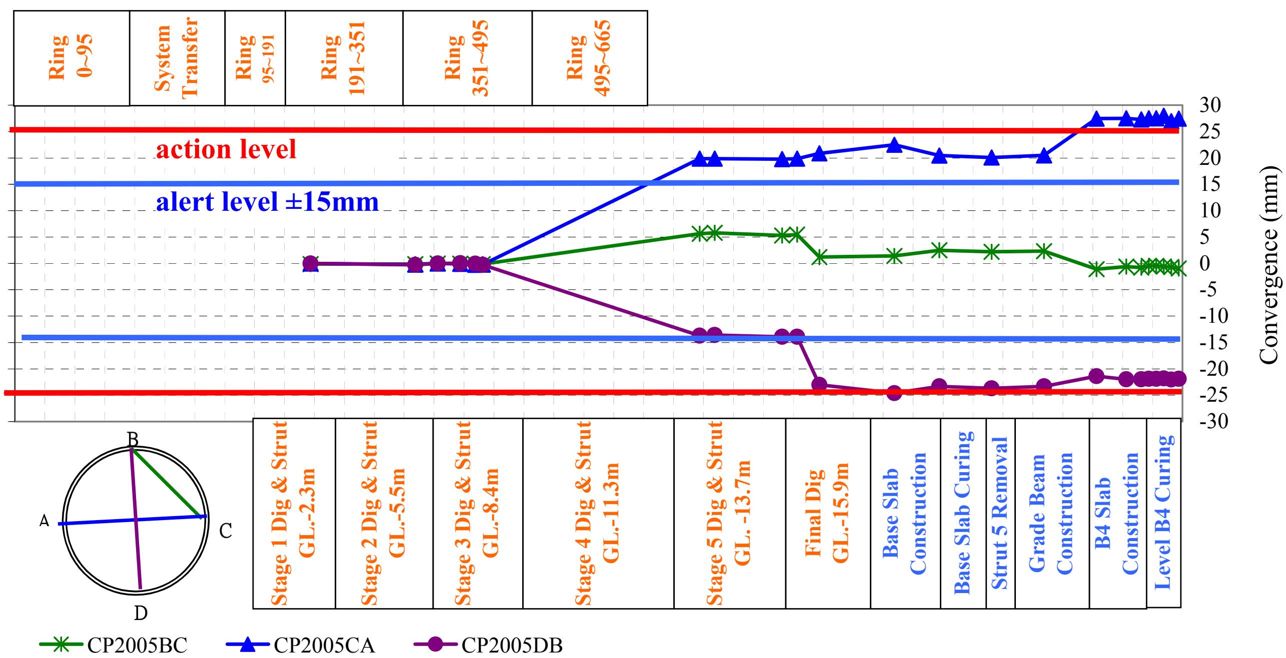


Figure 11 further shows the corresponding ground settlement and lateral deflection of ground and diaphragm wall measured at the construction of B4 slab. From the figure, the maximum settlement and wall deflection are on the order of 34mm and 55mm, respectively, at the time when maximum horizontal convergence (27.5mm) is detected. By checking the centerline coordinates for the down-track tunnel, shows substantial latitude deviation, with a maximum of about 45mm, to the south within the range of excavation site. Such a result implies excavation effects on the down-track tunnel.

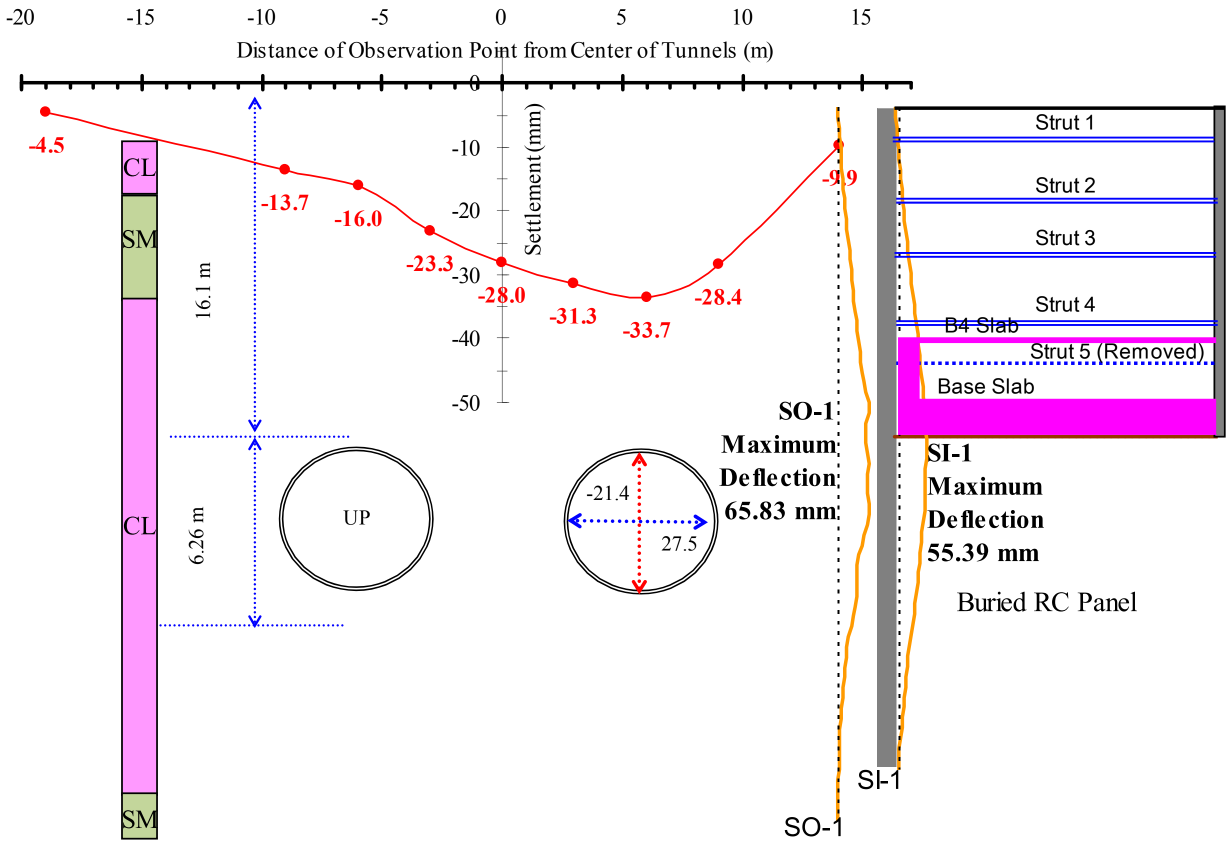


Figure 11: Ground settlement and lateral deflection of ground and wall at the construction of B4 slab

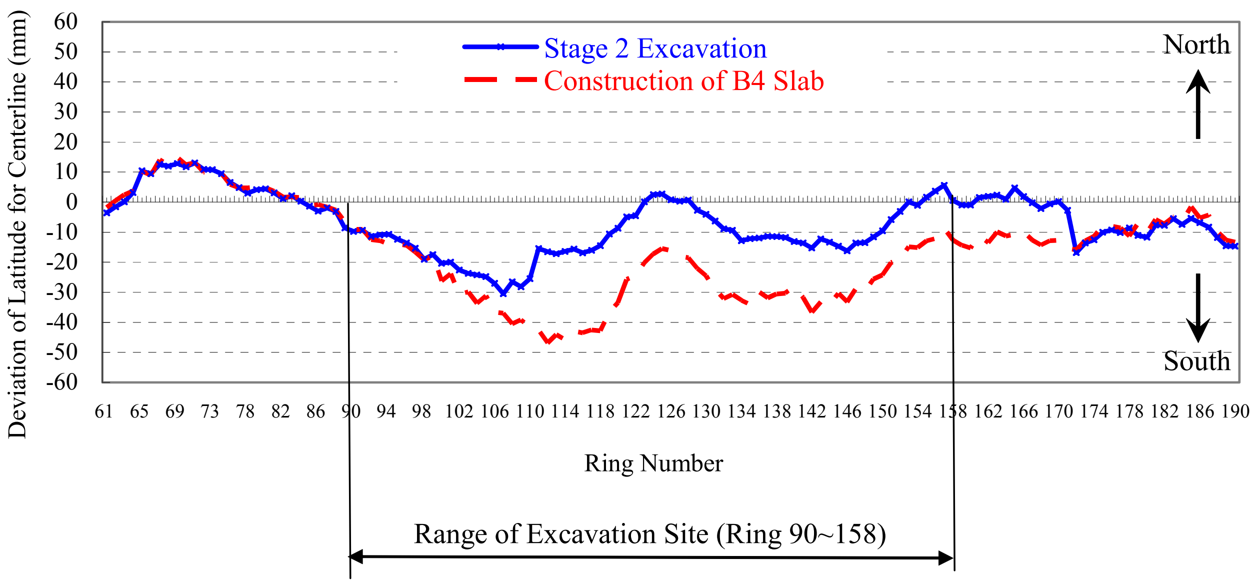


Figure 12: Deviation of latitude for tunnel centerline

3.2 Mitigation and analyses

In response to the excessive convergence observed at Ring 134, several protection and mitigation measures are implemented, including:

(1) Steel frames propped for the section of substantial deviation (Figure 12 and Plate 1).
(2) Additional instruments installed to monitor subsequent variation of convergences and cracks on the linings (Figure 9 and Plate 2).
(3) Frequency of monitoring increased to twice a day after the occurrence of excessive convergence, dropped to once in two to three days as convergence becomes stable, and increased to once a day as the up-track tunneling passes by.
(4) Speed for up-track tunneling dropped to 8m a day for the range of excavation site.
(5) Addition of reshoring for the excavation site.

Plate 1: Tunnel propped with steel frames

Plate 2: Cracks on linings

At the same time, a series of 2-D PLAXIS analyses is carried out to simulate the soil-substructure interaction before and after the mitigation measures are activated. Figure 13 depicts the 2-D meshed model and Table 3 summarizes the soil profile and parameters used for analysis. From the table, it is noted that the down-track tunnel is drilled in the clay layer of soft to medium consistency where the design depth of excavation is situated and the maximum wall deflection is induced. Since the setting is similar to that in Case I, it is omitted herein. Representative results are described in the following paragraphs.

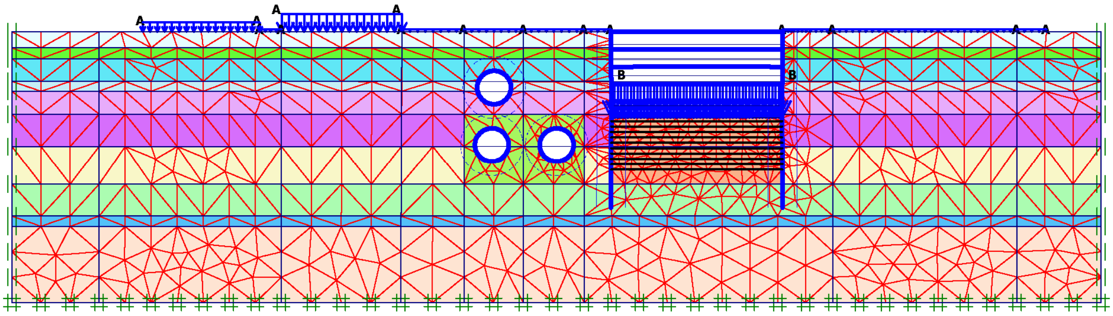


Figure 13: The 2-D meshed model for Case II

Table 3: Soil profile and parameters used for Case II analysis

Depth (m)	Soil Classification	SPT-N	γ (kN/m^3)	ϕ' (°)	s_u (kP$_a$)
3.0	SF	3	18.9	30	-
5.0	CL	2	18.8	31	30
9.2	SM	6	19.5	31	-
11.0	SM	6	21.8	33	-
21.2	CL	4	18.5	31	60
28.2	CL	6	18.8	30	70
34.1	CL	17	19.8	30	92
36.0	SM	25	20.8	33	-
>36.0	GM	>50	22.1	40	-

Figure 14 presents the computed wall deflection for the final stage of excavation, in comparison of the corresponding measurements in wall (SI-1) and ground (SO-1). As illustrated by the figure, the computation predicts a maximum deflection of about 68mm with a toe movement of about 23mm. The measurements however show a smaller value of maximum deflection (about 55mm in wall and 65mm in ground) under the presumption of zero displacement at the bottom of the inclinometers. The results would be in a better agreement if toe movement is considered in the readings.

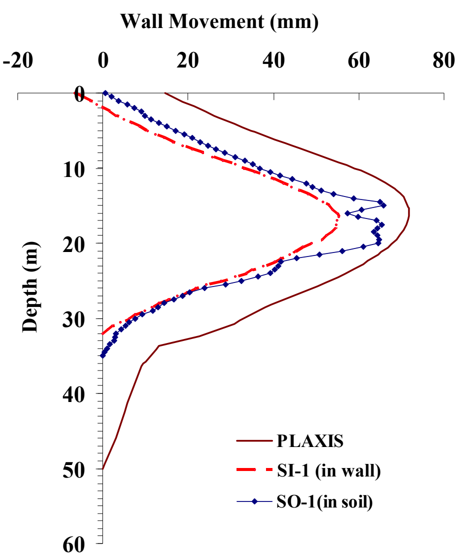
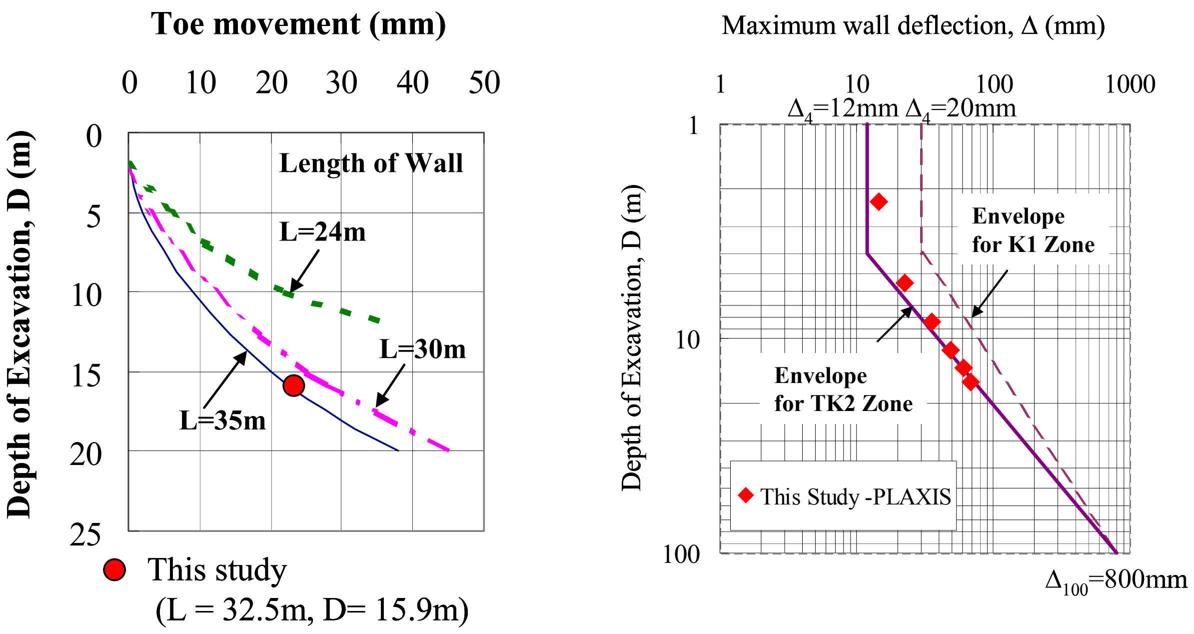


Figure 14: Computed and measured deflection of diaphragm wall at final stage of excavation

Figure 15 shows the comparison of analysis results with excavation experience in Taipei. In Figure 15(a), the computed toe movement for the case compares fairly with those proposed for the central areas of the Taipei Basin (Hwang et al. 2007). In Figure 15(b), on the other hand, the computed maximum wall deflections almost follow the corresponding reference envelope proposed by Hwang & Moh (2007). Should analysis results correspond well to the wall behavior on the site, it appears that the installed buttress and RC panel are not as effective as expected.

Figure 16 presents comparison of computed and measured convergence before and after the implementation of mitigation measures. The convergence becomes stable even during the drilling of the up-track and common duct tunnels, indicative of efficiency for the measures. A fair agreement is obtained between analysis results and measurements.



(a) Toe movement (b) Reference envelope

Figure 15: Comparison of numerical analyses with Taipei's excavation experience

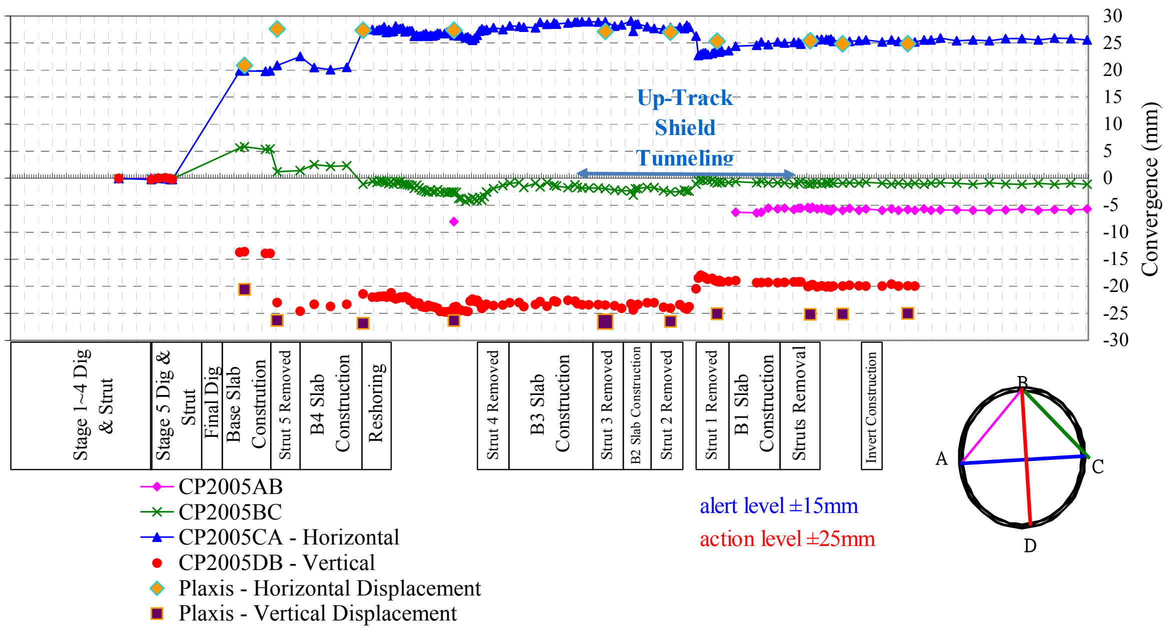


Figure 16: Comparison of computed and measured tunnel convergence before and after mitigation implementation

4 CONCLUDING REMARKS

Several conclusions can be drawn for the case studies of the paper as follows:

(1) To reduce excavation-induced wall deflection and its consequent effects on adjacent structures or facilities, buried RC panel and buttress have been widely applied to the urban excavation in Taiwan.

(2) The efficiency of the panel or buttress can be recognized through comparison of the induced wall deflection with respect to the corresponding reference envelope that is developed under the condition of green field and no ground improvement or auxiliary supports (e.g. Hwang & Moh 2007).

(3) Though 2-D or more sophisticated numerical schemes provide access to look into the soil-substructure interaction between the deep excavation site and the adjacent underground constructions, it is always noted that a quality monitoring system is of equal importance to the subject. It provides invaluable feedback data that could support and enhance not only analyses or design but engineering judgment as well.

ACKNOWLEDGEMENTS

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A Case History of Diaphragm Wall Trench Failure Counter Measures Applied in Taipei Loose Ground

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ABSTRACT

The Taipei Rapid Transit System (TRTS) Luzhou Line adopts the practice of diaphragm wall trench stable analysis in order to analyze the safety factor of excavation trenches affected by loose ground and slurry condition. Part of a TRTS station located in a park is a suitable site to execute a trial test to confirm the analysis results and effectiveness of the different counter measures. A number of counter measures for preventing trench failure, such as "enhancement of the slurry quality", "installation of steel plate to spread the construction equipment load", "increasing the fluid water head difference between slurry level and ground water level", "using different grouting patterns", etc., were applied step by step and evaluated for their effectiveness in order to apply the most suitable counter methods for the diaphragm walling near adjacent buildings so as to attain safe and economical aims.

1 BACKGROUND

The phenomenon of underground retaining wall trench failure is normally due to improper operational disturbances of the soft ground in high ground water alluvium. For this project, as a precaution, many diaphragm wall trench collapse case histories of other job sites were noted and referenced prior to construction; while on the other hand, there have been several reported successes of diaphragm walling near the project site. Nonetheless, this project adopts basic counter measures to improve the stability of the diaphragm wall trench, and in addition, through a diaphragm wall trench stable analysis, analyzes the safety factor of the excavation trench affected by loose ground and slurry condition, in order to develop dewatering measures and different grouting patterns to keep the trench stable.

2 PROJECT DESCRIPTION

The Luzhou Line of the Taipei Rapid Transit System (TRTS) travels through the districts of Sanchong and Luzhou in New Taipei City (refer to Figure 2-1), surrounded by river and floodway, and prone to flooding since these districts are in low-lying areas (refer to Figure 2-2). In fact, serious trench collapses of the diaphragm walling occurred at O45 and O47 stations (refer to Figure 2-3), while the diaphragm walling of O43, O44, and O46 stations proceeded smoothly. For the CL700A construction contract, the construction and effectiveness of various counter measures applied on O47 and O46 stations as performed by the main contractor and same diaphragm wall subcontractor were inspected and reviewed. The CL700A construction project includes O47 and O46 stations plus their connecting tunnels, and the ground condition of the job site is typical Taipei alluvium, which consists of clay and sand interlayer as shown in Figure 2-4. The ground water level of the site is at GL-1.0m to GL-2.5m, and in general, the serious trench collapses occurred at loose sandy sub-layer (SL 5).

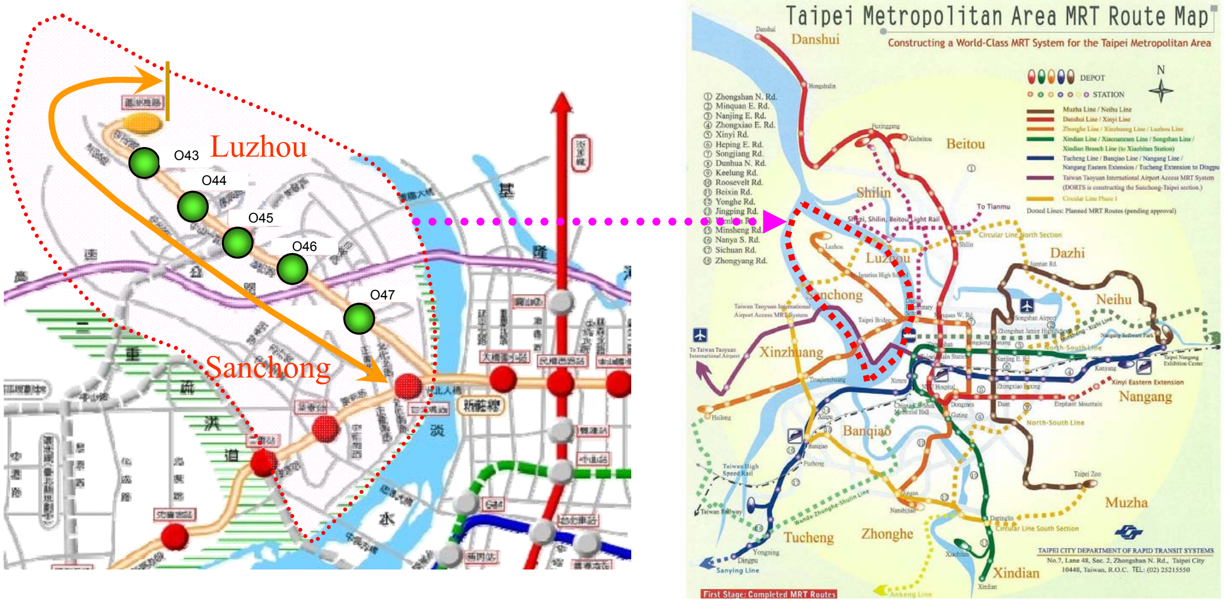


Figure 2-1: Location of TRTS Luzhou Line

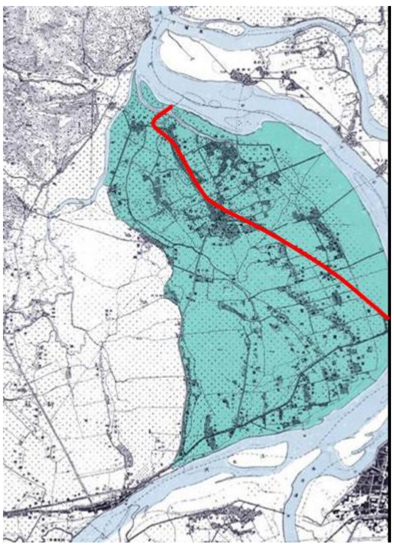


Figure 2-2: Low elevation of job site (Flood Area -Historic Map)

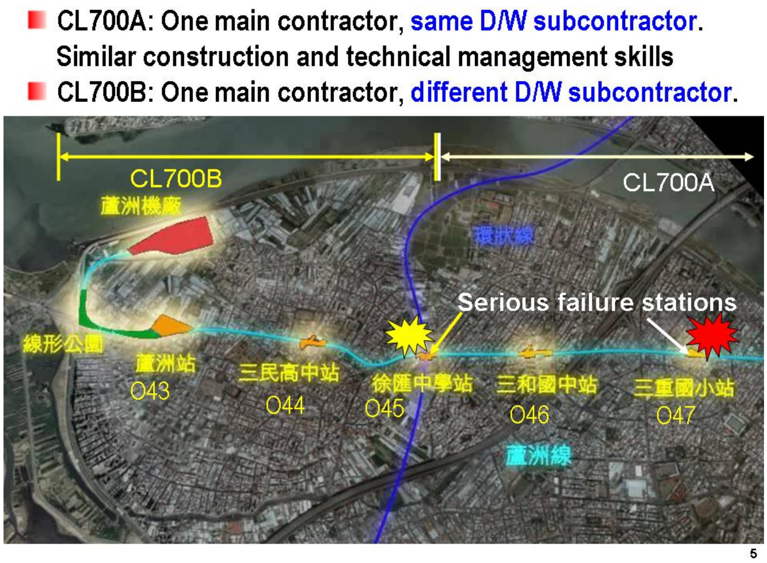


Figure 2-3: Serious diaphragm wall (D/W) trench collapses and their contractor at Luzhou Line

3 DIAPHRAGM WALL INSTALATION

A 0.6m thickness and 27m depth diaphragm wall is adopted for the 13.8m excavation depth of the O47 station entrance. Considering to increasing the stiffness of retaining wall in order to reduce the wall deflection due to station excavation, a 1.2m thickness and 39m depth diaphragm wall is applied for the 20.8m excavation depth of the main station. A total of 114 diaphragm wall panels for O47 station are arranged as shown in Figure 3-1, and a total of 28 diaphragm wall panels (24% of the 114 panels) were causing serious collapse during wall installation. Various types of counter measures were applied step by step in seven construction stages (refer to Figure 3-2), and their effectiveness is reviewed and described as follows.

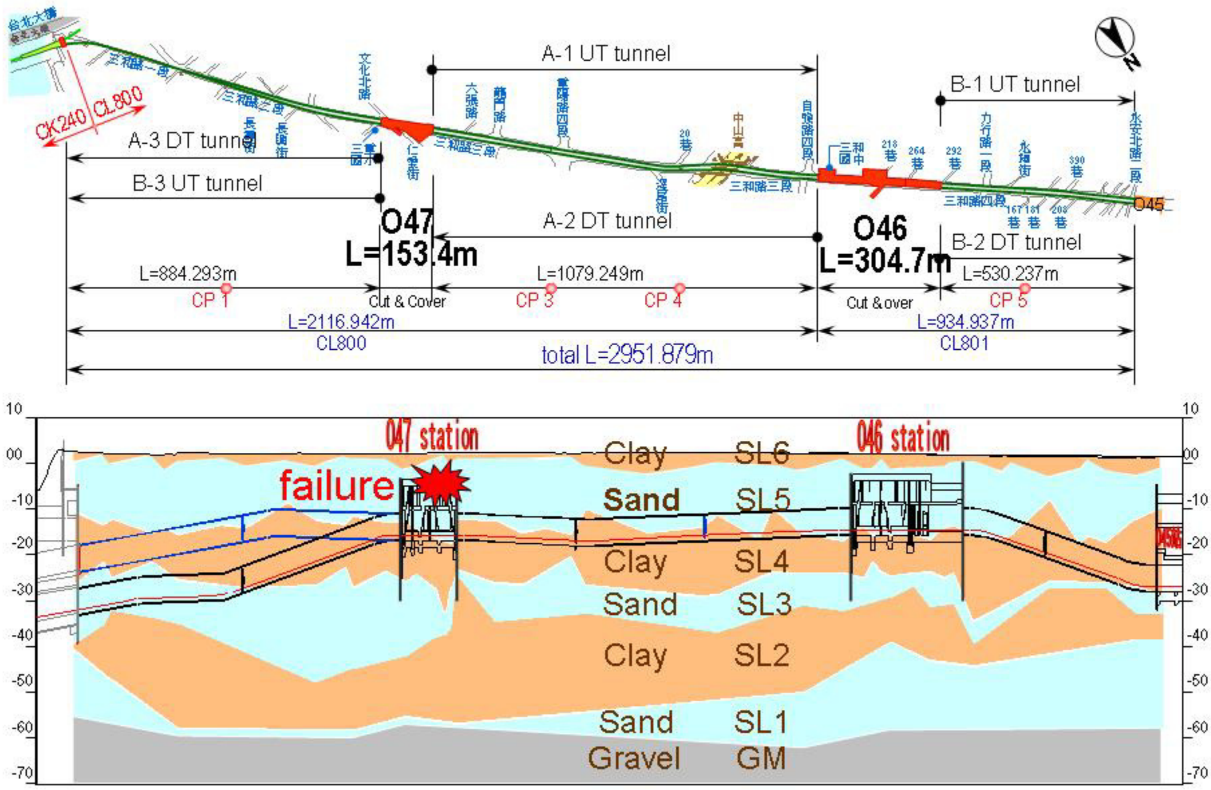
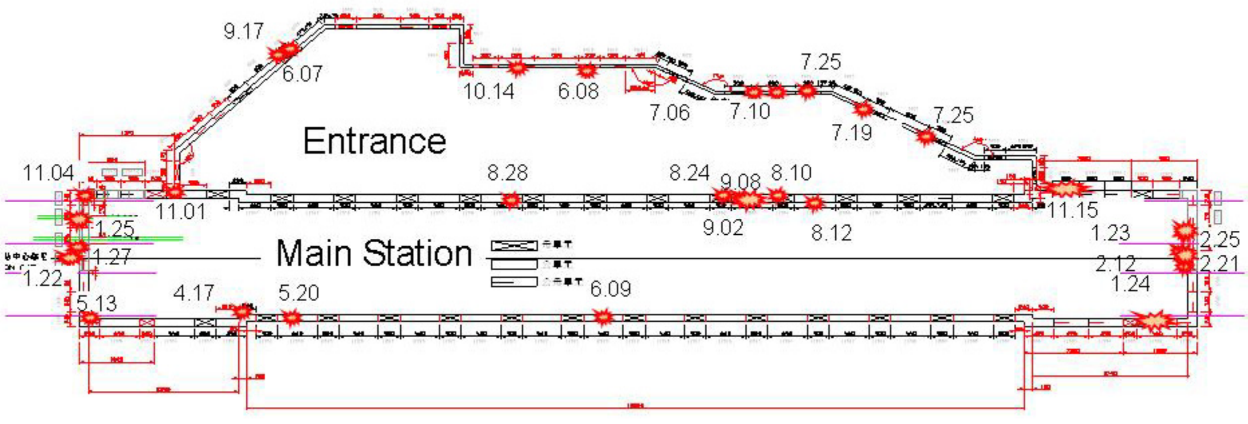


Figure 2-4: Ground conditions of CL700A



Collapse panels / Total panels: 28 / 114 (24%)
Diaphragm wall construction period:
2002.6~2003.6

Thickness=0.6m, Length= 148.4m, Panel length=3~6m
Thickness=1.2m, Length=468.5m, Panel length=3~5.6m

Figure 3-1: Diaphragm wall panel arrangement and collapses of O47 station

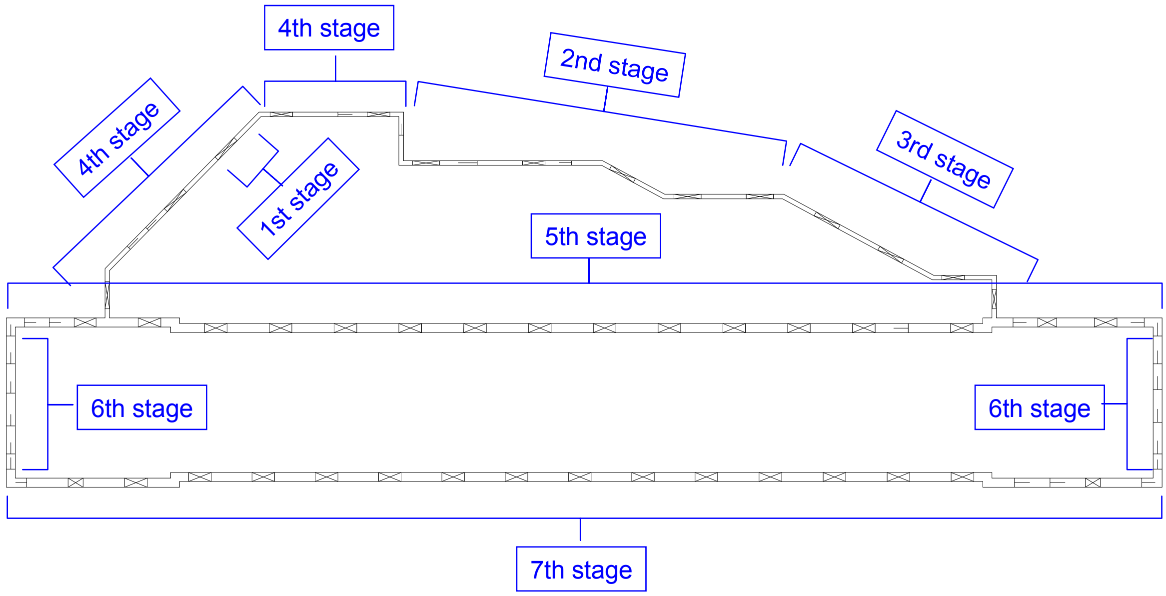


Figure 3-2: Different construction stages of O47 station diaphragm walling

3.1 Normal operation (1st stage)

Normal diaphragm wall operation was adopted on the site at the initial stage and slurry viscosity was generally controlled at 28 - 36 sec. The first two diaphragm wall panels of the trench collapsed, which caused adjacent ground settlement; hence, the guide wall was moved and diaphragm wall installation was suspended. Several factors, including ground conditions and operation management might affect the trench stability as summarized and shown in Table 3-1. However, emergency actions, such as grouting and backfill, were applied immediately after trench collapse on site. It is difficult to identify the actual impact factors which caused the trench collapse in accordance with the available information obtained from the site.

Table 3-1: Factors Affecting Trench Stability

Item	Easily Cause Trench Failure Factors
Ground condition	High ground water alluvium, improper diaphragm walling operation will easily cause trench failure at: (1) soft ground, such as loose density sandy soil or sensitive cohesive soil; (2) high permeability soft ground.
Ground water level	The low different pressure head between slurry and ground water level.
Slurry quality	Improper slurry gravity control.
Long installation period	Long length panel, deep wall depth.
Panel shape and size	Difficult operation on corner panel disturbs ground, and needs a long installation period.
Excavation depth	Deep wall needs a long installing period.
Operating skill	Unstable operation disturbs ground.
Surcharge loading	Non-uniform load near construction equipment. Job site near road, or railway.
Others	Partiality loading, ground movement, etc.

Cost expenses for grouting the full loose sandy layer at both sides of the diaphragm wall before walling were evaluated and summarized in Table 3-2. Besides expensive grouting, other counter measures such as (1) enhancement of slurry viscosity and quality control, (2) reducing the interface waiting period between different construction activities, such as rebar cage installation and tremie concreting, (3) increasing the different pressure head between slurry and ground water level, and (4) stabilizing the diaphragm wall operation techniques in order not to disturb the ground, were also adopted. In addition, the most suitable grouting pattern, such as grout thickness, space, and depth, need to be determined before construction at areas near adjacent buildings so as to attain safe and economical aims.

An empty space far away from the existing adjacent buildings was available and suitable to execute trial tests for various counter measures which would thereby provide useful information about their respective effectiveness. A number of different counter measures, therefore, were determined to be executed step by step in order to select the most proper counter measure and to determine the required grouting pattern to be applied for diaphragm walling near an adjacent building.

Table 3-2: Cost for full grout at all loose ground and actual counter measures

Station	Full Grout At All Loose Ground Fee (NT$)	Actual Fee Applied on Site (NT$)
O46	62,000,000	0
O47	30,000,000	16,000,000(53%)
Total	92,000,000	16,000,000(17%)

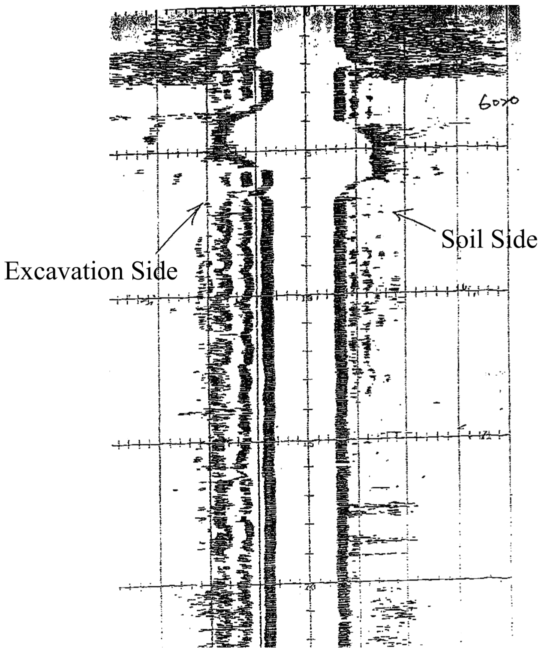
3.2 Basic counter measures: (2nd stage)

Besides normal diaphragm wall construction management, basic counter measures to improve construction management, including: (1) slow and stable trench excavation in order not to disturb sensitive soil; (2) increasing polymer slurry viscosity from 28 - 36 sec. to 40 - 45 sec.; (3) corner panel installation first; (4) shortening the construction interface waiting period between rebar cage installation and tremie concreting; and (5) besides the normal working R.C. slab, install steel plate on both sides of excavation panel to uniform the construction equipment surcharge loading in order not to cause ground movement. By adopting the above mentioned measures, all of the panels could be completed; however, all of the panels experienced trench failure during installation (refer to Figure 3-3). Consequently, it was confirmed that grouting and other measures would be required to prevent trench failure for the soft ground.

3.3 Diaphragm wall trench stable analysis

Serious diaphragm wall trench collapses occurred during construction of O47 station, while for O46 station, the diaphragm wall installation proceeded smoothly despite applying the same basic counter measures. Even though the ground condition of O46 station is similar to the ground of O47 station (refer to Table 3-3), the ground water table is 1.8m and 1.0m below the ground surface for O46 station and O47 station, respectively (refer to Figures 3-4 and 3-5). In general, the slurry was controlled at around 0.4m below the guide wall (ground) surface in order to prevent slurry to flow through and pollute the nearby roads for normal operation (refer to Figure 3-6). As a result, the 1.4m (=1.8m-0.4m) different pressure head between slurry and ground water is kept for O46 Station. On the other hand, there is a different pressure head between slurry and ground water of 0.6m (=1.0m-0.4m) for O47 Station (refer to Figures 3-5 and 3-7).

A trench stable analysis model (refer Figure 3-8) was introduced to analyze the safety factor of excavation trench affected by ground and slurry condition for the project. The analysis (refer to Figure 3-9) shows that the different pressure heads between the slurry and ground water was significantly affecting the safety factor of trench stability. So, keeping the different pressure heads between the slurry and ground water table as high as possible will be a major and economical counter measure to prevent trench failure (refer to Figure 3-7).



Trench failure, but diaphragm wall can be completed.

Trench collapse, causing adjacent ground settlement. Guide wall movement, construction shall be stopped.

