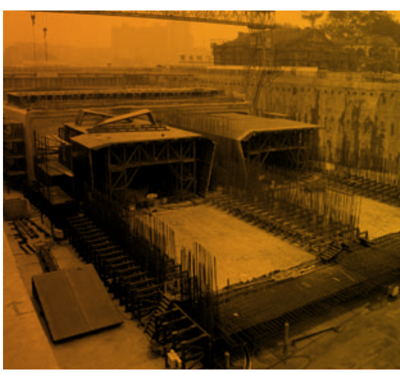
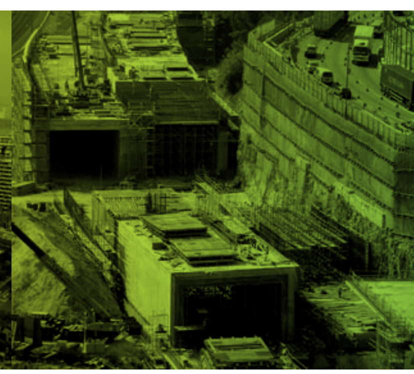
6 May 2010 / **Hong Kong**

Geotechnical Aspects of Deep Excavation



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Geotechnical Society

Proceedings of the 30th Annual Seminar
Geotechnical Division, The Hong Kong Institution of Engineers

Geotechnical Aspects of Deep Excavation

6 May 2010
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Captions of Figures on the Front Cover

Top figure: KDB200 - Cut and cover tunnel at Salisbury Road (Courtesy of Lambeth Associates Ltd.)

Bottom figures: Left to right:

1) Guangzhou immersed tube tunnel dry dock (Courtesy of AECOM Asia Co Ltd.)
2) Deep excavation of basement for commercial development in Shanghai (Courtesy of AECOM Asia Co Ltd.)
3) Deep excavation of PARC-1 development, Seoul, South Korea (Courtesy of Ove Arup & Partners Hong Kong Ltd.)
4) T3 Trunk Road development (Courtesy of AECOM Asia Co Ltd.)

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Foreword

This is the 30th Annual Seminar of the Geotechnical Division. Over the years, the Division has organized many popular seminars on various subjects. These seminars provided a platform for local practitioners and researchers to share their experience. The papers published in the proceedings are a valuable asset for knowledge management of the local geotechnical expertise.

This year, the Organising Committee has chosen the subject of deep excavation as the theme of our Annual Seminar. I believe deep excavation under complex ground conditions in the urban area is the most challenging area of geotechnical works. Such project demands close collaboration between professionals of different disciplines. It often calls for sophisticated investigation, modeling and analysis. Design has to be done diligently with due attention paid to detailing. The construction requires careful planning and monitoring. There is ample opportunity for innovation. Against this background this Seminar aims at bringing together experts on research and study, design, construction and performance review of deep excavation, for active interaction. I trust this seminar will achieve its objective.

On behalf of the Geotechnical Division, I would like to thank our Guest-of-Honour, Mr C K Au, JP; the Keynote Speaker, Ir Dr John Endicott; and the authors of papers for their valuable contribution to this seminar. The supports of our sponsors are also gratefully acknowledged. Lastly, I must thank the Organising Committee, under the leadership of Ir Tony Cheung, for their strenuous work in making this seminar possible.

Ir W K Pun
Chairman, Geotechnical Division (2009/10 session)
The Hong Kong Institution of Engineers

May 2010

Acknowledgements

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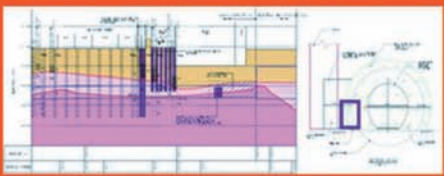

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Earth Lateral Support, Myths and Reality

John Endicott
AECOM Asia Co. Ltd., Hong Kong

ABSTRACT

In Hong Kong there have been many deep excavations for deep basements and underground railway structures that have been constructed over the last 35 years or so. There are many engineers who are familiar with the work and the engineering is well regulated with many guides to design and construction practices and rules and formal or informal practices. Checking procedures are well established.

The questions that are addressed in this paper are how well is the engineering process understood, how closely do the calculations that are carried out relate to reality, are popular conceptions myths and, as far as the ground is concerned, what is reality?

These questions are not hypothetical or academic. Engineers are taught to observe, to apply scientific understanding and then to apply the experience and insight to new projects. Where engineering is pushed forwards, say the excavations become deeper, it is important that the factors that apply are properly understood.

The paper critically reviews some design assumptions and back analyses, and performance of earth retaining structures for deep excavations

1 INTRODUCTION

Fundamental to deep excavations is Excavation with Lateral Support (ELS) and to this is coupled ground movement of which disturbance of nearby properties is a general concern and base heave within the excavation can adversely affect temporary works and piling. 35 years ago when I first worked in Hong Kong there were few basements and not many were deeper than two floors. Since then many deep basements have been built. In Hong Kong there must have been hundreds of deep excavations which fortunately have mostly been successful. There are few cases of collapse. Many engineers have been involved in the design and construction of deep excavations. The issue that I will address is “do we really know what is going on?” Superficially it appears as if the process is safe, but is it economical? Do the complex calculations that we do for purposes of design, or to satisfy a checker, bear any resemblance to reality?

Fundamental to the technology, in the 1770’s Coulomb first calculated active pressure “force” and later Rankine proposed active and passive earth pressures for soils without cohesion. Coefficients Ka, Kp have been used for years. To what extent have they been verified? I recall several years ago a few instances of measurement of contact pressures on walls were reported but I am not aware of many recent publications of such measurements. Likewise few reliable back analyses have been published. As a consequence, in my view, a number of myths have arisen. On the other hand, with so much emphasis on easily achieved complex numerical modeling, engineers run the risk of living in a virtual reality. In this paper I will draw upon some experiences from the past and then present some more recent examples of myths that have arisen and what might be reality.

2 SOIL PRESSURES

In the 1960’s Ralph Peck measured forces in struts for deep excavations (Peck 1969). Because there were no quick aids to calculations, he summarized the results as envelopes of maximum loading which amount to about 1.6 times the active soil pressure. With Terzaghi he developed trapezoidal pressure diagrams and these were used by many designers for working out strut forces and bending moments in the wall elements for braced excavations. The first edition of Geoguide 1 published in 1982 refers to the method. Whereas the method can be used to estimate maximum strut forces, the method is inappropriate for estimating deflections. For multi-strutted excavations the wall deflects quite a lot before each strut is placed and the Terzaghi and Peck diagrams give no indication of the magnitude of this effect. Likewise the strut forces obtained from using the envelope do not apply to any one stage of excavation because the struts do not normally develop their maximum load simultaneously.

3 WALL MOVEMENTS TO DEVELOP SOIL REACTIONS

In the late 1970’s computers became commonly available and in order to model the stiffness of the ground for design of ELS a number of “beam on springs” programs such as DIANA and SHEETPILE were written. In these models the ground was represented as a series of horizontal springs in contact with the walls. Some of the early models assumed that after excavation the wall was notionally held in place with "at- rest" pressures to either side and that, as the wall moved forwards into the excavated area, the ground pressure on the embedded section of wall built up until the passive pressure was reached. By contrast, it was known that when soil is allowed to swell in a consolidation apparatus the lateral soil pressures do not drop as quickly as the relief of overburden and, after a modest vertical strain, with no lateral strain, the lateral pressures exceed the vertical pressures and become equal to the passive pressures under the prevailing overburden pressure see Figure 1 (Endicott 1982). Soil fails one dimensionally in swelling. The analyses that assumed that after excavation the at-rest condition applies followed by a soil spring equal to the coefficient of lateral subgrade reaction was a myth. It was corrected.

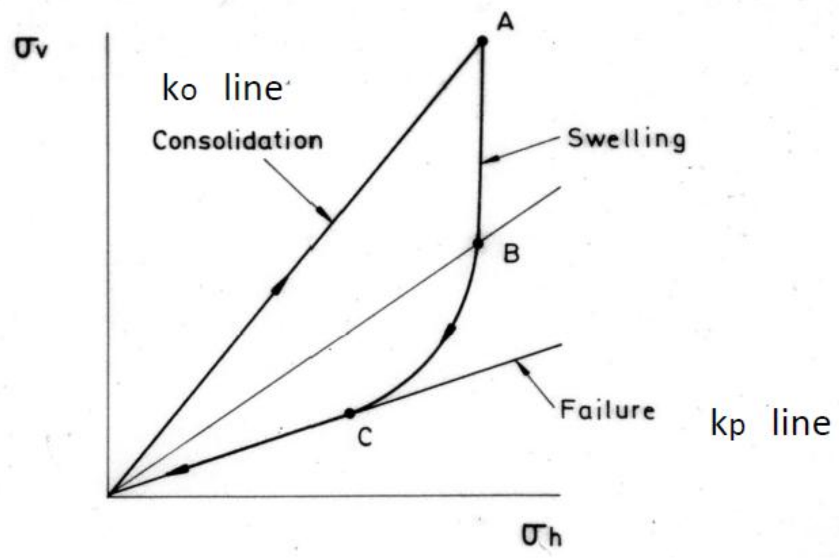

Figure 1: One dimensional consolidation and swelling (after Endicott 1982)

4 FIELD MEASUREMENTS OF CONTACT PRESSURES

By the time that finite element computer programs were available, in the early 1980's, some field monitoring of contact pressures on the face of deep walls showed pressures that were significantly less than those calculated by using the simple Rankine passive pressure times the overburden. Recognising that diaphragm walls by and large are constructed under bentonite slurry, people argued that there is a weak layer between the wall and the soil. They introduced slip elements with material weaker than the soil. Why introduce slip elements? What happens at the face of a wall? Logic tells one that on the centre line of a symmetrical case there is no shear and vertical and horizontal stresses are principal stresses. At yield the Rankine passive pressure could apply at the centre line of a symmetrical cross section. Examination of some computer analyses showed that only on the centre line of an excavation the vertical stress is a principal stress. Shear stress is developed on walls and the horizontal stress is much less than the passive pressure. In some instances the face of the wall is a plane of maximum shear stress and the principal stresses are at 45 degrees to the plane of the wall, the vertical stresses and the horizontal stresses are equal, see Figure 2 (Endicott & Cheung 1991). Some numerical models calculate reduced stresses due to rotation of axes of principal stress and it is not necessarily a need to introduce weak elements alongside the wall. The question remains, is there a weak layer on some walls or does the computer analysis with maximum shear stress represent reality?