Figure 3-3: Definition of trench failure and trench collapse on this project

Table 3-3: Soil conditions of O47 and O46 stations are similar

O47 Station Design Parameters						O46 Station Design Parameters					
Layer	classic	N	γ_t(t/m^3)	C'	ϕ '	Layer	classic	N	γ_t(t/m^3)	C'	ϕ '
6	CL	3	1.85	0	27.0	6	CL	4	1.85	0	27.0
5	SM	8	1.90	0	32.0	5	SM	9	1.90	0	32.0
4	CL	6	1.87	0	29.0	4	CL	6	1.87	0	29.0
3	SM	14	1.92	0	34.0	3	SM	15	1.92	0	34.0
2	CL	18	1.93	0	30.0	2	CL	18	1.93	0	30.0
1	SM	27	2.00	0	34.0	1	SM	27	2.00	0	34.0
GM	GW	>100	2.10	0	36.0	GM	GW	>100	2.10	0	36.0

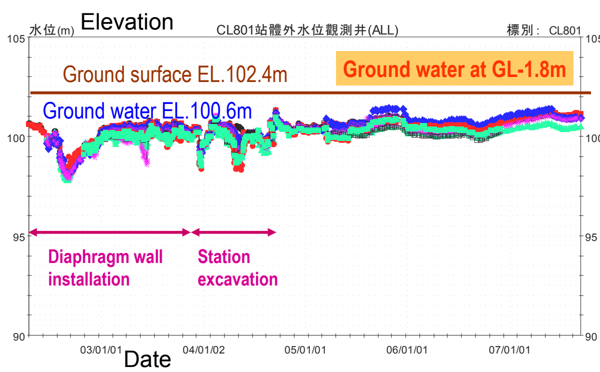


Figure 3-4: Ground water condition outside of O46 station

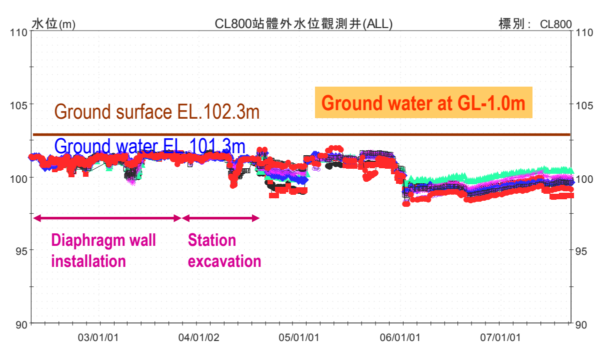


Figure 3-5: Ground water condition outside of O47 station

Figure 3-6: Slurry controlled at 0.4m below guide wall surface for O46 station.

Figure 3-7: Slurry raise from 0.4m to 0.05m below guide wall surface for O47 station.

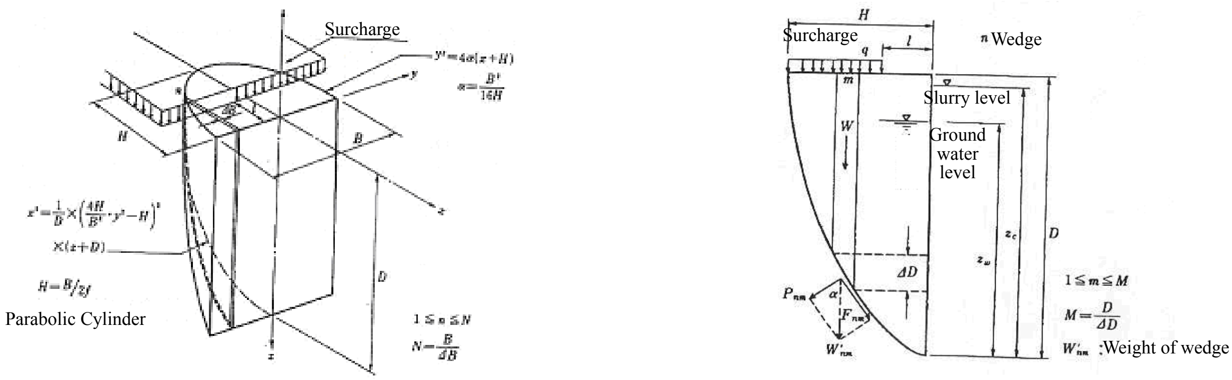
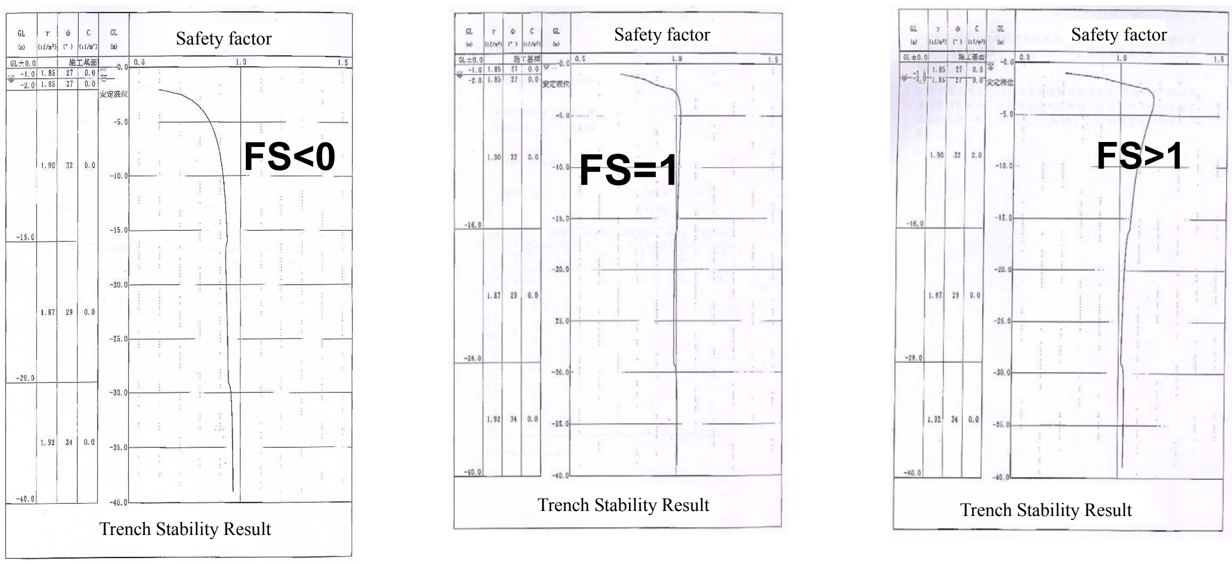


Figure 3-8: Diaphragm wall trench stable analysis model



Slurry gravity=1.01T/m^3
Fluid water head difference : 0m

Slurry gravity=1.01T/m^3
Fluid water head difference : 0.5m

Slurry gravity=1.01T/m^3
Fluid water head difference: 1m

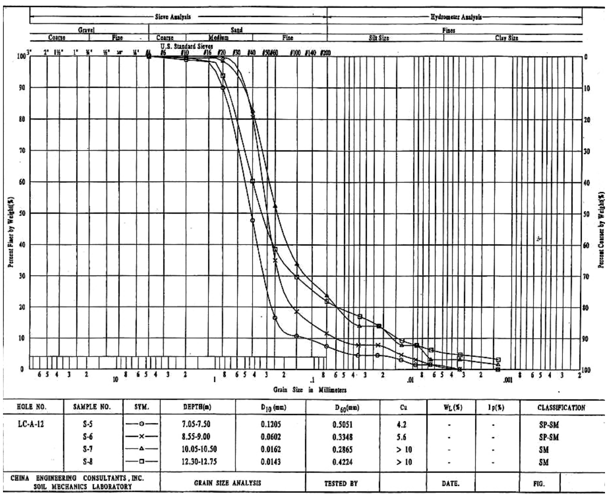
Figure 3-9: Safety factor is seriously affected by the fluid water head difference between the slurry and ground water for O47 station

3.4 Required grout pattern, dewatering wells, and their effectiveness

There are three ways: (1) raise the elevation of guide wall; (2) temporary installation of sand bags beside panels to raise the guide wall; and (3) temporary de-watering around excavation diaphragm wall panels, which can raise the different pressure heads between the slurry and ground water table. Taking into consideration the difficult at-road interfaces for diverting and maintaining the traffic conditions, as well as the limitations of ground settlement due to temporary 1 to 2m de-watering, wells, therefore, are a suitable counter measure to enhance trench stability for the site. In addition to the basic counter measures, a number of wells, with a diameter of 45 cm CCP (Chemical Churning Pile) grouting pattern were applied and evaluated for their effectiveness during different construction stages as summarized and shown in Table 3-4.

At the 3rd stage, CCP grout was adopted at the outside of the primary diaphragm wall panel of deep wells with an interval space of around 20m to lower the ground water level. But, trench collapse occurred for all panels that were without grout. Furthermore, the ground water of O47 station back-flowed quickly after pumping stopped, while the ground water of O46 station can easily be lowered down. The grade size distribution of sandy soil shows that the sandy layer is classified as well graded soil for O46 station, and uniform soil for O47 station (refer to Figure 3-10). This explains why the permeability of O47 station is high and as per Table 3-1, the higher permeability loose sand is a type of ground condition which is easily subject to trench failure. As a result, more point wells were needed for O47 station to lower down the ground water table, whereas proper ground water control can replace part of the grout in order to achieve the economical aims for preventing trench collapse.

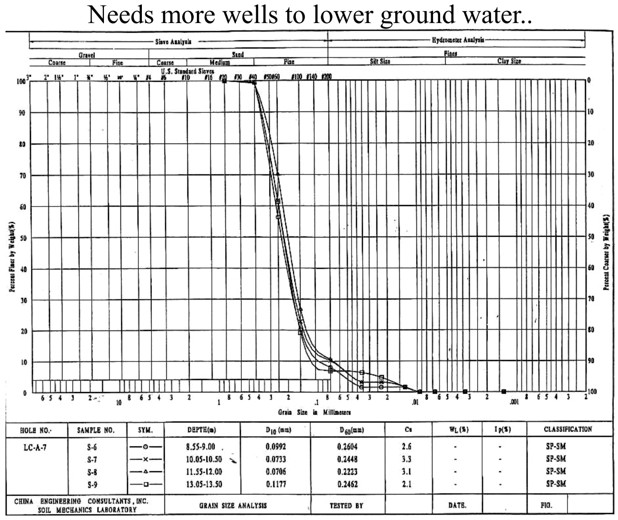
O46 station: ground water can be easily lowered down.



Coefficient of Uniformity Cu=5 – 8, Cu>=9;

Well graded Soil

O47 station: ground water backflows quickly after the pumping is stopped.
Needs more wells to lower ground water..



Cu=1 – 4; Uniform Soil

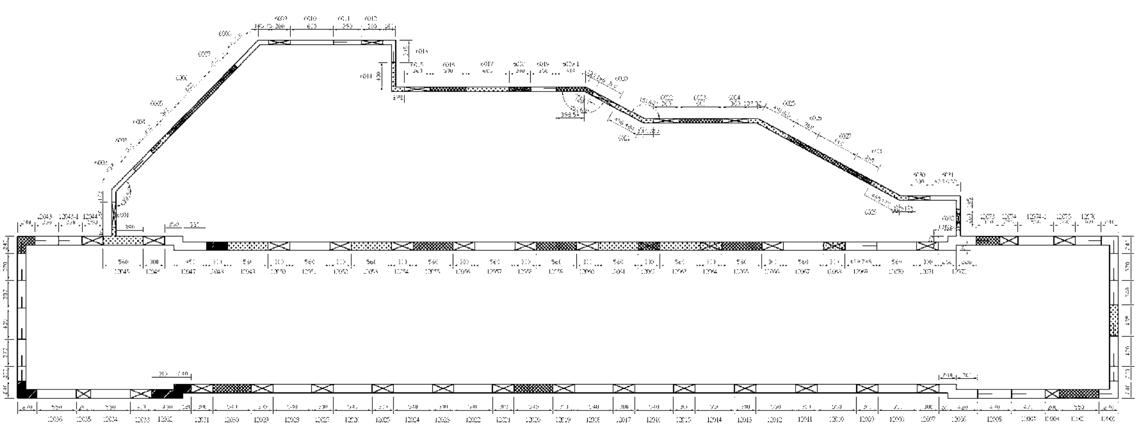
Figure 3-10: Grade size distribution of sandy soil of two stations

Table 3-4: Effectiveness of different counter measures applied on O47 station

Stage	Counter Measures	Result and Conclusion
1st (2002.6)	Normal operation: Slurry viscosity controlled at 28~36 sec.	Trench collapse, enhance construction management.
2nd (2002.6-2002.7)	Basic Counter Measures: A. Install steel plate to spread the construction equipment load, stable operation, and shorten the construction waiting interface period. B. Improve the slurry quality by increasing slurry viscosity from 28 - 36 sec. to 40 - 45 sec.	Grout and other measures are required to prevent trench failure.
3rd (2002.7)	A. Install release well spaced every 20m inside the excavation zone to lower ground water level. B. L=7.5m φ45cm CCP grout at outside of primary panel.	Panels without grout were failure. Both sides of primary and secondary panels need grout.
4th (2002.7-2002.9)	A. L=7.5m φ45cm CCP grout at both sides of all panels. B. Increase more point wells to control the ground water level.	Double rows CCP needed for corner panels after two corner panel failures. The wells are unable to lower down the water table near panels for high permeability sandy layer.
5th (2002.8-2002.12)	A. L=10m and L=18m (to clayey layer) φ45cm CCP grout at park side and road side, respectively. B. Shorten the space between wells to 10m.	Both L=10m and 18m CCP grout with dewatering near panels can prevent failure. 18m CCP is not required. Close panel, trench failure due to ground water table rises immediately after Typhoon raining period.
6th (2003.1-2003.3)	Additional low pressure grout under complicated utilities at station end wall.	Through many trench collapses due to utilities relocated, with additional grout to complete diaphragm walling.
7th (2003.5-2003.6)	A. L=10m φ45cm CCP grout both sides of all panels at nearby existing adjacent buildings. B. Install point wells every 10m space, temporary lower water level around excavation panel.	Except some panel failure due to abnormal disturbance under existing utilities, all panels at adjacent buildings were successfully completed.

4 DEFECTIVE PANEL AND INSPECTION

Several defective diaphragm wall modes and construction records were reviewed before station excavation. Defective diaphragm wall panels include: (1) trench collapse; (2) trench failure during excavation detected in accordance with the ultrasonic test results (Figure 3-3); (3) trench failure due to disturbing the ground during installation of the rebar cage; (4) buried soil defective wall panels, where trench failure and soil buried in the wall owing to unusual disturbances during tremie concreting causes the concrete table to rise suddenly due to trench failure; and (5) tremie concreting defective panels, which when estimated in accordance with unusual concreting amounts (larger or less than 5% of anticipated), may need to remove the extra portion. A summary diagram of all defective panels for O47 station as shown in Figure 4-1 was set as the inspection key point by the site engineers.



: Trench collapse during diaphragm walling (maybe buried soil cause leakage)

: Additional grout treatment adopted before station excavation

: Tremie Concreting defective panel

Figure 4-1: Defective wall panels diagram for O47 station

5 CONCLUSIONS

Primary panels, panels under existing utilities, and close panels are panels which are easily subject to failure. The corner panel should be installed early, and to enhance slurry quality, slow and stable diaphragm walling operations are the basic counter measures to prevent trench collapse. Trench stability analysis shows that the different pressure heads between the slurry and ground water level will significantly impact the stability of the trench. Also, the required grout depth can be noted from the analysis results. In order to achieve the required different pressure heads between the slurry and ground water, point wells are convenient and economic measures which can effectively increase the trench stability. Furthermore, only applying CCP grout for the diaphragm walling under existing utilities will not be sufficient. And finally, a defective panel diagram shows the key points for site inspection, whereas a well-prepared and quick-response rescue mobilization program can reduce the amount of damage if an accident takes place.

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Two Case Studies of Collapsed Temporary Excavation using Contiguous Bored Pile Wall

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ABSTRACT

Failure investigation of temporary excavation can be deployed in a systematic manner using the principles of forensic engineering. This paper focuses on the investigation works for two (2) temporary excavation failures in Klang Valley area, Malaysia. Both excavations utilised temporary steel struts propped against partially completed basement structure as lateral shoring supports along the peripheral retaining wall. The two investigations show that it is undoubtedly necessary to have appropriate and sufficient materials sampling and testing as part of the investigation processes if conclusive evidence regarding the failure cause is to be determined. The method, combined with the experience of the investigators, adequate evidences of material defects and numerical modelling of actual construction processes by finite element analyses, provide useful insight in exploring the probable causes of the collapse of temporary excavation and identify the major cause(s) accounted for the collapses. The excavation failure investigation methodology presented in this paper can serve as a guide for the investigation of similar failures and also as a lesson learnt for future excavation projects.

1 INTRODUCTION

Engineering failure investigation is a detective process of determining why and how things went wrong. Forensic engineering can be defined as the application of the engineering sciences to the investigation of failures or other performance problems, with a focus on uncovering causes so that improved facilities can be engineered.

The first stage in determining the failure causes is the investigative synthesis, where all the information gathered is recorded in a logical manner, typically in a report format and in chronological order. The following details are generally required from the Client for undertaking a excavation failure investigation study:

i. Subsurface Investigation (SI) factual Report.
ii. Geotechnical design report prepared by the consultant (if available)
iii. Original topography of the site (before earthworks)
iv. Earthworks plan and cross-sections
v. Permanent retaining walls and temporary strutting design analyses and calculations by the consultant or/and contractor
vi. Detailed construction drawings of the walls and its excavation and strutting sequences
vii. As-built drawings of the basement walls
viii. Construction sequences adopted at site
ix. Weather records of the site
x. Instrumentation monitoring reports
xi. Site records and reports of the incident

From the listed information, it is then necessary to determine which information supports or refutes each of the possible failure hypotheses. This may be initially be done by considering general failure causes, such as those related to natural disasters, act of sabotage, material defects, design, construction sequences, or the environment.

Once the probable causes of failure are identified, the information and knowledge learned during the investigation should be listed, for inclusion in the final investigation report. This should include any unusual aspects of the failure, and also any recommendations as to how the failure could be prevented in the future, through improved design, construction controls or good maintenance practices, or through an increased knowledge of the materials used and their properties.

It is normally impossible to conclude with complete certainty what the cause of the excavation failure being investigated was. Instead, there are often multiple factors that contributed to and with one most probable factor triggered the failure, in which probable causes are normally stated in the context of failure investigation. The common probable causes of excavation failure could be one or more than one of the following factors:

a. Natural Disasters : fire, earthquake, tsunami, tremor, wind, rainfall and flood
b. Act of Sabotage: explosive substances
c. Material Defects: reused steel strutting sections with poor conditions, concrete properties
d. Design: modelling and design parameters, robustness and ductility
e. Construction: sequences of works, excavation depth
f. Maintenance: drainage system, no timely review of instrumentation results

In general, the investigation procedure in geotechnical failure are listed below but not limited to:

i. Check safety factor of the original design
ii. Check the as-built construction for any deviations from original design
iii. Identify design shortcomings, material defects, workmanship deficiencies, if any
iv. Interview design team, construction management team, site personnel and eye witnesses
v. Consult other experts if required, for matters beyond the investigator's expertise or knowledge of the facts
vi. Identify possible collapse scenarios and rationalise conflicting facts or evidences
vii. Determine the major contributory and triggering factors that cause the collapse
viii. Conduct advanced non-linear analysis /tests to ascertain the collapse mechanism
ix. Confirm the collapse mechanism with those from facts and evidences
x. Write report

This paper discusses the approach taken to investigate the two case studies of distressed open excavation with contiguous bored pile wall (CBP) and steel strutting system at Klang Valley area in Peninsular Malaysia. During the course of investigation, interesting processes of gathering factual evidences, verification of conflicting facts, analysis for the sequence of events and finally arriving at conclusive findings with convincing evidence will be discussed in the paper. Last but not least, it is extremely useful to learn from these two failure cases to improve construction safety of future projects.

2 CASE HISTORY 1 (Liew & Khoo 2008)

2.1 Project background

This case history involved construction of a two-storey basement in an urban area. The scope of investigation was to evaluate the conditions of a distressed temporary shoring structure, investigate the probable causes and subsequently to propose remedial options.

Figure 1 shows the location of the project site and the adjacent land lot which had been affected due to ground distresses.

Temporary shoring structure consisting of CBP wall propped by raking struts against lower basement slab was proposed by the contractor to provide peripheral support for a 10.5m deep excavation at western boundary of the project site. The designed CBP wall was 16m long 750mm diameter bored pile with top cut off level at RL 54.6m. In order to facilitate the CBP installation, 12m long temporary steel sheet piles (type FSP IIIA) were installed at retained ground at RL59.0m with about 0.7m offset from project site boundary to provide sufficient working platform and as the temporary shoring support for the exposed 4.4m temporary excavation (from RL59.0m to RL54.6m). Figure 2 shows the details and cross-section of the proposed alternative temporary shoring system and permanent retaining wall.

After completion of CBP wall installation, excavation was continued from RL54.6m to RL52.0m. Ground distresses at the adjacent retained platform in the forms of ground subsidence, tension cracks and excessive deviation of the CBP wall were observed. From the site observation, deviation of CBP wall was likely caused by the over-excavation of the temporary passive berm with localized deep pile cap excavation in front of the wall without timely installation of the planned raking strut. The incidence had affected the adjacent property lot with considerable ground distresses and also resulted in structural damages to the CBP wall.

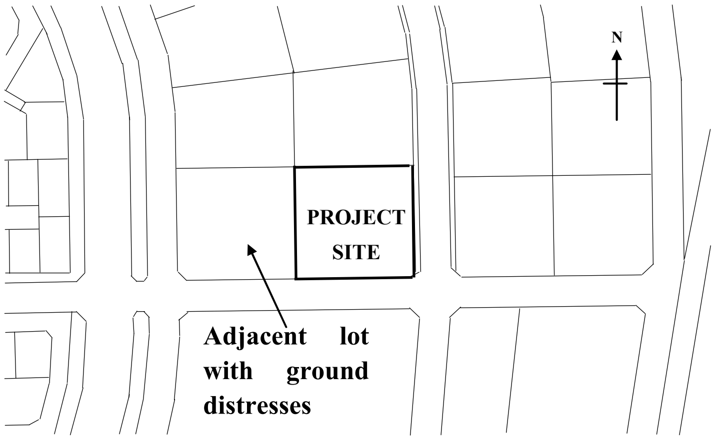


Figure 1: Site location

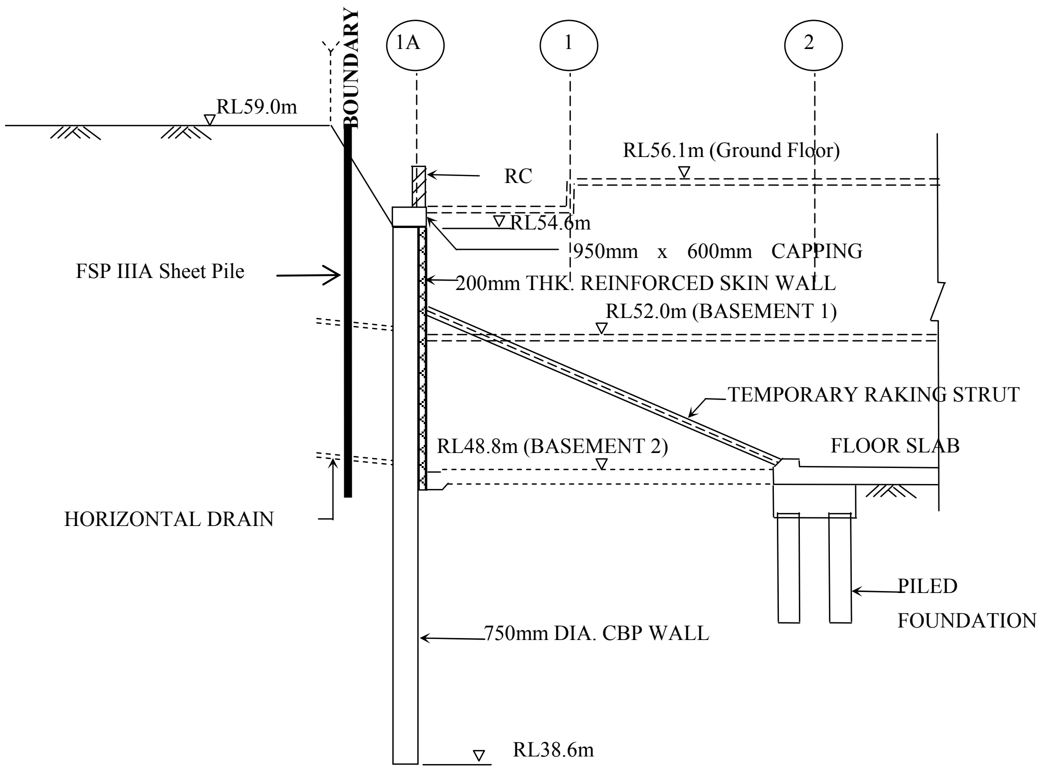


Figure 2: Cross-section of the proposed alternative retaining walls

2.2 Site conditions

From the earthworks as-built drawings, the ground level before excavation was relatively flat with levels ranging from RL54.8m to RL56.5m and a steep soil slope (1V:1H) of about 3m high sloping from adjacent lot (at RL 59.0m) towards the proposed site. Based on the topographical survey plan of adjacent lot, which was believed to be the condition before the earthworks, part of the proposed site (i.e. at north-western side) is of sloping terrain from approximately RL65m to RL56m. There was a 6m high cut slope existed within this sloping ground as indicated in Figure 3.

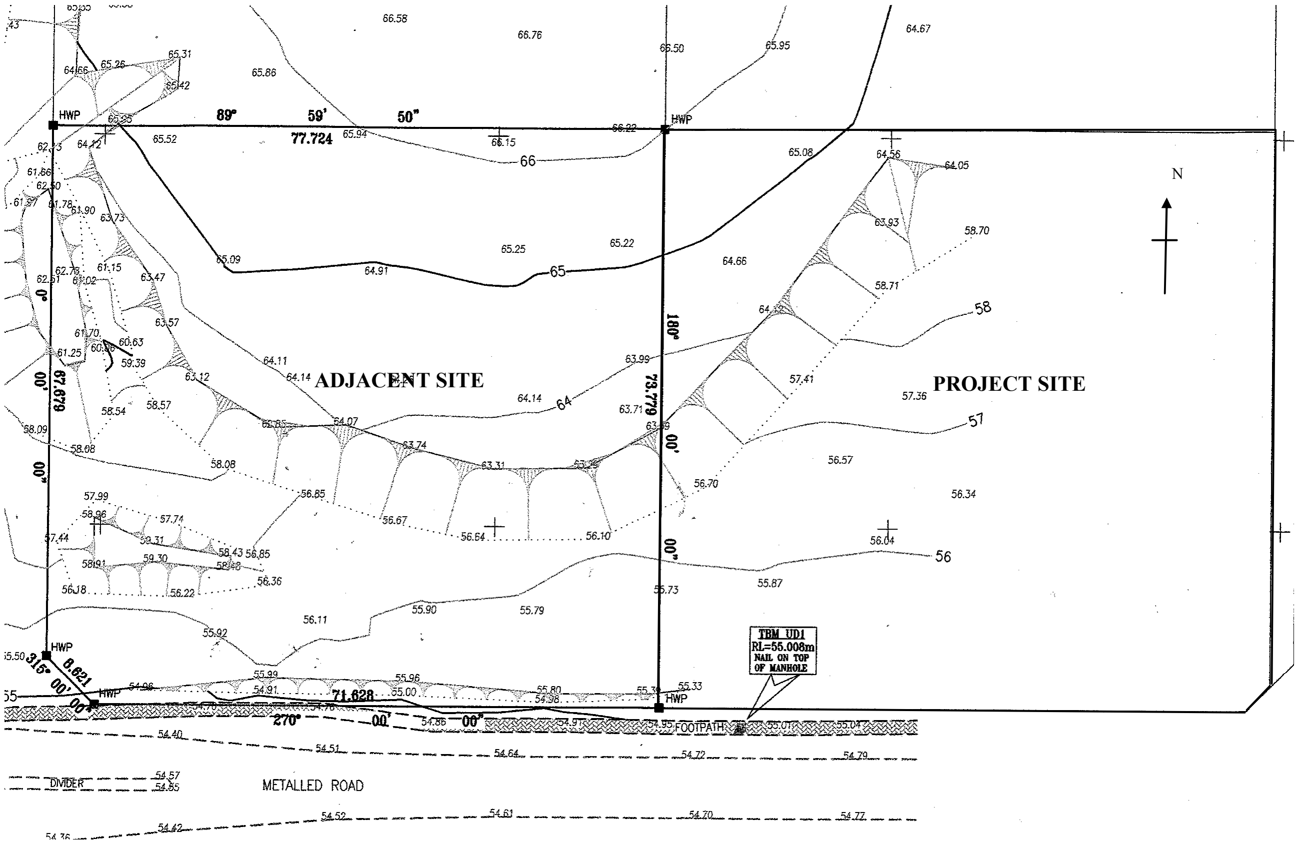


Figure 3: Topographical survey plan of adjacent lot (subjected to disturbance before reaching the finished level)

Based on the pre-development topographical condition as shown in Figure 4, the contour lines for both the adjacent lot and the project site range from RL54m to RL47m. As such, it was evidenced that earthworks had previously been carried out at these areas to raise the building platform level to RL 59.0m and RL 56.0m for the adjacent lot and the project site respectively. Both of the sites are on filled platforms. Particularly, the distressed area was primarily located at the valley where thicker fill was placed. High potential of saturation of fill due to perched groundwater seepage after filling in the previous valley terrain can be expected if subsurface drainage is not provided. In addition to the topographical map, assessment on the piling information was carried out to reveal the soil consistency profile within the site as shown in Figure 5. It was found that the distressed ground and retaining wall areas correspond well with the expected thicker fill and deeper weathering profile at the valley area.

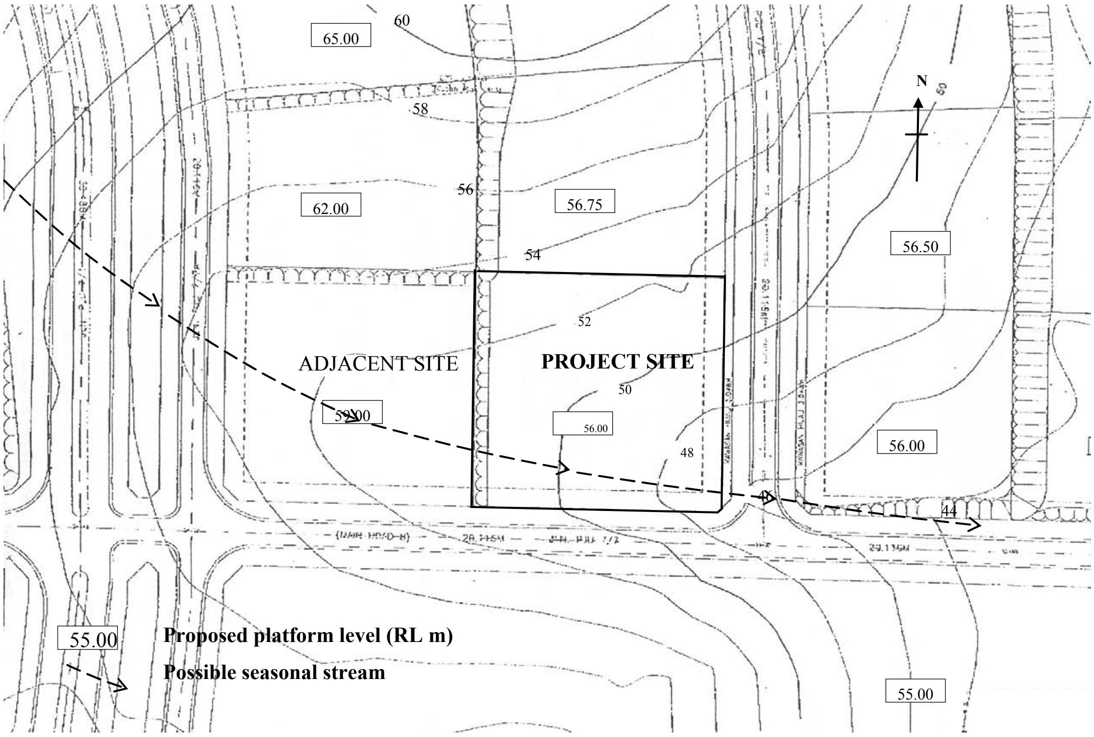


Figure 4: Contours of original ground and subsequently proposed earthwork levels

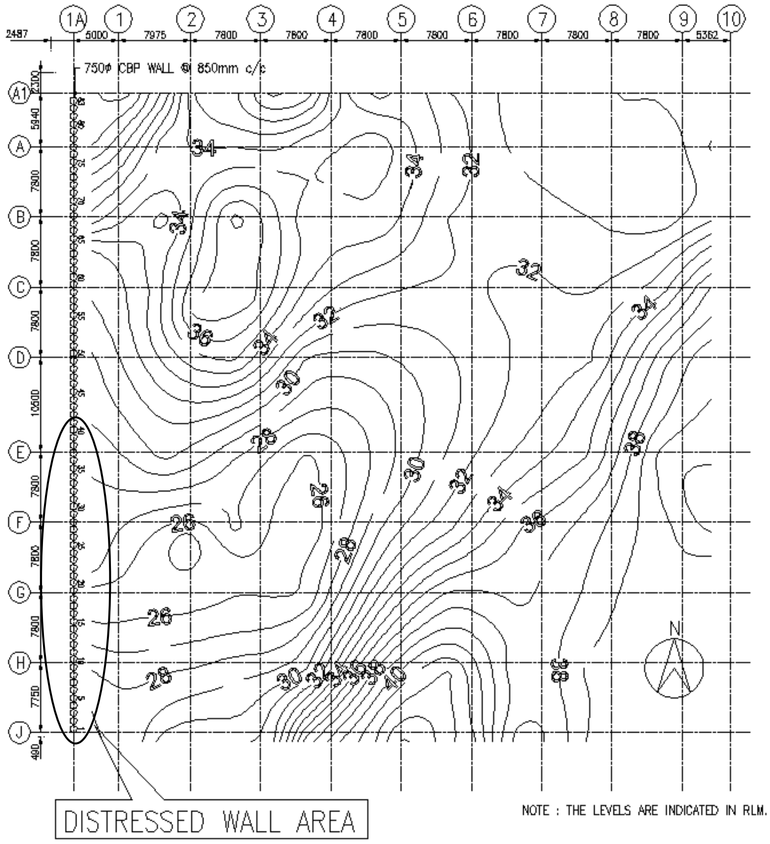


Figure 5: Interpreted contours of hard stratum from piling penetration information

2.3 Site inspection and mapping

It was observed that continuous tension cracks appeared at varying distance away along the sheet pile wall of approximately 73m long. The tension cracks were more distant away from the sheet pile wall at the southern end and became closer to the sheet pile walls toward northern end. At the time of inspection, more extensive excavation was carried out at the southern portion than the northern portion. The platform for the backyard car park of the adjacent lot had shown ground subsidence with tension cracks as shown in Figure 6 and Figure 7.

Figure 6: Site conditions of adjacent lot after the incidence of wall movement and ground distresses (Southward view)

Figure 7: Site conditions of adjacent lot after the incidence of wall and ground distresses (Northward view)

Site mapping had been carried out on the observed tension cracks and tilted sheet pile wall. Much more tension cracks were observed at the southern region. In this particular location, the sheet pile wall was seriously deviated outward relatively to the initial wall alignment. Efforts were made to map the crack lines by using measuring tape and slope meter. Figure 8 shows the details of mapped tension cracks. Generally, the worst tension cracks were measured with crack width of up to 400mm and shear drop of 500mm between the two dislodged earth blocks. At the critical location (Gridline G), the major crack line was measured at approximately 6.2m from the original fence line. While, the furthest crack line was measured at approximately 12m from the original fence line. In addition, the overall tilt angle of subsided platform at this area was crudely measured to be about 12˚ as shown in Figure 6. Sheet pile wall had also moved outward about maximum 1.2m from the initial wall alignment (at Gridline G). On the other hand, the measured pile top deviation of contiguous bored pile wall is also shown in Figure 8.

During the emergency repair work, the vibration effect of re-installing the temporary sheet piles wall by vibro-hammer had caused further tension cracks at the backyard car park platform. Therefore, it was suspected that the platform was of filled ground, which might not be well compacted as the effect of vibration might have caused soil densification and aggravated the creep movement.

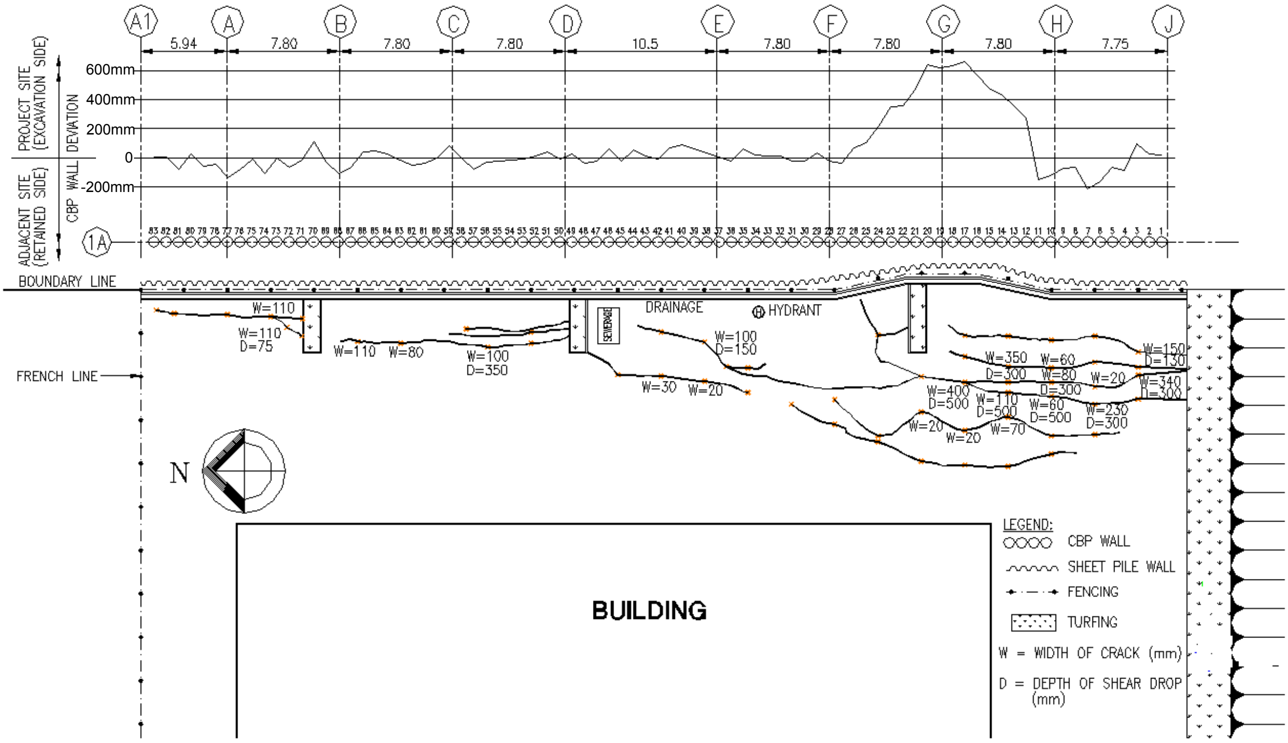


Figure 8: Crack mapping layout and CBP wall movement

2.4 Subsurface investigation

Before the wall and ground distresses, two stages of subsurface investigation (SI) works were carried out at the proposed site. The second stage SI works is the additional SI conducted at the perimeter western boundary for the alternative basement wall design by the contractor. At that time, no much attention was given in identifying the weak deposits between the original ground and the platform backfill. The SI layout is shown in Figure 9.

After the incidence, additional three boreholes were sunk within the distressed wall area to investigate the subsurface profile and to install instruments for substructure construction monitoring. In particular, a layer of 6-9m thick of very soft to soft sandy/silty clay (SPT-N $\leq$ 4) was encountered between RL52m and RL43m as detected in the few boreholes near to the distressed wall area. Generally, SPT-N values of the subsoil range from 3 to 6 at the top 13m of the subsoil and gradually increase with depth thereafter. This implies that the

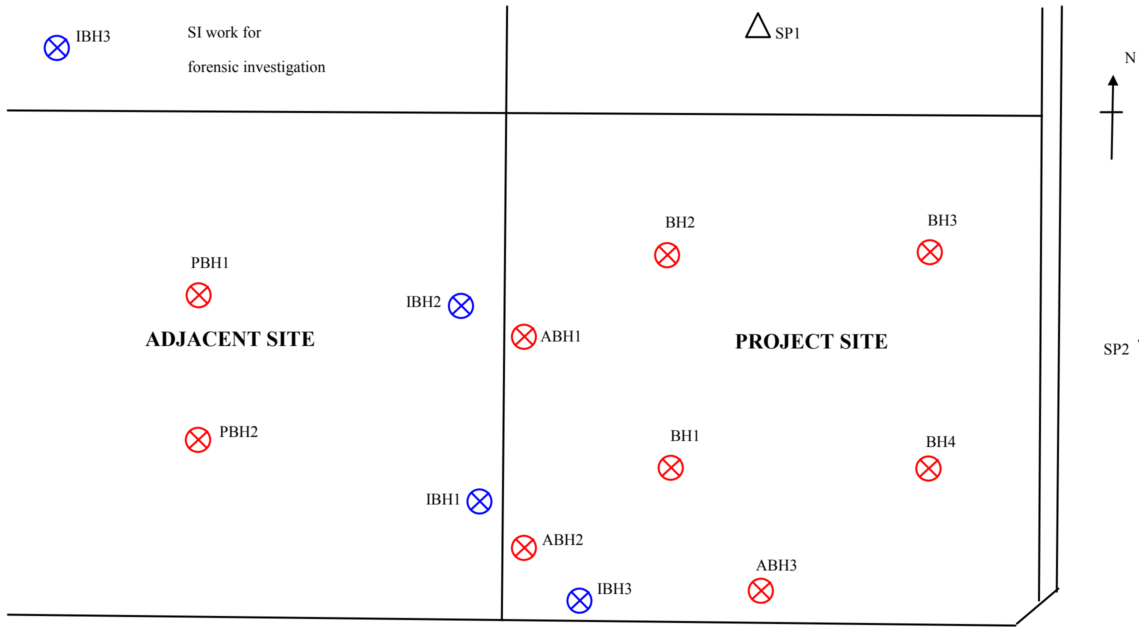


Figure 9: Layout of subsurface investigation boreholes

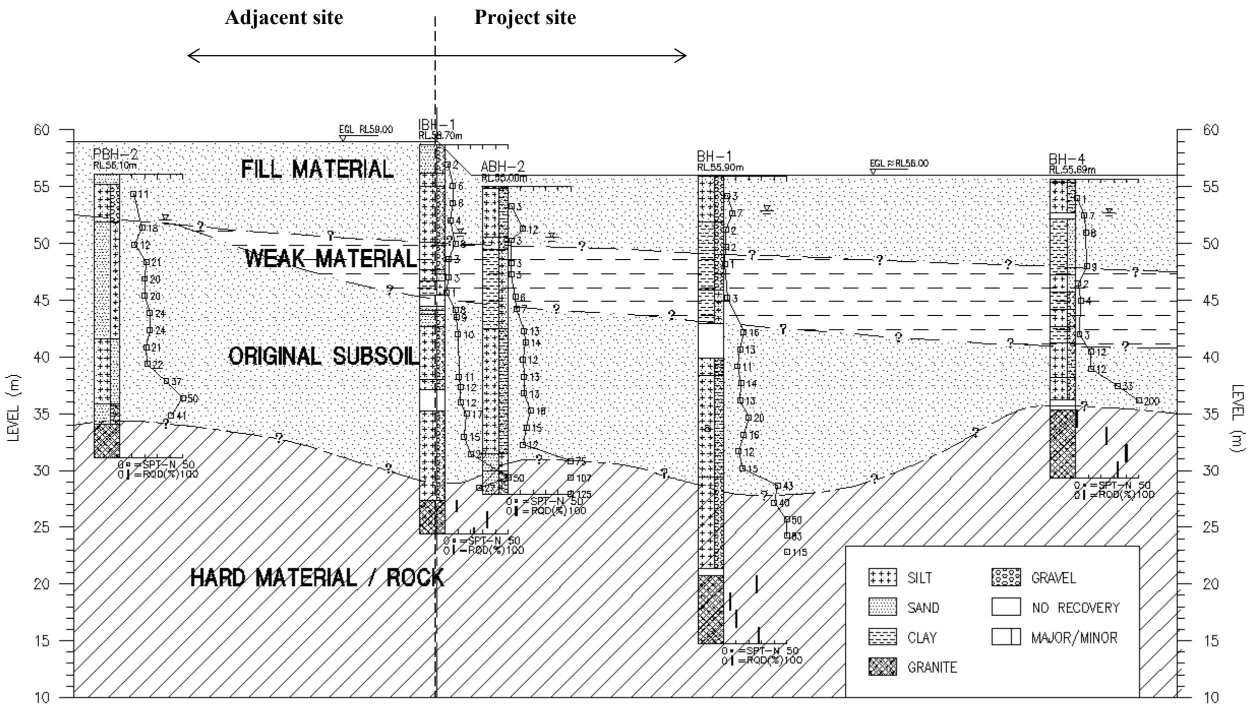


Figure 10: Interpreted subsurface profiles at the distressed wall area

2.5 Back analysis

In order to confirm the probable causes of ground distresses and wall movement, Finite Element (FE) analysis using computer software “PLAXIS” was performed independently based on the interpreted subsurface profile as shown in Figure 11. The construction sequences of excavation were simulated in the FE analysis.

At the analysis stage where over-excavation in front of the wall was carried out, the analysis results revealed that the retained earth platform displaced excessively in the horizontal and vertical (settlement) directions with the temporary sheet pile retaining wall moving forward. As part of the lateral resistance to the temporary retaining walls by the passive berm was removed before installation of raking strut, over-excavation of this passive berm had reduced the lateral resistance to the sheet pile wall and subsequently mobilised the structural strength of the retaining walls beyond serviceability state condition reaching towards the ultimate limit state condition. The excessively displaced temporary sheet pile wall had induced additional lateral force to the installed contiguous bored piles (CBP) walls. The high induced flexural stress unavoidably damaged the CBP pile. The results of FE analyses (see Figure 11) reasonably well agreed with the measured wall movements and ground deformations (e.g. tension cracks, settlement and depression).

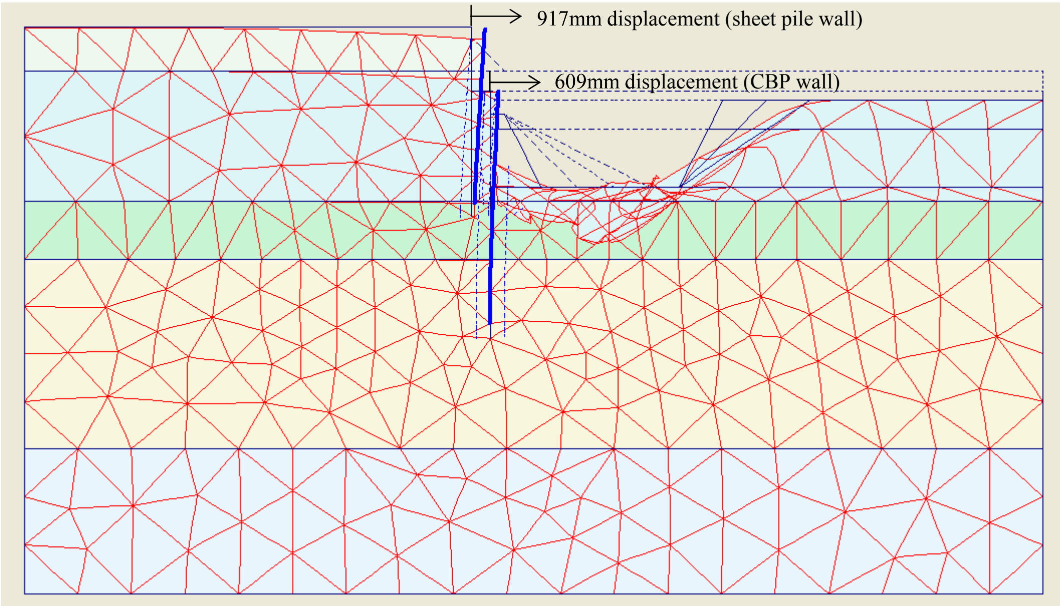


Figure 11: Results of finite element analysis

2.6 Remedial design

The immediate remedial measure was to temporarily backfill the excavation adjoining the distressed area to the top of CBP wall with temporary stabilising berm (1V:1H). A variety of permanent remedial options have been explored. Finally, internal strutting against permanent basement structures was adopted to provide a safe and cost effective solution.

The remedial works carried out included installation of additional row of 18m long sheet piles behind the deviated CBP area to stabilise the deteriorating retained ground. Temporary strutted cofferdam was constructed to facilitate the localized lift pit excavation. Two layers of temporary horizontal strut were used to prop the sheet pile wall against the partially completed permanent basement structures as shown in Figure 12. Excavation was only allowed to be carried out in stages after the struts were put in place. Once the final excavation level (B2) was reached, the deviated CBP wall was cut off at that level and verified with integrity testing using (both low and high strain dynamic pile tests) to confirm the structural integrity. Fortunately, most of the damages of the deviated CBP wall were well above the B2 level. Cast in-situ reinforced concrete wall was constructed with the exposed starter bars from the intact CBP wall. The finished permanent basement structure is shown in Figure 13.

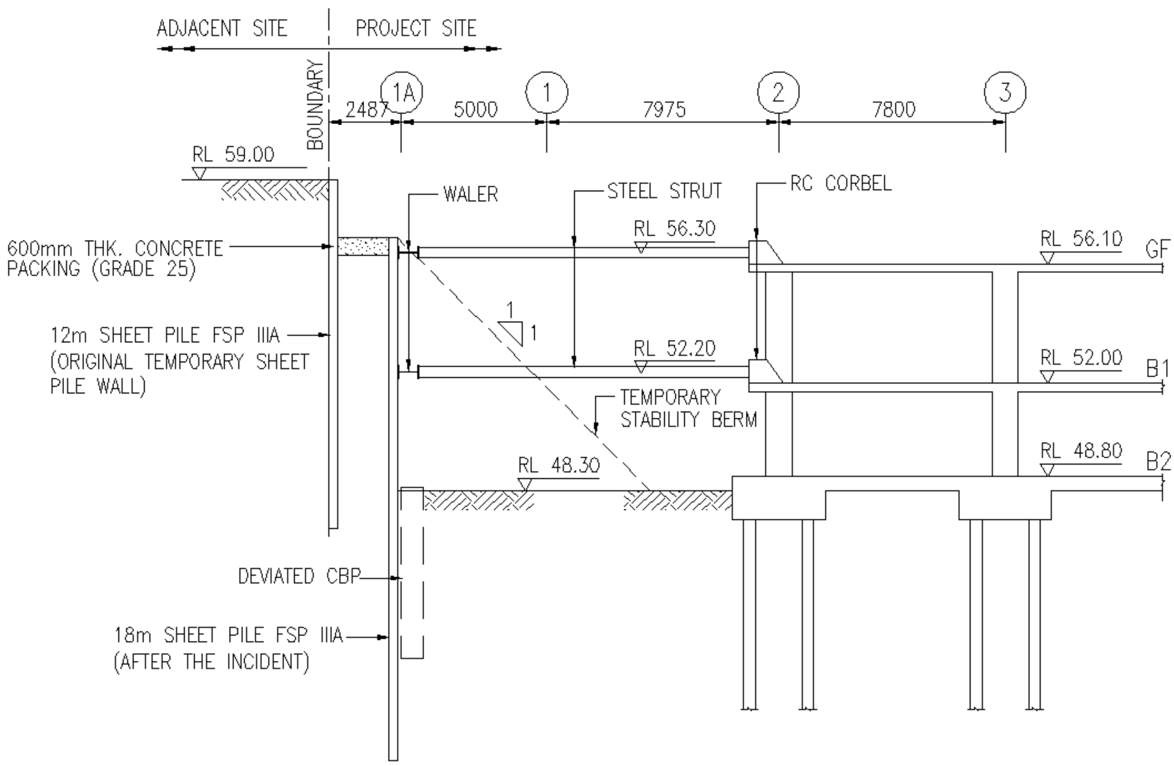


Figure 12: Cross-section of remedial works

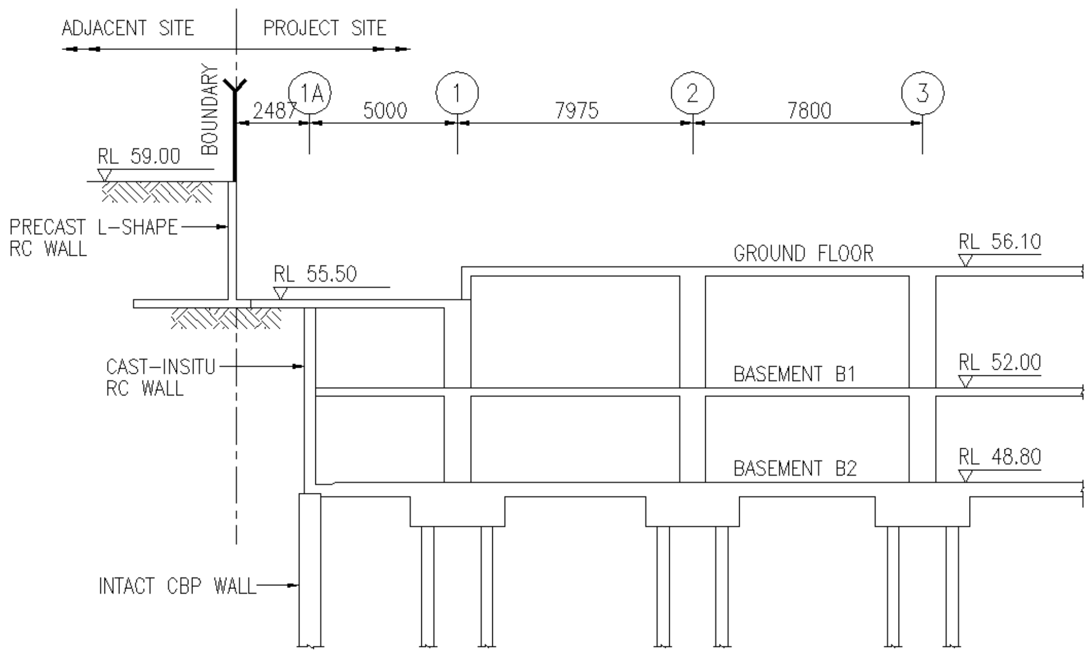


Figure 13: Cross-section of permanent basement structure

2.7 Summary of findings and lessons learnt

The investigation results and lessons learnt from this case study revealed the following findings:-

a. The distressed ground was located over a natural valley where thicker fill was placed over the previous soft deposits without proper engineering treatment to form building platform. Soft deposits at the lower part of the valley and potential concentrated underground seepage are common in hilly terrain and should not be overlooked. Desk study of pre-development ground contours to identify potential geotechnical problems is highly recommended.
b. Filling over valleys without proper site clearing, removal of unsuitable soft deposits and compaction could result in highly unstable backfill for any open excavation. It is important to thoroughly investigate the subsoil condition beneath the fill. Otherwise, proper treatment to the low-lying ground before the development earthworks shall not always be assumed.
c. Occurrence of tension cracks during initial open excavation and installation of sheet piles suggested that the underlying subsoil and at the valley area are inherently vulnerable to ground disturbance and hence are prompted to distressing.
d. The existence of soft compressible material at the valley area was further confirmed during additional subsurface investigation and localized deep pile cap excavation when reaching the final excavation level.
e. Original topographical features are the important design consideration for excavation stability and remedial strategy. In these case studies, natural valley with soft deposits was not detected during design stage causing cost escalation as a result of costly remedial work.
f. Perched groundwater regime can occur in backfilling over natural valley leading to unfavourable behaviour of backfill.
g. Excessive removal of passive berm with localized pile cap excavation without installing the planned strutting resulted in loss of wall support and led to excessive ground movement.

3 CASE HISTORY 2

3.1 Project background

This case history involved construction of two-level basement in city area using CBP wall with temporary strutting. About 40m stretch of CBP wall collapsed, after the water pipe burst incident at the back lane reported 3 hours ago as shown in Figure 14.

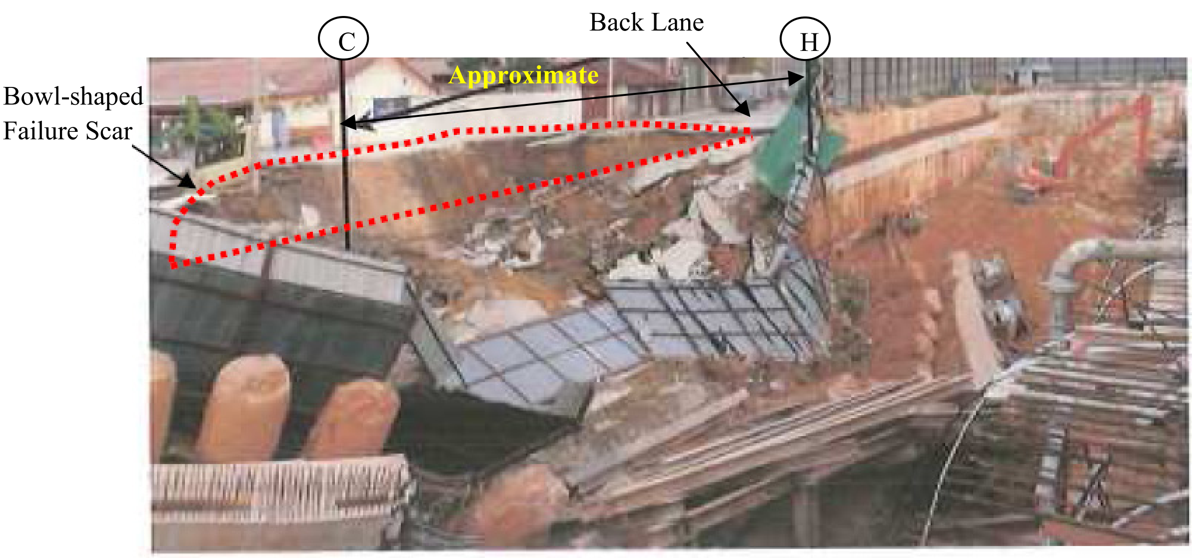


Figure 14: Collapsed CBP between Gridlines C and H

Figure 15 shows the overall retaining wall section for the above-mentioned development. The affected walls in this investigation were 1000mm diameter CBP wall installed at 1075mm centre-to-centre (c/c) and unreinforced CBP wall of 1200mm diameter installed at 1500mm c/c at a distance of 2.4m behind this 1000mm diameter perimeter CBP wall. Another row of CBP wall was installed for localised lift core pit excavation.

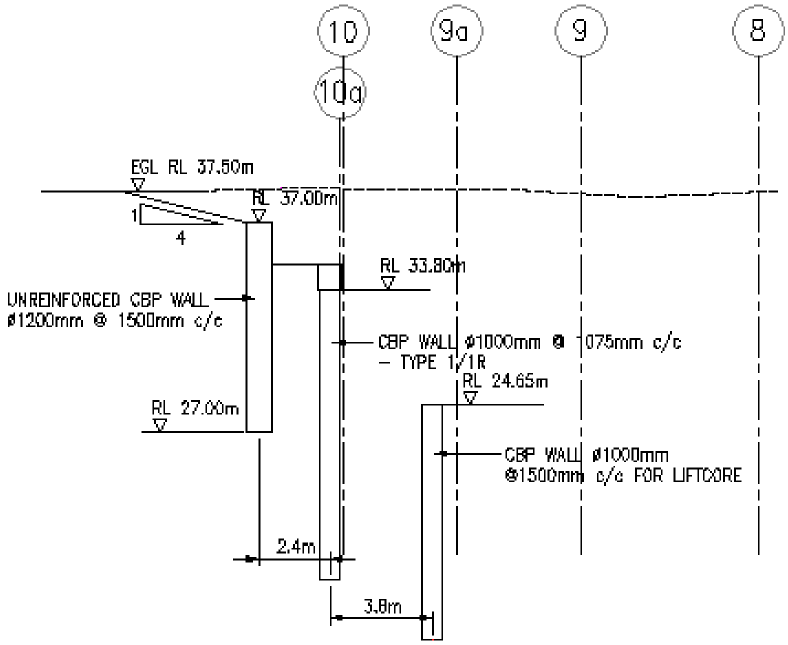


Figure 15: Section of CBP Walls

3.2 Review of information and technical clarification meeting

A technical clarification meeting was held between the contractor and consultant (Professional Engineer engaged by the contractor for temporary works) with the presence of Loss Adjuster to discuss, clarify and confirm on the supplied information, technical aspects of the design and discrepancies identified in the information provided.

During the site visit, it was observed that central portion (Gridlines C to H) of CBP wall had collapsed; single inclined struts at Gridlines G and H were observed buckled and disconnected from steel corbels while steel corbels at Gridlines D, E and F mounted at Basement 1 slab were completely sheared off. The unreinforced CBP wall and perimeter CBP wall fell rotationally and leaning towards the excavation face as shown in Figure 16.

Based on the photographs taken before the wall failure and information provided by the contractor, the as-built strutting layout was re-constructed and presented in Figure 17. Figure 17 shows that horizontal tie beams were installed to provide lateral restraint for single inclined strut but no bracing system was provided to properly transfer and balance the restraint force. The lateral restraint system provided by horizontal tie-beam was incomplete in accordance with BS5950-1:2000 where Clause 4.7.1.2 in the BS5950-1:2000 states that a restraint should be connected to an appropriate triangulated bracing system for the overall internal stability of the strutting system.

Figure 18 shows post-installed bolts were adopted for steel corbels at Gridlines D, E and F and cast-in bolts were used for steel corbels at Gridlines G and H. The steel corbel was lifted-up with snapped post-installed bolting connection after wall collapse indicating that the steel corbel connections for struts at Gridlines D, E and F are the weakest link of the entire strutting system.

Other than that, Figure 19 also shows that the cut holes at steel corbel for Strut F were not complied with the allowable dimension permitted by Table 33 of BS 5950-1:2000. This again would lead to uneven distribution of strut loads onto bolting connection possibly resulting into progressive failure of bolts which may further reduce the overall capacity of the strut.

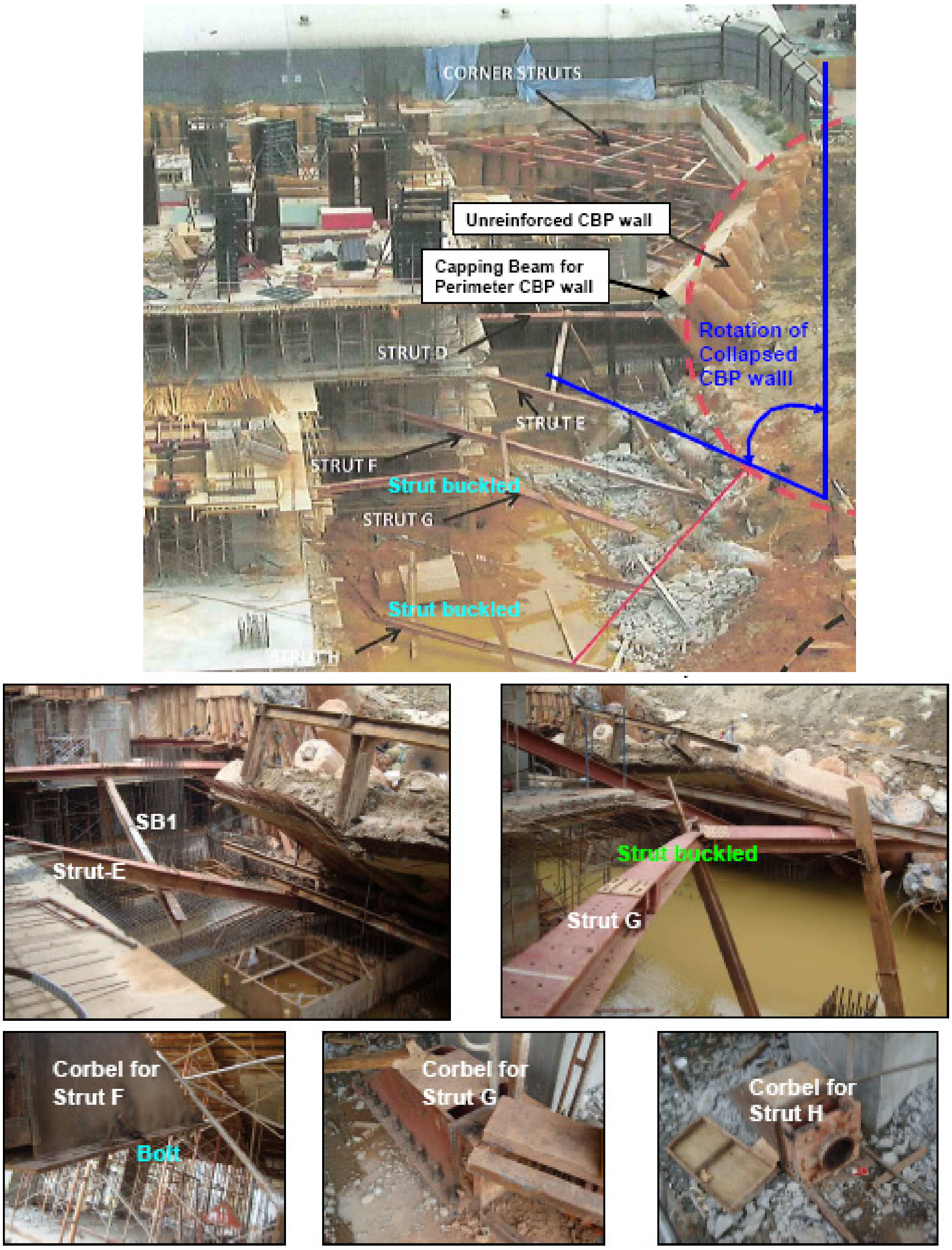


Figure 16: Site condition and strutting details after CBP wall failure

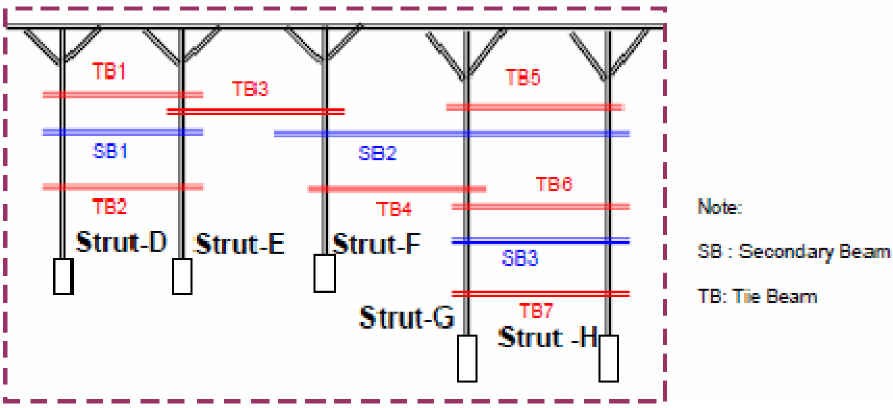


Figure 17: As-built strutting layout

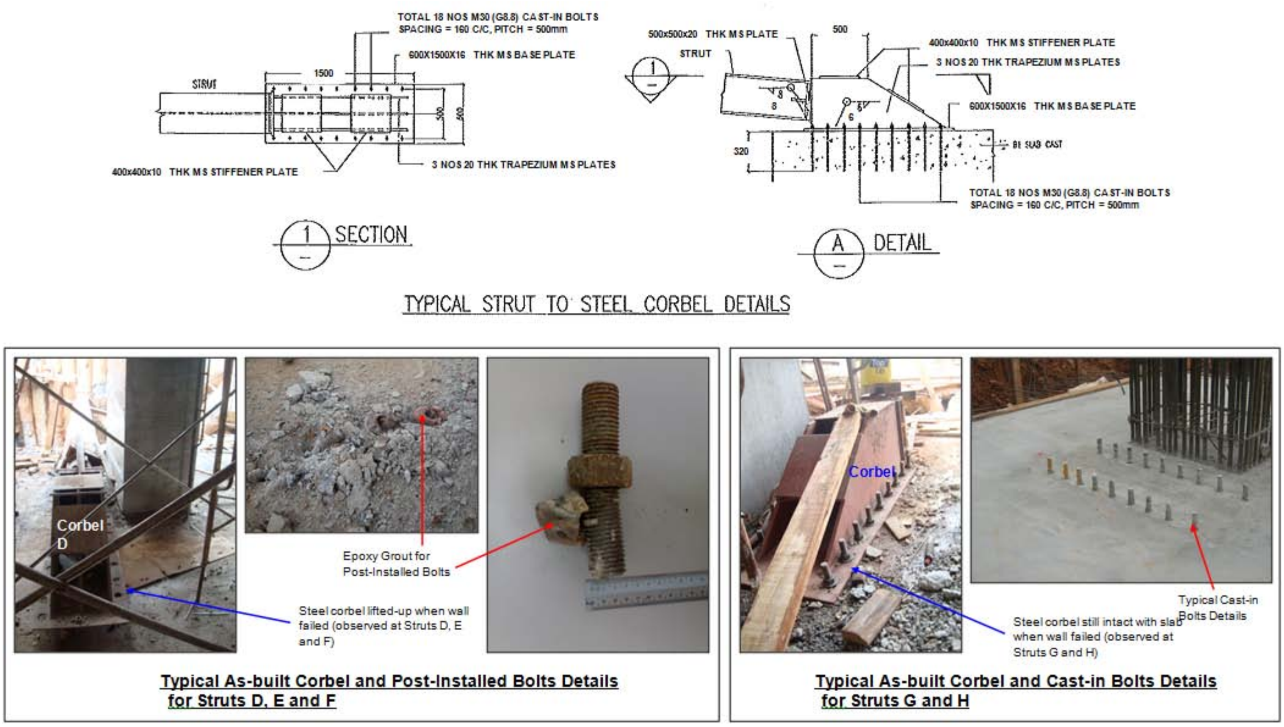


Figure 18: As-built corbel and anchored bolts details

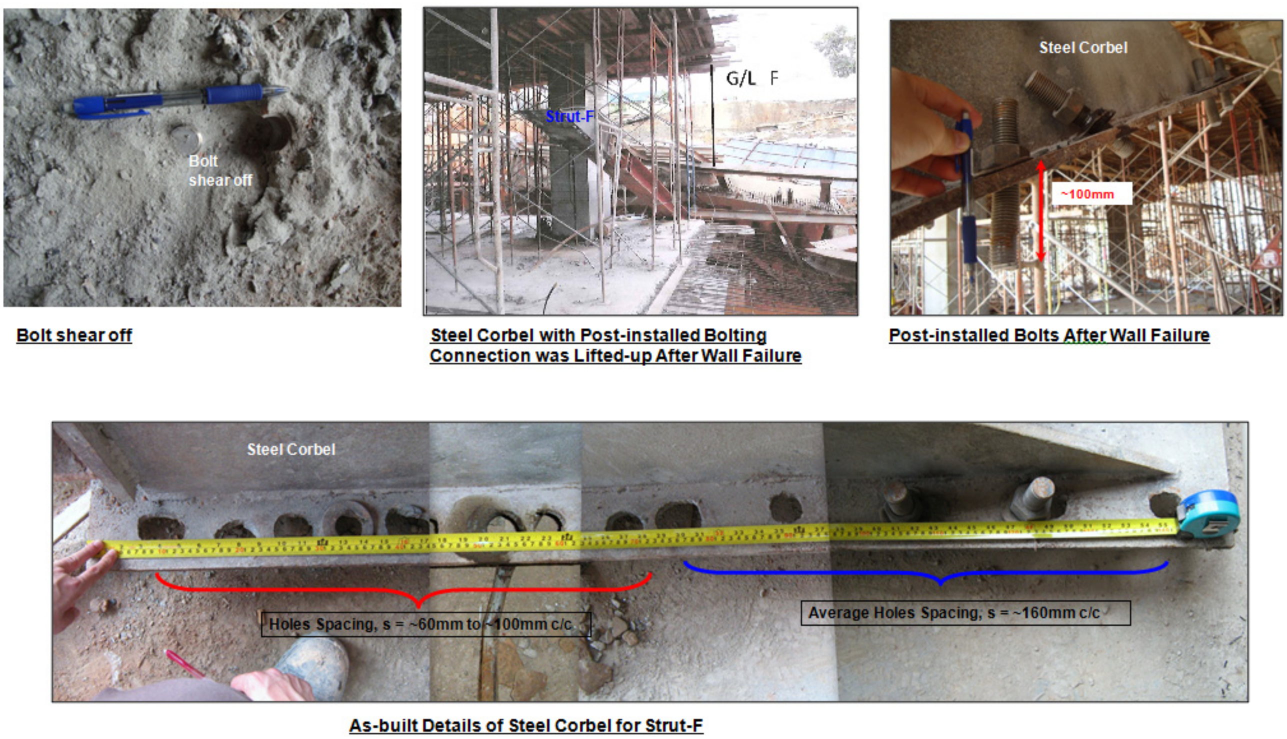


Figure 19: Steel corbel with post-installed bolting connection after wall failure

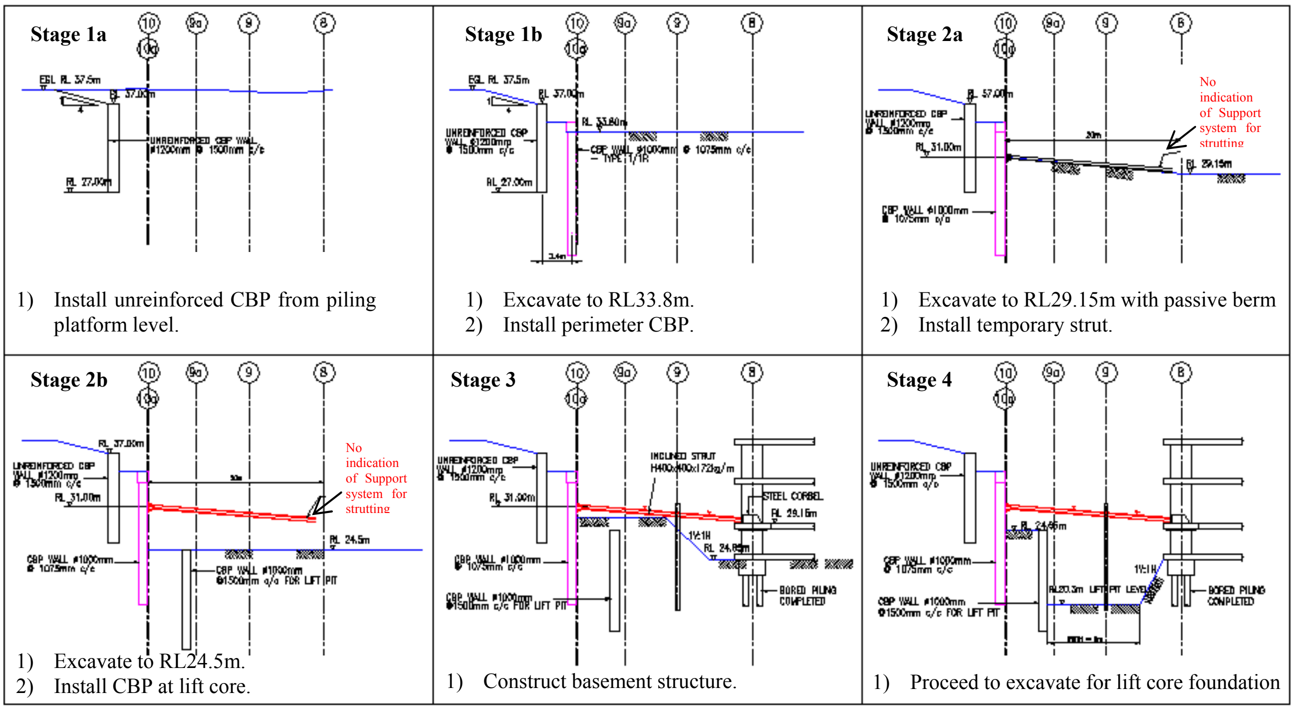


Figure 20: Modelled sequences of works in FE analysis by the consultant (same as FE analyses Cases O and P1 as discussed in)

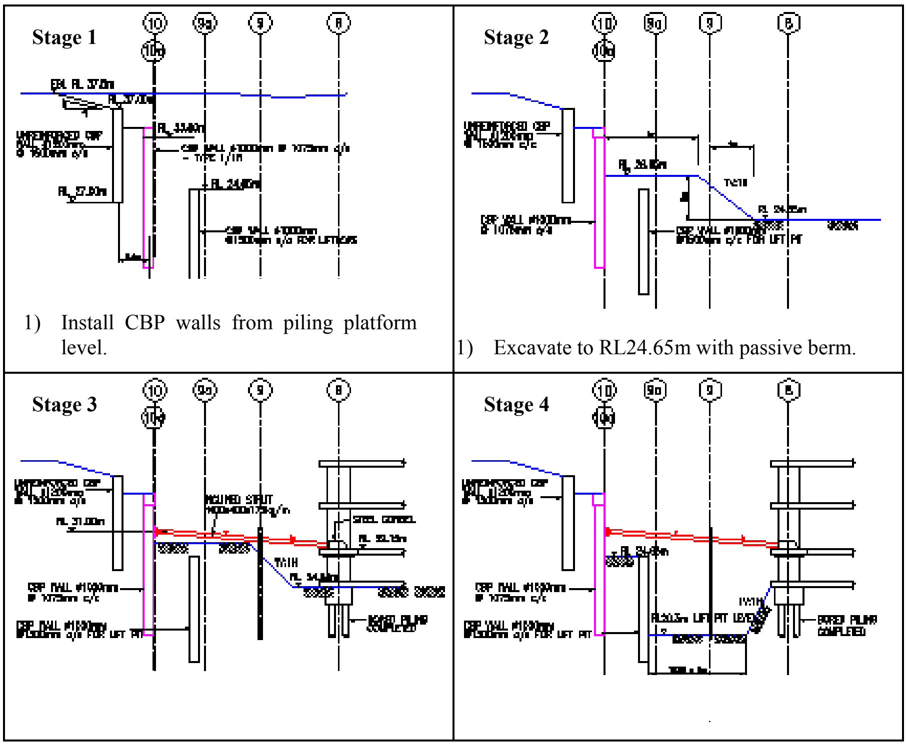


Figure 21: Sequences of works executed by the contractor at site (Same as FE analyses Cases P2 and P3 as discussed in)

3.3 Subsurface investigation

Locations and subsoil profiles for the relevant boreholes located near the collapsed CBP wall are shown in Figure 22 and 23.

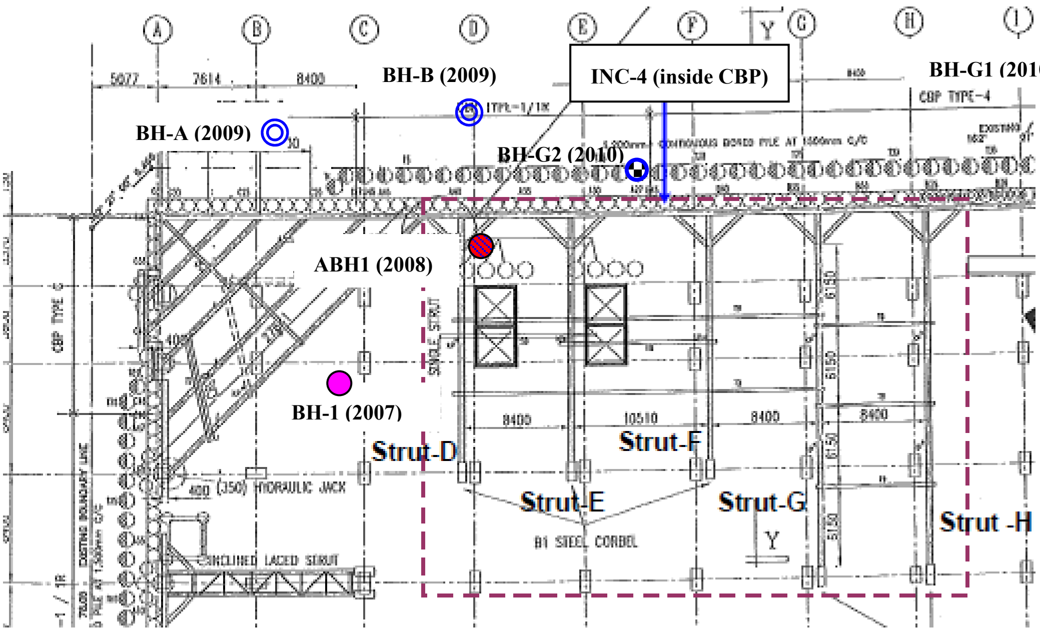


Figure 22: Borehole locations

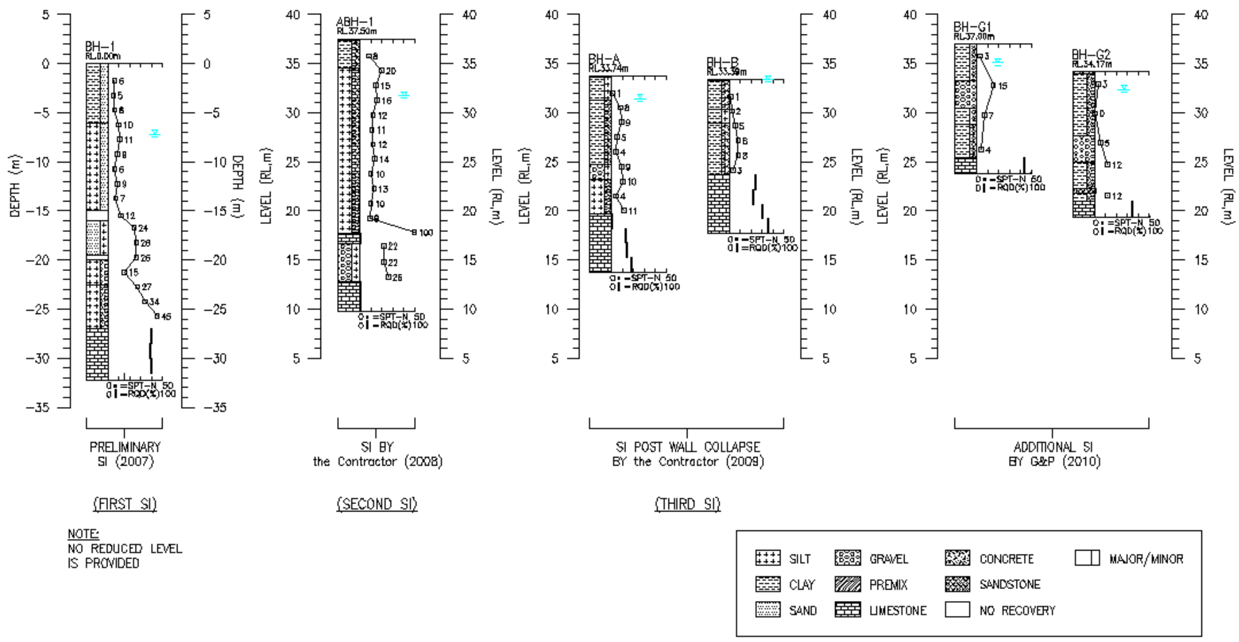


Figure 23: Subsoil profiles

No laboratory strength test was specified for determination of effective stress parameters in first, second and third SI works. As such, laboratory testing was performed in fourth SI work to determine the effective stress parameters of subsoil and the interpreted results of the relevant engineering parameters were summarised in .