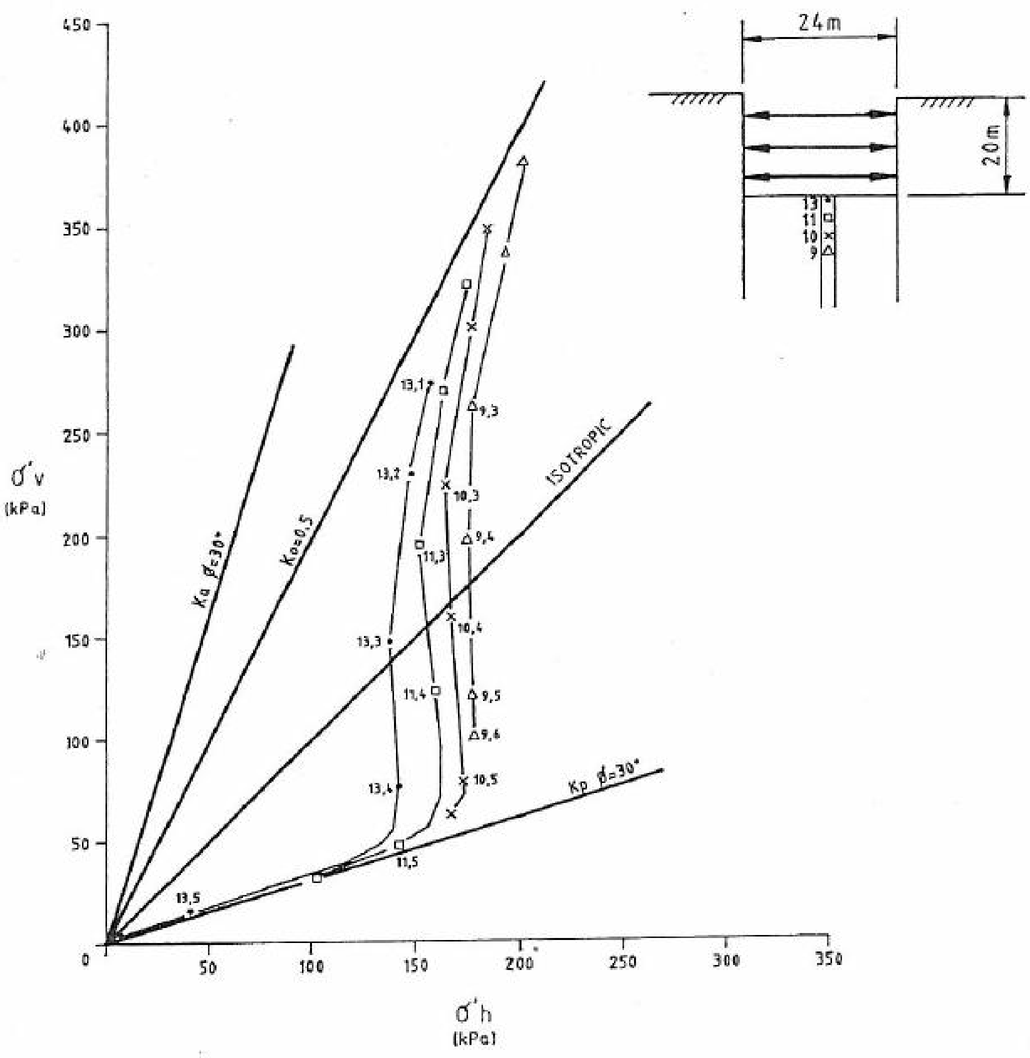

Figure 2: Stress path during excavation, centre line

5 PRE-STRESS

Pre-stressing the struts was introduced as a measure to reduce deflections. It was widely adopted in design specifications. Why must we use 90%, or any prescribed value? For example, why can't the designer relax the first strut after second strut is in place and thereby develop a hogging moment in the wall at the location of the second strut? The effect lower down the wall is to reduce the sagging bending moment and correspondingly less deflection results. (Endicott 1982)

Because design standard prescribe a high level of pre-stress there is little appreciation of how it works. It is shocking that before the collapse of the MRT excavation at the Nicoll Highway in Singapore lots of people who should have known better did not notice a 70% loss of pre-stress in the first hours after placing struts as early as months before the collapse. (COI 2005)

6 NUMERICAL MODELLING

Nowadays there are several powerful computer programs that can be used to numerically model deep excavations. Are they used properly?

At the Inquiry into the collapse of the Nicoll Highway in Singapore, three experts said "my PLAXIS analysis fails by base heave, that was the reason for the collapse". Examination of inclinometers at the failed area showed that there was no kick- in of the toes of the diaphragm walls even when attempting to reinforce the strutting because of gross deformation higher up.

Another issue arose about the program. The user's manual at the time advocated using the undrained setting with the Mohr Coulomb model with effective stress parameters. Without guidance, one would be tempted to follow the user's manual. However, the undrained setting means that when the soil is sheared there is constant volume. Effective stress parameters mean that if there is constant volume the isotropic stress does not change. For wet clays (normally consolidated or lightly over-consolidated) the model is incorrect. For isotropically consolidated clays the strength is over-estimated by about 60% see Figure 3. If the strength is important in the calculation then this model is not good. At one point when I was reviewing the checking of some designs for LTA in Singapore, the designer had used PLAXIS, with both the drained and undrained setting, the checker says which one will you use, the designer says to checker "you chose".

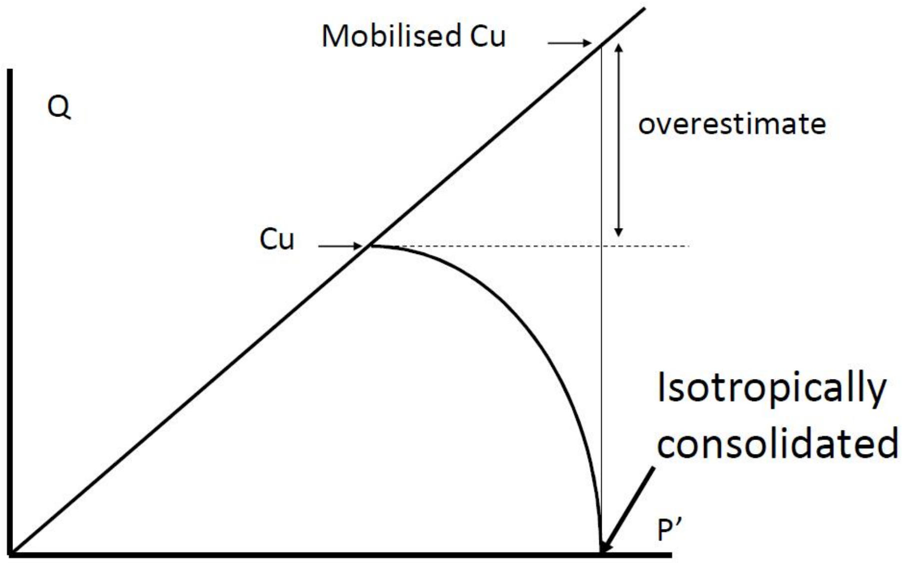

Figure 3: PLAXIS undrained setting Mohr Coulomb model effective stress parameters

Another expert said "Back analysis just does not work. I have tried many times, I can adjust the parameters to match the deflections but the strut forces are all wrong". He was assuming that the struts behaved elastically. His back analysis told him what was happening. The monitoring showed that the walls converged by more than the increase in the force in the struts. The strutting system in the field was not behaving elastically and as long as he modeled the strutting as elastic members he would not get a good fit to the observations. His back analysis, incomplete though it was, told him what he did not want to admit, that the struts were failing.

Back analysis means adjusting many parameters to match observations. Given a multi-variate problem; then there are many solutions. If one of the factors is wrong then it might well be possible to fit the observations but one or more of the other parameters will be wrong and forward prediction to the next stage of the excavation could be very unreliable. If one does not know how the program works and if one does not know how the soil behaves then getting the right design is random with a chance of inadequacy or failure. Mathematical models answer questions; they do not solve the problems.

7 WHY DO WALLS FAIL?

What is the reality? Fortunately very few deep walls fail in Hong Kong. One example from 1981 was at Queen's Road Central (personal recollection). There the removal of the lower strut when there was no penetration of sheet piling below excavation level and only one level remaining raking shore on a wide waler resulted in closure of Queen's Road for a month. However a search of the records revealed that the old water main in Queen's Road had been leaking, it was repaired and during the few weeks before the collapse the piezometers showed unexplained rises in ground water pressure. Who will know what was the reality of that failure?

From experience overseas in nearby locations I have seen evidence of inadequate toe penetration, loss of pre-stress, failure of the waling connection, as inferred for the Nicoll Highway Collapse. These cases are not unique.

8 WHAT IS REALITY?

Reality is what actually happens in the ground. It is not necessarily what is contemplated by a prescriptive design. One can get insight into what really happens by quality monitoring of deep excavations. However the monitoring may have errors, it may be infrequent or incomplete, but with care it can be understood.

Another type of reality comes with physical models. Physical models, i.e. using real soil, can be very useful. Compared to a prototype they may suffer from scale effects and representativeness however physical models are in themselves a reality. For example they do not suffer from an inherent risk with numerical models when accidentally incorrect parameters result in answers which might seem plausible. Certainly using soil models on a centrifuge with controlled conditions can be used to calibrate numerical methods and the relevant soil parameters. Field trials, such as pile load tests can also provide the basis for validation of assumptions about the ground conditions. However the magnitude of deep excavations mitigates against field trials but adequate instrumentation, properly interpreted can assist the engineer in understanding what is going on.

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Engineering Geological Considerations for Construction of Deep Basements in Hanoi, Vietnam

A.D. Mackay
Snowy Mountains Engineering Corporation, Hong Kong
W.T.N. Chow
Hyder Consulting Hong Kong Limited

ABSTRACT

Vietnam has undergone rapid economic growth during the last decade. As the peripheries of its major cities are often physically constrained by mountainous terrain or limits of the existing infrastructure, there has been a trend to create additional space by excavating deeper level basements, often reaching about 20 metres below the existing ground surface. Hanoi in particular is situated within the Hung river delta, formed by variable fluvial deposition, which has abrupt changes in physical properties and pore water pressure relatively short horizontal and vertical distances. These ground conditions increase the potential to encounter adverse conditions with increased excavation depths, and require improved risk management mitigation strategies, more accurate site investigation data and improved construction techniques to overcome the adversities. In addition Hanoi is situated within a region of moderate tectonic activity also impacting on deep excavation. This paper outlines the main issues affecting excavation as the depth increases, referencing an example of a potential development in central Hanoi.

1 INTRODUCTION

Due to the space demand in Hanoi there has been a need to construct deeper level basements to cater for the additional amenities. As Hanoi is located in the Hung River delta, the ground conditions have had a variable deposition history resulting in ground and groundwater variations and a greater potential to encounter adverse ground conditions with increased excavation depth. In the past the delta covered a far greater area ranging from the Ha Leong Mountains further inland extending approximately 200 kilometres along the coastline, see Figure 1. As many deeper excavations have encountered difficulties with increased excavation depths, many of the more responsible developers have carried out detailed and accurate ground investigations prior to commencing construction and have used a more cautious risk management approach to safety and construction methodology. This paper outlines the geology of the Hanoi delta and provides an example of a recently proposed deep basement development in central Hanoi, which is considered to be representative of the ground conditions and typical factors effecting deep excavations in the region.

2 GROUND CONDITIONS HUNG RIVER DELTA

The Hanoi delta generally has a consistent ground surface level, rising a few metres above sea level. The Hung River delta was formed in five geological deposition cycles during the recent Quaternary era. These deposition periods are referred to as the Riss, Mindel and Munz glacial periods; and the Wurm and Wurm 1 inter-glacial periods (Nghi 2006). The thickness of the Quaternary deposits, overlying the conglomeritic bedrock termed the Vin Bao formation, is about 80 metres (Vu et al. 2001). A summary of the thickness, constituents and the geological period for each formation is provided in Table 1.

Table 1: Superficial deposit and bedrock (Nghi 2006; Vu et al. 2001)

Glacial / Inter-glacial / period	Formation (reference)	Description	Approximate Depth (m)
Munz / Late Holocene	Thai Binh ($aQ_2{}^3tb$)	Greenish grey clay and peat, derived from alluvial, lagoon and coastal swamp deposition	0 – 10
Wurm 1/ early – mid Holocene	Hai Hung ($mQ_2{}^{1-2}hh$)	Greenish grey clay and peat, derived from alluvial, lagoon and coastal swamp deposition. Each deposition has an abrupt / distinctive boundary towards the centre of the delta.	10 – 20
Mindel / Mid – Up Pleistocene	Vinh Phuc ($aQ_1{}^{3b}vp$)	Coarse grained sand, derived from river bed deposition, grading into flood plain (silty clay) and lagoon (clay) deposition towards the uppermost levels. Distinct changes between sand, silt and clay (alluvial) and silty clay, lagoon deposition is apparent towards the centre of the Hung River delta. Towards the modern coastline the facies change to oxbow and coastal swamp deposition with a greater presence of peat.	20 – 40
Wurm / Low - Mid Pleistocene	Ha Noi ($aQ_1{}^{2-3}hn$)	Cobbles and gravels, derived from deposition from flow from adjacent mountains. The deposition is widespread throughout the Hung River delta.	40 – 60
Riss / Lower Pleistocene	Le Chi ($aQ_1{}^1lc$)	Cobbles and gravels, grading into sand, silt and clay towards the uppermost levels. The deposition is widespread throughout the Hung River delta.	60 – 80

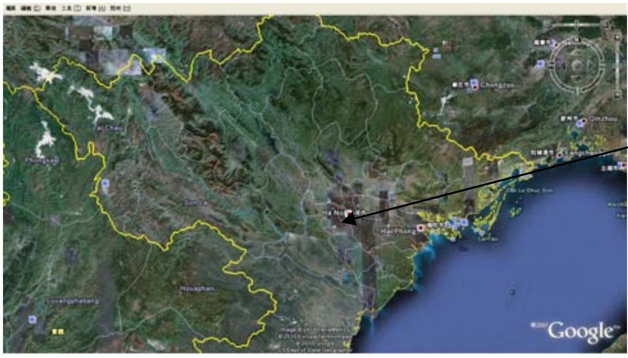

Figure 1: Hanoi, Hung River Delta, North Vietnam.

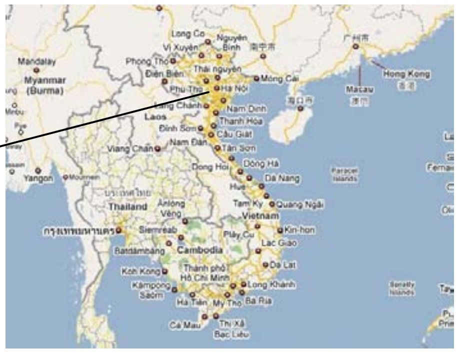

Figure 2: General location of Vietnam

3 SEISMICITY

Hanoi is located in a region of low to moderate seismic activity. More recently Eurocode standards, referenced TCXDVN 375 (2006), have been adopted for Vietnam usage. Notwithstanding many local experts, such as Ngo et al. (2008), highlight that there is a lack of historical earthquake data and knowledge of how local ground conditions respond to seismic activity to provide any meaningful basis for these standards. The Hung River delta in particular is considered to be the most seismically active region in Vietnam, refer to Figure 3 for the seismic intensity in Vietnam (Ngo et al. 2008), with seismic activity generated from active north-west to south-east trending faults, such as the Lo River, Vinh Ninh, Songhong and Chay River fault zones, refer to Figure 4 (Pubellier et al. 2008). The quaternary deposits of the Hung River delta, overlying the fault zones, are relative weak. As a result ground motion generated along the faults and passing into the overlying ground is amplified prior to reaching the ground surface (Ngo et al. 2008). Table 2 shows the reduction in ground motion shear wave velocity within the more recent deposits, corresponding to an increased amplification.

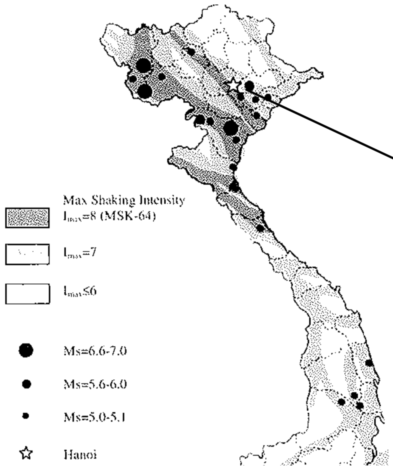

Figure 3: Seismic acceleration, Vietnam (Ngo et al. 2008)

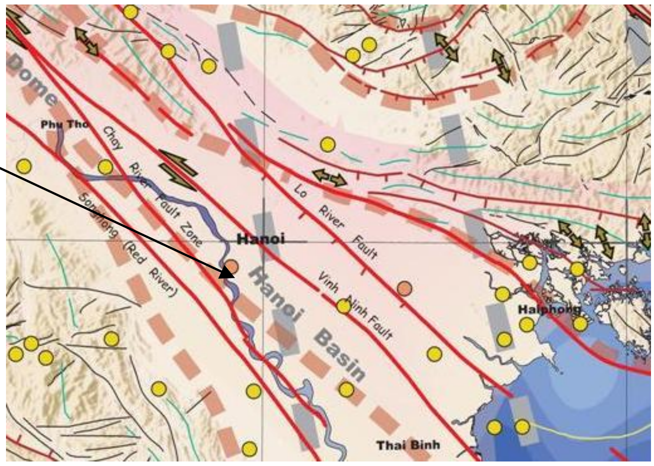

Figure 4: Fault and seismic record locations, Hung River Delta, Vietnam (Pubellier et al. 2008)

Table 2: Approximate soil shear wave velocity, Hanoi (Ngo et al. 2008)

Formation	Approximate Depth	Shear Wave Velocity	Aquifer
Thai Binh	0 – 10m	100 – 200 m/s	
Hai Hung	10 – 20m	200 – 300 m/s	Holocene Aquifer
Vinh Phuc	20 – 40m	300 – 400 m/s	Holocene - Pleistocene Aquiclude
Ha Noi	40 – 60m	400 – 450 m/s	Pleistocene Aquifer
Le Chi	60 – 80m	450+ m/s	

4 GROUNDWATER CONDITIONS WITHIN THE HUNG RIVER DELTA

The abrupt changes in grain sizes associated with the variable deposition cycles of the Quaternary have led to the development of an unconfined aquifer, mainly located within the Thai Binh formation of the Holocene period and a confined aquifer within the Vinh Phuc, Ha Noi and Le Chi formations of the Pleistocene period (Nghi et al. 2006). The Pleistocene aquifer is capped by the fine grained deposits within the uppermost levels of the Vinh Phuc formation. A lower level aquiclude is also present within the uppermost layers of the Ha Noi deposits, however, as erosion has removed much of the fine grained material, the extent of this aquiclude is reduced and localized. The Holocene aquifer is recharged from surface water and the Hung River.

5 IMPLICATIONS FOR DEEP BASEMENT EXCAVATION, HANOI, VIETNAM

Due to the high level water table and relatively weak ground conditions encountered in Hanoi, diaphragm walls, which provide a relatively robust retaining structure for deep level basements and groundwater cut off, have traditionally been installed to support deep basement excavation in Hanoi. To accelerate construction programs the top down excavation method is typically adopted in Vietnam; successive floor slabs are installed to provide support reaching the basement invert level. A summary of the potential risks and mitigation measures for deep basement design is summarized in Table 3.

Table 3: Summary of potential impacts for deep basement excavation in Hanoi

Deep basement / approximate depth (m bgs)	Potential Concern	Mitigation
Invert level at the Hai Hung (Holocene) / Vinh Phuc (Pleistocene) boundary (20)	Abrupt change from fine to coarse grained deposition; potential liquefaction of the coarser particles.	Ensure adequate groundwater drawdown; suitable groundwater cut off and surrounding recharge to mitigate potential settlement of sensitive structures in close proximity.
Invert at the upper levels of Pleistocene confined aquifer (20)	May pass into the confined aquifer	Ensure adequate groundwater drawdown from both the upper and lower confined aquifer.
Tectonic activity (all levels)	Impact on the structural stability	Basement design needs to incorporate suitable, robust acceleration loads. Structural flexibility to be incorporated.
Increased pH and Na – Cl ratios (20)	Corrosive effects on the foundation	Adequate concrete reinforcement cover. Concrete additives needed to provide resistance. Waterproofing to prevent precipitate concentration.