Table 1: Summary of laboratory test results for shear box and C.I.U. test

Borehole (Soil Sample)	Depth of sample	Soil Type (Based on BS 5950)	Shear Box Test		Consolidated Undrained Triaxial Test (C.I.U.)	
			c' (kPa)	ϕ' (°)	c' (kPa)	ϕ' (°)
BH-G1 (UD1)	2.50m b.g.l	Silt	4	35	Not enough soil sample for C.I.U. test	
BH-G1 (UD2)	9.00m b.g.l	Silt	4	25	1	32
BH-G3 (UD1)	1.50m b.g.l	Sandy Clay	4	28	Not enough soil sample for C.I.U. test	
BH-G3 (UD3)	4.50m b.g.l	Silt	3	31	Not enough soil sample for C.I.U. test	

Figure 24 shows upper and lower bound envelopes of shear strength parameters from drained shear box were c' =4kPa and ϕ' = 35° for former while c' =4kPa and ϕ' = 27° for the latter. For this investigation, moderate strength parameters of c' = 4kPa and ϕ' = 30° were selected for the overburden soil layer. Same effective friction angle of 30° was adopted in the consultant's design except for higher apparent cohesion c' value of 10kPa was adopted.

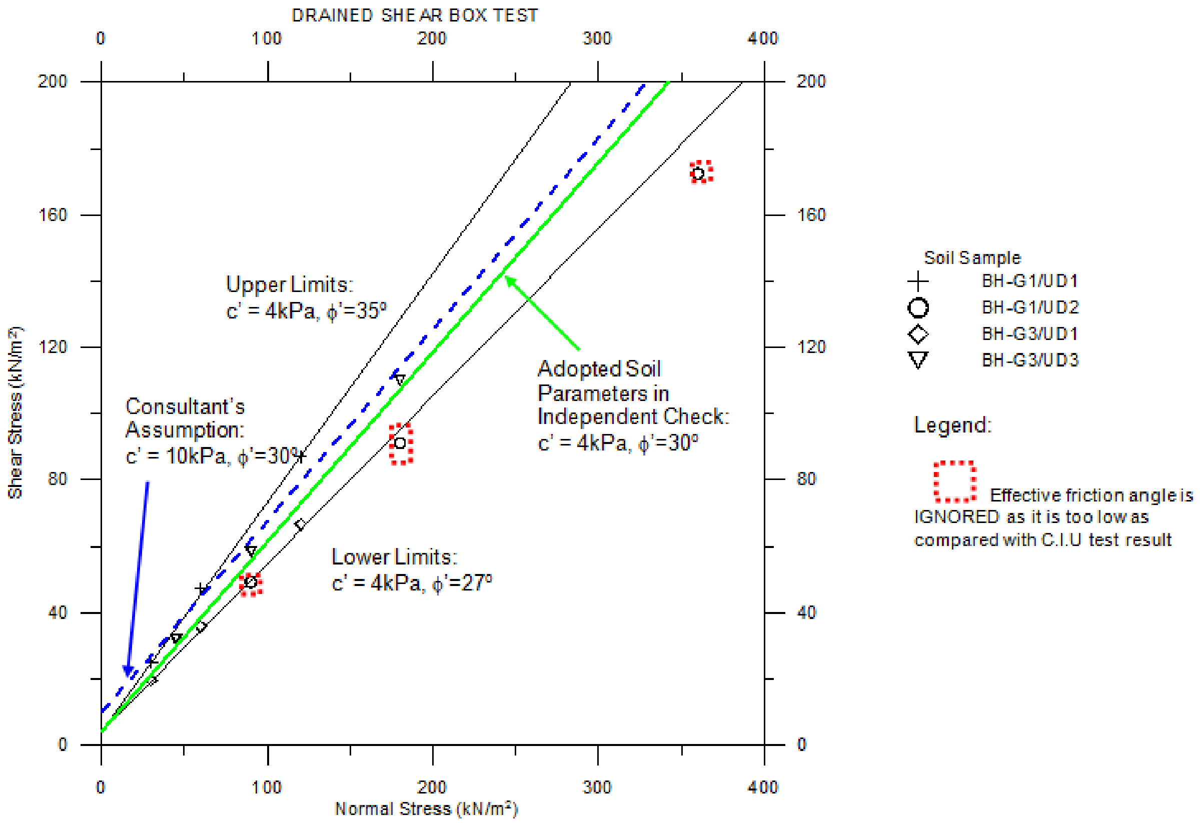


Figure 24: Interpreted shear strength parameters from drained shear box test

It is evidenced that the adopted high apparent cohesion is not justified from the interpreted strength test results above. The apparent cohesion has significant impact to the assessment of lateral earth pressure. A small increase in the apparent cohesion will reduce significantly the lateral earth pressure. Hence, it would be necessary to be prudent in assessing the cohesion parameter.

From the available water standpipe readings, the groundwater table fluctuated between RL 33.16m and RL 33.67m. For design check and back analysis purposes, initial groundwater is adopted at RL33m which is same as consultant's design assumption for normal design groundwater table and within the reasonable range of the measured data.

3.4 Finite element (FE) results

For fair assessment on the consultant's design, the adopted parameters such as shear strengths (c', ϕ'), deformation modulus (E), permeability (k) and initial groundwater table in the first checking (Case O) were similar to the consultant's analysis. In order to study the effect of soil properties (shear strength and deformation modulus) and construction sequences to the performance of retaining wall system, the FE analysis cases as shown in are performed.

Table 2: FE analysis cases

Case		*Strength Parameters	**Groundwater Level	Remarks
Modelled sequences of works in FE analysis by the Consultant (Figure 20)	O	c' = 10kPa, ϕ'= 30° Note: Consultant's adopted strength parameters	RL 33m	Checking on base design where **passive berm** of 4m wide and 5m high was **not modelled**
	P1	c' = 4kPa, ϕ'= 30° Note: Moderately strength parameters interpreted by G&P	RL 33m	Parametric study 1 adopting moderately strength parameters where **passive berm** of 4m wide and 5m high was **not modelled**
Actual sequences of works executed by the Contractor at site (Figure 21)	P2	c' = 10kPa, ϕ'= 30° Note: Consultant's adopted strength parameters	RL 33m	Parametric study 2 adopting Consultant's strength parameters where **passive berm** of 4m wide and 5m high **was modelled** according to actual construction sequences implemented by the Contractor at site
	P3	c' = 4kPa, ϕ'= 30° Note: Moderately strength parameters interpreted by G&P	RL 33m	Parametric study 3 adopting moderately strength parameters where **passive berm** of 4m wide and 5m high **was modelled** according to actual construction sequences implemented by the Contractor at site

* For overburden soil only. Properties for rock layer remained unchanged.
** The initial groundwater table behind the retaining wall is assumed to be constant at RL33m. After every stage of excavation, the ground water level within the excavation site is re-defined to the surface of the new formation level to model the drawdown of water table within the excavated area. Hence the steady stage seepage on the active and passive sides of the retaining wall at each construction sequence is considered.

Summary of design checks based on the normal design water level at RL33m for all the analysis cases is tabulated in and reveals the following findings:

a. Design of unreinforced CBP wall for all cases of the section is adequate for moment resistance. However, the moment capacity of unreinforced CBP wall will drop drastically if any cracking was formed when any of the structural thresholds are exceeded and hence the back calculated moment of unreinforced CBP wall can be unreliable.

b. Design of perimeter CBP wall for all cases is adequate except Case P3 in which the design is inadequate for shear resistance.

c. Design of lift core CBP wall for all cases is adequate except Case P3 in which the design is inadequate when the actual construction sequences at site was used, which is significantly deviated from the assumed construction sequence in FE model and the apparent cohesion for overburden soil is reduced from 10kPa to 4kPa as justified from the interpreted strength parameters.

d. Design of strutting members is adequate based on extreme strut force predicted by the consultant. However, design of strutting members for Cases O, P1, P2 and P3 is either inadequate or unsafe based on extreme strut forces predicted in the independent analyses by the investigator.

e. There are an increase of approximately 24% in strut force and seven times larger in the maximum predicted wall deflection for perimeter CBP when the apparent cohesion for overburden soil is reduced from 10kPa to 4kPa and actual construction sequences are deviated from site execution. These findings reveal the apparent cohesion is a very sensitive factor on the performance of CBP wall.

f. Adoption of high apparent cohesion in FE analysis by the consultant had caused the underestimation of strut force and virtually led to insufficient provision of strutting members (as per findings from Cases P1 and P3).

g. Large wall movement predicted in Case P3 reveals the adoption of moderately strength parameters in retaining wall design analysis would be able to identify the potential distress of existing utilities due to large ground movements which had happened in this project.

Table 3: FE analysis results

Wall Type	Max. ultimate Capacity	Max. Results from FEM (Note: normal groundwater level was RL33)	Consultant's Analysis c' = 10kPa, ϕ'= 30° (Without passive berm)		Case O (Independent Analysis)- c' = 10kPa, ϕ'= 30° (Without passive berm)		Case P1 (Independent Analysis)- c' = 4kPa, ϕ'= 30° (Without passive berm)		Case P2 (Independent Analysis)- c' = 10kPa, ϕ'= 30° (With passive berm)		Case P3 (Independent Analysis)- c' = 4kPa, ϕ'= 30° (With passive berm)	
			Extreme Value from FEM	Remark	Extreme Value from FEM	Remark	Extreme Value from FEM	Remark	Extreme Value from FEM	Remark	Extreme Value from FEM	Remark
Unreinforced CBP wall (1200mm size @ 1.5m c/c)	-	Deflection	40.8mm	-	36mm	-	53mm	-	77mm	-	814mm	-
	256kNm	Service Bending Moment per pile.	132kNm	Adequate	93kNm	Adequate	50kNm	Adequate	61kNm	Adequate	80kNm	Adequate
Perimeter CBP wall (1000mm size @ 1.075m c/c)	-	Deflection	30.4mm	-	36mm	-	41mm	-	81mm	-	619mm	-
	1700kNm	Service Bending Moment per pile	640kNm	Adequate	856kNm	Adequate	851kNm	Adequate	626kNm	Adequate	1122kNm	Adequate
	1233kN	Service Shear Force per pile	700kN	Adequate	391kN	Adequate	428kN	Adequate	462kN	Adequate	1069kN	Inadequate
Liftcore CBP wall (1000mm size @ 1.5m c/c)	-	Deflection	2.6mm	-	9mm	-	9mm	-	12mm	-	17mm	-
	1260kNm (Note: 16T25)	Service Bending Moment per pile	135kNm	Adequate	247kNm	Adequate	217kNm	Adequate	626kNm	Adequate	1257kNm	Inadequate
Strut Force (For strut spacing of 8.4m)	3375kN	Maximum induced force per strut	1856kN	Adequate	2890kN	Inadequate	3452kN	Unsafe	2654kN	Inadequate	3301kN	Inadequate

Notes:

The extreme induced force is compared with ultimate capacity and service (safe working load) capacity on checking the adequacy of design. In brief, the adequacy of design is classified as follows:

(1) **Adequate** Design: Extreme induced force ≤ Service capacity;

(2) **Inadequate** Design: Service capacity < Extreme induced force ≤ Ultimate capacity and;

(3) **Unsafe** Design: Extreme induced force > Ultimate capacity;

Where service capacity = Ultimate capacity / 1.4

3.5 Back analysis

In the second part of this investigation, the construction sequences were modelled based on furnished information which was clarified and confirmed by the consultant and contractor during several rounds of clarification meeting. Back analysis had been carried out based on the instrumentation data for Inclinometer INC-4 installed inside the collapsed CBP wall. Attempt was made to calibrate the predicted wall deflection with instrumentation data of Inclinometer INC-4 to assess the behavior of the CBP walls and investigate the possible mechanisms of strut failure and/or wall collapse.

From Figure 16, it was sufficiently evident that all the post-installed bolts at struts D, E and F were completely sheared off. Hence, these struts were modelled in FEM back analysis as elasto-plastic anchor element.

Repetitive water pipe burst incidents at the back lane had contributed to massive movement of CBP wall prior to the CBP wall collapse incident as highlighted in Figure 25. Therefore, water pipe burst incidents happened on Day 206 and Day 227 were taken into consideration in FEM analysis for best fitting the inclinometer readings. Figure 26 and Figure 27 show the FE mesh and predicted CBP wall deflections for back-analysis.

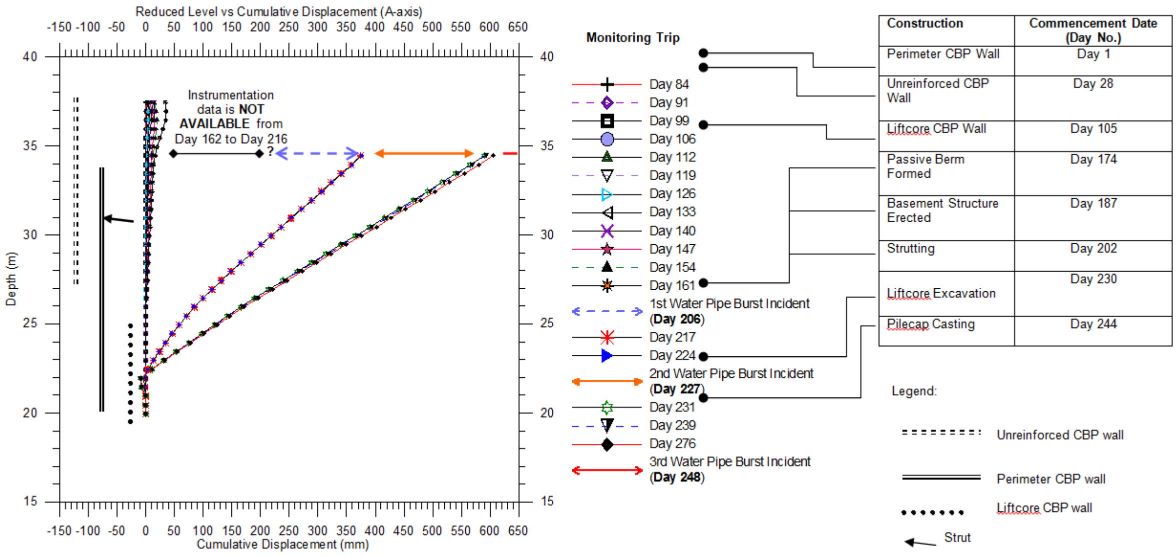


Figure 25: Monitoring results for INC-4 and summary of construction activities

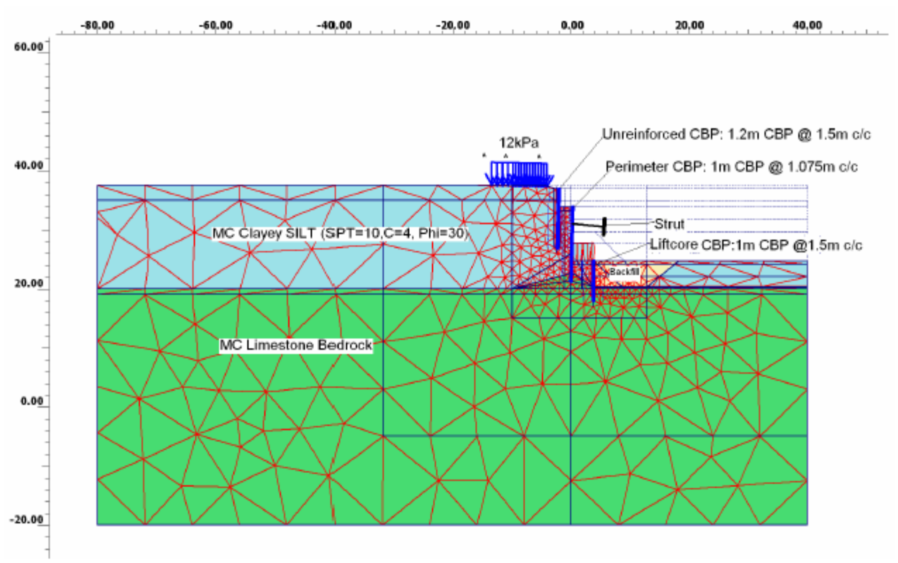


Figure 26: FE mesh of back-analysis

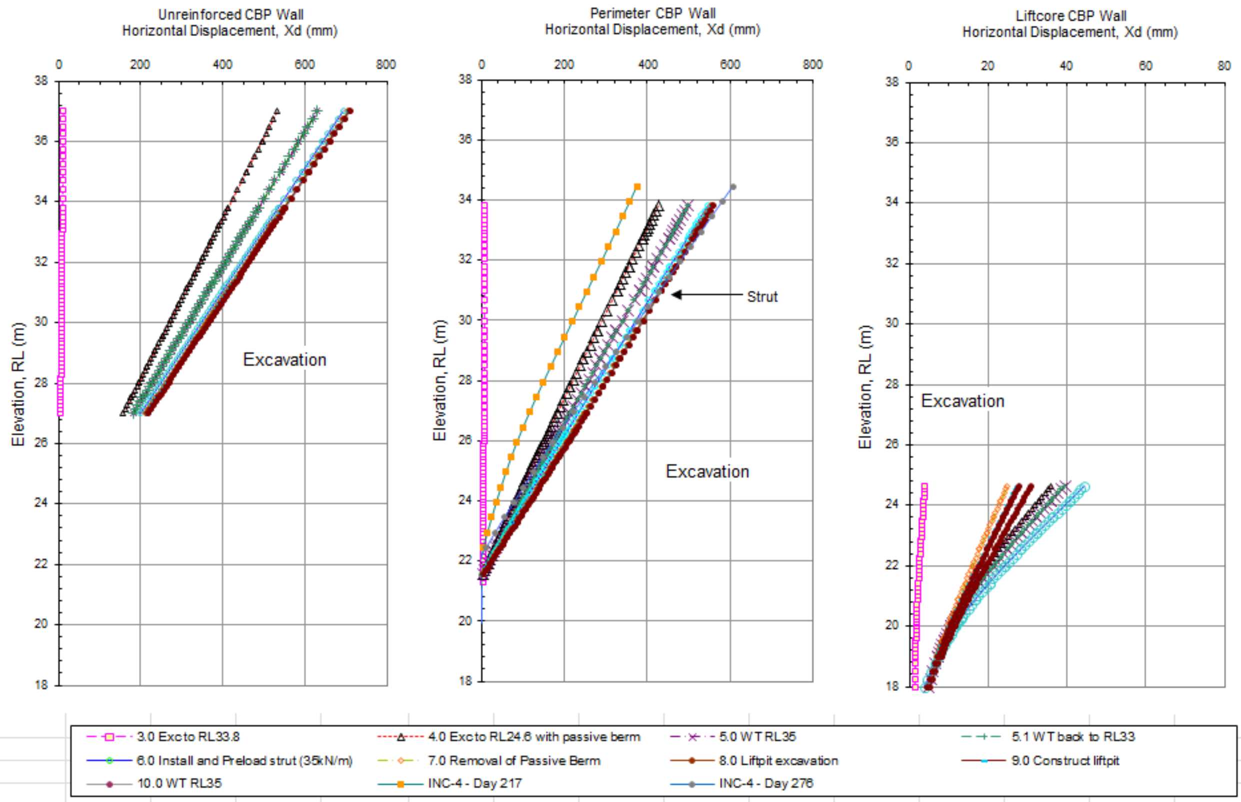


Figure 27: Modelled CBP wall deflections Vs monitoring results for INC-4

From the assessments mentioned in the preceding section, it would be possible to deduce the following sequence of events and development of wall collapse mechanism as below:

a. The original excavation design with optimistic strength parameters has suffered excessive ground movements, which subsequently led to potential distresses of water pipe after experiencing the intolerable deformation.
b. The water leakage caused increase of lateral wall pressure and induced more lateral wall movement and more leakage, hence the same for the deformation of the retained ground.
c. The strut forces also increased as a result of the increase in lateral wall pressure.
d. The steel corbel connections at Struts D, E and F, especially the large hole cutting for accommodating the poorly positioned bolts, would have resulted in uneven distribution of shear force from the struts. Hence, the total shear resistance with probable progressive failure of bolts would be lower than the simple summation of the shear resistance of individual bolts (3375kN). With the increasing strut load, the bolts at the steel corbels sheared off and led to total loss of support for the CBP wall.
e. It is likely that Struts E and F had given way first and affected the Struts D (bolts at steel corbel of Struts D, E and F were sheared off) and the other two longer Struts G and H (Struts G and F buckled and dislodged from the corbels).

Figure 28 and 29 illustrate the important chronological events related to the CBP wall collapse.

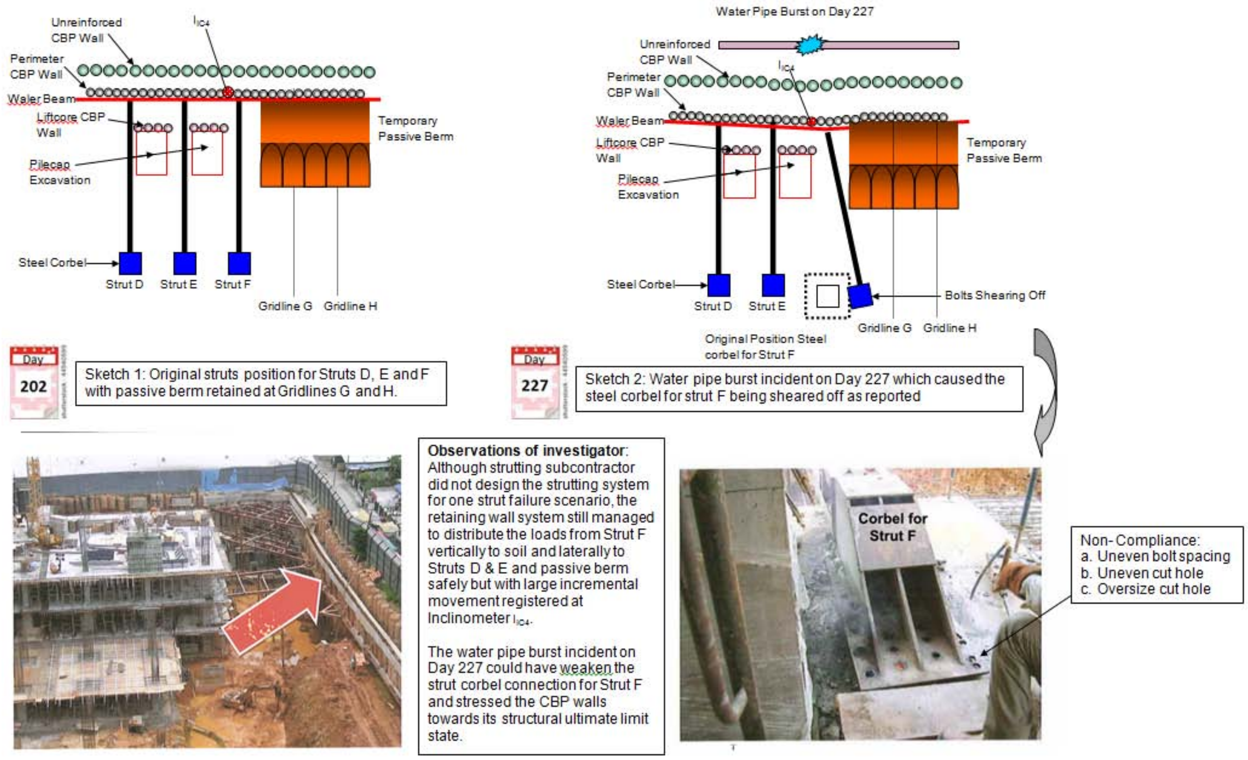
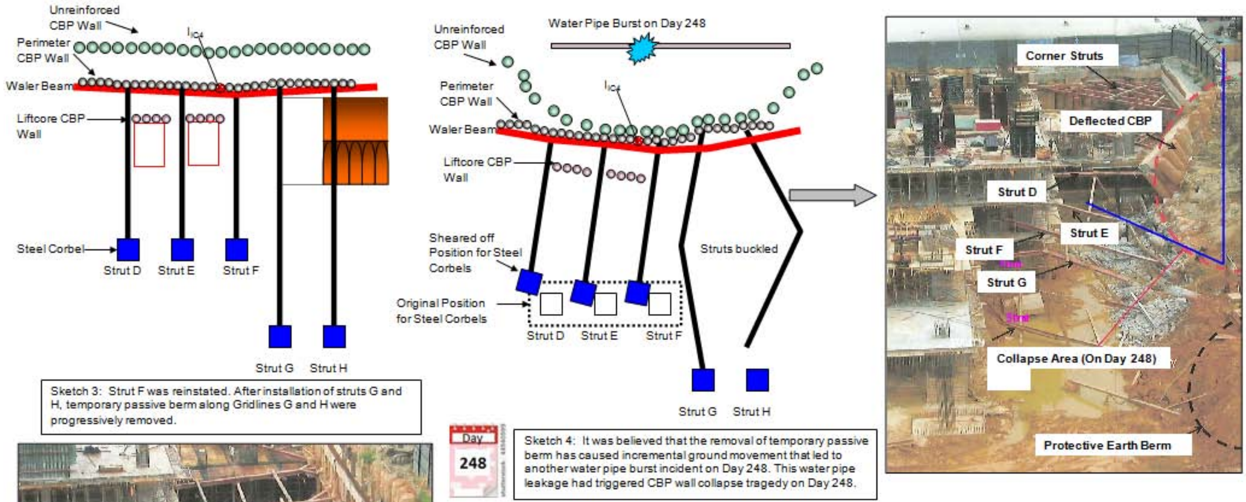


Figure 28: Flow of important events related to CBP wall collapse (Part 1 of 2)



Possible flow of wall collapse on Day 248:

1) Water pipe burst
2) Steel corbel connection at Strut F sheared-off
3) Failure of strutting system to re-distribute the failure load to adjacent struts
   a. Steel corbel connections at Struts D and E sheared off due to sudden increased in strut force
   b. Struts G and H buckled due to sudden increase in strut force
4) Failure of CBP walls due to loss of lateral supports (struts)
5) CBP wall failed rotationally and retained earth at active soil wedge in to the excavation site

Figure 29: Flow of important events related to CBP wall collapse (Part 2 of 2)

3.7 Summary of findings

The findings of the independent investigation for the CBP wall collapse are summarised as follows:

a. The apparent cohesion adopted by the consultant is on the optimistic side without verification from laboratory test which led to unexpected large wall displacement and increase in strut forces when experiencing increase of water pressure behind the wall as a result of water pipe burst incident.
b. The actual executed construction sequences at site by the contractor were not conformed to consultant's modelled construction sequences (Figure 20 and Figure 21).
c. Improper lateral restraint bracing system and connection between the steel corbels and Basement 1 slab by strutting sub-contractor were observed (Figure 17 and Figure 19). The uneven hole cutting at the steel corbels would result in differential mobilisation of the bolt shearing resistance and led to lower overall shearing resistance of the support due to progressive failure.
d. No timely review of the retaining wall and strutting designs after the early incidents of the water pipe leaking though the leaking incident had caused distresses to one of the struts (Strut F) and had shown alarming sign of excessive wall movement immediately after the leaking incident.

3.8 Lessons learned and conclusion

Based on the findings from independent assessment as listed above, the contributory factors of the wall collapse for Case History 2 are:

a. adoption of optimistic cohesion parameter by the consultant,
b. inconsistency of design intent and site execution between consultant and contractor (Figure 20 and Figure 21),
c. improper lateral restraint bracing system and non-compliance on hole cutting at steel corbel by strutting sub-contractor (Figure 17 and Figure 19), and no timely review of the retaining wall and strutting designs.

In addition to the above, the triggering factor of the wall collapse was the increase of water pressure due to repetitive water pipe burst incidents happened at the back lane.

In most of the excavation failure cases, timely review by the consultant is important to prevent the failure as the tell-tale sign revealed by the monitoring results should alert the consultant to implement the contingency plan once any of the threshold limits of the excavation system is breached.

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Soil-Structure Interaction (lateral) of Wall Support System: A Geotechnical Approach

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ABSTRACT

This paper outlines the analyses carried out in providing the understanding of the behaviour of the embedded wall support system with the soil loads. The significance of this behavioural understanding is pronounced for a strutting type of retaining wall support system as compared to the ground anchor system or the reinforced concrete slab system. The significance is also evident for embedded wall with zero or low rigidity in the lateral direction, like the sheet-pile wall and the contiguous bored pile wall (with or without interlocking joints between them). The analyses illustrated the soil-structure interaction between the soil and the embedded wall support system. Thus, it is a geotechnical approach in analysing the wall support system of an embedded wall i.e. a geotechnical solution for a structural problem.

1 INTRODUCTION

The analysis of the strut and waler is commonly carried out by the conventional structural framing approach using a structural frame. As shown in Figure 1 below, a Uniform Distributed Load (UDL) is applied on the top of the frame (i.e. the waler or waling) and supports are placed at the end of the struts.

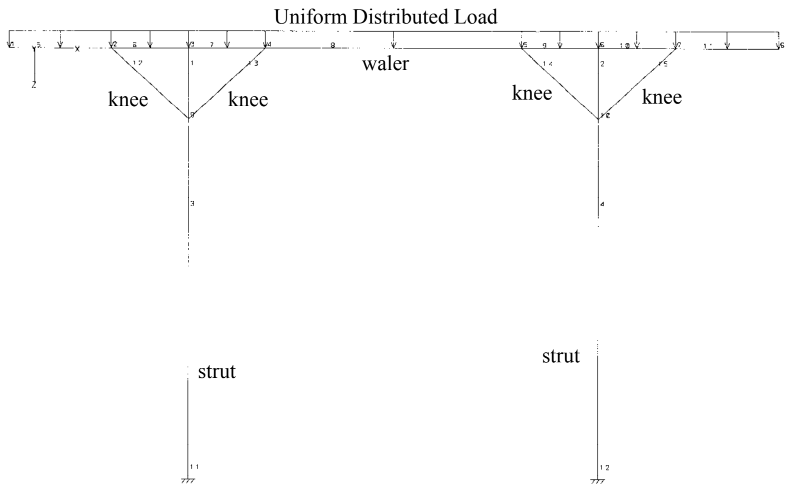

Figure 1: Typical structural frame configuration with UDL load on the waler of a strutting system

Based on the 'attributed length' method of the structural framing approach, the uniform distributed load is distributed to the struts and knee (or splay) according to its attributed or associated length. Hence, the knee or splay having longer attributed length will be subjected to a higher proportion of the UDL load. This results in the knee being designed as a major component of the strutting system. This deviates from the concept where the knee acts to spread the strut load wider along the waler, and functioned only as an enhancement to the design of the strutting system. In this concept, it is supposed to be only a minor component of the strutting system. This deviation from the supposed concept leads to the investigation into the understanding of the behaviour of the embedded wall support system.

Subsequently, structural software was used in the analysis to better estimate the forces. However, the analysed behaviour of the free-standing structural frame did not reflect the expected behaviour of the actual field condition (details of the findings are elaborated in Section 3.1 below). Hence, the investigation proceeds in the direction away from the conventional structural approach, and subsequently leads to the soil-structure interaction between the soil and structure (wall and associated support), and an understanding of the behaviour of the embedded wall support system.

2 BACKGROUND

2.1 Soil structure interaction

Soil-structure interaction refers to the interaction between the soil and the structure. It is the behaviour of both the soil and structure acting together, under loading. The loadings could originate from the soil and/or the structure, and includes secondary loads derived from subsequent interaction between the soil and structure.

For an embedded wall, the soil-structure interaction is complex as it can be influenced by the sequencing of work, the type of wall support system, the wall rigidity, and the type of soil. And, the influence of the type of soil will be characterised by its properties of lateral earth pressure as well as the stress relaxation upon removal of earth on the excavation side of the embedded wall.

In the context of this paper, the structure would be the embedded wall with the wall support system on the excavation side of the wall. The rigidity of the wall structure is anticipated to have significant influence on the soil-structure interaction. It has formed part and parcel of many softwares for the design analysis of embedded walls with props. However, the analysis is carried out in a vertical plane, and there is limited study or investigation into the influence of the wall rigidity in the lateral direction. In consequences, there is a lack of investigation into the soil-structure interaction in such cases. This lack of investigation is likely caused by the complication with the presence of laterally non-rigid walls like sheet-pile wall, contiguous bored pile wall and soldier pile wall. Moreover, it would appear to be a three-dimensional problem. Thus, this paper attempted to provide some insight into the understanding in this aspect.

2.2 Embedded walls

In the context of this paper, it is necessary to recognise or identify the laterally non-rigid type of embedded wall. Basically, this type of embedded wall has zero or low rigidity or stiffness in the lateral or horizontal direction. Such embedded walls are formed out of individual piles aligned contiguously in a row in forming a wall. The piles could be interlocked as in the case of a sheet-pile wall, and could also be non-interlocking as in the case of a contiguous bored pile wall. Hence, the stiffness or rigidity in the direction across all the individual piles is low or zero. In effect, the influence of the associated wall support system on the soil-structure interaction between the soil and this type of laterally non-rigid embedded wall is significant.

In comparison, an example of laterally rigid walls is a diaphragm wall. Although diaphragm walls are also formed from jointed panels, the lateral stiffness or rigidity is relatively much higher than those walls with zero or low rigidity. And, diaphragm walls can have lateral rigidity with proper steel reinforcements across the joints. Hence, the diaphragm wall is assumed to have high lateral rigidity. The actual difference in behaviour would require a separate investigation to be carried out in studying the discontinuous lateral rigidity or stiffness of diaphragm wall panels. Therefore, in comparison, the configuration of the associated wall support system on a laterally rigid embedded wall would have minimal or low influence on the soil-structure interaction response.

In view of the minimal or low influence on the soil-structure interaction of the laterally rigid embedded wall, the emphasis of this paper is on the laterally non-rigid embedded wall.

2.3 Wall support systems

There are typically four types of wall support systems; the strutting system, the reinforced concrete slab system, the ring beam system, and the ground anchor system.

The strutting system is a system that consists of steel members used to physically provide a continuous support or contact to the wall. The strutting configurations are made up of struts, waler and knee as shown in Figure 1. Thus, this limited rigidity of the strutting system coupled with a laterally non-rigid wall will have a significant influence on the soil-structure interaction between the strutted wall and soil.

The reinforced concrete slab system is the reinforced concrete slabs that are casted continuously along the wall, like the permanent floor slab abutting the wall. This type of wall support presents a continuous support on the retaining wall. As a result, the slab provides a fully rigid wall support system for a laterally non-rigid embedded wall. Hence, there is no variable influence on the soil-structure interaction between the wall and soil.

The ring beam system is a support system used for retaining circular cofferdam. It functions by way of resisting the hoop compressive stresses that is generated from the circular cofferdam. In effect, it also provides a continuous support on the retaining wall, and similarly to the reinforced concrete slab, there is no variable influence on the soil-structure interaction between the wall and soil.

The ground anchor system is a system whereby walers are also required to effectively provide continuous support or contact to the wall. These walers are subsequently supported by the ground anchors at regular intervals. The limited rigidity system coupled with a non-rigid (lateral) wall will also have a significant impact on the soil-structure interaction between the anchored wall and soil. However, the configuration is relatively simple and the waler could be represented with a simply supported continuous beam. And, the ground anchor would act as comparable, regular supports for the continuous beam. This representation is less complicated than the strutting system. Thus, the analysis carried out for strutting support system would encompass the ground anchor support system as well.

Therefore, the analysis of the strutting system is the key analysis required to determine the soil-structure interaction response for the wall support systems. And, this wall support system is the emphasis of this paper.

3 ANALYSES

3.1 Structural framing system

The first investigative work was carried out more than a decade ago, with the structural frame using a structural software STAAD III as shown in Figure 1. A uniform distributed load of 100kN/m was applied and the results of the analysis were unsatisfactory. The maximum deflection occurred at the cantilevered end of the waler at about three times higher than the deflection at the mid-span. Furthermore, the short portion of the strut that was perpendicularly in contact with the waler (in-between the two associated knees) was in tension, which was a structural response associated to the cantilevered end of the waler. As it is not possible to get such behaviour in actual condition, for this reason, the structural framing system did not simulate well with the actual condition. Revisions were carried out like the removal of loadings at the cantilevered end. However, this did not work either as other problems cropped up such as having bending moments in the struts. It was concluded that the structural frame system was effectively a stress-control or a force-control mechanism. The stress or force was applied and the corresponding deflection is obtained. A strain-control or a displacement-control mechanism would have been more appropriate.

An attempt was carried out to simulate a strain-control condition by reversing the loadings with support and viceversa, as shown in Figure 2 below. The 100kN/m UDL load was converted to 2 point loads acting axially at the ends of the two struts. And, the supports were linear springs aligned at regular intervals on the waler side. This arrangement was similar to the loading condition for a strip footing. This result did resemble the actual condition better, albeit on the rough side. And, the supports were independent linear springs which could not exhibit the ‘dishing’ effect of soil when point loads are distributed via the waler to the supports.

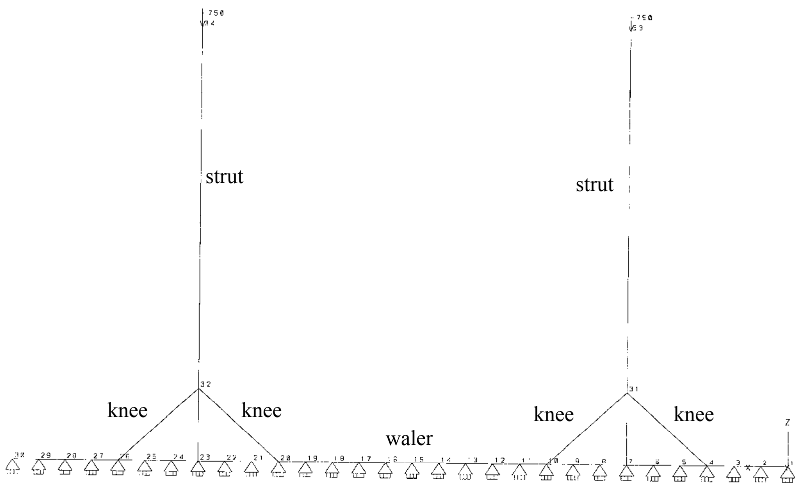

Figure 2: Reverse loading arrangement to simulate strain-control condition

3.2 Two dimensional (2D) analysis - finite element method

A two dimensional finite element analysis was conducted to provide a better insight into the soil structure interaction response. PLAXIS 2D software was used for this analysis. A model of a sheet-pile wall with strutting system as shown in Figure 3 was analysed and the structural response of the strut and waler were observed. The deflected shape is also shown in Figure 3 while the bending moment diagram is shown in Figure 4.