To ensure sufficient data is obtained to address the potential concerns listed in Table 3, it is imperative that a high quality ground investigations (GI) is carried out prior to commencing the works and at subsequent stages in the design and construction process. As bedrock is generally situated at about 80 metres below ground surface (m bgs), pile installations, passing through the deeper level basements and supporting multi storey structures, may need to extend into or immediately above bedrock. As the diaphragm wall installation is typically “floating” within superficial deposits below the deep basement invert level the installation may need to extend to 1.5 times the basement excavation depth, or 50m bgs. The boreholes carried out to verify the ground conditions therefore need to be advanced to below bedrock level in the vicinity of the piles and to 55 to 60 m depth, to verify the thickness of the coarse grained Hanoi formation, in the vicinity of the diaphragm wall installation. Traditionally GI techniques used to verify ground conditions for shallower foundations have typically extended to about 50 m bgs, providing indicative fine / coarse grained boundaries, typically located at the Vinh Phuc – Ha Noi formation boundary. To allow rapid borehole advancement bentonite has typically been used for support, resulting in disturbed sampling, indicative test results and an inability to provide reliable groundwater level and chemical test data.

6 CASE STUDY, CENTRAL HANOI

A 35 storey development with a 5 level basement covering an approximate 4 hectare plan area was proposed within Hanoi, Vietnam; refer to Figures 5 for the general location. The basement perimeter was anticipated to be a diaphragm wall to be constructed using top-down excavation techniques. The super-structure was to be founded on barrette piles passing through the basement. A detailed GI was proposed comprising drillholes scheduled to extend below the anticipated founding levels, estimated to be about 55 metres below ground surface (m bgs) for diaphragm wall and bedrock level, about 80 m bgs, for the piled foundations. The data obtained from the GI was interpreted to verify the ground and groundwater conditions, ascertain appropriate ground models, and estimate suitable geotechnical parameters and to carry out the foundation design.

Figure 5: Approximate site location, Hanoi, showing proximity to the Hung River and surface water

In order to gain a thorough knowledge of the ground conditions a suitable range of GI techniques were adopted, which provided feedback at different stages of the GI contract and ranged in complexity and accuracy for the data provided as summarized in Table 4.

Table 4: Summary of GI techniques adopted

GI technique	Methods
Exploratory Stations	Boreholes, dynamic cone penetration tests and trial pits. Boreholes were advanced using alternate in-situ tests and undisturbed (U100) samples
In-situ tests	Standard Penetration, permeability, pressure-meter, in-situ density and hand shear vane
installations	Casagrande and standpipe piezometers
Laboratory tests	Classification (particle size distribution, atterberg limit, natural moisture content and density); compaction and deformation (proctor and oedometer); shear strength (triaxial) and split and detailed description of undisturbed samples.

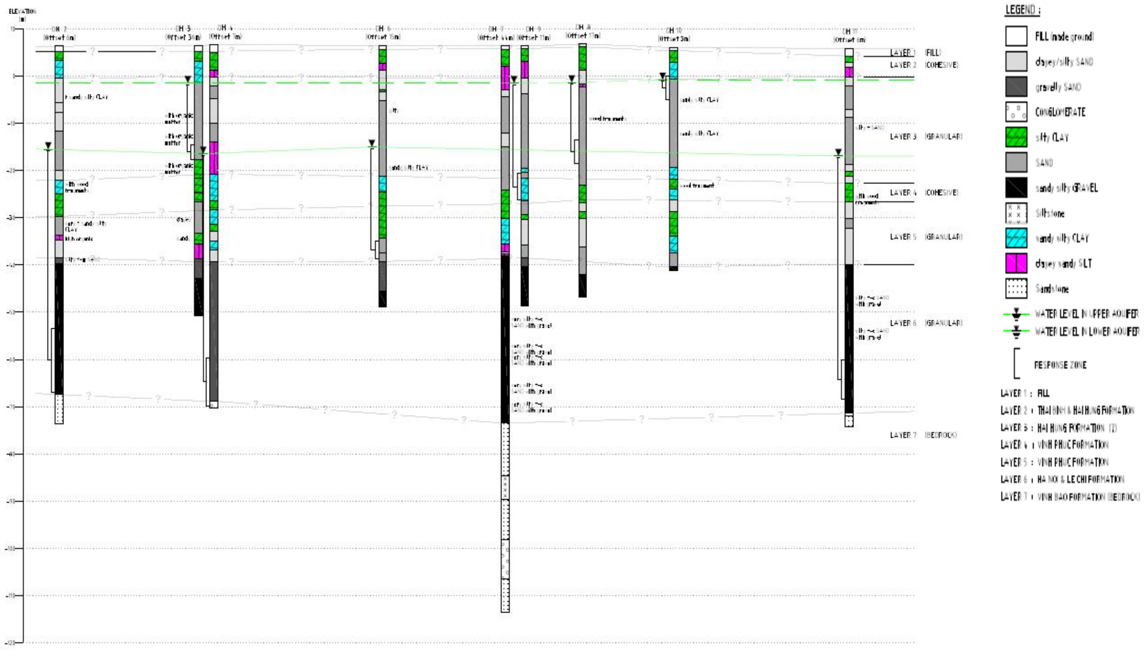

Figure 6: Geological cross section Key

The GI verified the ground conditions indicated from the available literature and revealed relatively consistent depths for the main formations across the site. A section taken across the site, incorporating the findings of the majority of the boreholes is presented in Figure 8 and a summary of the findings, compared to that indicated in the literature search is provided in Table 4.

Table 5: Summary of the ground encountered during the Chua Boc ground investigation

Layer / ground condition, from GI data	Level	Depth	Base of formation, available literature	Basic description main constituents, BS 5930
Fill	+6 - +4 mCD	0- 2 m bgs		Silty clay
Thai Binh / Hai Fung	+4 - -1 mCD	2 – 7 m bgs	10 m bgs	Silty clay and sandy silty clay
Hai Hung	-1 - -2 mCD	7 – 8 m bgs	20 m bgs	sand
Vinh Phuc	-2 - -30 mCD	8 – 36 m bgs		Silty clay
Vinh Phuc	-30 - -40 mCD	36 – 46 m bgs	40 m bgs	sandy silty clay / clayey sandy silt
Ha Noi	-40 - -70 mCD	46 – 76 m bgs	60 m bgs	Sandy silty clay / sand
Bedrock	Below -70 mCD	Below 76 m bgs		Rock

As presented in Figure 6 and Table 5 the Le Chi formation was not identified during the GI. The groundwater response also showed a considerable difference, approximately 15m, between the upper unconfined and lower confined aquifers, possibly due to the lower aquifer requiring greater time to reach pore water pressure equilibrium (Mackay et al. 2008).

7 CONCLUSION

There is a current demand for taller structures and deeper basements in Hanoi, Vietnam. Based on the available literature adverse ground conditions, mainly resulting from the presence of a confined aquifer located at about 20m below ground surface (m bgs), may impact deeper level excavations. In order to provide suitable information for the foundation design and initiate a risk assessment, a high quality ground investigation (GI) with drillholes advanced to bedrock level, typically located 80m bgs, is required. A high quality GI carried out for the proposed high rise, deep level basement at central Hanoi, verified the findings of the literature search and was successful in providing representative data for input to design and construction. The use of casing to support the hole instead of bentonite was considered to be an important factor in improving the quality of the GI and advancement to deeper levels. The piezometer records were successful in showing that the pore water pressure response in the lower confined aquifer was minimal at this location.

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Cofferdam Design and Construction Considerations for Tunnel Boring Machine Launches and Recovery

A.D. Mackay
Snowy Mountain Engineering Corporation, Asia

ABSTRACT

Ideally when tunnels are formed using Tunnel Boring Machines (TBMs) considerable space is required for the launch and recovery. Due to the space constraints typical of Hong Kong's urban environment, deep shaft excavations are required to facilitate the TBM launch and recovery to minimize impact. These include restricted available area for launch and recovery, limited headroom, sensitivity of the adjacent sub-surface and surface structures, and minimizing disruption to the surface activities. Methods adopted in Hong Kong for shaft excavation support in soil, have traditionally included diaphragm wall installation for permanent works and contiguous pipe pile installation, strut and waling lateral support or sheet piles for temporary works. In addition grout is typically injected to provide a groundwater cut off extending around and beneath the installation. Within the TBM eye, formed to allow launch and recovery to take place, the reinforcement is minimized. Ground improvement, usually by grout injection, is required to extend beyond the TBM eye to provide a groundwater cut-off and to stiffen the ground during launch and retrieval. The grout injection extends beyond the length of the TBM shield and surrounding the outer diameter of the TBM to allow a seal to be installed after the shield has passed the shaft wall. This paper provides an overview of deep excavation in Hong Kong and summarizes documented examples of shaft excavations used both for launch and recovery, including the China Light and Power (CLP) Tsz Wan Shan and Tuen Mun Nullah crossing, the Kowloon Southern Link and the Kai Tak Transfer Scheme tunnels. An example of a past TBM launch and retrieval at Hung Hom is also provided, which was carried out in adverse ground conditions.

1 INTRODUCTION

Due to the hilly terrain and restricted space available in Hong Kong (HK), deep excavations are routinely formed adjacent to sensitive structures, through variable material, within limited space and subject to rigorous and stringent government approval procedures. These procedures often require lengthy approval periods from the HK Government and need robust solutions and construction methods in order to gain consent. Presently there are numerous tunneling projects about to commence or on-going in many urban environments in HK, all of which will require associated cofferdam excavation for the TBM launch and recovery and other associated works in congested areas. To allow for the reduction in space required for these excavations, innovative construction solutions are required to ensure project success. This paper provides a short review of TBM cofferdam excavations carried out in Hong Kong and an outline of the details of an example of a TBM recovery and launch, carried out within the same shaft.

2 DEEP EXCAVATIONS IN HONG KONG

The initial deep basement excavation in HK extended to about 25m below the ground surface for the New World Trade Centre development in Tsim Sha Tsui, 1976 and was supported by tensioned tie backs extending from the retaining wall (Mackay 2010; Endicott 2007). Despite the initial success this type of excavation support, subsequent basement excavations using similar support methods resulted in failure. As a result a ban of their use was implemented by the HK Building Authority in 1978. Following this, failures, including collapses or excessive displacement, were documented by Man et al. (1992) and Ho et al. (2007); two examples of these are summarized in Table 1.

Table 1: Summary of past documented deep excavation (Ho et al. 2007; Man et al. 1992)

Date	Location	Remarks	Caption
1981	Queens Road, Central	Large sized collapse occurring at the uppermost levels of the support and extending to 9m depth. This was reported to be a due to the sheet piles not being driven to the correct design penetration and removal of the temporary props to allow permanent works to commence.	
1991	Man Lau	Medium sized collapse caused by inadequate shoring between the soldier piles.	

It was noted that a major cause for this instability was unexpected geological and hydro-geological conditions combined with inadequate planning. The importance of developing reliable hydro-geological and geological models was therefore emphasized and further changes made to the Building Ordinance in 1991, leading to highly robust design and construction support requirements for deep excavations. Subsequently the Geotechnical Engineering Office (GEO 2007) highlighted that "a considerable level of confidence in the robustness of any deep excavation scheme", in particular understanding the variability of the soil type and properties; estimating representative permeabilities, identifying potential non-uniform ground treatment on the soil properties and the presence of large voids and high permeability, generally associated with pre-existing marine structures.

3 TBM COFFERDAM EXCAVATIONS IN HONG KONG

TBM tunnel launch and retrieval cofferdams in HK typically require minimal space; as a result unique design and construction considerations are required, considering factors such as equipment loading and placement, the TBM eye, and space arrangements for back up and launch equipment and set up within the working space. Examples are provided below.

3.1 The Kowloon Southern Link

The Kowloon Southern Link joined the existing West Rail to the Tsim Sha Tsui East Rail Station and was partly excavated using TBM for 1.1 km for the southern contract (KDB 200) running along Canton Road before turning in a tight 225m radius run beneath Salisbury Road to the reception chamber (Hake et al. 2008). Due to the size (9.2m outer diameter), and depth of the tunnel (invert level at 20m bgl), the TBM launch cofferdam was formed using diaphragm walls with four layers of struts and waling for support. Plates 1 and 2 present the TBM launch and retrieval shafts respectively (Wallis 2006; Taylor 2009) and Figures 1 and 2 oblique and plan views of the alignment (Taylor 2009; Tam et al. 2009).

Plate 1: Kowloon Southern Link TBM launch.

Plate 2: Kowloon Southern Link TBM retrieval.

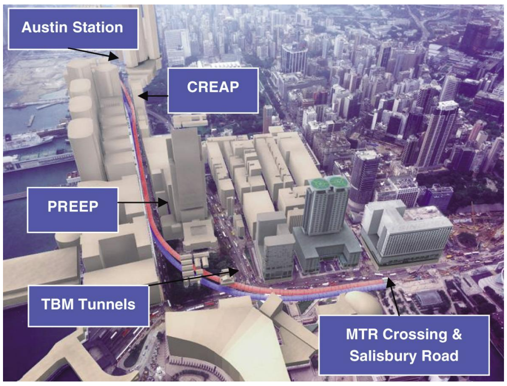

Figure 1: Kowloon Southern Link alignment.

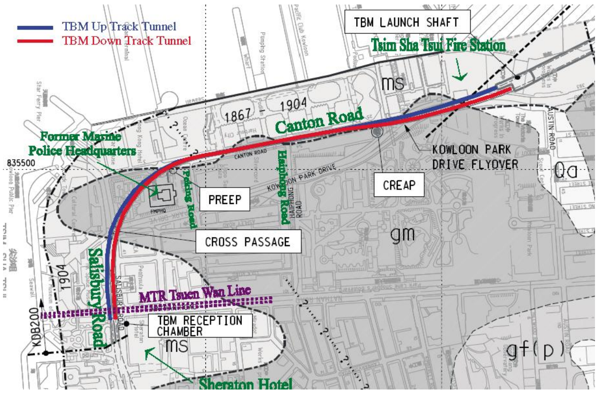

Figure 2: Kowloon Southern Link alignment.

The TBM launch was from a 22m deep shaft invert through a plastic reinforced eye formed within the wall panel. The ground was improved by the installation of jet grout columns extending a sufficient length from the eye to allow the wall to be sealed once the TBM shield had passed. The retrieval shaft was formed to extract the TBM disc cutter only (see Plate 2). The remaining TBM equipment was extracted through the tunnel. Due to the shaft's proximity to the Mass Transit Railway Corporation Ltd (MTRCL) Tsuen Wan Line, See Figures 1 and 2, and the minimal separation between the tunnel invert and the Tsuen Wan Line crown, horizontal pipe piles were installed from within the shaft to strengthen the ground between both tunnel alignments (Taylor 2009).

3.2 The Kai Tak Transfer Scheme

The Kai Tak Drainage tunnel was constructed to relieve potential flooding of the Mong Kok area and comprised a 4.4m inside diameter, 1.5km length tunnel, running from the Waterloo Road to the Kai Tak nullah, refer to Figure 3 for the alignment (Salisbury et al. 2004). The tunnel was formed using a 5.2m diameter TBM and intercepted six shafts, referenced A to F, from east to west respectively, along the alignment (Hake 2004; Salisbury et al. 2004); the details of each Shaft is summarized in Table 2. All shafts were supported by sprayed concrete with reinforced concrete installed at the base.

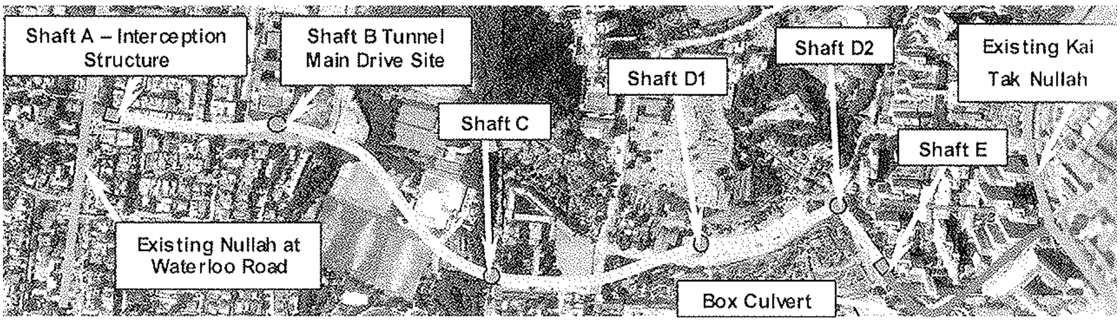

Figure 3: Kai Tak Tunnel Transfer alignment.

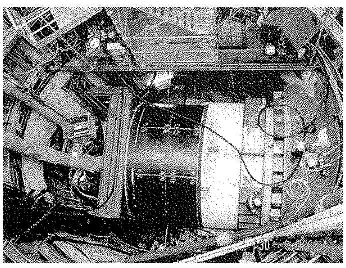

Plate 3: Shaft B short launch

Plate 4: Shaft D2

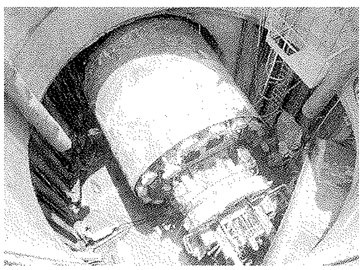

Plate 5: Shaft D2 TBM turn

Table 2: Summary of shaft construction, Kai Tak Transfer Tunnel (Salisbury et al. 2004)

Shaft	Depth (m)	General Arrangement	Approximate plan area (m^2)	Remarks
A	8	Square	50	Located at the box culvert / tunnel intercept
B	12	Ovalized	Enlarged	Enlarged to launch TBM to the east, to shafts C to E, and to Shaft A to the west, See Plate 3.
C	12	Circular	50	
D1	12	Circular	50	
D2	9	Circular	Enlarged	Enlarged to allow a 90 degree TBM turn, See Plate 5.
E	3	Square	50	Located at the box culvert / tunnel intercept

3.3 The China Light and Power Cable Tunnels

The CLP Cable tunnels, formed from 2004 to 2006, comprise the Tsz Wan Shan, Tuen Mun Nullah Crossing, Kwai Chung and Chi Ma Wan tunnels. The tunnels were constructed as part of the HK electrical network upgrade, refer to Figure 4 for the locations. The Kwai Chung tunnel was excavated using drill and blast techniques and as the Chi Ma Wan tunnel, located on Lantau in a more rural location, allowed more space to be available for the TBM launch and recovery. Both the Tsz Wan Shan and Tuen Mun River crossing tunnels were excavated using TBMs with restricted space for launch and recovery shafts, refer to Figure 5 and Plate 6 respectively.

Figure 4: CLP Cable tunnel locations

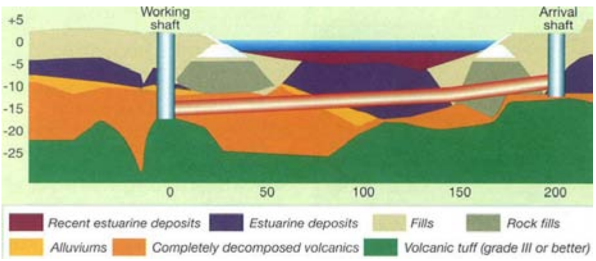

Figure 5: Tuen Mun Nullah Crossing alignment

Plate 6: Tsz Wan Shan launch

The Tsz Wan Shan Cable Tunnel TBM was launched from a pipe pile cofferdam with grout cut-off surround and an invert level at about 15m below ground surface, adjacent beneath a heavily trafficked road, see Plate 6. The tunnel was formed using a 3.8m diameter TBM and ran for 610m through variable CDG with corestone inclusions, and colluvium. The Tuen Mun Nullah crossing also used a 3.8m diameter TBM driven through completely decomposed granite (CDG) beneath fill and soft marine mud and estuarine deposits. The TBM was designed with compact equipment back up to allow an efficient launch and retrieval from shafts formed by diaphragm wall installation to depths of approximately 25 and 15 m respectively, see Figure 5 (Wimalasena 2005).

3.4 The Hung Hom Freight Yard Cofferdam

A Cable tunnel was formed beneath the beneath the MTRCL Hung Hon Freight Yard, east Tsim sha Tsui, comprising a 300m length, 3.8m outside diameter with the invert rising from 12m below ground level (m bgl) to 6m bgl from east to west respectively, see Figure 6 for the tunnel location. Due to a tunnel inundation that occurring approximately mid-way along the alignment during the TBM drive, a recovery and re-launch cofferdam was excavated and, in order to minimize disruption to the surface operations within the Hung Hom Freight Yard, the space was minimized as much as practicable. Refer to Figure 7 for the cofferdam location and adjacent sensitive structures.