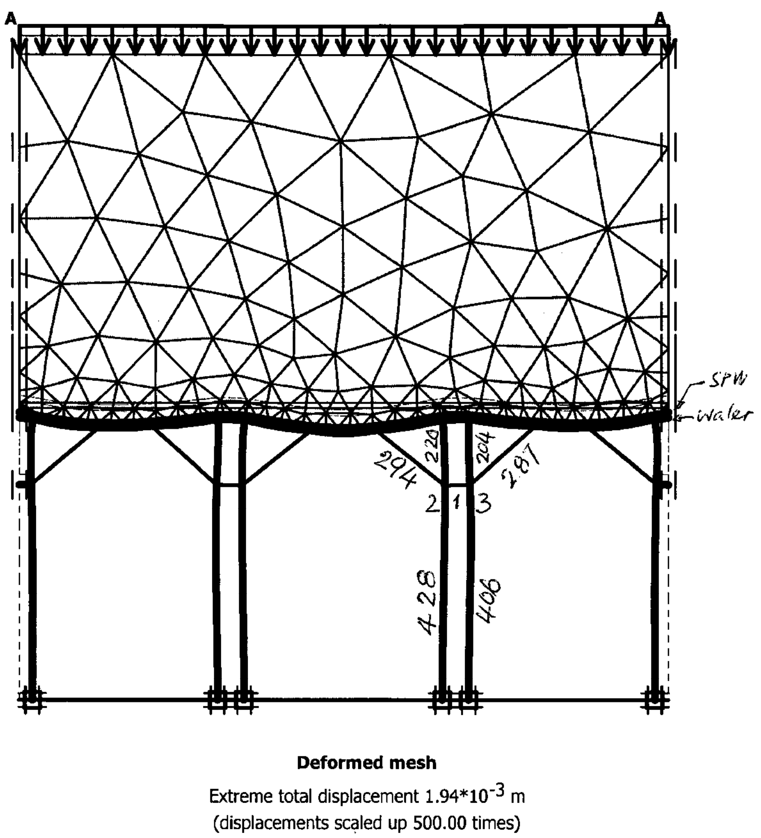

Figure 3: Model of a strutted sheet-pile wall with soil

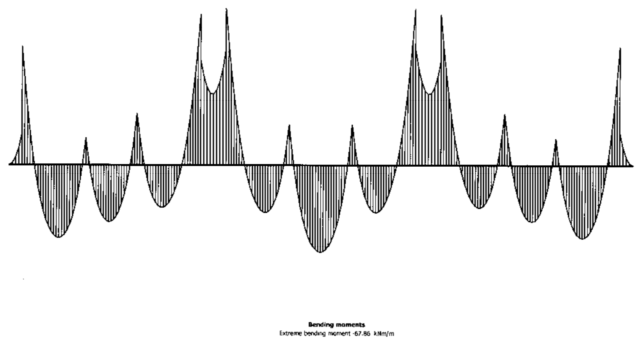

Figure 4: Bending moment diagram for the waler of the strutted sheet-pile wall model shown in Figure 1

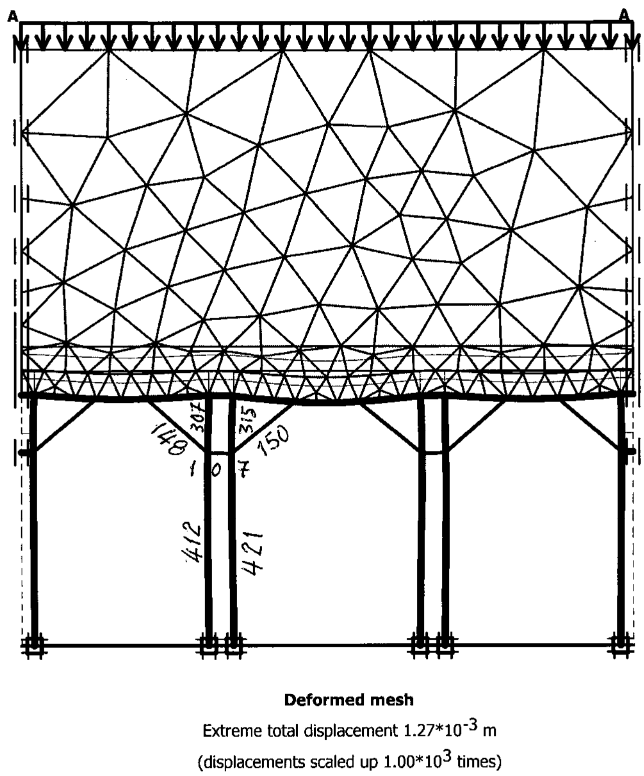

Figure 5: Model of a strutted diaphragm wall with soil

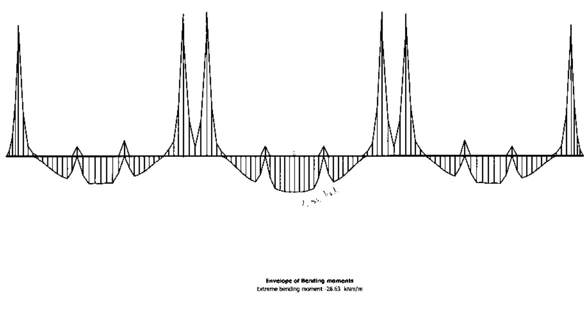

Figure 6: Bending moment diagram for the waler of the strutted diaphragm wall model shown in Figure 5

For the strutted sheet-pile wall, the axial forces in the knee was analysed to be higher than the corresponding tip of the strut. This is a result of the flexible or non-rigid sheet-pile wall, and the waler offered limited stiffness or rigidity. For comparison, the analysis for a diaphragm wall was presented in Figures 5 and 6 above. It can be seen that the knee axial force is only about half of the corresponding strut force. Thus, this shows that the stiffness or rigidity of the wall in the lateral direction plays an important part in the distribution of load forces.

3.3 Two dimensional (2D) analysis – finite difference method

Recently, the 2-D Plaxis model in Section 3.2 was recreated to provide more details on the soil-structure interaction response. A model is generated using a 3-D software FLAC3D as shown in Figure 7 below. Similarly, the same UDL loading of 100kN/m is applied on one end of the soil, and the strutting system acts as the reaction on the other end.

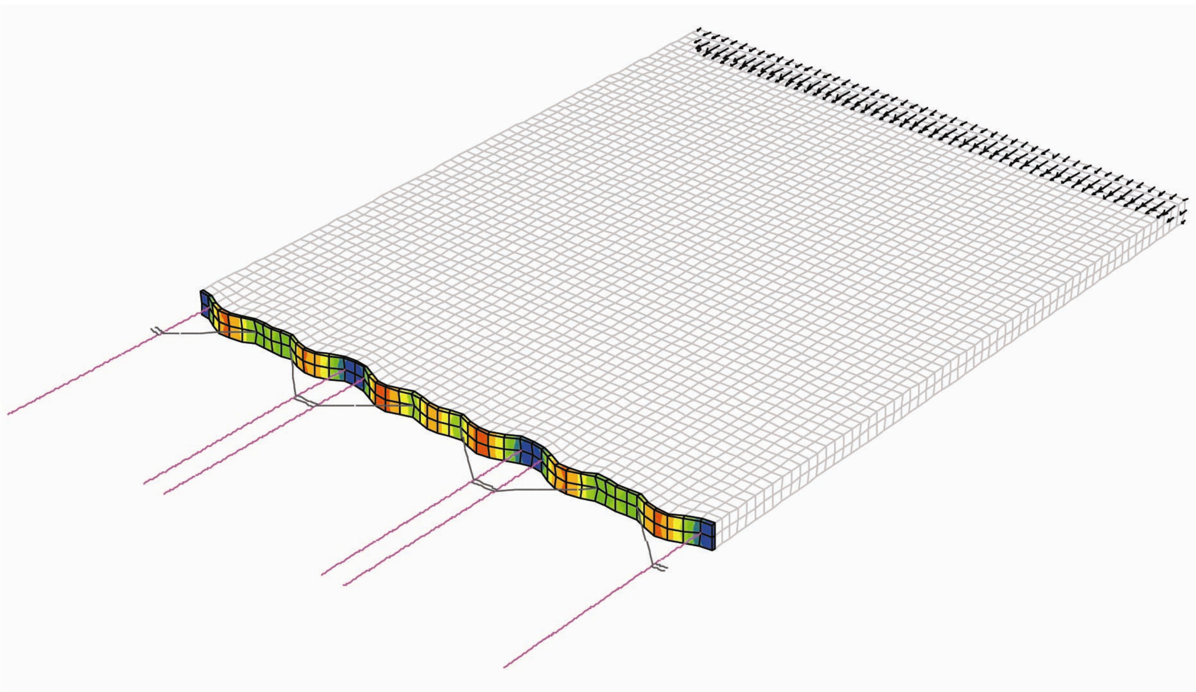

Figure 7: Model of a strutted sheet-pile wall with soil using 3-D software. Note that waler deformation is exaggerated

As shown by the exaggerated deformation of the waler in Figure 7 above, the stiffer strut supports deflected less than the mid-span of the waler. A better representation of the deflection is as shown in Figure 8 where deflection or movements are indicated by colour contours. In Figure 8 below, there is some form of arching effect of soil. Note that the stiffer support is indicated by the blue contours, and the maximum deflection or movement is indicated by the red contours. Hence, it can be seen that the soil bridged from one stiffer strut support to the next stiff strut support. And, the higher deflection of the soil occurred in between the two struts. The maximum deflection actually occurred between the strut member and the knee.

Figure 9 is another illustration on the movement of soil. The vectors indicate direction of movement away from the stiffer strut supports. And, the maximum vector happened at the span between the strut member and the knee.

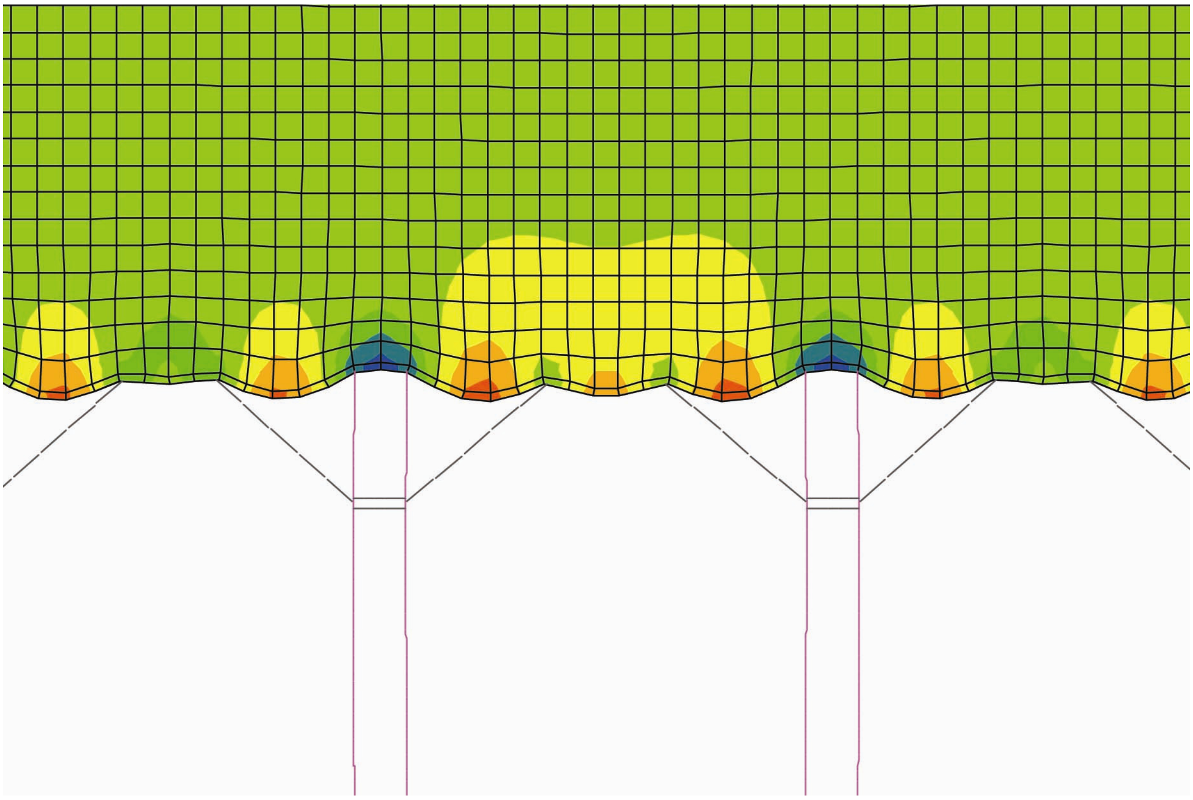

Figure 8: Arching effect of soil mainly between the stiffer strut supports. Note that waler deformation is exaggerated

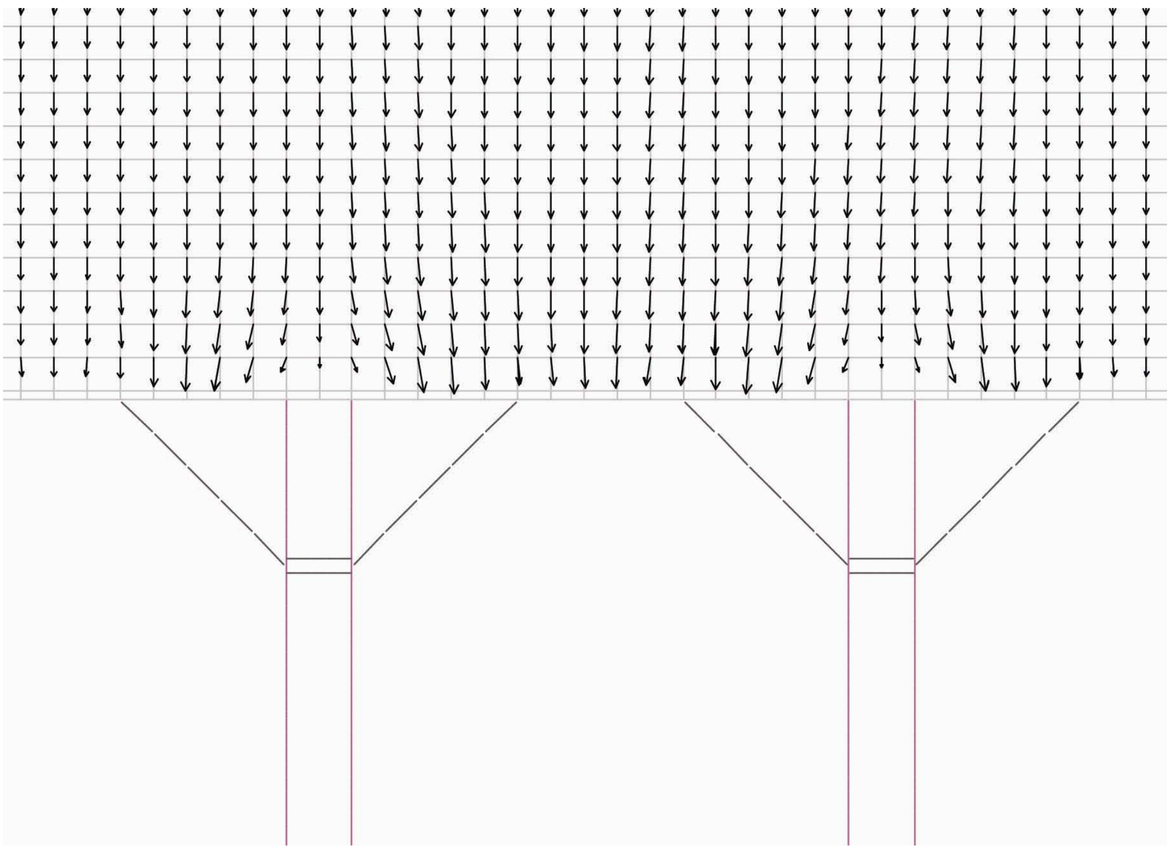

Figure 9: Vector movement of soil indicated to be away from the stiff strut supports

Summing up, Figures 8 and 9 illustrated the soil-structure response of the soil and the structure. The stiffer parts of the strutting have less soil deflection than the less stiff parts in between them. These stiffer parts correspond to higher stress and the less stiff parts correspond to lower stress.

3.3 Three dimensional (3D) analysis – finite difference method

A 3-D model of the strutted embedded wall, as shown in Figure 10, was created to further investigate the soil-structure interaction of the strutted embedded wall system with respective to the strutting system. The wall is modelled after a sheet-pile wall inclusive of the pinned connection of the interlocking joints. This would simulate the laterally non-rigid wall structure.

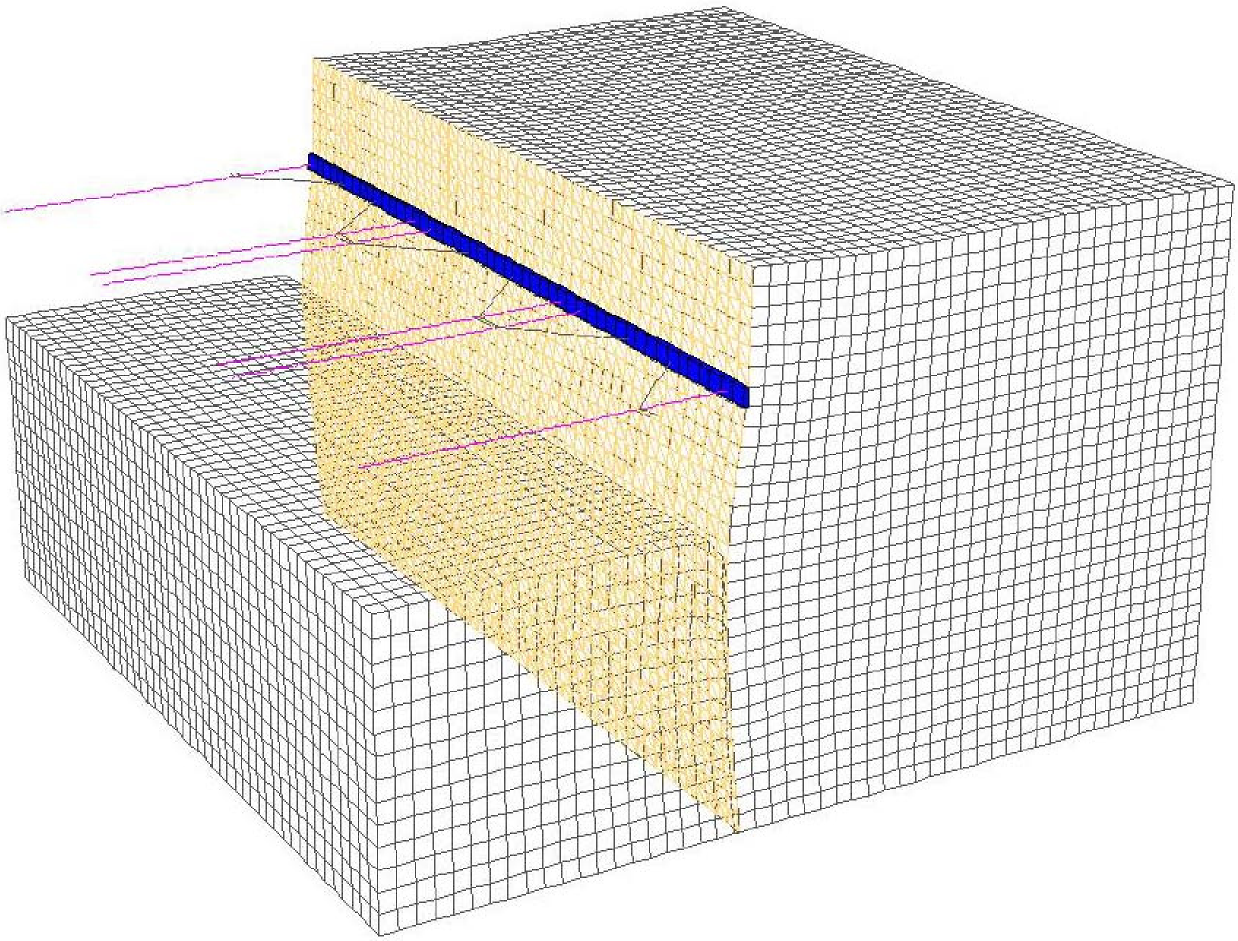

Figure 10: A 3-D model showing the wall, waler, struts and soil, as well as exaggerated deformation

As shown in Figure 10 above, the deformation (exaggerated for better observation) of the wall occurred at the excavation level, as well as heaving at the excavation soil side. This is typical of a normal strutted embedded wall excavation.

At the strutting level, the soil movement behind the strutting system is shown in Figure 11 below, which is distinctly different from the 2-D case shown in Figure 8. The least soil movement still occurred at the strut support (red colour). However, the large soil movement (blue colour) occurred at a distance away from the wall, which occurred generally across the full width of the model. And, the maximum soil movement occurred at the middle of the model. This is a result of larger movement of soil towards the highest wall deflection zone at the lower excavation level. This is illustrated in Figure 12 below. Similarly, in Figure 13 below, the vector movement of soil did not indicate any significant pattern when compared to the vector plot in figure 9.

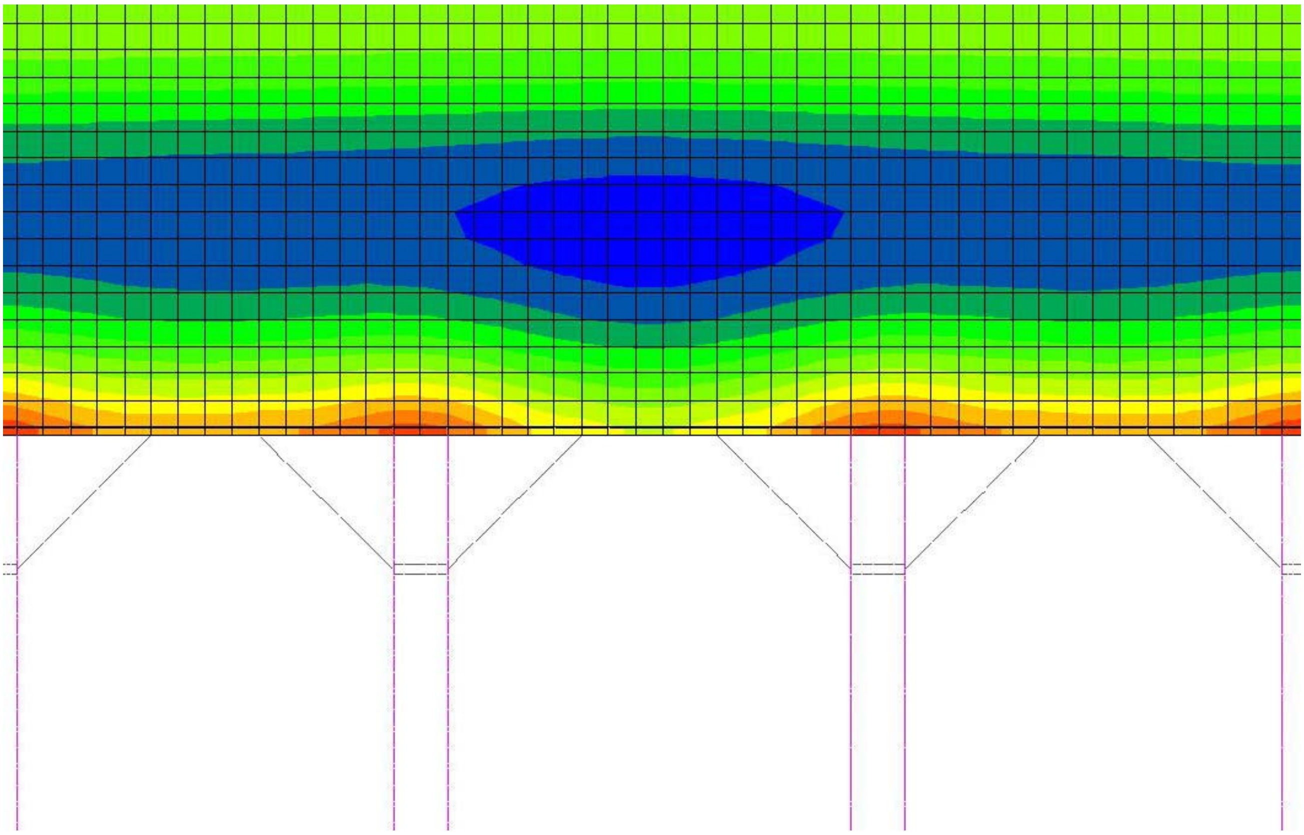

Figure 11: Distribution of movement in soil at strutting level for the 3-D model

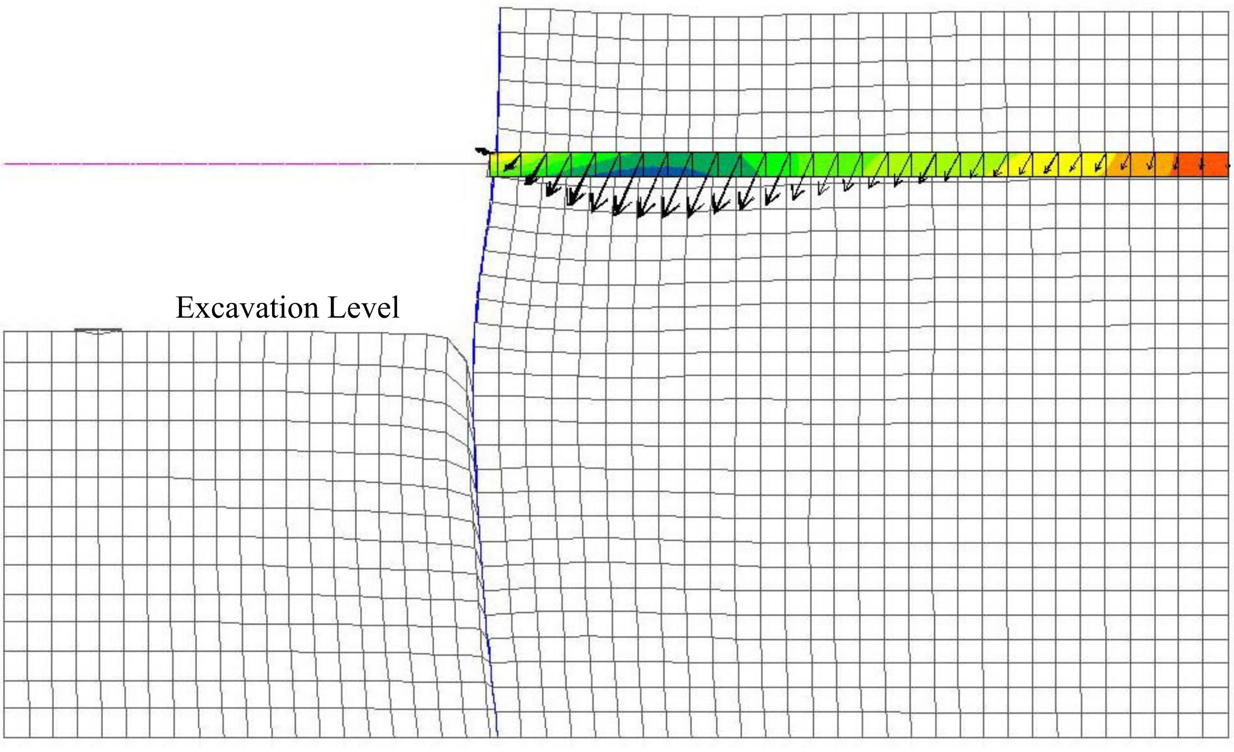

Figure 12: Actual movement of soil at strutting level of the 3-D model

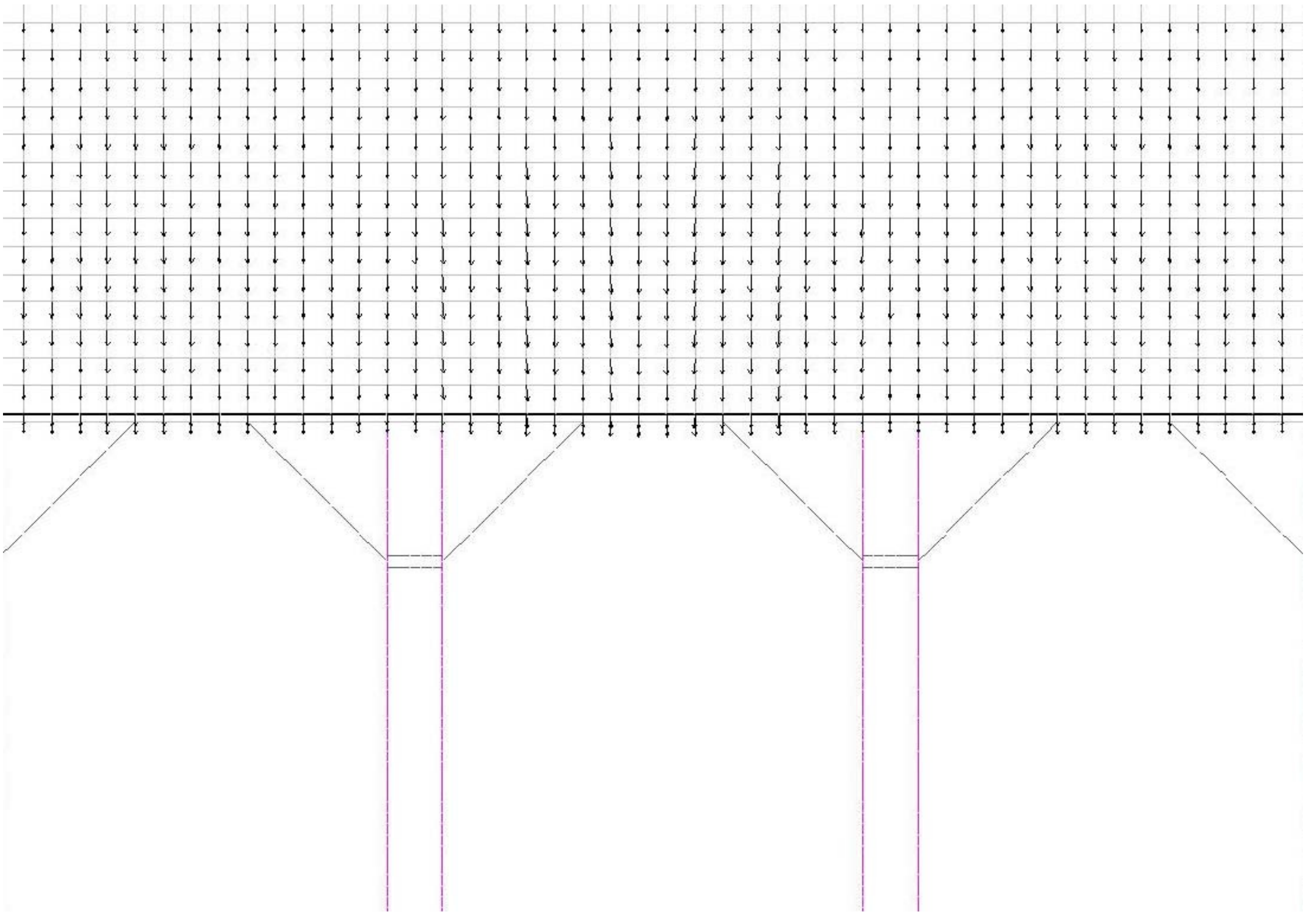

Figure 13: Plan view of the vector movement of soil at the strutting level of the 3-D model

This difference in soil movement pattern with the 2-D case indicates the influence of 3-D effect, and demonstrates the complex nature of the soil-structure interaction of a strutted embedded wall in actual field condition.

4 DISCUSSION

4.1 Knee vs strut forces

Table 1 tabulated the ratio of the knee component forces and the strut-tip forces (that is in contact with the waler) with respect to the total strut forces for the analytical methods mentioned in this paper.

Table 1: Tabulation of the ratio of strut component forces for various mentioned analytical methods

	KneesComponent*/strut	Strut-tips**/strut
Structural Frame STAAD III (Figure 1)	117.9%	-17.9%
UDL support STAAD III (Figure 2)	78.4%	21.6%
PLAXIS 2D Sheet pile Wall (Figure 3)	48.6%	51.4%
PLAXIS 2D Diaphragm Wall (Figure 5)	25.2%	74.8%

Notes: *KneesComponent = component of both knees in strut axial direction
**Strut-tips = end part of the strut or struts in connection with the waler

With the type of configuration in Figure 1, the structural frame approach indicated that the two knees component forces contributed to 117.9% of the total strut force, while the strut-tip 'contributed' to -17.9% of the total strut force. Effectively, the strut-tip is subjected to tension instead of the normal compression, as a result of the cantilever action at the waler end. Note that the knee component/strut ratio would be much reduced should the frame is modelled with more number of struts and without cantilever ends.

In the UDL support type of configuration (Figure 2), the use of the linear spring support is to simulate a pseudo soil subgrade reaction. The resulting knee component force ratio and the strut-tip force ratio are 78.4% and 21.6% respectively. These ratios still indicates a higher knee component forces which do not relate well with the stiffer strut-tips which associated with least soil movement as shown in Figure 8. And, it differs greatly from the corresponding ratios analysed based on the Plaxis 2D sheet-pile wall model (see figure 3). Thus, the structural framing approach used would be a relatively conservative approach and care has to be taken in creating the frame model.

For the type of configuration using PLAXIS 2D finite element, the sheet-pile wall (see Figure 3) analysed indicates ratios of 48.6% and 51.4% for the knee component forces and strut-tip forces respectively. This differs greatly from the analysis for a diaphragm wall (see Figure 5) with ratios of 25.2% and 74.8% for the knee component forces and strut-tip forces respectively. This demonstrates the difference in the soil-structure interaction with respect to the influence of the lateral stiffness or rigidity of the wall.

4.2 Waler Forces

Table 2 tabulates the analysed bending moment and displacement of the waler, with respect to the analytical methods mentioned in this paper.

Table 2: Bending moments and displacement of the waler

	BMwaler mid-span	BMwaler support	Disp-waler mid-span	Disp-waler Cantilever End
Structural Frame STAAD III (Figure 1)	148.5kNm	200kNm	4mm	14mm
UDL support STAAD III (Figure 2)	36.1kNm	76.8kNm	7mm	7mm
PLAXIS 2D Sheet pile Wall (Figure 3)	37.5kNm	67.9kNm	1.9mm	-
PLAXIS 2D Diaphragm Wall (Figure 5)	28.6kNm	7.0kNm	1.3mm	-

The bending moments and displacement for the structural frame (see Figure 1) are very much higher than the UDL support (see Figure 2) and the Plaxis 2D Sheet pile wall (see Figure 3). This relates to the relative conservativeness of the structural frame approach on the waler.

Also, the difference in the analysed results for the Plaxis 2D sheet pile wall (see figure 3) and Plaxis 2D diaphragm wall (see Figure 5) shows the influence of the lateral rigidity of the wall in the soil-structure interaction.

5 CONCLUSIONS

The soil-structure interaction has been illustrated for a laterally non-rigid strutted embedded wall. The stiffer strut supports on the soil (waler) is clearly shown, which associated with the least movement part of the soil. This corresponds to the higher stress area. The 'arching' pattern is also presented in the 2-D case, which bridged from support to support. However, this pattern is complicated with the 3-D effect.

The influence of the lateral rigidity on the soil-structure interaction is also illustrated with differing results between the sheet-pile wall and diaphragm wall.

The structural framing approach for the wall support system is relatively conservative as it did not consider the soil-structure interaction. Also, it has to be used with caution in preparing the frame model, in order to avoid problems like tension or bending moments in struts, cantilever waler. Moreover, the wall with its lateral rigidity needs to be incorporated as a structural element in the framing system, to better simulate the field condition.

The recommended method of analysis for the wall support system is the geotechnical approach which is the use of finite element method or the finite difference method, as it takes into consideration the soil-structure interaction which better simulate the actual field condition.

Deep Excavation for Basement via Soil Nailing Method

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ABSTRACT

Deep excavations for basement are common part of development to utilize underground space in densely populated areas. Protection of adjacent buildings and properties is a primary design concern nowadays for underground construction and thus, correct selection and design of suitable retaining wall system are critical for the success of the project. Common conventional retaining wall system used for deep excavation in Malaysia includes diaphragm wall, secant pile wall, contiguous bored pile wall, soldier pile wall or sheet pile wall supported either by internal strutting, temporary ground anchors, semi top-down or top-down, etc. The Authors had introduced soil nailing technique to replace the conventional retaining wall system for deep basement excavation in Malaysia. If the site condition is suitable for soil nailing technique, it will contribute to significant savings in cost and time of construction compared to conventional retaining wall system.

This paper presents two case histories on the use of soil nailing technique for deep basement excavation in Malaysia. The first case history presents an excavation of up to 30m deep for the construction of 7 levels of basement for a commercial development. The site is underlain by metasedimentary formation. The geotechnical challenge is to design and construct a deep basement of up to 30m deep with close proximity of low-rise structures (less than 5m) adjacent to the deep excavation. Experience learnt in characterizing the soil and weathered rock properties is also presented especially on determination of geotechnical parameters for weathered rock mass where proper sampling and testing of materials are difficult. Monitoring works on the completed soil nailed slope using settlement markers and inclinometers had demonstrated the effectiveness of the system where lateral movement and settlement are well within prediction. The case history demonstrates the effectiveness of soil nailing technique for deep basement construction even with close proximity of adjacent structures.

The second case history presents an excavation of up to 20m deep for the construction of 5 levels of basement in a mixed development underlain by granite formation. The site is adjacent to low-rise buildings at the boundary to the north and roads to the east and west sides of the site. This paper presents the detailed planning, coordination and interaction between geotechnical engineers and Architect to come out with the most cost effective and construction friendly solution that is at the same time fulfills the Architectural requirements. This case history demonstrates the importance of cooperation between Architect and geotechnical engineers in producing innovative solutions for the benefits of the project.

1 INTRODUCTION

1.1 Deep excavation for basement construction

Efficient use of space is a major design concern in urban development and as such, underground space is commonly utilized for basement parking, mechanical and electrical (M&E) rooms, etc. With high-rises, the depth of basement excavation is also significant in order to cater for the required numbers of carpark bays by local Authority. As such, like any major cities, the depth of basement excavation in Kuala Lumpur also increases as the country progresses and this continuously advances design concepts and capabilities of machineries to construct deeper basement. Some Malaysian experience in the design and construction of retaining wall and support systems for deep basement construction have been discussed by Tan & Chow, 2008.

However, due to scarcity of land in Kuala Lumpur City Centre, major developments are increasingly being carried out in peripheral areas of Kuala Lumpur such as in Petaling Jaya, Mont' Kiara, Puchong and Shah Alam. These areas present unique challenges compared to conventional development in Kuala Lumpur City Centre as its land price and selling price are lower compared to Kuala Lumpur City Centre and the land available for development is bigger. This situation presented unique challenges and opportunities for innovative design as the selling price is not high enough for conventional basement excavation technique (e.g. diaphragm wall) to be economically feasible. However, with larger development area, some flexibility in basement layout and design allow techniques such as soil nail to be employed even for deep excavation with basement of up to 30m deep.

1.2 Soil nailing technique

Soil nail as stabilization measure for distressed slopes and for new very steep cut slopes has the distinct advantage of strengthening the slope without excessive earthworks to provide construction access and working space associated with commonly used retaining system such as reinforced concrete wall, reinforced soil wall, etc. In addition, due to its rather straightforward construction method and is relatively maintenance free, the method has gained popularity in Malaysia for highway and also hillside development projects.

In recent years, due to the advantages of soil nail which can be constructed in areas with difficult access and minimizes earthworks, soil nail system has also demonstrated its applicability for deep excavation works for basement construction. The use of soil nail system has resulted in cost savings to deep excavation project and also enables basement construction to be carried out in a relatively unobstructed working space.

The basic concept of soil nailing is to reinforce and strengthen the existing ground by installing closely-spaced steel bars, called 'nails', into a slope as construction proceeds from 'top-down'. This process creates a reinforced section that is in itself stable and able to retain the ground behind it. The reinforcements are passive and develop their reinforcing action through nail-ground interactions as the ground deforms during and following construction.

Various international codes of practice and design manuals such as listed below are available for design of soil nail:

a) British Standard BS8006: 1995, Code of Practice for Strengthened/Reinforced Soils and Other Fills.
b) HA 68/94, Design Methods for the Reinforcement of Highway Slopes by Reinforced Soil and Soil Nailing Techniques.
c) U.S. Department of Transportation, Federal Highway Administration, Manual for Design and Construction Monitoring of Soil Nail Walls (FHWA, 1998).
d) CIRIA C637: Soil nailing – best practice guidance.

A review of the various design methods for soil nail has been carried out by Chow & Tan, 2006. In this paper, a brief discussion on the use of soil nail for deep excavation works is presented highlighting its relative advantages and also its limitations and how it can be overcome. Two case histories are also presented to illustrate the principles discussed.

2 SOIL NAIL FOR DEEP EXCAVATION

Soil nail offers significant advantage for deep excavation works especially in areas outside of major city centre which requires deep basement in order to maximize land use. Development features which are most suited for the use of soil nail to replace conventional system such as diaphragm wall are as follows:

a) Areas just outside of major city centre where land price and selling price is not at the top most tier.
b) Sizeable land for development with typical land area exceeding 6 acres. Larger land area will give more flexibility in terms of layout and carpark planning in order to optimize basement construction.
c) Best suited for areas with different elevations across the site where conventional retaining wall such as top-down construction may induce unbalanced forces onto the permanent structures. Typical land profile is illustrated in Figure 1. Nevertheless, soil nail system can also be adopted for relatively flat land and in Malaysia, it is significantly cheaper compared to conventional system such as diaphragm wall with top-down construction and is also faster as the area will be relatively free for subsequent basement construction.

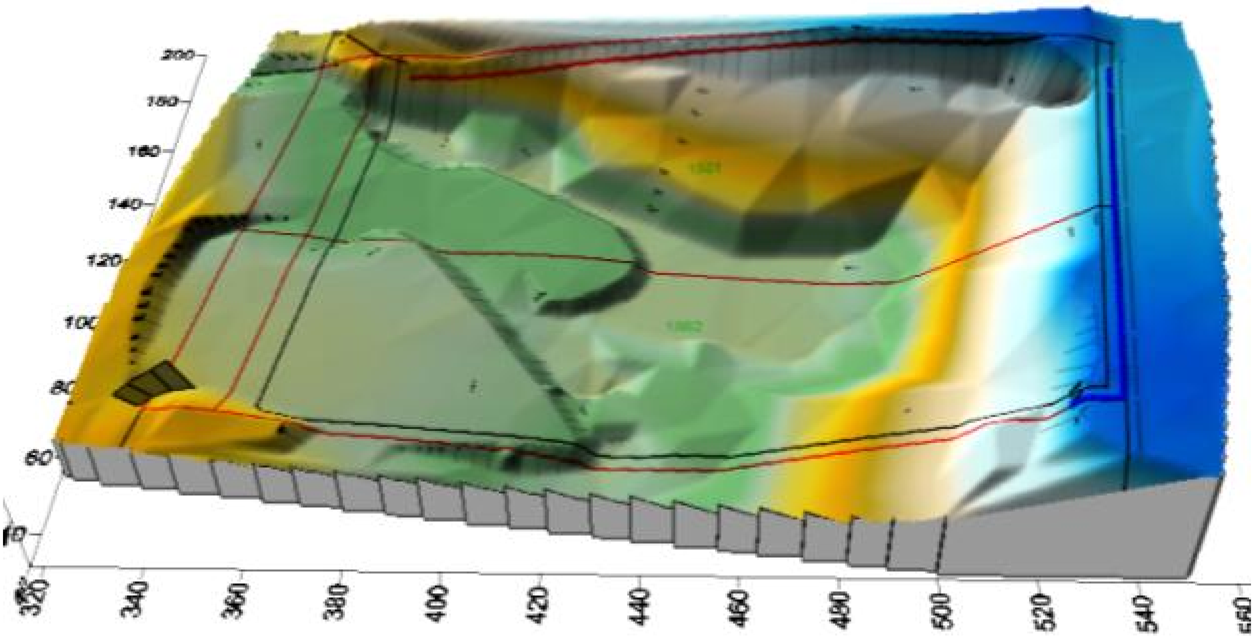

Figure 1: Land profile suitable for soil nail system due to different ground elevations surrounding the site.

Some of the advantages of soil nail system compared to conventional retaining wall system associated with deep basement are:

a) Does not require large working space for the works. For example, diaphragm wall would require area for large machineries, storage for bentonite and recycling of bentonite fluid.
b) Relatively cheaper.
c) Relatively cleaner site as disposal of drilled/excavated materials will be less.
d) Straightforward construction as it does not involve other trades. For example, top-down construction would involve concreting and structural works, installation and prestressing of temporary ground anchors for anchored wall system, etc.

Similar to any system, there are limitations to the system as follows:

a) Requires close coordination and cooperation between Architect, Structural Engineer and Geotechnical Engineer. The system is generally different compared to conventional design where basement walls are usually located at the edge of the boundary, right up to the statutory building setback requirements. As such, Architect and Engineers would have to work together in order to optimize the basement layout to suit the ground profile and the design. This would results in optimized solution for the development which is a compromised solution between ideal Architectural, Structural and Geotechnical considerations. As such, involvement of geotechnical engineer should start from early development planning stage.
b) May require interfacing with foundation works as the system works best by not disrupting the superstructure design (changes to accommodate soil nail system to suit the ground profile are confined to basement carpark only). Some columns for superstructures would be situated at berms within the soil nail system in order to support the superstructure.
c) Requires considerable design effort as the design needs to integrate with Architectural, Structural and Earthworks requirements while at the same time fulfilling safety and serviceability of adjacent structures affected by the excavation works.

The above limitations can be overcome if proper planning is carried out and would results in significant savings in terms of cost and time to the project as demonstrated in the two case histories discussed in the following sections.

3 CASE HISTORY 1: 30m DEEP EXCAVATION FOR MIXED COMMERCIAL DEVELOPMENT (SOLARIS DUTAMAS)

3.1 Description of project

The proposed development, Solaris Dutamas, consists of 3-5 storey commercial units, office blocks of up to 42 storey high and 33-storey service apartments with 6 levels of basement on an 18-acres land. Figure 2 shows picture of the proposed development while Figure 3 shows completed view of the basement works taken recently (superstructure works completed).

Figure 2: Overall perspective of proposed Solaris Dutamas.

Figure 3: Completed basement and superstructure works.

3.2 General geology and subsoil profile

The site is underlain by metasedimentary rock formation known as Kenny Hill Formation. The bedrock consists of sandstone/siltstone with quartzite and phyllite. The weathered soils are generally characterized by the presence of sandy SILT/CLAY.

Typical simplified borelog profiles at the site are shown in Figure 4 for relevant boreholes at the Northern boundary of the site where the highest soil nail of up to 30m is constructed. In general, the residual soil layer is very thin (generally less than 10m) before deep weathered metasedimentary materials with SPT-N > 50 is encountered. The significant thickness of the weathered metasedimentary materials presented challenges in characterizing its behavior due to sampling difficulties and its variability. For design purposes, soil properties were carried out using equivalent Mohr-Coulomb (c', ϕ') parameters based on Hoek-Brown failure criterion. The soil nail slope is continuously re-assessed during construction as the slope surface is exposed and rock mass characterization was carried out by experienced geologist/geotechnical engineer.

Figures 5 and 6 show typical weathered metasedimentary materials exposed during soil nail construction.

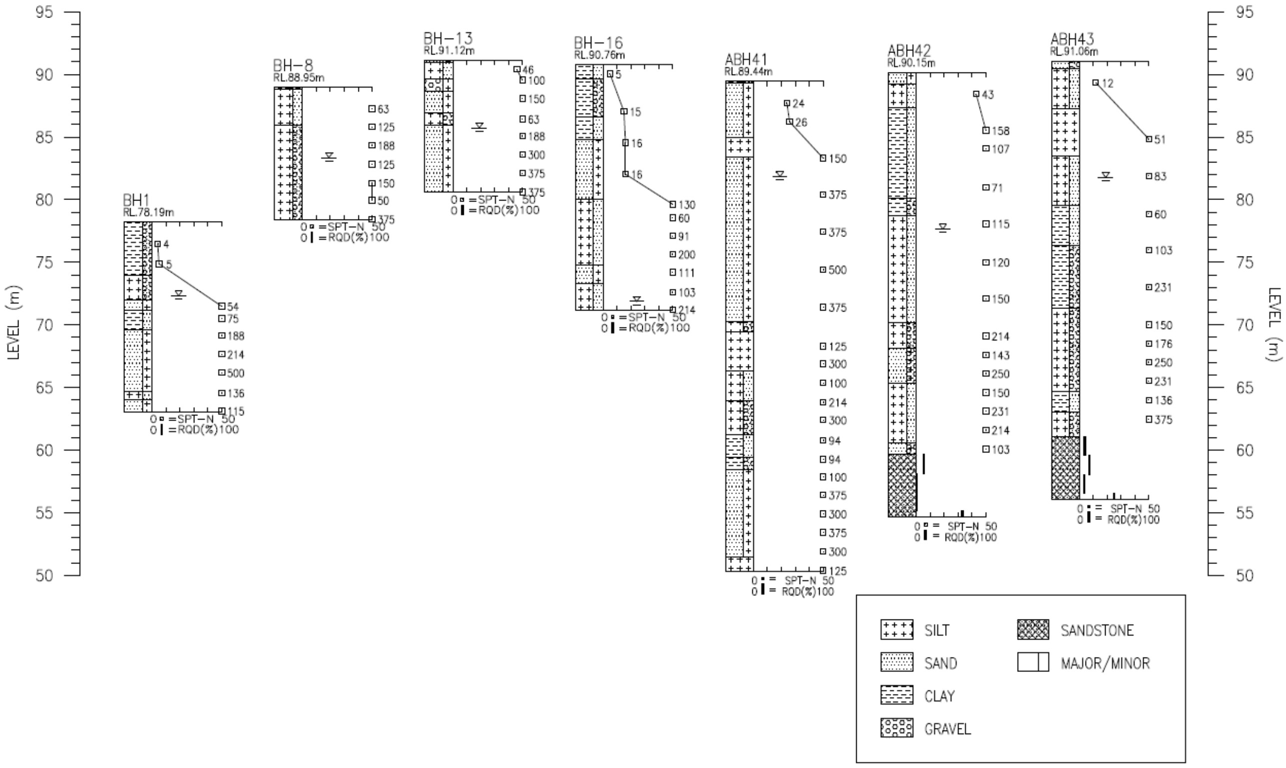

Figure 4: Typical simplified borelog profiles at Northern part of the site with soil nail wall up to 30m high.

Figure 5: Typical weathered metasedimentary materials exposed during soil nail construction (Grade V/VI).

Figure 6: Typical weathered metasedimentary materials exposed during soil nail construction (Grade III/IV).

3.3 Design of soil nail system for deep excavation

The original retaining wall system consists of ϕ600mm contiguous bored pile (CBP) wall with temporary ground anchors as shown in Figure 7. However, review of the contiguous bored pile system shows that the cost of the CBP wall will be high as it needs to drill through the Grade III/IV material in order to form the CBP. The weathered material cannot be left unsupported as it is highly fractured and is susceptible to degradation with time. As such, soil nail system is introduced as alternative to CBP wall and it is believed that the use of soil nail system for basement construction with such high retained height of up to 30m is being introduced for the first time in Malaysia.

Figure 8 shows the alternative soil nail system which was successfully constructed. The alternative soil nail system requires some minor modifications to the basement layout and also proper planning of construction sequence and foundation system to support the superstructure. Figures 9 and 10 show pictures taken during soil nail construction where the close proximity of existing buildings (less than 5m) to the soil nail slope can be observed. Completed bored piles (with protruding starter bars) within the soil nail slope can also be seen. Figure 11 shows picture taken when basement construction has been completed.

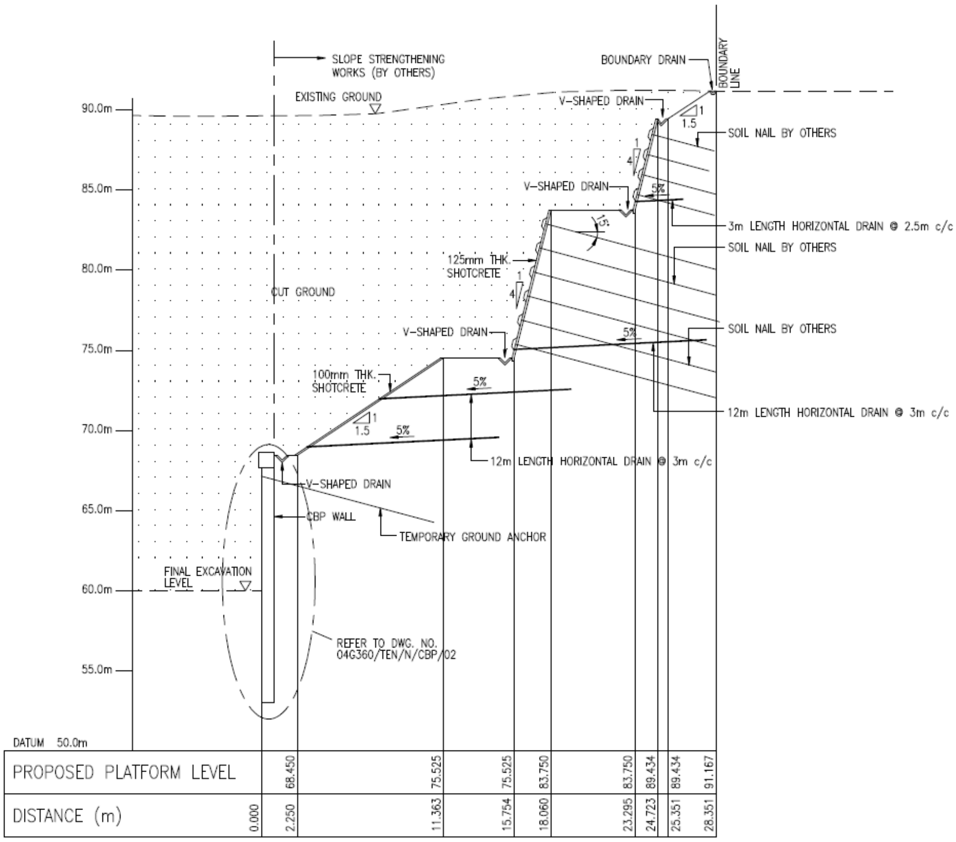

Figure 7: Original design of retaining wall system using conventional CBP wall with temporary ground anchors.

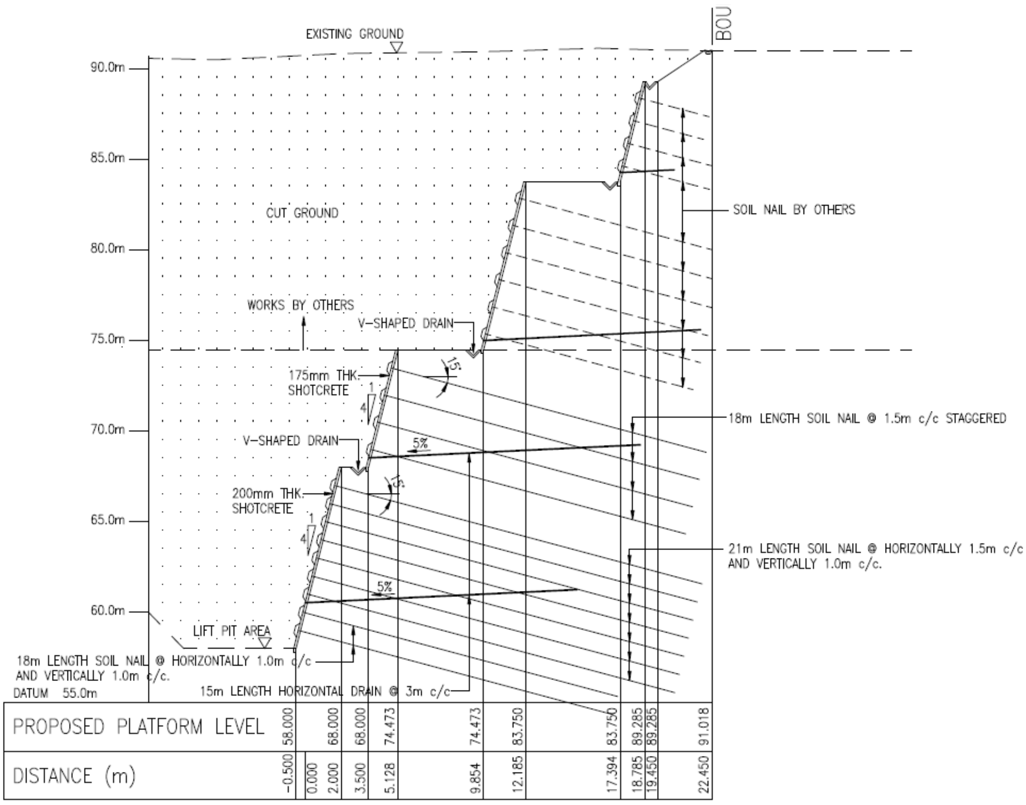

Figure 8: Alternative soil nail system to replace conventional CBP wall with temporary ground anchors.

Figure 9: Picture taken after completion of soil nailing works and basement construction on-going.

Figure 10: Picture taken during soil nailing works.

Figure 11: Picture taken after completion of basement construction.

The design of the soil nail slope was carried out based on the recommendations of FHWA, 1998. Broadly, the design needs to ensure:

a) Internal failure
   i. Face failure
   ii. Pullout failure
   iii. Nail tendon failure
b) External failure – Global failure surface
c) Serviceability to prevent distresses to adjacent existing buildings

FHWA, 1998 provides a rational and detailed approach for the design of soil nail system which ensures safety with respect to internal failure and external failure. Particular attention was paid to design of adequate thickness of shotcrete due to the significant height of the soil nail slope with steep angle (4V:1H) and also in deriving correct parameters especially on nail load diagram for stability analysis. The importance of nail load diagram is briefly discussed below.

In Figure 12, it can be seen that the nail load diagram consists of three zones, A, B and C. Zone A is governed by the strength of the facing, T_F and also the ground-grout bond stress, Q. If the facing of soil nails is designed to take full tensile capacity of the nail, then the full tensile capacity of the nail can be mobilized even if the critical slip circle passes through Zone A. However, to design the facing with full tensile capacity of nails instead of lower T_F is not economical for high slope (e.g. more than 15m). Zone B is governed by nail tendon strength, T_N and Zone C is governed by ground-grout bond stress, Q. From the diagram, it is clear that the mobilized nail resistance should not exceed the nail load envelope developed from the three failure criteria discussed above. Therefore, the nail resistance as input for slope stability analysis should refer to the nail load diagram (Figure 12) corresponding to the available bond length for the critical slip plane (Figure 13).

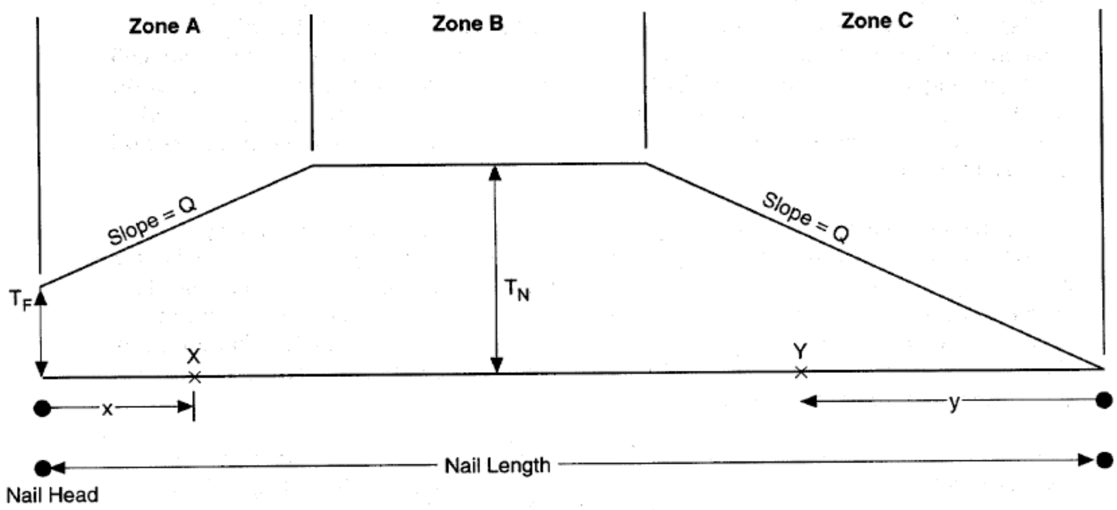

Figure 12: Nail load diagram (FHWA, 1998).

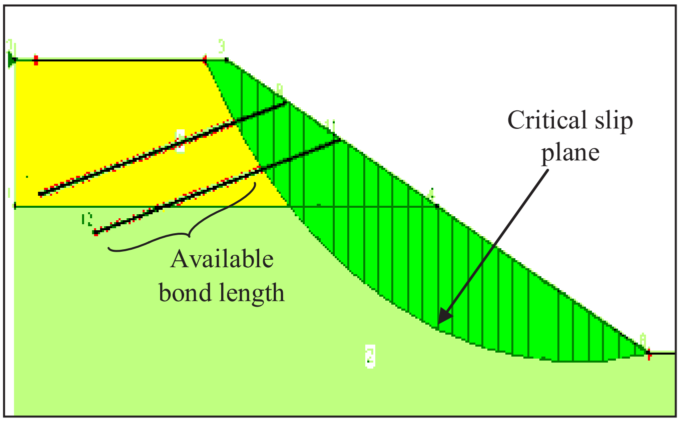

Figure 13: Available bond length from slope stability analysis.

Some slope stability analysis software may have the capabilities to automatically adjust the nail resistance based on the ground-grout bond stress and nail tendon strength. However, extra caution needs to be exercised as some of the software does not cater for the reduction of nail load at Zone A and assumes strength of T_N. As illustrated in Figure 14, failure to cater for the reduction of soil nail resistance in Zone A may lead to overestimation of the available nail resistance in slope stability analysis for critical slip circle that passes through this zone.

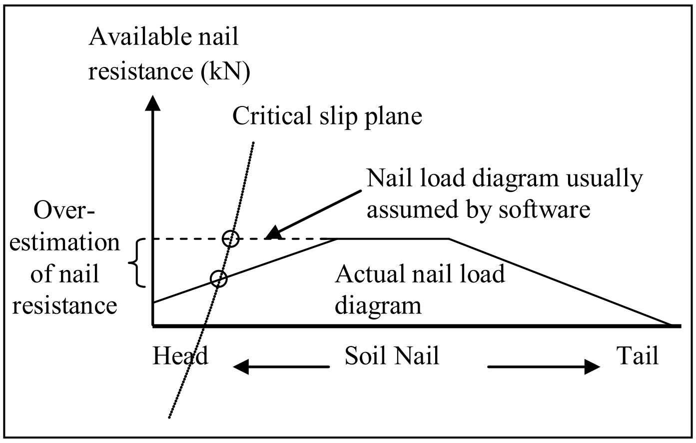

Figure 14: Overestimation of nail resistance if shotcrete facing strength not taken into consideration.

As the soil nailed wall will be left exposed and is part of the permanent structure, the minimum factor of safety (FOS) required is 1.4. The details of the soil nail satisfying ultimate limit state requirements are:

a) Nail diameter of 25mm to 32mm (Yield strength of nail = 460 N/mm^2)
b) Grout hole diameter = 125mm.
c) Nail length from 4m to 21m.
d) Shotcrete thickness of 100mm to 200mm.

During the design process, one of the major challenges in designing the soil nail system is characterization of the highly fractured/weathered rock mass (Figure 6). Based on reported range of basic friction angles (Barton & Choubey, 1977) for rock materials which is reproduced in Table 1 below, the effective stress shear strength parameters adopted for the weathered sandstone is ϕ' = 35° and c' = 15kPa. The basic friction angle is not adjusted for roughness and surface irregularities as site observation has confirmed the surfaces of the weathered rock is quite smooth. A relatively low effective cohesion, c' = 15kPa is adopted as it is imprudent to adopt high c' value especially for long-term stability considerations.

Table 1: Basic friction angles for a range of rock materials (Barton & Choubey, 1977).

Rock type	ϕ_b dry Degrees	ϕ_b wet Degrees
Sandstone	26 – 35	25 – 34
Siltstone	31 – 33	27 – 31
Limestone	31 – 37	27 – 35
Basalt	35 – 38	31 – 36
Fine granite	31 – 35	29 – 31
Coarse granite	31 – 35	31 – 33
Gneiss	26 – 29	23 – 26
Slate	25 – 30	21

As the soil nail work progresses, review of the weathered rock mass parameters were also carried out based on site observations by geotechnical engineer/geologist using Hoek-Brown criterion as shown in Figure 15. Results from Figure 15 indicate the adopted shear strength parameters for the weathered rock is appropriate. It is noted that the equivalent Mohr-Coulomb parameters obtained from Hoek-Brown criterion are ϕ' = 35° and c' = 77kPa but as explained above, the use of lower c' for long-term slope stability is recommended.

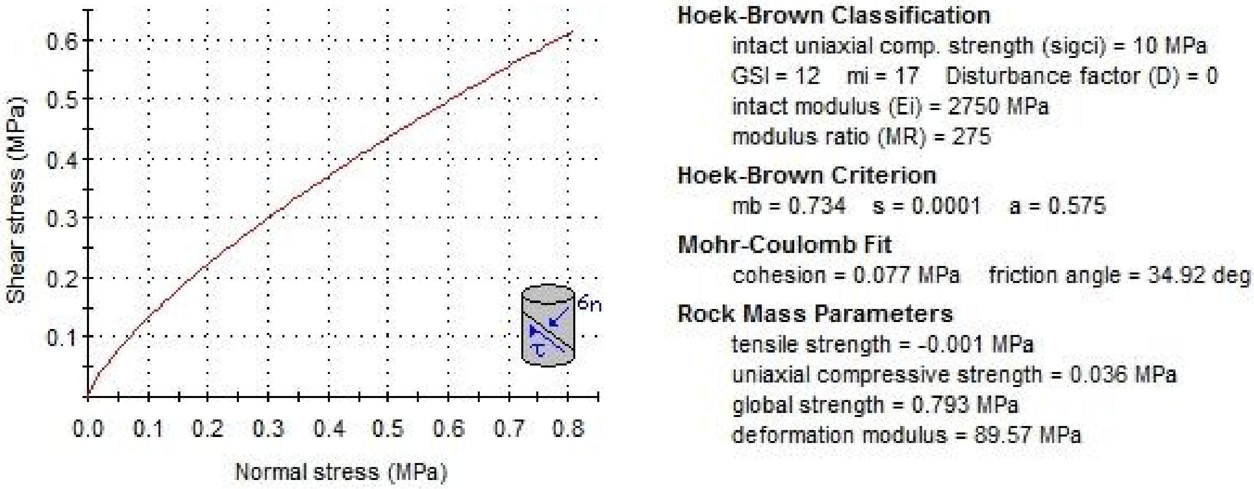

Figure 15: Rock mass parameters for weathered sandstone based on Hoek-Brown criterion (Output from Roclab software).

Due to the close proximity of adjacent low-rise structures, serviceability limit states were also checked using finite element method. The analysis was carried out using the commercial software, PLAXIS. Typical model set-up in PLAXIS is shown in Figure 16.

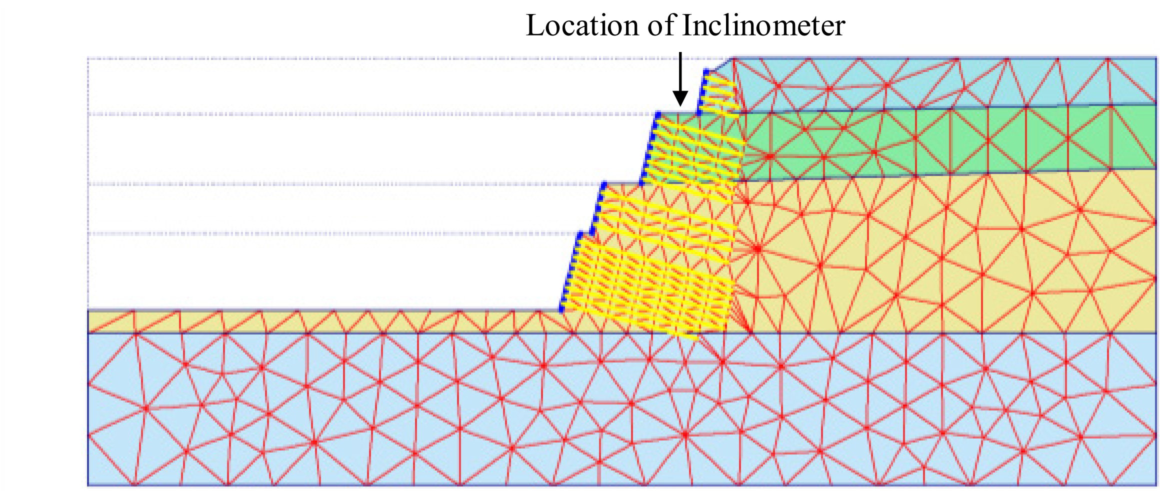

Figure 16: Typical finite element model in PLAXIS for soil nail system.

Figure 17 shows the measured ground movement using inclinometer together with the predicted ground movement from PLAXIS. It can be observed that the ground movement measured at site generally agrees with the prediction although the predicted movement is larger which is expected as slightly conservative deformation parameters were adopted during design. Based on the prediction, the soil nail slope is not expected to cause any significant distress to the adjacent structures and this is subsequently confirmed by monitoring results and visual assessment on the adjacent structures.

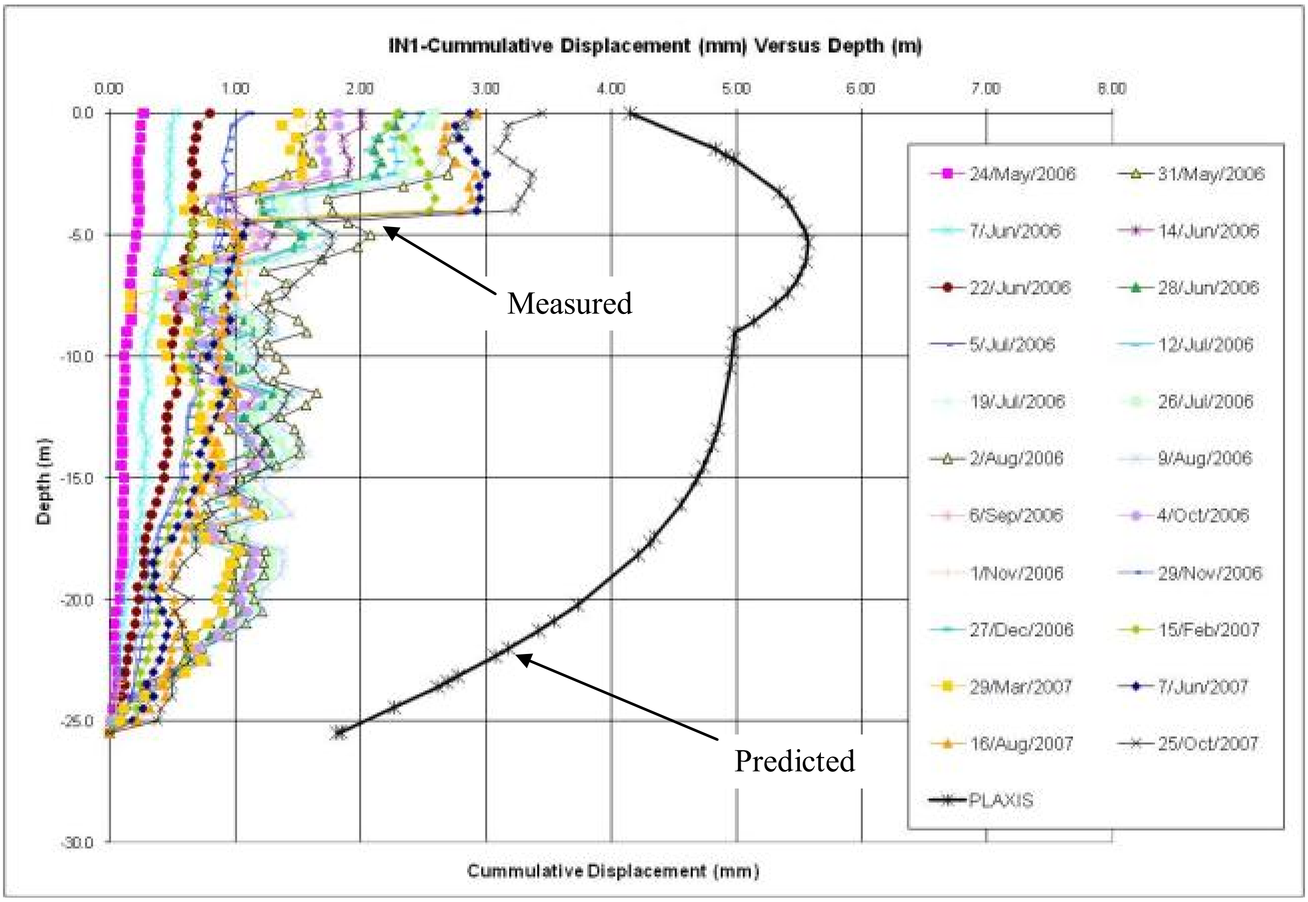

Figure 17: Predicted and measured ground movement of soil nail slope.

The soil nails were also subjected to verification and proof test based on the following acceptance criteria in accordance with FHWA, 1998:

a) For verification tests, a total creep movement of less than 2mm per log cycle of time between the 6 and 60 minute readings is measured during creep testing and the creep rate is linear or decreasing throughout the creep test load hold period. (Note: The creep criterion has been established to ensure that the nail design load can be safely carried throughout the structures' service life (up to 100 years) without causing movement that could damage the structures)
b) For proof tests, a total creep movement of less than 1mm is measured between the 1 and 10 minute readings or a total creep movement of less than 2mm is measured between the 6 and 60 minute readings and creep rate is linear or decreasing throughout the creep test load hold period.
c) The total measured movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the test nail unbonded length.
d) A pullout failure does not occur at the maximum test load. Pullout failure is defined as the load at which attempts to further increase the test load simply result in continued pullout movement of the test nail.

Based on verification tests carried out on preliminary soil nails, the ultimate capacity of the ground-grout bond ranges from 3 to 5 times SPT-N (in kPa). As such, the 3*SPT-N correlation was routinely adopted for preliminary design in Malaysia and optimization to 5*SPT-N will be carried out if pull-out test results justify higher values.

Typical result of a pull-out test is shown in Figure 18.

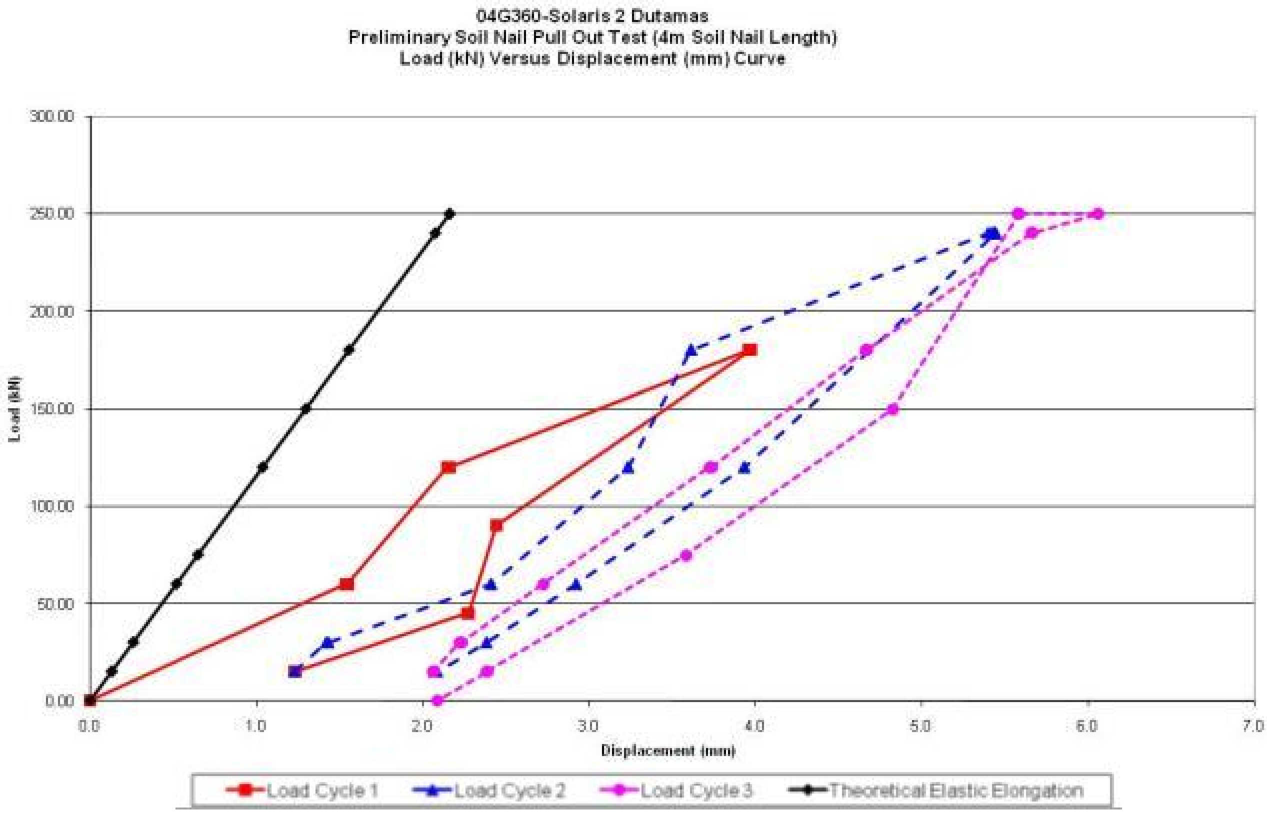

Figure 18: Typical verification pull-out test results.

In summary, the performance of the soil nail system is satisfactory and is superior compared to conventional retaining wall system for this particular project site resulting in significant time and cost saving. The cost savings resulting from the soil nails system is approximately RM5 Million.

4 CASE HISTORY 2: 20m DEEP EXCAVATION FOR MIXED COMMERCIAL DEVELOPMENT AT MONT' KIARA

4.1 Description of project

The proposed development consists of 1 block of 37 storey Hotel and SOHO and 1 block of 19 storey serviced apartments with 5 levels of basement which requires excavation using soil nail of up to 20m deep.

4.2 General geology and subsoil profile

The site is underlain by the Kuala Lumpur Granite formation and the granite bedrock has been detected during the soil investigation works. The texture and composition of the granitic rock generally ranges from coarse to very coarse-grained. The overburden materials consist mainly of completely weathered residual soils, which are derived from the weathering of granitic rock. Loose fill is also detected at certain parts of the site which is reflected in the low SPT-N blow count (SPT-N < 10). Typical borelog profiles are shown in Figure 19.

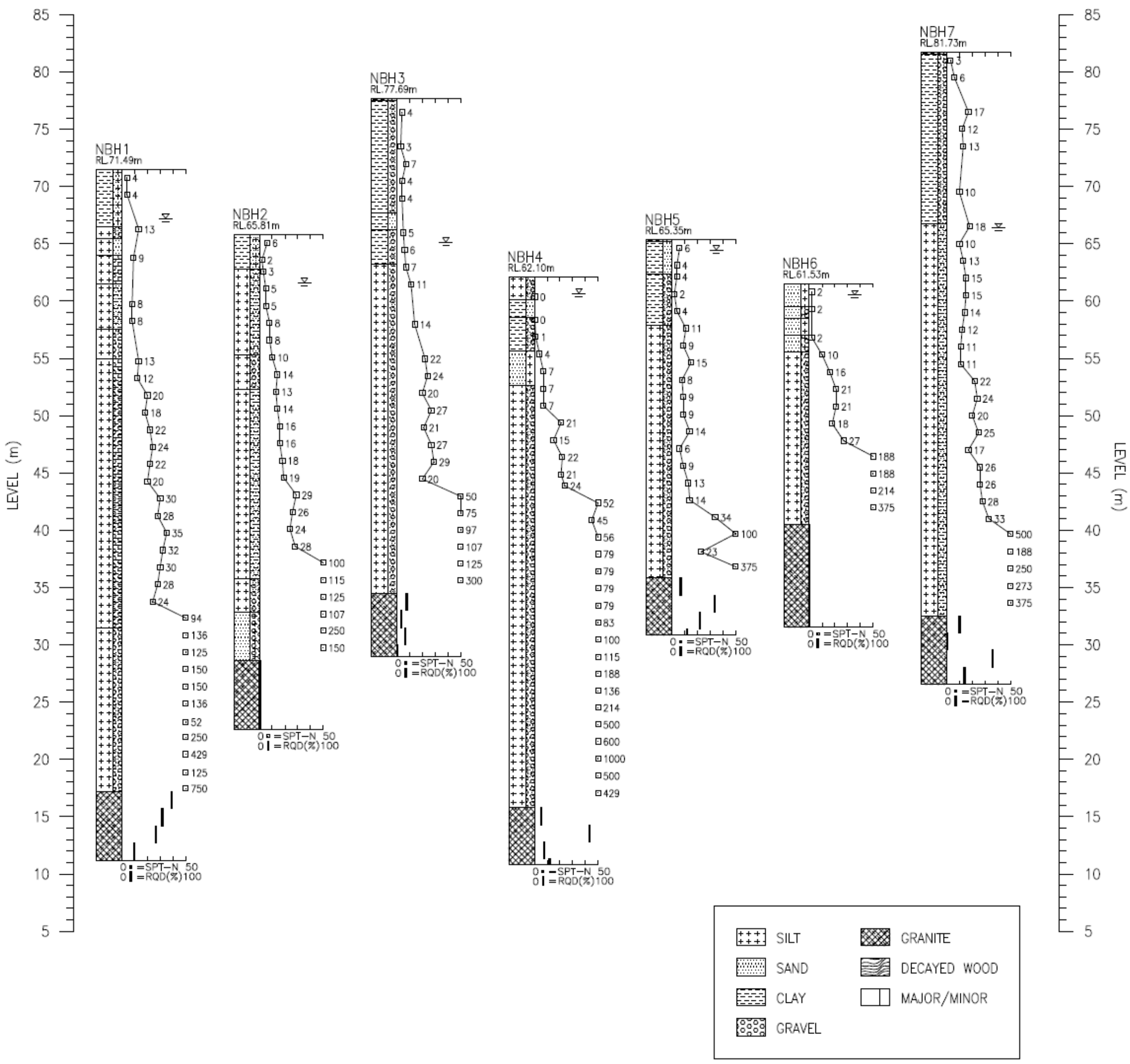

Figure 19: Typical simplified borelog profiles (Case History 2).

4.3 Design development

The original design requires retaining wall of up to 21m deep which is propped against the future basement structure as illustrated in Figure 20. Unbalanced forces will be induced onto the building frame as the supported height of the retaining wall differs significantly with one side of the ground significantly higher than the other side.

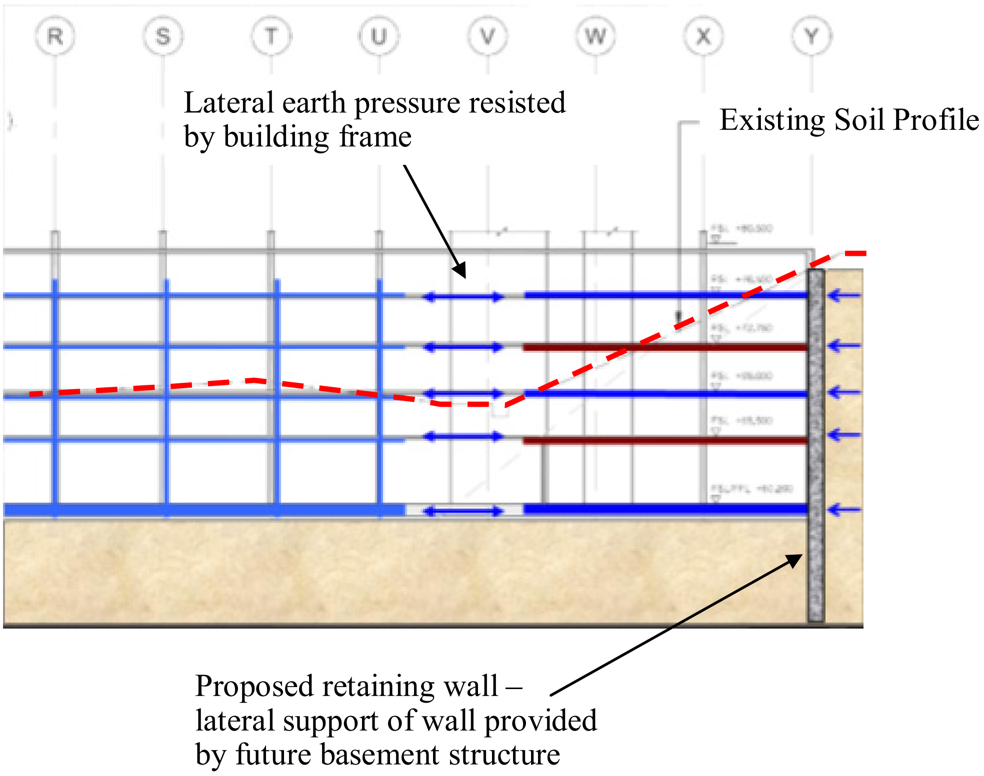

Figure 20: Conventional retaining wall system supported by building structural frame.