Figure 6: Seawall section

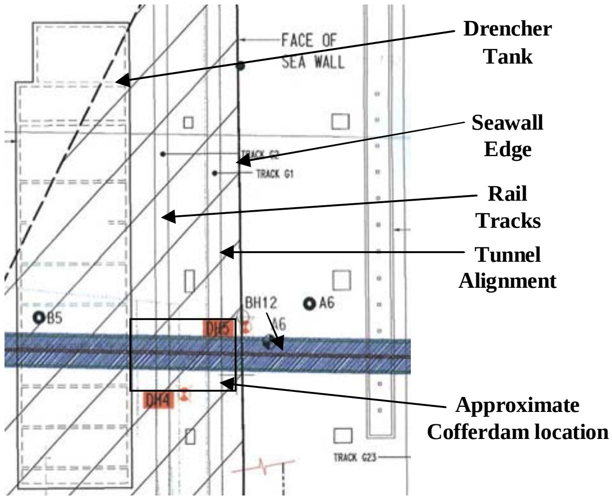

Figure 7: Location of sensitive structures adjacent to the cofferdam

The ground conditions along the alignment included reclamation fill, overlying superficial deposits, which in turn overlay completely and locally moderately decomposed granite (C/MDG). A buried masonry seawall was located approximately mid way along the alignment with the TBM drive passing through the underlying grouted pell mell rubble foundation. The pell mell rubble wall was located immediately in front of the TBM excavation during excavation (Massey et al. 2007).

In order to retrieve and relaunch the TBM a cofferdam was excavated immediately in front of the TBM cutter head and was supported by pipe pile installation extending to about 25m bgl (-19mPD), founded within CDG, with a grout curtain groundwater cut-off extending to 30m bgl (-25mPD), into MDG. The cofferdam design accounted for soil compressibility, based on the Young's Modulus (Es) = 1 * Standard Penetration Test (SPT) "N" value (GEO 1990), ground strength and variability of the hydrogeological and groundwater characteristics. The cofferdam was formed using 80 pipe piles, 610mm diameter installed with sufficient spacing to form an inside area of 13 by 7m. The pipe piles generally had a concrete infill; however, as presented in Figures 8 and 9, granular backfill and concrete infill with H pile reinforcement were installed to allow pile removal for the TBM re-launch. Grouting comprised initially of polyurethane injection surrounding the TBM cutter head and silicate and bentonite-cement surrounding the TBM and the grout curtain cofferdam surround, refer to Table 1 for the installation process. Successive grout injections were carried out until the specified grout volume or pressure had been achieved.

The sensitive structures adjacent to the cofferdam, as presented in Figure 7, included rail tracks, a drencher tank to 6m depth (+4mPD to -1.6mPD) and the freight yard structures located at about 7m horizontally from the edge of the cofferdam. Adjacent utilities included foul and storm water drains and electric and telephone cables.

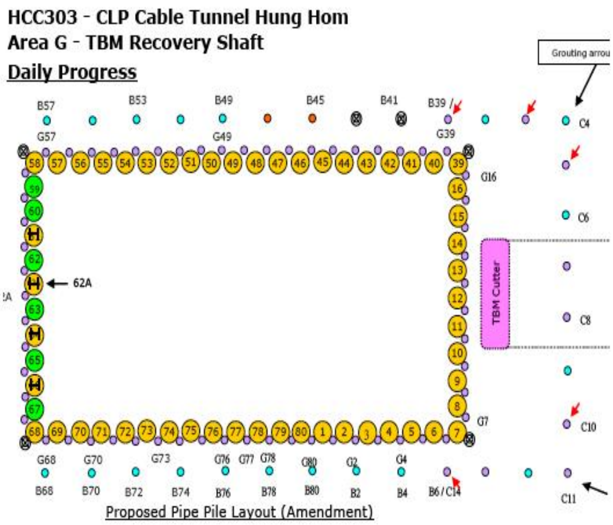

Figure 8: Cofferdam pipe pile and grout installation

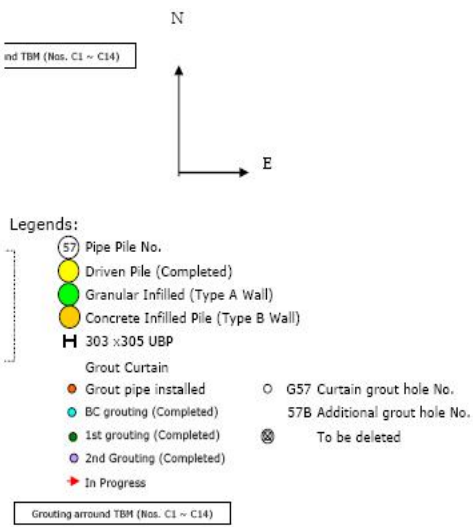

Figure 9: Pile and grout curtain infill

Table 3: Locations of bentonite cement and silicate cement grout locations

Date	Proposed 58 Nos. of Pipe Pile + 1 No. 62A (Additional pipe pile)								
	Production								
	Daily Progress				Accumulated Total				
	Curtain Grouting (Total 59 Nos.)		Grouting arround TBM (14 Nos.)		Curtain Grouting			Grouting arround TBM	
	Bentonite-cement	Silicate-cement	Bentonite-cement	Silicate-cement	BC (Completion)	SC (2nd grout	SC (3rd / Re-grouting)	Bentonite-cement	Silicate-cement
21-Mar-05 (Mon)	Completed	G1, G6, G8, G9, G11, G13, G15, G72, G75, G76, G77	Nil	Nil	Completed	59 Nos. (100%	On-going	71%	0%
22-Mar-05 (Tue)	- ditto -	G1, G4, G5, G6, G42, G52, G72, G75 & G77	C2 & C9	C7, C11 & C13	- ditto -	Completed	On-going	86%	21%
23-Mar-05 (Wed)	- ditto -	G3, G73, G74, G75, G76, G78 & G80	C2, C3, C5 & C9	C1, C3, C5, C8, C10, C13	- ditto -	- ditto -	Completed	100%	36%

Following grout injection the TBM cutter head was removed through the cofferdam. To allow this the pipe piles were initially removed and successive installation of horizontal sheet piles carried out. These were then each welded onto the TBM shield, immediately behind the cutter head (Plate 7), to allow subsequent detachment and removal. The remaining TBM was left in place. Following the cutter head removal, debris was removed to allow segment placement within the tunnel.

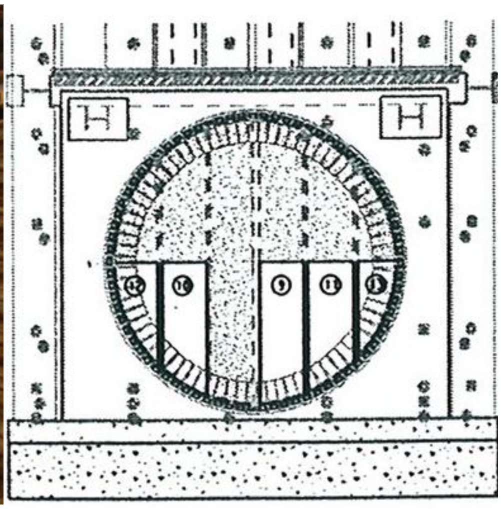

Plate 7: Sheet pile installation around TBM cutter head.

Figure 10: Pipe pile removal for TBM launch.

Immediately prior to the TBM re-launch from within the cofferdam pipe piles were removed successively as presented in Figure 10, after which the TBM was immediately jacked forward against the exposed face and excavation commenced. The segments within the cofferdam were erected using a chain block and tightening mechanism for both temporary steel ring support, to allow jacking of the TBM past the grout block, and permanent concrete ring erection thereafter. The cofferdam was then carefully backfilled around the permanent lining.

4 CONCLUSIONS

Tunnel construction in Hong Kong typically passes through congested and busy urban areas and, as a result, severe limitations for TBM launch and recovery are imposed. The Hung Hom Cofferdam is an example of a successful TBM recovery and launch in a highly congested location with limited access and adverse ground conditions, which could be used as a reference for similar tunneling construction works in future projects.

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The views expressed in this paper are those of the author and not of any other parties.

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Some Observations Associated with Design and Construction of Excavation and Lateral Support in Mainland

Paul K.F. Yau & Angus H.L. Sum
Meinhardt (C & S) Ltd.

ABSTRACT

Deep basement excavation and construction become commonplace in Mainland China. Design and construction of deep basement in urban area is always a challenge. The key challenges are to design and to construct the deep basement such that not only the operation of the adjoining public, buildings, and underground railway could be maintained without any disruption, but also the construction works would be carried out in an economical way to suit the local design and construction practices.

This paper presents briefly some observations on the design and construction aspects for deep basement excavation in Mainland China. It outlines some key considerations behind ground investigation work, geotechnical design approach, code of practice, excavation methodology, construction monitoring and supervision requirements. A case history of deep basement excavation in Shenzhen is also presented in this paper. The performance and lesson learnt have provided information which may lead to the adoption of a more practical design and construction solution for the engineers practicing in Mainland China.

1 INTRODUCTION

In Mainland China, over the last decade, there are many deep basement excavation projects to cope with the rapid growth of developments. Deep excavations are commonly constructed to maximize the land use for development in dense urban area. Many Hong Kong engineers are engaged in the design and manage engineering projects in Mainland China. As the construction cost and time of the excavation and lateral support works are always significant to the overall development, therefore, a cost effective excavation and lateral support work for deep basement construction should be implemented. The selected scheme should be within the budget, while it should also be designed within the allowable tolerance of the nearby utilities, buildings and underground properties as well as achieving in safe and fast track manner. As a professional engineer practicing in Mainland China, we should thoroughly understand the ground conditions, design methodology and practices of the design of excavation and lateral support works for basement excavation works in Mainland.

This paper presents some observations and overviews on the design and construction aspects for deep basement excavation in Mainland China. It outlines some key considerations behind ground investigation work, geotechnical design approach, code of practice, excavation methodology, construction monitoring and supervision requirements. A case history in Shenzhen of using 800mm thick diaphragm wall supported by 4-level circular reinforced concrete struts by means of bottom up excavation is also presented. The performance and lesson learnt have provided information which may lead to the adoption of a more practical construction solution for the engineers practicing in Mainland China.

2 OVERVIEW OF DESIGN PRACTICES IN MAINLAND CHINA

According to the codes of practice of Mainland China, the general design practices and considerations of the excavation and lateral support works can be briefly described in the following sections:-

(a) Category of the excavation works

The scale of the excavation works should be categorized into 3 main classes. Appropriate safety factors should be adopted for the different categories of the excavation work. The classification is depended on the depth of excavation, significance of the excavation works as well as sensitivity of the adjacent ground, utilities and buildings. The scale of the excavation can be categorized into three classes. They are summarized in Table 1.

Table 1: Category of the excavation works of excavation and lateral support design (based on JGJ 120-99 & SJG 05-96)

Category of Excavation	Criteria	Safety and Importance Coefficient, γ_o
I	1. Depth of excavation is greater than 14m; and 2. Presence of historic building, modern excellent buildings or sensitivity utilizes/services adjacent to the excavation.	1.1
II	1. Except Category I and III	1.0
III	1. Depth of excavation is less than 9m; and 2. No sensitivity buildings and utilities adjacent to the excavation.	0.9

One should note that the control limit to the induced movement for each category of excavation is also categorized. These control limits are summarized in Table 2.

Table 2: Control limit of the different categorized excavation class (based on SJG 05-96)

Category of Excavation	Control Limit of Horizontal Displacement of Temporary Shoring/Walling System	
	Bored pile wall, diaphragm wall, soil nailed wall	Sheetpile wall, soil mixing pile wall
I	0.0025h	N/A
II	0.005h	0.01h
III	0.01h	0.02h

*where h is the excavation depth.

The above control limits should be reviewed in accordance with the condition of adjacent properties / buildings / utilities to be affected by the excavation works.

(b) Ground Investigation Works

Ground investigation works will be undertaken at the early stage of the development as a common practice in Mainland China. The ground investigation work is to reveal the ground, geology and ground water condition for the design of foundation and geotechnical works. In normal practice, the investigation work will be designed and will be carried out by a qualified site investigation specialist institute. The institute will base on the proposed building layout to plan a ground investigation proposal according to local codes of practice. Upon completion of the ground investigation works, the institute will prepare a comprehensive ground investigation report and the contents of investigation report will include an outline of the topography, geology, ground profile, ground properties and groundwater conditions of the site. The recommendations on the foundation types, capacity as well as excavation methods for deep excavation including any necessary ground improvement works will also be given in the investigation report which should be formed part of the statutory document to support the design of the development.

Geotechnical design parameters are given in the ground investigation report for the engineering design. However, it is appeared that the recommended shear strength parameters are not specified whether they are applied for total or effective stress design analysis.

(c) Design Guidance and Methodology

There are many codes of practice related to the design and construction of the excavation and lateral support works to suit the different provinces of Mainland China. These codes of practice serve as guidance for the designer to follow in forms of Chinese national standards, Provincial government standards and Municipal standards.

The limit state approach of using un-factored parameters is adopted for the analysis of the design of the excavation and lateral support work in Mainland China. Ultimate limit state for the stability checking includes overturning, kicking out and piping failure checks. Serviceability limit state for the deformation checking includes wall deflection, ground and building deformation assessment. Those designs should also be based on the required factor of safety and the importance coefficient (γ_o) related to the category of excavation as stated on Table 1.

From many observations in Mainland China and guidance as stated on codes of practice, beam on spring elastic analysis method for the retaining wall design is commonly adopted for the design of the excavation and lateral support works. The designers would prefer to adopt the total stress approach for the design of deep excavation. A well known commercial computer program developed by a local institute give a comprehensive design solution for both ultimate and serviceability design checking. For the lateral support system, the mainland engineer commonly use structural computer programs for the frame analysis to design the lateral support system. It is also one of the statutory requirements that an expert review of the design and construction methodology for deep excavation works should be conducted for the application of construction permit.

(d) Excavation Methodology

-Temporary Walling System

There are many temporary wall types widely adopted in Mainland China. The choice of wall type should depend on the category of excavation. The commonly used wall types in Mainland China are summarized in Table 3.

Table 3: Type of walling system for the different categorized excavation class (according to JGJ 120-99)

Category of Excavation	Type of Walling System
I	Diaphragm Wall, Bored Pile Wall, Secant Bored Pile Wall, Hand Dug Caisson Wall
II	Bored Pile Wall, Secant Bored Pile Wall, Soil Mixing Wall
III	Cement Mixing Wall, Open cut with soil nail/anchors, Sheet Pile Wall

-Temporary Shoring System

Temporary shoring system including steel I-beam, steel tubular beam, soil anchor and reinforced concrete strut are commonly used in Mainland China. As the cost of steel is high, reinforced concrete struts are normal accepted by the local contractor as the cost of concrete is much cheaper. Moreover, the advantage of reinforced concrete strut is their relatively high stiffness and man-made on site. However, sufficient time should be allowed for reinforced concrete setting. The removal of reinforced concrete struts involving some blasting techniques should be considered in the design and construction in the subsequent basement erection. Soil anchor is also a common temporary shoring system in Mainland China. The beauty of using soil anchor for the excavation is that a large open working space can be provided for the subsequent excavation, foundation and sub-structure works. However, the feasibility of using soil anchor as a shoring system depends on the existence of foundation/structures of the nearby buildings. As the rapid growth of the development in Mainland China, it is expecting that the use of soil anchor for the excavation and lateral support works will be decreasing in the future.

(e) Instrumentation Monitoring

Many types of instruments are listed and required in the local code of practices of Mainland China to suit for the different categories of the excavation. The types of monitoring instrument are summarized in Table 4.

Table 4: Type of monitoring instruments (based on JGJ 120-99)

Type of Monitoring	Instrumentation	Category of Excavation		
		I	II	III
Deflection on Temporary Wall	Inclinometer	***	***	***
Deformation on adjacent building/utilities	Settlement / Tilting markers	***	**	*
Groundwater monitoring	Standpipe/piezometer	***	***	**
Deformation on Shoring/King Post	Settlement markers	***	**	*
Internal Stress on Temporary Wall	Strain Gauges	***	**	*
Internal Stress on Shoring Support	Strain Gauges	**	*	*
Deformation on soil stratum	Inclinometer / Extensometer	***	**	*
Heaving at the bottom of Excavation	Heaving Markers	***	**	*

Notes: *** Compulsory, ** Recommend, * Optional

The monitoring works are essential to ensure that the deep excavation works would be carried out in a safe manner. As stated on the codes of practice, appropriate ranges of alert, alarm and action value are also given for general guidance. For sensitive excavation work such as Category I excavation works, the monitoring works should also be carried out by a qualified independent third party surveyor directly employed by the developer. All independent monitoring results should be submitted to the Local Quality Bureau regularly as the safety control purpose.

(f) Role of the Design Engineer and Site Supervision

The construction of the excavation and lateral support works should be supervised by representatives from an independent qualified supervisory firm appointed by the developer according to the local statutory requirements. The role of the independent supervisory engineer is to ensure that the construction works would be carried out in accordance with the design and construction requirements, as well as relevant government quality standards. However, the role of designer during the construction stage turns into an advisor role. In some cases, Hong Kong developer may appoint the designer representative on site to support the local supervisory engineer and to make necessary design review for some significant deep excavation projects. This is different from the general approach adopted in Hong Kong where the geotechnical design engineer shall carry out on-site review and provide qualified supervision to the construction.

3 SOME CASE HISTORIES OF DEEP EXCAVTAION CONSTRUCTION IN MAINLAND CHINA

Some selected case histories of deep basement excavation projects in Mainland China are tabulated in Table 5.

Table 5: Summary of case histories of deep excavation method in Mainland China

Location and level of Basement	Excavation Depth (m)	Deep Excavation Method	Typical Ground Conditions	
4-level basement in Shenzhen	22	Phase I: 800mm thk Diaphragm wall supported by 4-layer circular reinforced concrete strut Phase II: 800mm thk Diaphragm wall using top down excavation	Silty clay, silty sand overlying decomposed rock; Groundwater at 3m below ground.	
2-level basement in FoShan Guangzhou	14	1.0m diameter bored pile wall supported by 3-layer soil anchors with ground treatment, and open excavation with soil anchors	Silty clay and sands overlying decomposed rock; Groundwater at 2m below ground.	
4-level basement in WuXi	22	1.3m diameter bored pile wall supported by 4-layer reinforced concrete strut and ground treatment	Clay and silty clay/ sands; Groundwater at 1m below ground.	
3-level basement in Beijing	20	1.1m diameter bored pile wall supported by 5-layer soil anchors	Sands, silty clay and gravels; Groundwater at 3m below ground.	
3-level basement in Shanghai	15	800mm thk diaphragm wall supported by 3-layer concrete struts with ground treatment	Shanghai clay; Groundwater at 1.5m below ground.	
Railway development in Shenzhen	18	1.2m diameter bored pile wall supported by 3-layer steel tubular strut	Silty clay, silty sand overlying decomposed rock; Groundwater at 2m below ground.	

The construction practice, which varies significantly, is dependent on the technical, cost, programme and practices in different provinces of Mainland China. The method of construction includes bottom up and top down construction, and the choice of suitable construction method of deep excavation is mainly controlled by the ground conditions, depth of excavation, and the sensitivity of the adjoining ground, buildings and structures. Ground treatment works are also commonly adopted to act as a groundwater cut-off measure and to minimize the wall movement effect on the adjoining underground structures such as MTR station and tunnel. In the detailed planning, understanding of the local design and construction requirements, the authors have provided successful designs to achieve time and budget for many deep excavation projects in Mainland China. One of the examples is illustrated in the following section.

3.1 Design and Construction Practice for One of the Deep Basement Projects in Shenzhen

-Site Location and Ground Conditions

The site is about 100m x 130m in plan, which is located at the urban centre of the Shenzhen district. The development comprises 2 commercial buildings with 4-level basement. The site is surrounded by existing operating buildings supported either on shallow footings or piling. As the site is within the existing metro railway protection zone, the design of the deep excavation and lateral support works should be complied with the protection requirements as stipulate in the local railway authority guidance. The site is characterized by a geological stratigraphic sequence of artificial fill, silty sand and clay, and weathered granite of different weathering grades as illustrated in Figure 1. The granite bedrock is varied at about 30m to 55m below ground and the groundwater level is very high of about 3m below the existing ground level. The geological conditions across the site are illustrated in Figure 2.