As part of the value-engineering exercise carried out, an alternative system using soil nail is explored. However, the required space in order to accommodate the soil nail system within the site boundary requires re-designing of the entire basement. As such, the alternative soil nail system requires the Architect and the Geotechnical Engineer to work closely together in order to produce the optimum solution based on the building needs and existing site conditions. A typical section of the final product of the soil nail system is shown in Figure 21.

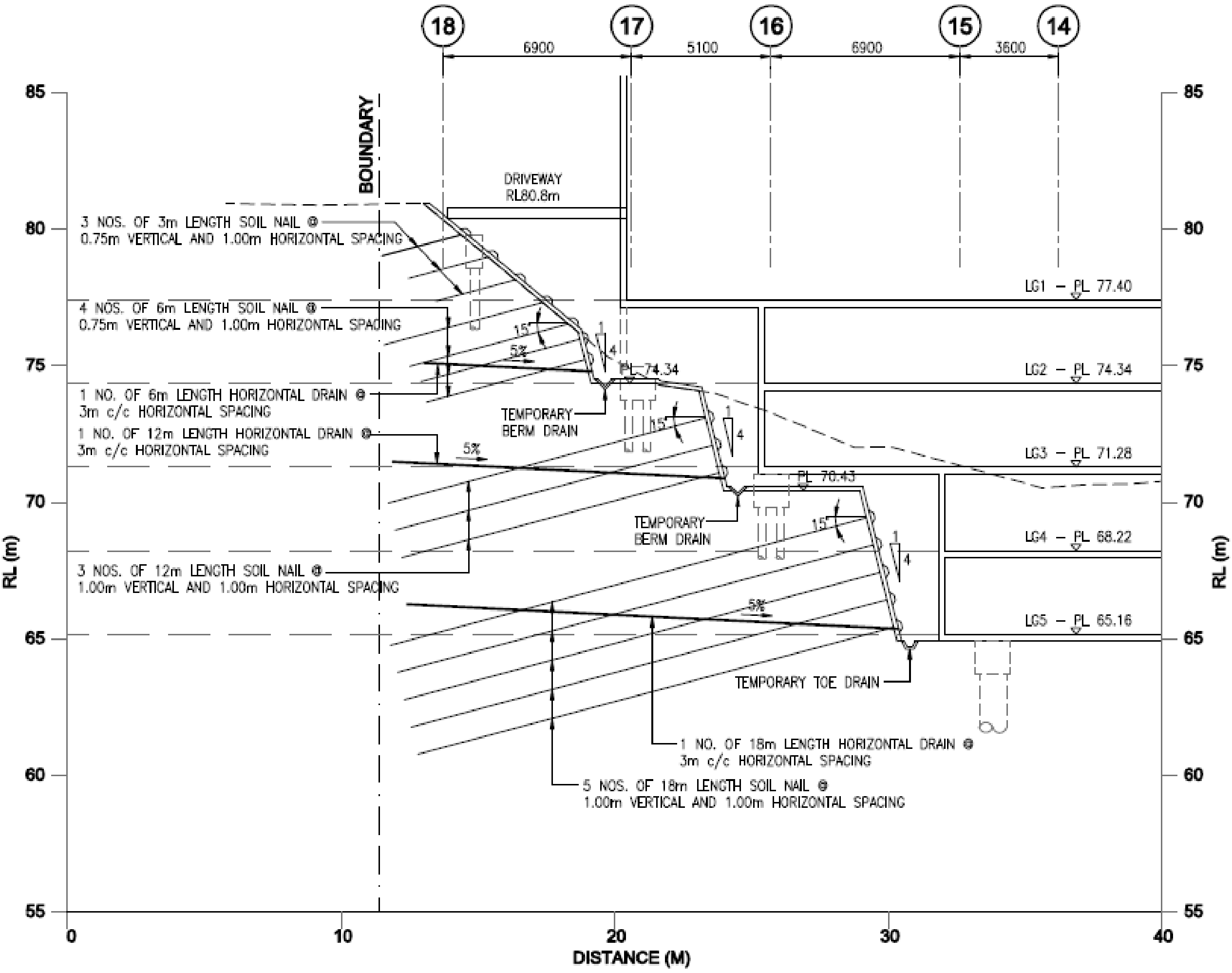

Figure 21: Alternative soil nail system for basement construction.

As can be seen from Figure 21, the basement for the alternative soil nail system is more complicated in order to accommodate the space required for the soil nail. The basement layout has to be adjusted and proper planning of foundation system is also required in order to support the superstructure. The alternative soil nail and foundation system are planned in such a way that the building design above the basement parking is not affected. The required time input for such design is therefore more than conventional design and the system requires careful coordination between the Architect and the Geotechnical Engineer to ensure the design is coordinated. For example, a 3-D SketchUp Model was drawn by the Geotechnical Engineer in order to assist the team in visualizing the soil nail system as shown in Figures 22 and 23.

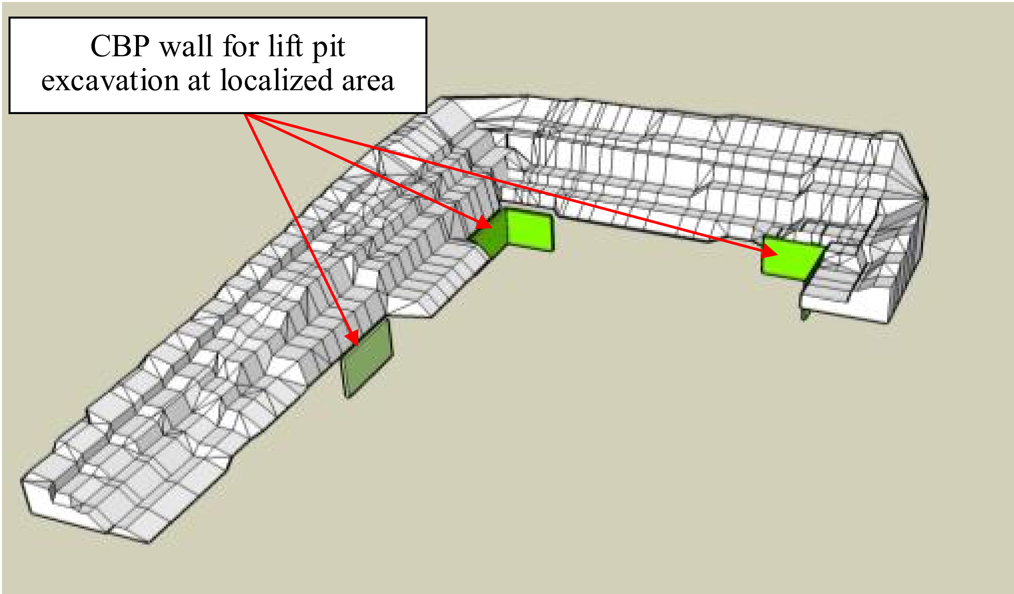

Figure 22: 3-D model of soil nail system.

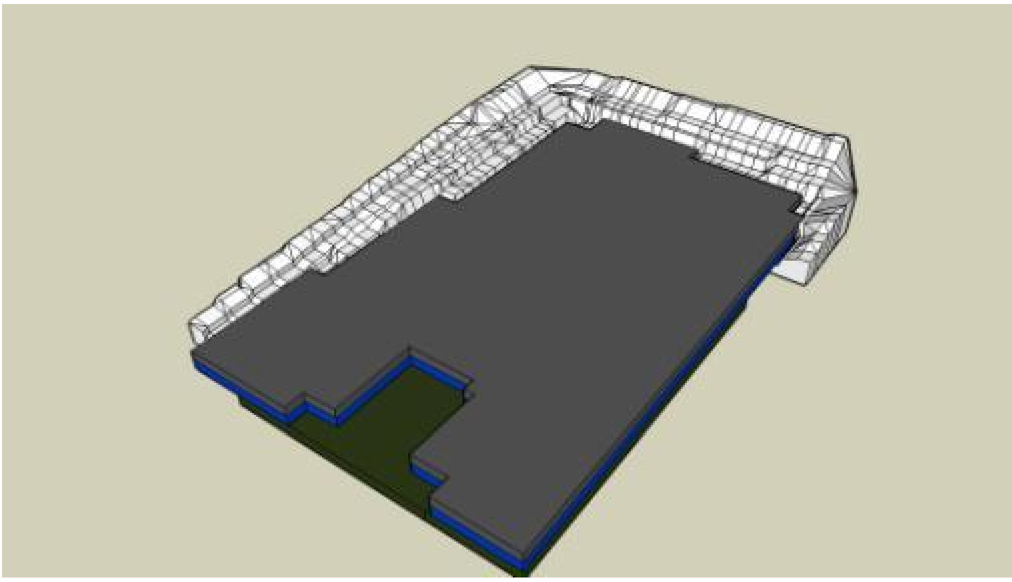

Figure 23: 3-D model showing layout of basement floors superimposed onto soil nail slope.

Due to the collaborative effort of the Architect and Geotechnical Engineer together with a supportive Client, the cost savings by adopting soil nail system to replace conventional retaining wall system is approximately RM19 Million. In summary, this case history illustrates the enormous benefits to a project brought by close cooperation of the Client, Architect and Engineer from the planning stage.

5 CONCLUSIONS

Soil nail system offers a viable and practical alternative for conventional retaining wall system typically adopted for deep excavation works such as diaphragm wall especially for project site with relatively large area (typically more than 6 acres). Soil nail system has demonstrated to offer significant cost and time savings in addition to offering advantages in terms of a robust system with relative ease of construction. In Malaysia, deep basement of up to 30m deep with close proximity of existing sensitive structures has been successfully designed and constructed using soil nail system as alternative to conventional retaining wall system and it is envisaged that more future projects using the same system will be adopted.

The system requires close cooperation between the Architect, Structural Engineer, Geotechnical Engineer and Client and therefore, the participation of the Geotechnical Engineer should begin from early development planning stage in order to optimize the basement design.

ACKNOWLEDGEMENTS

The success of the above projects would not have been possible without the contributions of fellow colleagues, Mr. Lee Seong Tatt, Mr. Choong Kean Wui and Mr. Lim Fang Liang and their contributions are gratefully acknowledged.

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Three-Dimensional Effect Around Corner of Deep Excavation

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ABSTRACT

Geotechnical engineers, being the designer of deep excavations, have been facing the challenges of minimising the impact on existing facilities near the excavations. The prediction of the ground settlement is one of the critical elements in the design of the excavation and lateral support system. It has been known from field measurements that the ground settlement around the corner of excavation can be significantly less than those far from the corner. If this effect can be assessed quantitatively, there may well be great cost saving when implementing measures to control the ground settlement. This paper aims to understand the corner effect of deep excavation better by conducting three-dimensional finite element method analyses on two model deep excavations in local site setting. The results were examined and observations made. The finite element method results were also compared with those derived using empirical method which was reported in the literature.

1 INTRODUCTION

1.1 Deep excavation in Hong Kong

Deep excavation as part of the construction for basement, railway station, cut & cover tunnel is very common in highly urbanised city such as Hong Kong. In Hong Kong, very deep excavations, exceeding 30 m in depth, are not uncommon. These excavations very often will have to be carried out in area surrounded by buried utilities and existing structures both above and below ground.

The geotechnical engineers in Hong Kong have been facing the challenges of minimising the impact on these existing facilities due to the excavation. And a reasonable prediction of the ground settlement is one of the critical elements in the design of the Excavation and Lateral Support (ELS) system for the excavation.

1.2 Prediction of ground settlement

Nowadays, computer programs are available to geotechnical engineers for predicting the ground movement associated with the excavation. These programs commonly adopt numerical methods, such as the finite element method (FEM) to examine the interaction between the ground, the ELS and the existing structures nearby.

In Hong Kong, the Buildings Department of the HKSAR Government provides a list of computer programs which have been pre-accepted for analysing excavation (Buildings Department 2011). Another example is the list of computer programs provided in the CIRIA Report No. C580 for excavation in U.K. (Gaba et al. 2003). In these two lists, two-dimensional analysis remains the main stream type of analysis. The common consensus is that the two-dimensional analysis will provide conservative results and thus, can be safely applied.

On the other hand, geotechnical engineers are also required to enhance the prediction on ground settlement for cost-effectiveness and sometimes for cases where the allowable movement of the existing facilities nearby is extremely tight.

It has been known from field measurements that the ground settlement around the corner of excavation can be significantly less than those far from the corner and it has been the subject of interest and studied by different researchers as described later in Section 2.

This paper aims to understand the corner effect of deep excavation better by conducting three-dimensional (3D) FEM analyses on two model deep excavations in local site setting. The results were examined and observations made. The FEM results were also compared with those derived using empirical method reported in the literature.

2 SOME PREVIOUS WORKS

The ground settlement around corners had been studied mainly with either 3D numerical methods or empirical methods calibrated with case history. The works by Ou et al. (2000) and Zdravkovic et al. (2005) fall into the former category. Fuentes & Devriendt (2010) provided a review of the development of empirical methods and proposed a new empirical method which had been calibrated with some case history.

3 3D FINITE ELEMENT METHOD

3.1 The models

Two model excavations are studied in this paper, which are considered typical in terms of local site setting and design practice of ELS. In both models, A and B, the depth of excavation is about 27 m below the existing ground surface. On plan, Model A is 45 m long and 30 m wide, and Model B, 75 m long and 30 m wide (Figure 1). The length to width ratio of Models A and B are 1.5 and 2.5 respectively. The choices of using different length to width ratios provide some interesting observations as described later.

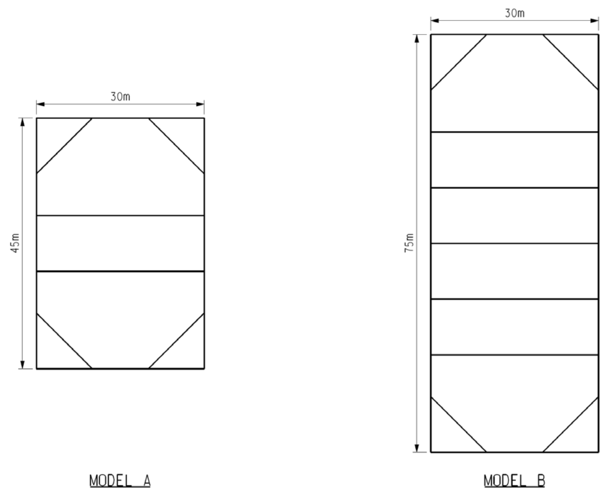

Figure 1: Plan view of model excavations (Models A and B)

The excavation will have to go through a number of soil strata (Figure 2). The soil strata in the models; fill, colluvium, alluvium and completely decomposed granite are commonly encountered in Hong Kong. The groundwater level in the model is assumed to be 1 m below the existing ground level.

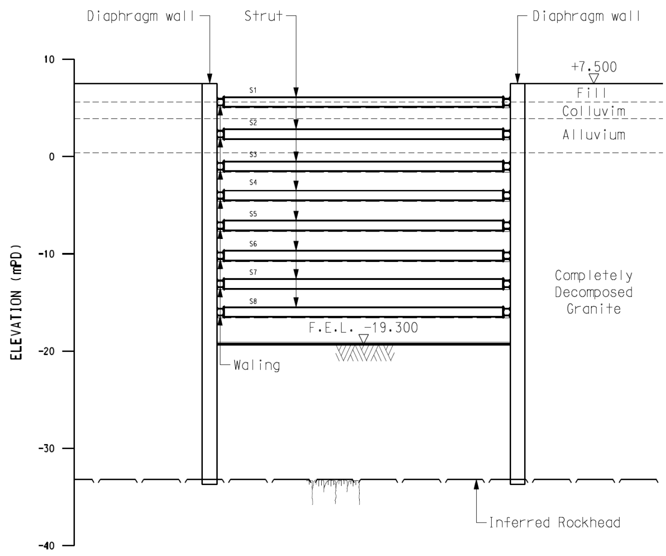

Figure 2: Cross-section of model excavations

The retaining wall is in the form of 1.5 m thick reinforced concrete diaphragm wall resting on bedrock at about 40 m depth. In order to control the ground movement induced due to the excavation, the wall will be heavily strutted, spacing at about 3 m to 4 m in the vertical direction (Figure 2).

3.2 Computer program and analysis

Computer program Plaxis 3D Foundation, version 2.2 is adopted to analyse the above models. The oblique view of the three-dimensional mesh can be seen in Figure 3. Each model, A and B is analysed with the soil within the excavation removed in stages. After each intermediate stage of excavation, the strut immediately above the current excavation level is installed before the next stage of excavation proceeds. The process continues until the final excavation level is reached. For simplicity, only the main struts of the ELS are modelled. In the analyses, only the ground settlement due to excavation is considered. Ground movement due to installation of the diaphragm wall and dewatering outside the excavation are excluded.

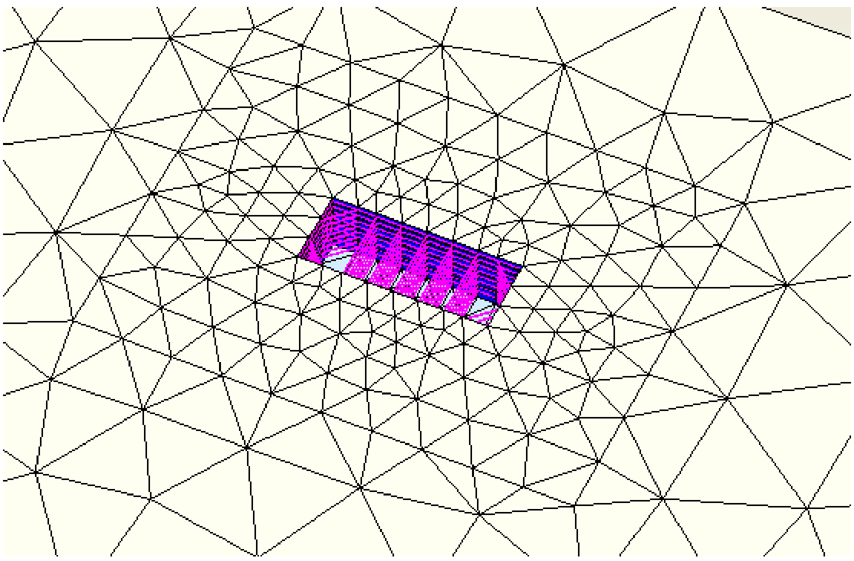

Figure 3: Oblique view of the 3D FEM mesh (Model B)

3.3 The results

Figures 4 and 5 shows the plan views of the ground settlement contours of Models A and B. As envisaged, the ground settlement is significantly less around the corners, for example, 6 mm at the corner against the maximum of 32 mm in Model A. It is also noted that the maximum ground settlement at the short side of the excavation is significantly less than that of the long side, 12 mm against 32 mm in Model A. This is probably due to the stronger influence of the stiff corner to the short side of the excavation. Another observation is that Model B which has a length to width ratio of 2.5 produces a higher ground settlement on the long side than Model A which has a length to width ratio of 1.5 (32 mm and 50 mm for Models A and B respectively). This is expected as a higher length to width ratio will produce a condition closer to the 2-dimensional condition, resulting in higher ground settlement. However, what is unexpected is that even with a length to width ratio of 2.5 (Model B), as far as ground settlement is concerned, the 2-dimensional condition does not seems to have fully developed, as otherwise the ground settlement contours would not vary at least for a certain length of the long side.

A comparison of the ground settlement around the corners of Models A and B is also made (Figure 6). The ground settlements from these two models are very similar and the ground settlement around the corner does not seem to be sensitive to the length to width ratio.

The effective lateral earth pressure profiles behind the wall were also examined. Figure 7 shows the effective lateral earth pressure profiles at different locations of the long side of excavation of Model B. It can be shown that the lateral earth pressure in general increases from the mid-point of the long side (Section 3-3) towards the corners (Section 2-2 and then Section 1-1). This is consistent with the observation that ground settlement around the corner is smaller and therefore the lateral movement of the wall therefore should also be smaller, resulting in less reduction in the lateral earth pressure from the at-rest value.

The bending moment of the diaphragm wall is also one of the critical elements in the design of ELS. Figure 8 shows the bending moment in the minor axis of the wall panel (as typically determined in the 2-dimensional analysis) in Model A. The plot shows a maximum bending moment of about 3100 kNm/m, roughly at 2/3 of the depth of the wall. At the corner, the wall panel is very stiff in that direction and the bending moment there is far smaller. To the contrary, the bending moment in the major axis is at its maximum at the corners as shown in Figure 9. This is due to the ‘closing-in’ of the corner. This maximum bending moment is about 5500 kNm/m which is about 77 % more than the maximum bending moment in the minor axis.

4 COMPARSION OF RESULTS BY 3-D FEM AND EMPIRICAL METHOD

A comparison is made between the results from the 3-dimensional finite element analysis for Model A and those derived using the empirical method suggested by Fuentes & Devriendt (2010). The method by Fuentes & Devriendt requires the input of parameters of p_1^* and p_2^* and the typical values of 67% and 25% respectively were taken in this exercise. Two other key parameters, d_A and d_B, were taken as 15 m and 22.5 m respectively. Figure 10 shows the ground settlement around the corners based on the two different methods.

Both methods show marked reduction of ground settlement at the corner. However, the finite element method predicts more rapid reduction of ground settlement on the long side of Model A than the empirical method, while the reverse is observed for the short side of the excavation. It should be noted that the current comparison is only preliminary and the input data (p_1^*, p_2^*, d_A and d_B) in the empirical method would have to be calibrated with case history to provide a more in depth comparison.

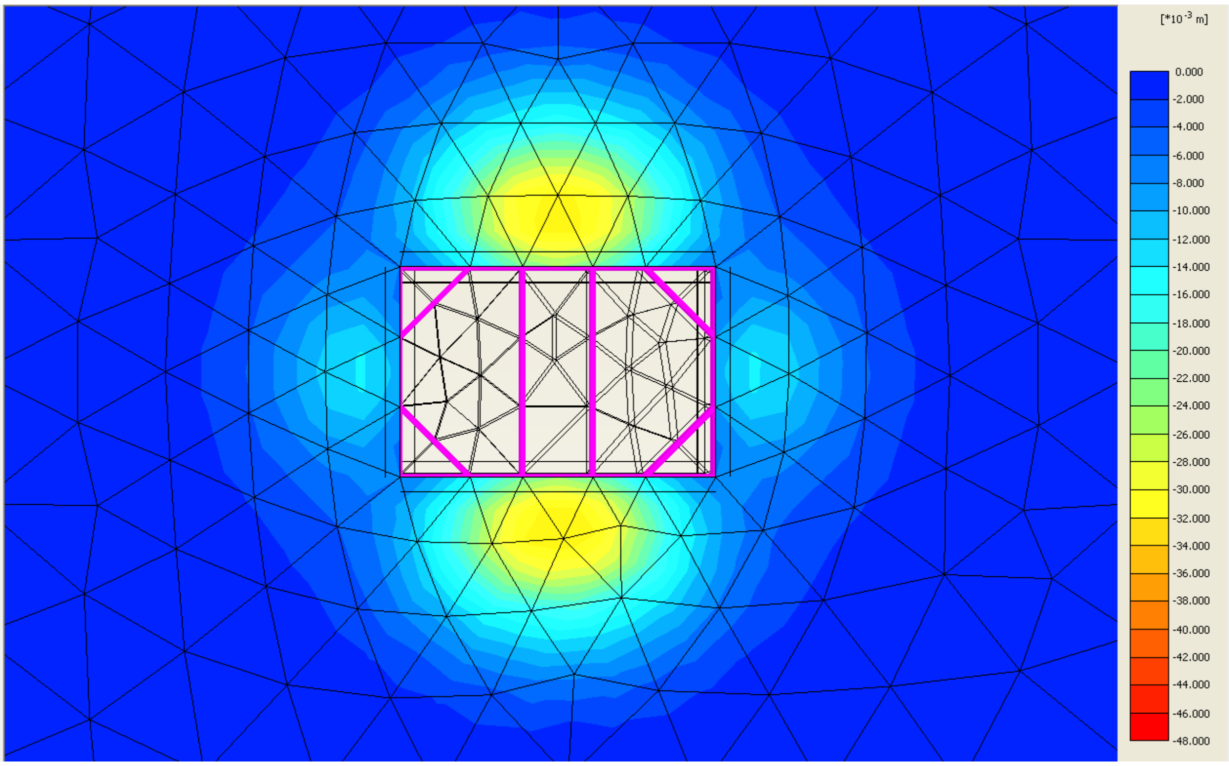

Figure 4: Ground settlement in Model A

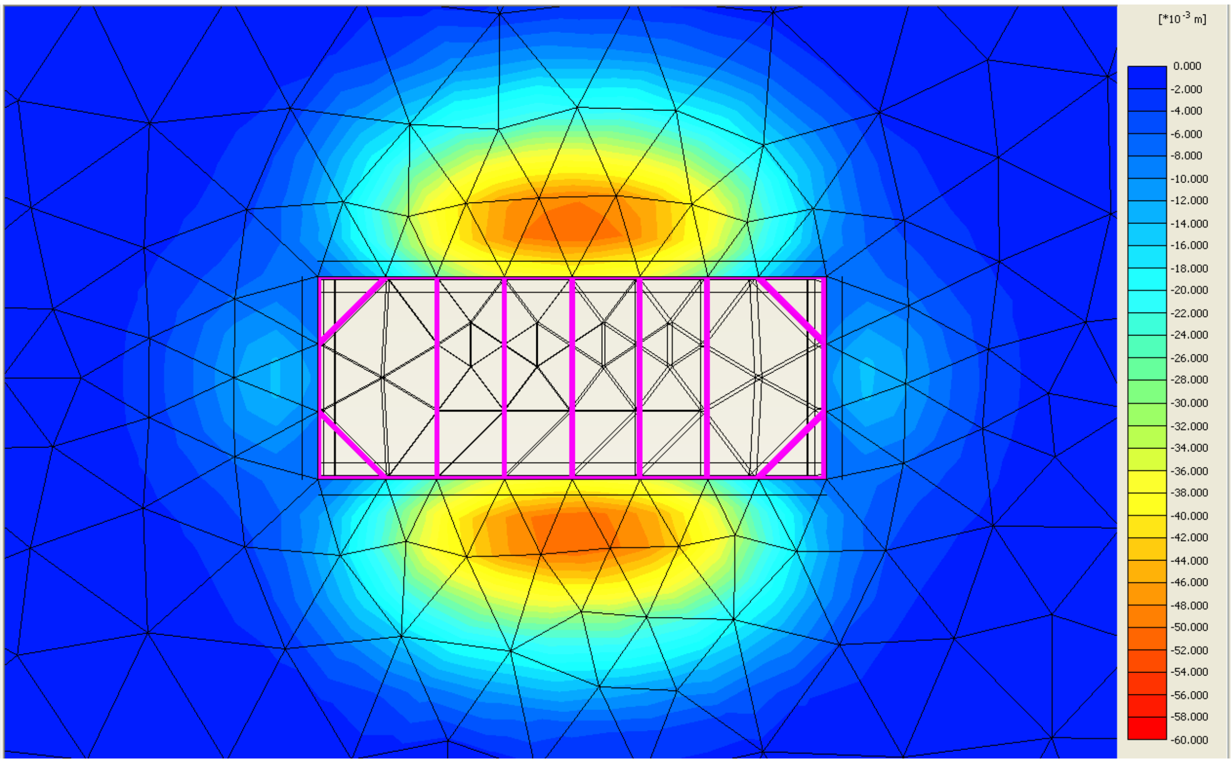

Figure 5: Ground settlement in Model B

3.3 The results

Figures 4 and 5 shows the plan views of the ground settlement contours of Models A and B. As envisaged, the ground settlement is significantly less around the corners, for example, 6 mm at the corner against the maximum of 32 mm in Model A. It is also noted that the maximum ground settlement at the short side of the excavation is significantly less than that of the long side, 12 mm against 32 mm in Model A. This is probably due to the stronger influence of the stiff corner to the short side of the excavation. Another observation is that Model B which has a length to width ratio of 2.5 produces a higher ground settlement on the long side than Model A which has a length to width ratio of 1.5 (32 mm and 50 mm for Models A and B respectively). This is expected as a higher length to width ratio will produce a condition closer to the 2-dimensional condition, resulting in higher ground settlement. However, what is unexpected is that even with a length to width ratio of 2.5 (Model B), as far as ground settlement is concerned, the 2-dimensional condition does not seems to have fully developed, as otherwise the ground settlement contours would not vary at least for a certain length of the long side.

A comparison of the ground settlement around the corners of Models A and B is also made (Figure 6). The ground settlements from these two models are very similar and the ground settlement around the corner does not seem to be sensitive to the length to width ratio.

The effective lateral earth pressure profiles behind the wall were also examined. Figure 7 shows the effective lateral earth pressure profiles at different locations of the long side of excavation of Model B. It can be shown that the lateral earth pressure in general increases from the mid-point of the long side (Section 3-3) towards the corners (Section 2-2 and then Section 1-1). This is consistent with the observation that ground settlement around the corner is smaller and therefore the lateral movement of the wall therefore should also be smaller, resulting in less reduction in the lateral earth pressure from the at-rest value.

The bending moment of the diaphragm wall is also one of the critical elements in the design of ELS. Figure 8 shows the bending moment in the minor axis of the wall panel (as typically determined in the 2-dimensional analysis) in Model A. The plot shows a maximum bending moment of about 3100 kNm/m, roughly at 2/3 of the depth of the wall. At the corner, the wall panel is very stiff in that direction and the bending moment there is far smaller. To the contrary, the bending moment in the major axis is at its maximum at the corners as shown in Figure 9. This is due to the 'closing-in' of the corner. This maximum bending moment is about 5500 kNm/m which is about 77 % more than the maximum bending moment in the minor axis.

4 COMPARSION OF RESULTS BY 3-D FEM AND EMPIRICAL METHOD

A comparison is made between the results from the 3-dimensional finite element analysis for Model A and those derived using the empirical method suggested by Fuentes & Devriendt (2010). The method by Fuentes & Devriendt requires the input of parameters of p_1^* and p_2^* and the typical values of 67% and 25% respectively were taken in this exercise. Two other key parameters, d_A and d_B, were taken as 15 m and 22.5 m respectively. Figure 10 shows the ground settlement around the corners based on the two different methods.

Both methods show marked reduction of ground settlement at the corner. However, the finite element method predicts more rapid reduction of ground settlement on the long side of Model A than the empirical method, while the reverse is observed for the short side of the excavation. It should be noted that the current comparison is only preliminary and the input data (p_1^*, p_2^*, d_A and d_B) in the empirical method would have to be calibrated with case history to provide a more in depth comparison.

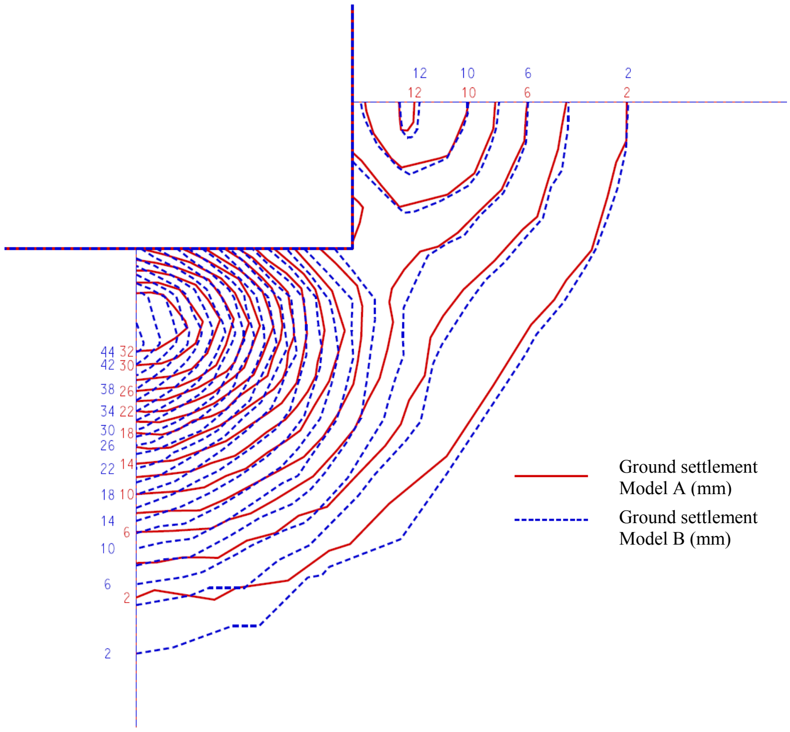

Figure 6: Ground settlements around corner in Models A and B in present 3D FEM analysis

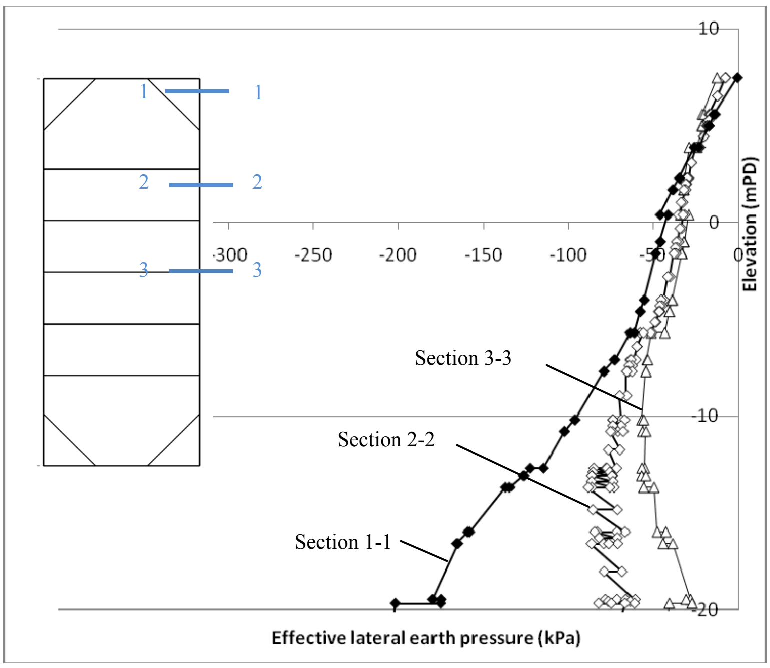

Figure 7: Effective lateral earth pressure behind wall at different locations (Model B)

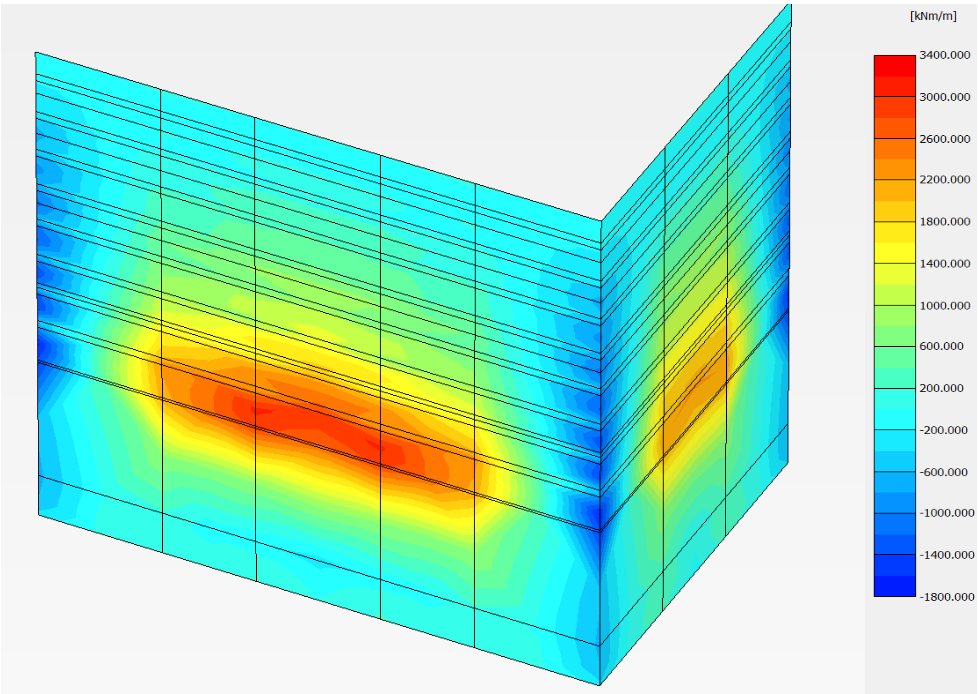

Figure 8: Bending moment of diaphragm wall in minor axis (Model A)

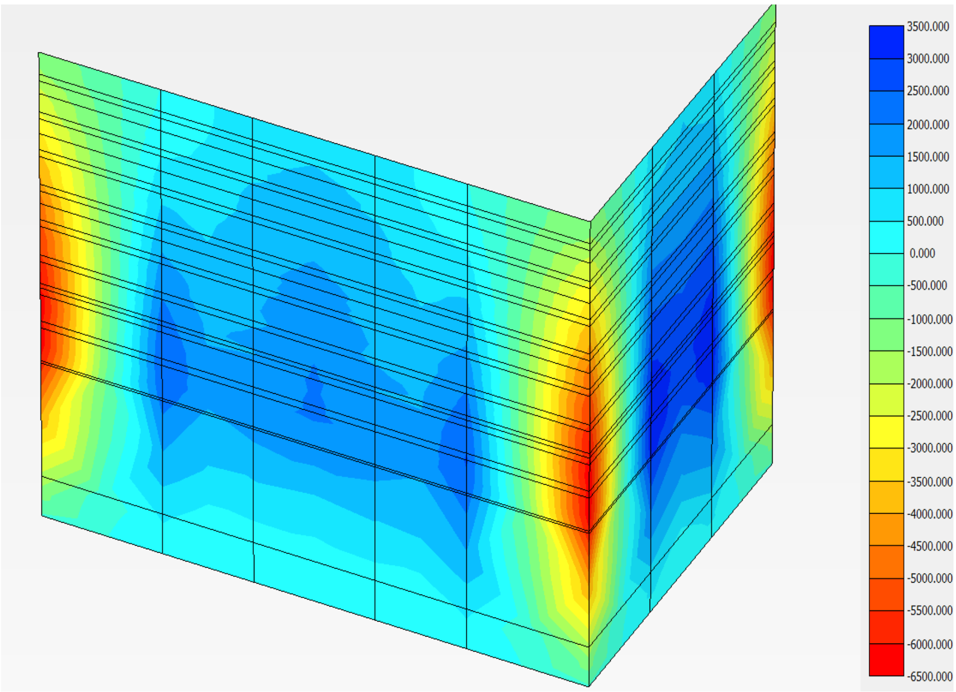

Figure 9: Bending moment of diaphragm wall in major axis (Model A)

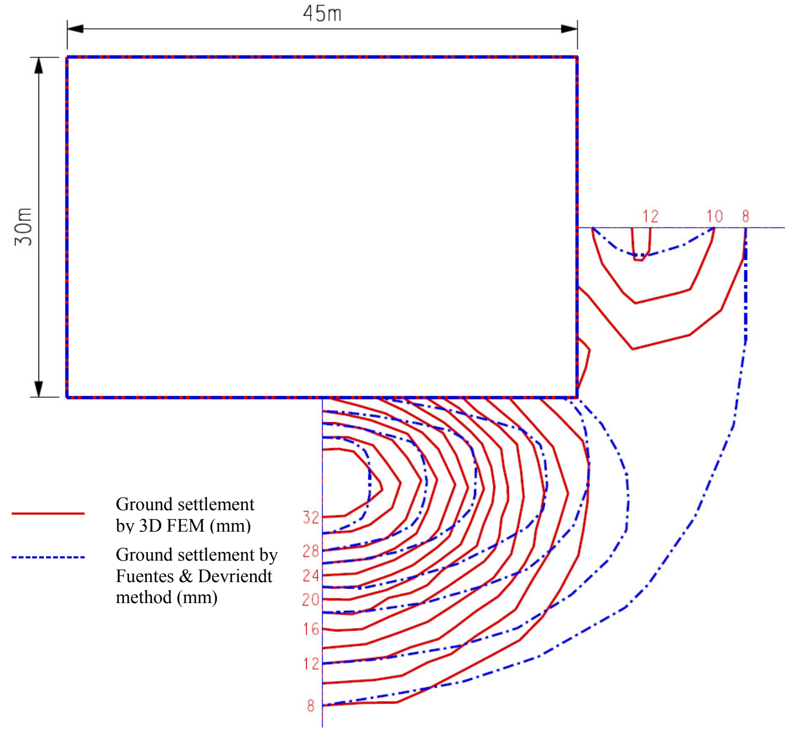

Figure 10: Comparison of ground settlement predicted for Model A with present 3D FEM and empirical method by Fuentes & Devriendt (2010)

5 CONCLUSIONS

This paper studies the corner effect of deep excavation by conducting 3D finite element method analysis on two model excavations in local site setting. The results were examined and also compared with those derived using empirical method suggested by Fuentes & Devriendt (2010). Some observations were made:

1. As expected, the ground movement around the corner can be significantly less than those far away from the corner. The results of the present 3D FEM analysis show that the ground settlement right at the corner can be as low as 19% of the ground settlement far from the corner. In addition, the corner effect can also influence the overall ground settlement behind the wall if the side of the excavation is relatively short, as demonstrated by Model A;
2. Two–dimensional condition may not fully develop even for a excavation with a length to width ratio (on plan) of 2.5; and
3. The bending moment (in the major axis) of wall panel at the corner can exceed the bending moment (in the minor axis) of wall panel far from the corner. This means that the amount of lateral reinforcement to be placed at the corner wall panel would need to account for this particular effect.

As this paper only studies the subject with two model excavations, further study is recommended to investigate whether the above observations are indeed common among other cases. Another area worth studying is finding the controlling factors which govern the ground settlement around the corner.

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Deep Excavations - Industry Challenges

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ABSTRACT

There are many challenges facing the construction industry in the region with improving safety standards the greatest challenge of all. Published statistics suggest, generally, safety standards on construction sites have reached a plateau with no significant improvement in recent years. To drive change, traditional views on roles and responsibilities need to be challenged starting with the designer. Mindsets that site safety is only the Contractors responsibility must change. Designers have a vital role to engineer out risks that could potentially result in accidents during installation, use and dismantling of temporary works. Focusing on making deep excavations easier to build safely, designers must maximise working space, engineering out risks such as accidental impacts on temporary works and promote prefabrication of earthwork supports. Effective communication of design information, obtaining frontline feedback from sites on buildability and harnessing the latest instrumentation and Building Information Modelling technologies are vital. Statutory frameworks must be sufficiently flexible to promote innovation and to keep pace with technological advances as a fundamental to raise industry safety standards.

1 INTRODUCTION

1.1 *Industry safety performance*

Safety standards in the construction industry are far from satisfactory. Government published safety records suggest over the last decade that the number of construction fatalities in Asia Pacific countries has generally reached a plateau (Figure 1). In Hong Kong, accident incident rates have fluctuated within a narrow range of 55 and 65 per thousand workers during the last 7 years (HKSAR Labour Department, 2010).

Falls from height and being trapped or struck by moving objects are consistently the most common causes of fatalities in Hong Kong, accounting for about 85% of industry deaths.

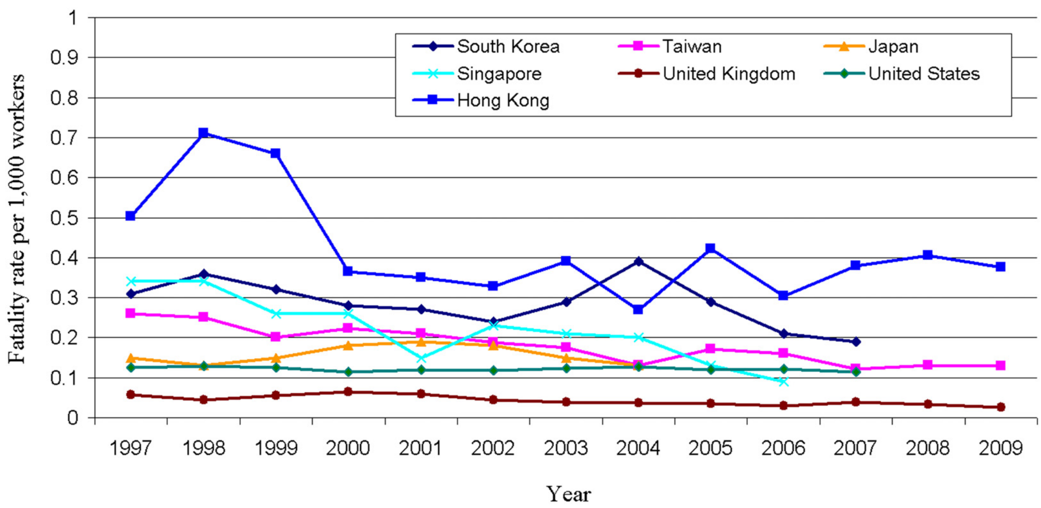

Figure 1: Fatality statistics by region

1.2 Layers of protection

People make mistakes so it is necessary to have several layers of defence to ensure that these mistakes do not translate into accidents. Robust safety management systems have several layers of protection with an accident only occurring when all of these layers are breached. The first layer of protection is design. Reliance on front line workers to protect themselves is the last layer (Figure 2).

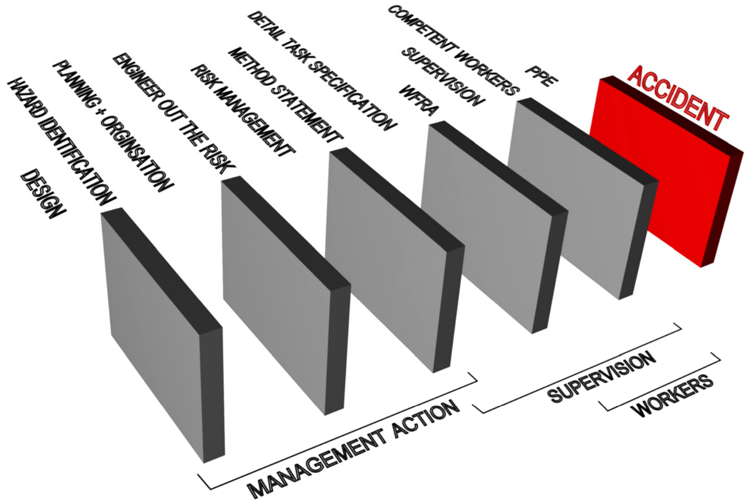

Figure 2: Layers of protection

Worldwide statistics suggest that 60% of major and fatal construction accidents could have been prevented by better application of the design process. It is suggested that a similar ratio will apply to construction projects involving deep excavations.

2 CONSTRUCTION SAFETY

2.1 Designing for safety

Maximising working space inside deep excavations is a key objective to promote safer construction. Using fewer widely spaced struts is favoured to create open working areas and to reduce the number of heavy lifts. Prefabricating struts and temporary working decks off-site is a further positive step in creating a safe construction site.

Plate 1: Maximising working space and prefabrication

Locations for spoil removal should be identified at an early stage so that potential risks of damage to earthwork supports from accidental plant impacts can be designed-out. For example, at mucking-out locations, fender panels can be introduced to protect struts or oversized structural members can be used locally for added protection.

Plate 2: Fender panels for the protection of temporary earthwork supports

Access and egress provisions for workers are rarely shown on design drawings. This is often left to "trade practice". Accidents due to falls from height and plant impacts could be mitigated if access and egress risks are designed-out with details of access walkways and stairs clearly identified on construction drawings.

The wider adoption of observational design approaches should be promoted for deep excavations. Observational approaches, when correctly applied, provide a rational means to remove often inherent conservatism in design parameters without compromising robustness. A further benefit is that observational approaches tend to advance understanding of soil-structure interaction by making designers monitor the actual performance of their designs against predictions.

Consideration of the removal of earthwork supports requires an equal emphasis to the installation. For example, if temporary concrete walings are used and they require demolition, designers should introduce slots, void formers and reinforcement breaks to aid removal. Lifting eyes and cut locations for earthwork support installation and removal should be pre-planned and shown on design drawings.

2.2 Communicating design information

Temporary works for deep excavations are often shown as "wished-in-place" on design drawings. Generally there is no information provided on how earthwork supports such as struts and walings or temporary access decks are to be installed, used or dismantled. This is usually left to "trade practice" and described in generic method statements that are generally of low value for risk management. A desired position would be for every step in the construction cycle to be clearly identified on design drawings, including access provisions for site workers and material delivery. If for operational reasons it is necessary to depart from the drawings, changes will be agreed with the designer to allow construction to progress safely.

It is not always possible to mitigate all risks through design. Residual risks should be identified on design drawings as "Designers Notes" and highlighted, through the use of colour or a text box, so that they stand-out above general notes that tend to be fairly generic.

When presenting information on design drawings it is favoured to use more images and fewer words. Plans, sections and developed elevations are typically shown on deep excavation drawings. Adding isometric views can be effective to show spatial arrangements and connection details for earthwork support systems. Including images of people and plant on design drawings is also a favoured practice since it can trigger designers and planners to re-evaluate working space provided.

Designers must encourage feedback on buildability from those engaged in the construction of their designs. Ideally dialogue should start during the early stages of design development and continue through the entire construction cycle. Traditional procurement approaches, where designs are produced with little or no contractor engagement, competitively tendered and awarded shortly before site works commence do not drive safer construction. Designs produced by designers integrated within construction teams tend to be more efficient and easier to build safely.

During the construction of deep excavations, design changes are often necessary due to unexpected ground conditions, utilities and because of differences between as-built conditions and construction drawings. Changes must be referred back to the designers to ensure that appropriate checks are conducted so that the work can proceed safely. Internet based systems, such as Gammon's GEMS, allow the electronic issue and endorsement of all design changes as part of broader platform for temporary works control. Systems that rely upon paper "change requests" tend to be less effective because they often suffer communication lags and changes are difficult to track.

2.3 Harnessing technology

There have been major advances in instrumentation for ground and structure monitoring, particularly during the last decade, with the development of ever more powerful microelectronics and communication networks. Innovation in instrumentation will continue to accelerate with more accurate devices available at reducing cost.

Development of new sensor technology, such as MEMS (micro-electro-mechanical systems) sensors, is expected to lead to an order of magnitude reduction in the cost of sensing devices. This, coupled with continuing advances in communication systems, will allow the evolution of current automated strain and tilt-sensing systems into ubiquitous wireless sensor networks.

Other developments, likely to improve the speed and ease of installation of monitoring devices for deep excavations, include the use of a digital camera and image processing techniques to detect movements of selected features in a two dimensional plane. Pairs of cameras linked to sophisticated processing systems will allow the use of terrestrial photogrammetric techniques to determine the movements of points in 3 dimensional space. An important advantage of these remote monitoring systems is expected to be that physical access to the points being monitored is not necessary.

It is common that architectural changes in the layouts of underground structures occur after the installation of the temporary earthwork supports. This can result in positional clashes that are often remedied on site by *ad hoc* modifications to the temporary works. Building Information Modelling (BIM) can be used for the early detection and designing-out of clashes, allowing construction to proceed in a more controlled, planned and therefore safe manner (Figure 3).

Plate 3: Clash between temporary and permanent works

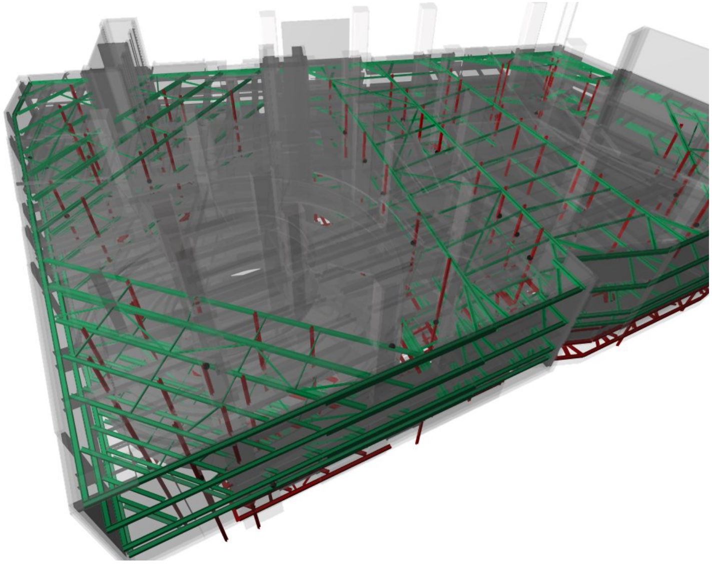

Figure 3: BIM model of Hennessy Centre, Hong Kong

Statutory frameworks must be sufficiently flexible to allow advances in instrumentation and modelling to be harnessed by industry. For example, advances in instrumentation should promote the wider application of observational design approaches for deep excavations. This is not happening. Most statutory frameworks are tailored towards having only one approved design outcome rather than a framework of design decisions based on field observations. Similarly, it is becoming common to have web-based access to instrumentation databases yet there are still onerous contractual and statutory requirements to have daily or weekly hard copy reports circulated.

3 CONCLUSIONS

Improving safety on construction sites is a major challenge for the industry across Asia Pacific. For all projects, including those involving deep excavations, designers have a leading role to make projects easier to build safely. A degree of historical reluctance on the part of designers, to bear a higher level of responsibility for the safety of those who build their designs, will need to be overcome. Raising the emphasis in education and training of temporary works design, since they are almost always safety critical, would be a positive step to change this mindset.

An industry culture that encourages designers and contractors to invest in the latest technologies should be promoted. In the context of deep excavations, instrumentation and modelling tools are developing at ever increasing rates. Harnessing such technology to deliver safer and more efficient construction must be a shared objective of all those involved in the industry.

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Circular Excavations Using Diaphragm Walls

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ABSTRACT

Circular excavations using diaphragm walls are very attractive as they do not require any internal propping which allows both the excavation and the subsequent construction of the below ground structure to be progressed without hindrance. Clearly there are design issues however in deciding what thickness of diaphragm wall is required and what limitations need to be imposed to prevent the possibility of instability due to egg shaping of the wall in plan. This paper initially presents a series of case histories in Hong Kong and Singapore. In all the cases the walls performed very well with very small lateral movements being observed during the excavations. It then goes on to discuss the design methods using the Singapore project as an example as this case history is the most recent and the most challenging. It is demonstrated that while hand calculations and/or 2 dimensional analysis gives a good start to the design, 3D analyses are necessary to be able to confidently assess the possibility of instability due to egg shaping. Discussion is also included as to the use of shear capacity between adjacent panels that is available due to the inherent inter-panel compression generated by the geometry of these walls

1 INTRODUCTION

Circular excavations using diaphragm walls are very attractive as they do not require any internal propping which allows both the excavation and the subsequent construction of the below ground structure to be progressed without hindrance. Clearly there are design issues however in deciding what thickness of diaphragm wall is required and what limitations need to be imposed to prevent the possibility of instability due to egg shaping of the wall in plan. This paper initially presents a series of case histories in Hong Kong and Singapore. It then goes on to discuss the design methods using the Singapore project as an example as this case history is the most recent and the most challenging.

2 CASE HISTORIES IN HONG KONG

2.1 Cheong Kong Center, Central

The Cheong Kong Center made use of a 1.2m thick circular diaphragm wall for the excavation to be progressed for the main core of this 62 storey building. The internal diameter of the shaft was 37m and the depth of excavation was 28m. It should be noted that rock was at the base of the excavation and the core of the building was founded on a large pad footing at the base of the excavation. Plate 1 shows an image of excavation when it was near completion in late 1997. There was a 1m deep capping beam constructed at the top of the diaphragm wall panels and it can be seen towards the base of the excavation that there were a series of ring beams all about 1m high by about 1m thick.

While the core could be constructed within the circular excavation the surrounding mega columns were each founded within their own 4m diameter machine dug caisson to enable 7m diameter circular footings to be founded onto rock as shown in Plate 2. It should also be noted that there was also a 1.2m thick perimeter diaphragm wall to support the overall basement that was formed later in the project. This meant that the circular diaphragm wall was completely removed as part as the basement was subsequently excavated.

Plate 1: View into the excavation for the core of the Cheung Kong Centre

Plate 2: View of the core and mega columns prior to the excavation of the overall basement at the Cheong Kong Centre

2.2 International Finance Centre 2, Central

The International Finance Centre 2 (IFC2) made use of a 1.5m thick circular diaphragm wall for the excavation to be progressed for the main core and the mega columns of this 88 storey building. The internal diameter of the shaft was 61.5m and the depth of excavation was 35m. Rock was encountered at the base of the excavation except on one side where there was about 5m of completely decomposed granite at the base. A single 6m thick raft foundation was formed under the core wall and the 8 mega columns. Several barrettes had been pre-installed to support the foundation at the zone where decomposed granite was encountered. Plate 3 shows a general view of the excavation when it was almost complete in 2000. In the same way as for the Cheong Kong Centre the circular diaphragm wall was demolished as the overall basement was subsequently excavated.

Plate 3: View of the excavation for the International Finance Centre 2

2.3 International Commerce Centre, West Kowloon

The International Commerce Centre (ICC) again made use of a 1.5m thick circular diaphragm wall for the excavation to be progressed for the main core and the mega columns of this 118 storey building. The internal diameter of the shaft was 76m and the excavation depth was 26m deep. Rockhead was very deep at this site and the building core and mega columns were founded on a 6m thick raft supported by 240 shaft grouted barrettes that extended about 50m below the excavation. The circular diaphragm wall was also used to help support the structure. Plate 4 shows a general view of the excavation that is adjacent to the MTR Kowloon Station and Plate 5 a view looking down on the 240 shaft grouted barrettes. It can be seen that there is a capping beam and two ring beams installed. It should also be noted that this wall was not removed except for the upper few metres.

Plate 4: Overview of the excavation for the International Commerce Centre

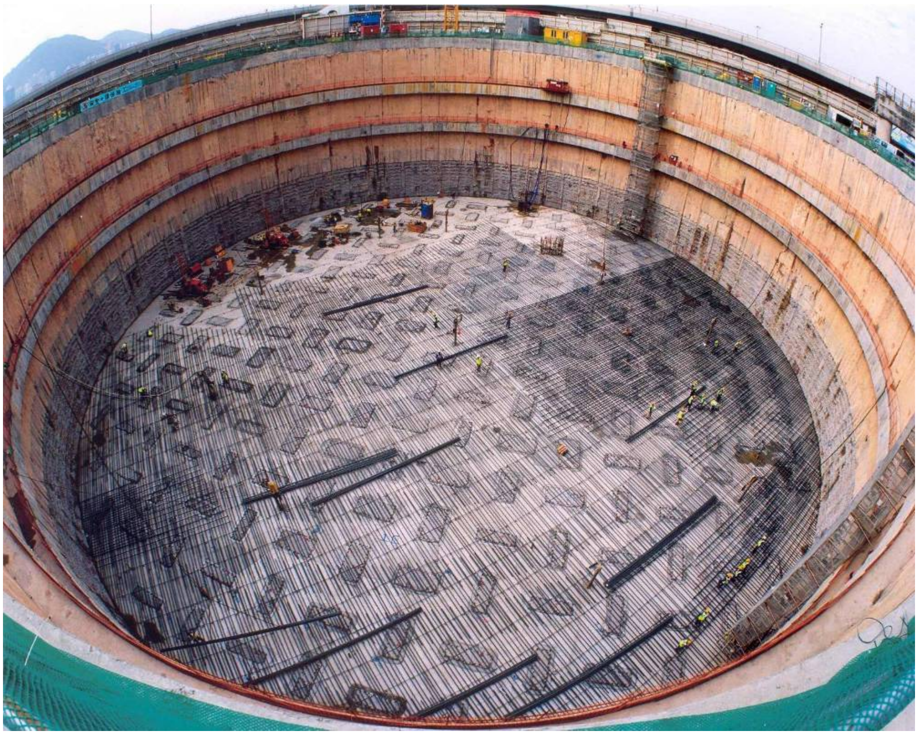

Plate 5: The 240 barrettes supporting the International Commerce Centre

3 CASE HISTORY IN SINGAPORE

The recently completed Marina Bay Sands Integrated Resort made extensive use of circular cofferdams. The project is divided into various zones as shown below in Figure 1. It should be noted that ground level at the site is at about 104mRL and the site is underlain by about 10m of sand fill over a variable thickness of soft clay over a hard material referred to as Old Alluvium. The surrounding water level and the ground water level within the site are at about 100.5mRL, about 3.5m below the ground surface. The contours on Figure 1 refer to the level of the top of the Old Alluvium and show that the depth to the top of this layer varies from a maximum of 45m at the Southern and north east corner to a minimum of about 20m at the north western part of the site. The entire project is founded on friction bored piles that extend into the Old Alluvium to generate the bearing capacity.

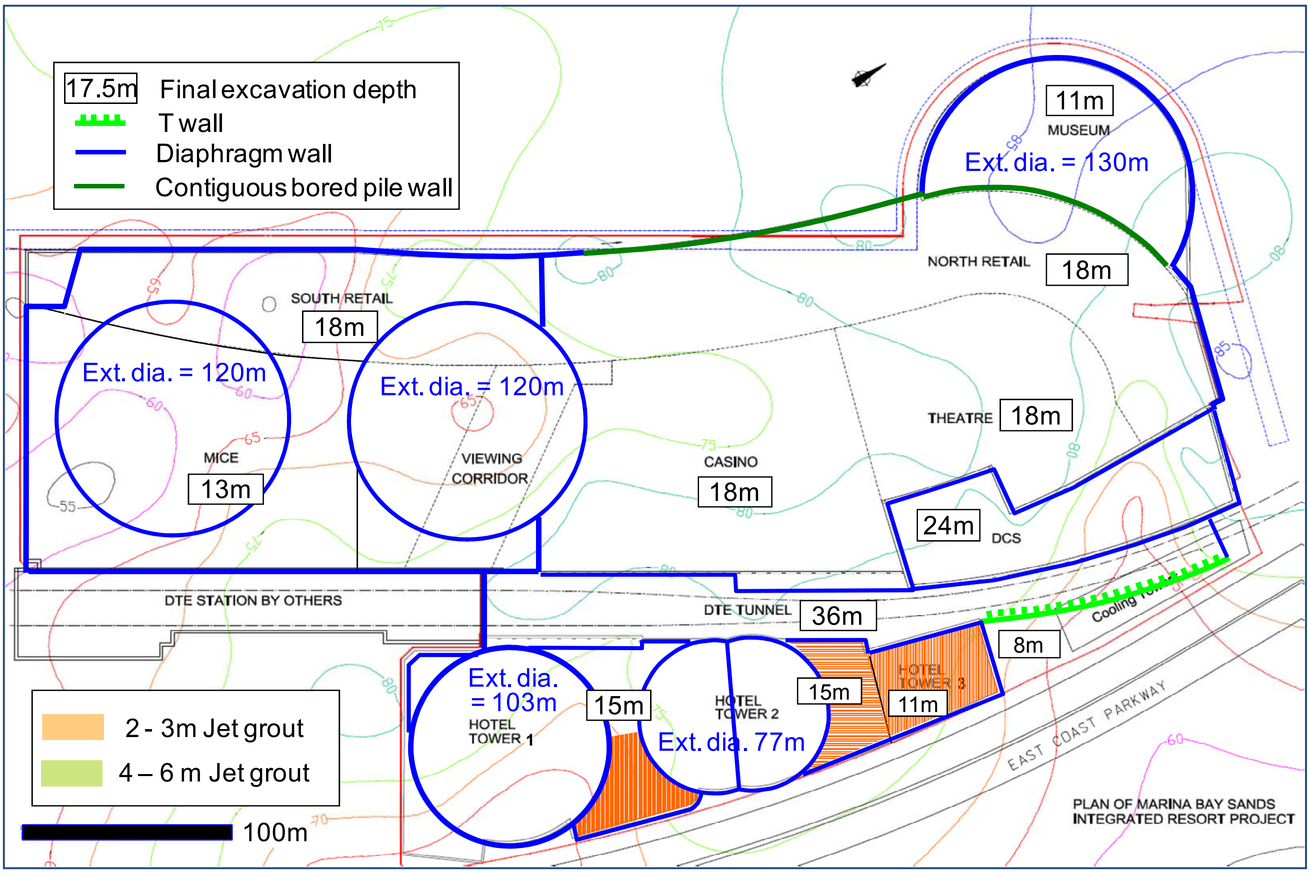

Figure 1: Plan of the Marina Bay Sands Integrated Resort

It can be seen on Figure 1 that there are a total of five circular excavations in the project. In the Hotel area to the eastern side there is a 103m diameter circular excavation and a 77m dual circular excavation affectionately referred to as the ‘Peanut’. Both of these were comprised 1.5m thick diaphragm walls and were excavated to a depth of 15m. At the south western part of the site there are two conventional circular excavations of 120m diameter that were excavated to depths of 13m and 18m. In the Museum area at the north western corner there is a 130m diameter partial circle that was excavated to 11m. An overview photograph of the project taken from the south west is shown in Plate 6 and a close up of the northern 120m circular excavation in Plate 7.

Please note that all of the diaphragm walls forming circular cofferdams that were internal to the perimeter wall were eventually removed down to the base of the excavation. This was achieved by cutting them into blocks using a wire cutter (Plate 8 shows a partially removed wall in the Hotel area). While in the Hotel area this was done as the external excavation progressed beyond them, the walls of the circular cofferdams on the south western part were kept to provide support to the propping that was supporting the perimeter 1.5m thick diaphragm wall. At the 13m deep excavation at the southern boundary, for example, a single level of props at a depth of 3m was provided to support the external wall. This prop (shown schematically in Figure 2 and in Plate 9) was supported by the 2m wide by 2m deep capping beam constructed at the top of the 120m diameter circular diaphragm wall. It is arranged in a truss to enhance the stability of the circular walls during the external excavation.

Plate 6: View of the Marina Bay Sands Integrated Resort taken from the south west showing the two 120m diameter circular excavations in the lower foreground, the Hotel Peanut in the upper centre and the 130m diameter outline on the left hand side

Plate 7: View within the excavation of the northern 120m diameter circular excavation

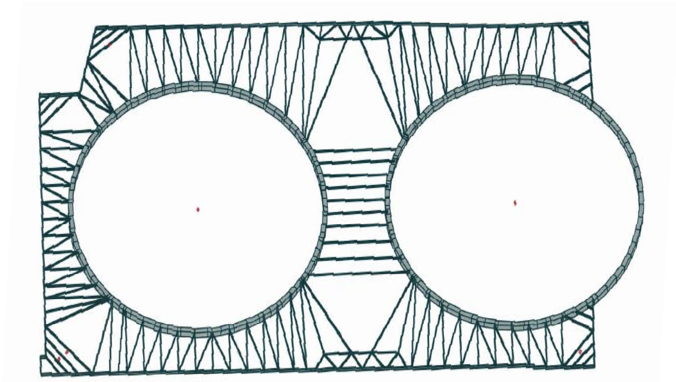

Figure 2: Schematic diagram of prop at 3m below ground level that spans between the two 120m diameter circular walls and the perimeter diaphragm wall

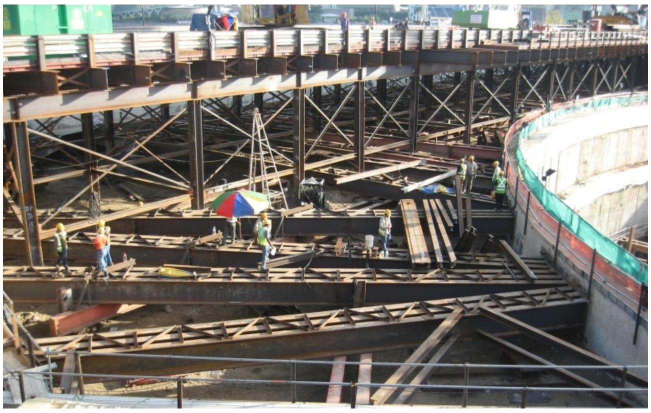

Plate 9: View of the prop between the perimeter wall and the 120m diameter circular wall

Plate 8: View of the Hotel 103m diameter circular diaphragm wall being removed by wire cutting into blocks

4 DESIGN CONSIDERATIONS

It is important to note that all of the circular walls described in the above case histories performed very well. In all cases the lateral displacement was small generally very much less than 20mm. One example is ICC cofferdam and the measurement is shown in Figure 3.

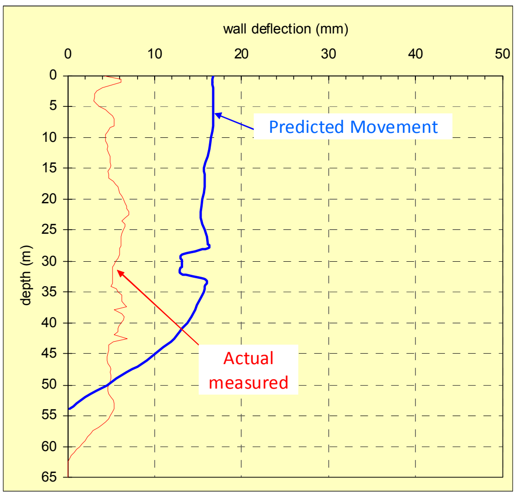

Figure 3: Comparison of predicted and actual lateral displacement of the circular cofferdam at ICC

In order to assure such performance, the key issues that need to be considered in the design of these circular excavations are outlined in the following sections. The depth of the toe of the wall is not discussed as it will generally be determined based on considerations of hydraulic stability of the soil in a similar way to that for straight walls.

4.1 Hoop compression

A critical element in the design of the walls is to ensure that the hoop compression capacity of the diaphragm walls panels is not exceeded. The compression stress in the wall is equal to total horizontal stress in the ground multiplied by the radius of the excavation and divided by the thickness of the wall. The highest compression stress occurs at the base of the excavation. Given that the walls move very little the horizontal stress in the ground acting on the outside of the wall can be assumed to be the at rest (Ko) condition. It follows that the compression force is closely related to the depth of the excavation multiplied by its diameter. Of the case histories presented above it can be determined that both the 120m diameter wall in Singapore and the IFC2 wall are working to a similar stress level being about 10% greater than the wall at the ICC.

Considerations must be made of the vertical tolerance of the walls. If two adjacent panels become offset from each other clearly the concrete compression stress at that location will increase. While it has been found that diaphragm wall panels tend to follow each other down this cannot be relied upon. Experience shows however that by using a hydrofraise/hydromill system with real time verticality monitoring a tolerance of better than 1 in 200 is readily achievable. While internal ring beams have often been used they are actually not effective in reducing the hoop compression in the wall. They are useful however if there is a defect in any of the walls.

Since the cofferdam is formed by discrete rectangular diaphragm wall panels, the cleaningness at the joints between the panels has significant influence to the hoop stiffness. A cutter joint by milling into the completed two primary panels at either side by a hydrofraise /hydromill is preferred compared to the use of conventional stop ends.

4.2 Variations in ground conditions

If the ground conditions change noticeable around the excavation this should be considered in the design. This can be achieved by either carrying out a full 3D finite element analysis directly or by a simplified procedure using a conventional 2D soil-structure interaction analysis combined with a 3D structural analysis. Figures 3 and 4 illustrate this latter procedure. A conventional *Oasys* FREW analysis was carried out to determine the forces in the wall and also to determine how stiff the external soil is to small lateral movements of the wall. These are determined at various sections around the wall and the soil springs transferred to the SAP 3D model of the wall as a whole. The net soil pressures (enhanced to allow for the soil springs) were then applied to the SAP model to predict the final deflected shape.

4.3 Control of excavation geometry within and outside the circular wall

If the internal excavation is perfectly horizontal at all stages then no out-of-balance forces within the cofferdam be generated. Clearly this is likely to be impractical however especially in the initial parts of the excavation where the preferred method of spoil removal is directly by vehicle from within the excavation (see Plate 10).

While this problem of uneven excavation either within or outside of the circular wall can be readily studied directly using 3D soil structure finite element analysis programs preliminary analyses can be carried out using 2D finite element analyses in plan. Figure 5 shows examples of studies carried out of the two 120m diameter excavations to investigate their effects on each other and also of using an excavation ramp. It must be emphasised that these 2D analyses are really quite conservative as they neglect the stiffening effects of the walls extending to deeper levels where the effects of the uneven excavations will be reduced.

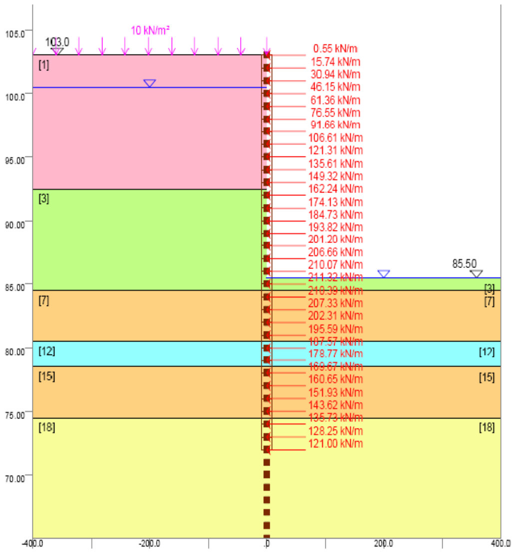

Figure 3: *Oasys* FREW analysis of a wall section

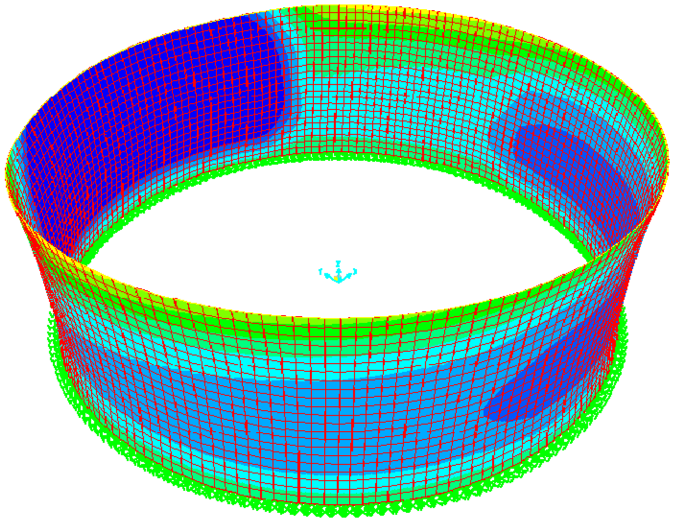

Figure 4: SAP analysis showing the inwards wall deflection.

Plate 10: View showing vehicle ramp within the northern 120m diameter circular excavation with Hotel Peanut and 103m circular excavation in the background

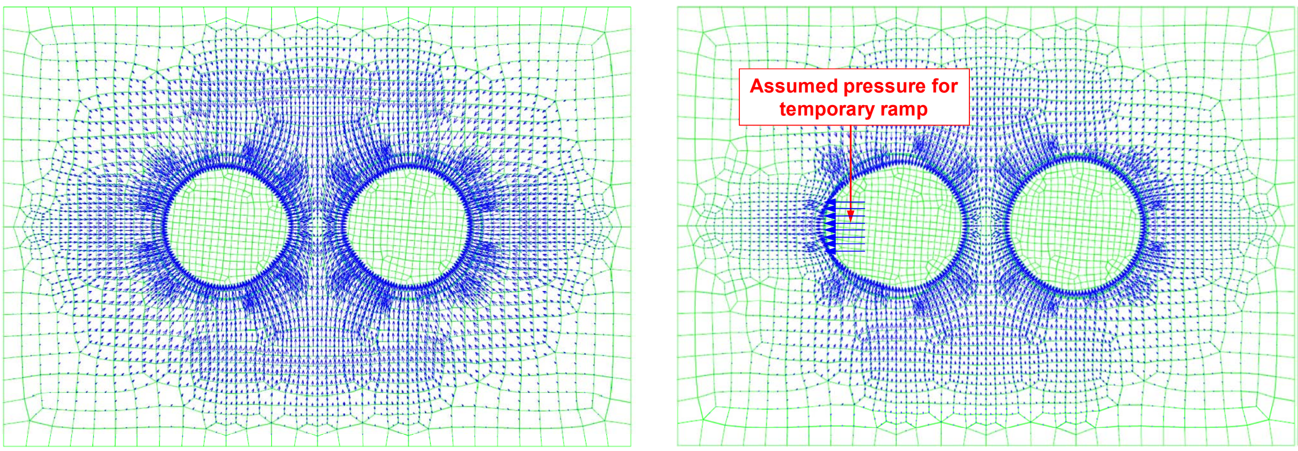

Figure 5: Examples of *Oasys* SAFE 2D plan analyses carried out to examine the effect of the two 120m diameter excavations on each other and to investigate the effect of a ramp within one excavation

There was a particular issue at the northern 120m diameter wall of the Singapore project in that the northern part, together with the attached wing walls, formed a boundary between two independent contracts and, while the internal excavation would be completed, it was uncertain which of the external excavations would be carried out first. The design therefore had to check both possibilities and a 3D finite element analysis using LS-DYNA was carried out to do this. Figure 6 shows the arrangement of the analysis.

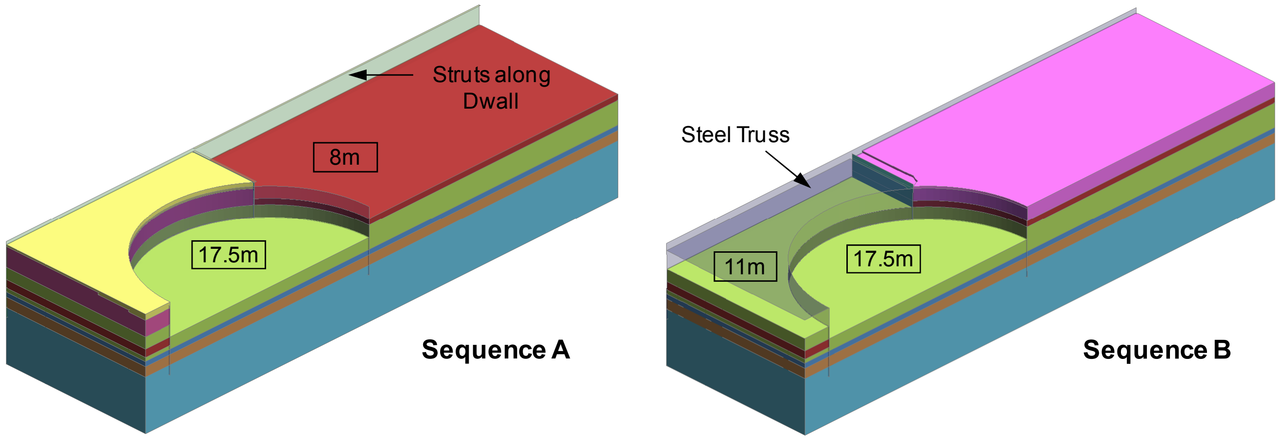

Figure 6: Layout of the *Oasys* LS-DYNA analyses carried out to examine the effect of the two 1external excavation sequences that may have been applied to the northern 120m diameter excavation

4.4 Use of diaphragm wall panels in shear

One feature of diaphragm walls that was made use of in the Singapore project was the use of the shear capacity of the walls. Often, due to the use of bentonite slurry in their construction, designers consider that the friction capacity between adjacent wall panels is not reliable and should be neglected. While this is reasonable if there is no compression forces between the walls it is considered to be overly conservative for circular walls where, due to their geometry, there can be significant compression as discussed previously Section 4.1.

The Singapore project relied heavily on friction between the wall panels at two locations, the Hotel Peanut and the Museum partial 130m diameter wall. In both cases it was assumed that the guidance within BS8110 for the friction angle across a plain concrete surface could be relied upon. SAP analyses were therefore carried out where the wall panel boundaries were explicitly modelled and the shear capacity was controlled by the compression force at each location. As SAP is a linear program this adjustment for the shear capacity at each location had to be iteratively carried out. Figure 7 shows the output of the SAP analysis for the Hotel Peanut. This design issue was also checked by Bachy Soletanche who were, quite reasonably, concerned that their walls may be overstressed by this excavation and used Plaxis Foundation 3D to directly model the problem and predicted similar shear stresses in the wall.

5 CONCLUSIONS

This paper has presented a series of case histories of the use of diaphragm walls to facilitate un-strutted circular excavations in Hong Kong and in Singapore. The walls behaved very well in all cases showing small lateral deformations and enabling rapid excavation and subsequent construction of the below ground structures.

The various design considerations that may be required for these walls are also outlined. It is demonstrated that while hand calculations and/or 2 dimensional analysis gives a good start to the design, 3D analyses are necessary to be able to confidently assess the possibility of instability due to egg shaping. Discussion is also included as to the use of shear capacity between adjacent panels that is available due to the inherent inter-panel compression generated by the geometry of these walls

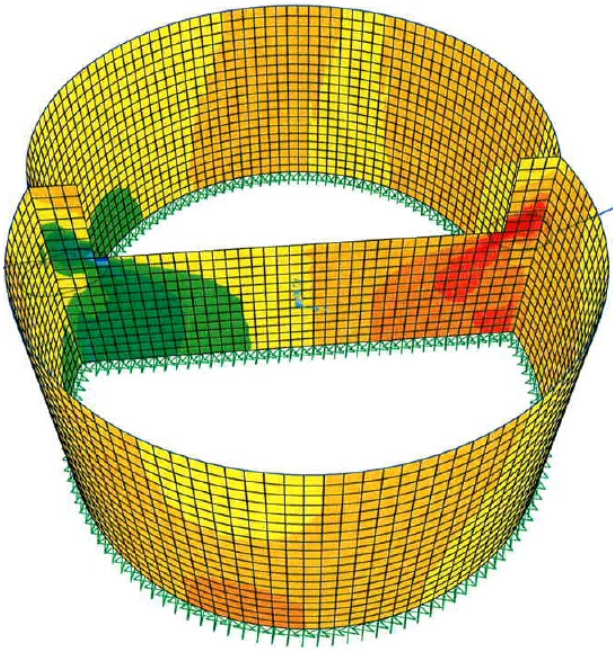

Figure 7: Output for in plane shear stress from the SAP analysis of the Hotel Peanut

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