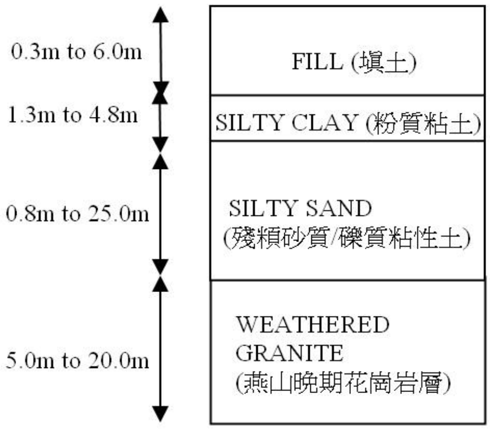

Figure 1: Geological straticgraphic sequence

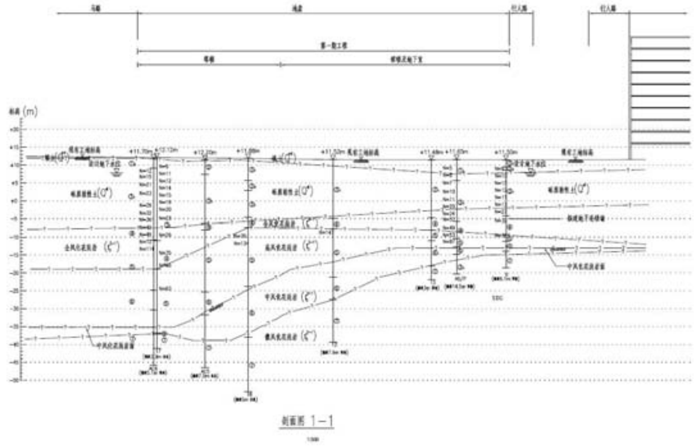

Figure 2: Geological cross section

-Selection of Construction Method for Deep Excavation

The excavation for the 4-level basement is about 22m in depth. The building development was divided into two phases to suit the site usage. A comprehensive option review on the construction methodology including technical, cost and time aspects was evaluated at the early stage of project. After detailed studies with the client, interview with the local design institute and specialist contractor, and the project quantity surveyor, 800mm thick diaphragm wall supported by 4-layer reinforced concrete circular beam using bottom up excavation method was the most cost-effective scheme and adopted for the 4-level basement construction for Phase I development. Upon the demolition of the existing building in Phase II and the completion of 4-level basement in Phase I, top down method supported by 800mm thick diaphragm wall are employed for Phase II development. The advantage of the diaphragm wall can act as a temporary and permanent structural wall. With using bottom up excavation method in Phase I, the superstructure foundation of hand dug caisson can be carried out after the completion of the excavation works. The significant cost and time for the excavation of the foundation to support the two high rise building structures can be optimized.

-Geotechnical Design Considerations

As lessons learnt from the several design and construction experiences in Mainland China, it is attention that the ground investigation report normally provides total stress parameters and deformation modulus parameters those are directly related to the laboratory results. For this project, it is recommended to the site investigation institute to undertake the sufficient drained and un-drained tri-axial tests, field tests and to provide a set of the total and effective stress parameters in the investigation report. The design analysis should be undertaken using total and effective stress parameters to demonstrate the deep excavation works achieving the certain accepted risk levels.

The diaphragm wall was terminated into bedrock to achieve stability and groundwater cut-off control issues. Reinforced concrete circular beam was used for the shoring elements to maximize the working space and the size of reinforced concrete circular beam was ranged from 850mm x 850mm to 1600mm x 2000mm. With the use of the circular shaped strutting, a large open working space is fully utilized to enhance the subsequent hand dug cassion foundation construction upon the completion of excavation works. The layout and a typical section of the excavation and lateral support works are shown on Figures 3 and 4 respectively.

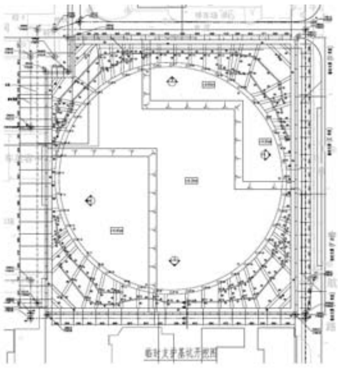

Figure 3: Layout of ELS works

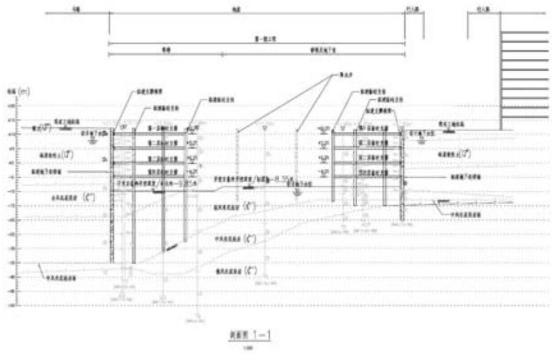

Figure 4: Cross section of ELS works

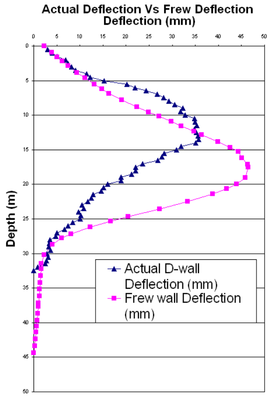

Figure 5: Wall deflection

In the analysis, Oasys FREW was used to compute the loads of the temporary wall and the shoring system by using the effective stress parameters. To assess the behavior of the reinforced concrete circular struts, structural computer program SAP2000 was used in considerations of the compatibility of the circular strut deformation and the estimated wall deflections from the FREW analysis. According to the assessment, the predicted wall deflection and ground deformation were 47mm and 25mm respectively. According to the monitoring results, the maximum wall deflection and ground deformation were only 36mm and 19mm respectively. Thus, it is revealed that the monitored movements were generally within the predicted values and the robustness of the geotechnical design was proven. The comparison of the wall deflection and strut forces as an example is illustrated on Figures 5 and 6. Back analysis has been conducted according to the observed wall deflections to verify the ground stiffness applied in FREW. The results of the back analysis showed that the Young Modulus E value of the fill and silty clay soils could be up to 1.5N whereas CDG could be up to 3N, which is consistent to the results in some case histories published in Hong Kong.

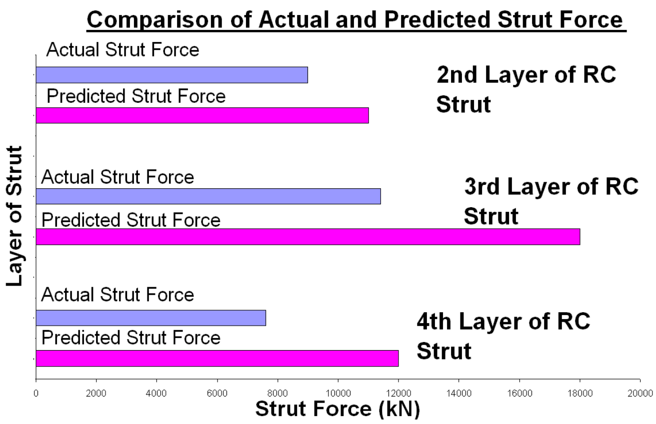

Figure 6: Calculated and measured strut force in RC circular struts

Plate 1: Progress photos for deep excavation and basement construction in Shenzhen

This project was selected by the Development Bureau of Shenzhen as one of the learning modeling sites of the deep basement excavation work in Shenzhen. A design workshop was conducted to share the project experience with the local Mainland engineers for the design and construction considerations of the safe and cost-effective deep excavation in Shenzhen. During the workshop, the major different cultures in particular on the caution of the geotechnical design considerations, design on-site review and qualified supervisions for the deep excavation design and construction practices between Mainland China and Hong Kong were highlighted.

4 CONCLUDING REMARKS

Some observations on the design and construction aspects of deep basement excavation in Mainland China have been illustrated in this paper. A case history of 4-level basement excavation and construction has been presented. Some lessons learnt on the design and construction issues for the deep excavation works in Mainland China are made and they may be helpful to the engineers working on similar projects in the future.

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Management of the Design and Construction of Kai Tak Cruise Terminal Cable Tunnel

S.P. Tang & Joseph Cheung
CLP Power Hong Kong Limited, Hong Kong

Charles Jim & Joe Lau
Atkins China Limited, Hong Kong

ABSTRACT

The new Cruise Terminal with two alongside berths suitable for mega vessels and the associated tourism-related facilities, included hotels, shopping malls and conference facilities, will be developed at the southern end of ex-Kai Tak Airport runway and to be commissioned from 2012. To facilitate the power supply for the development and avoid frequent excavation in the future runway park, a 300m long twin box cable tunnel emanating from the Kai Tak Cruise Terminal (KCT) Substation will be constructed to accommodate a total of 60 nos. 11kV and 18 nos. 132kV power cables for electricity supply to the future cruise terminal and it's vicinity areas. The project is challenging as the works have to be completed within 18 months. According to the lease requirement of the Short Term Tenancy, the works area has to be divided and handed over in 2 phases, where the first phase is to be completed within 9 months to permit possession by other utility contractor and another 9 months was allowed for the second phase. This paper presents the design management and key design and construction features of the cable tunnel project adopted to ensure timely completion of the project with due consideration on safety, performance, cost, quality and reliability.

1 INTRODUCTION

The alignment of the KCT cable tunnel was designed to run parallel with the existing pitched slope seawall about 20m to the northeast of the site and connects with the KCT Substation at the southeastern end to avoid conflicts with future development (See Figure 1).

Figure 1: Overview of the KCT Cable Tunnel project

Typical installation of underground cables involved the direct burial of cables within excavated trenches. The entire process of trench excavation, cable laying and reinstatement is time consuming and oftentimes opened or decked trenches cause inconvenience to the publics. Moreover, congestion of underground utilities often leads to works area problem amongst various utility undertakings including telecommunications, drainage, water, gas and etc. As such, adaptation of underground cable tunnel is considered the best option in addressing the above constraints, particularly in densely populated areas, with the following reasons:

- Avoid third party influence
- Allow easy maintenance
- Reduce disturbance to traffic and inconvenience to the public
- Reduce environmental impacts
- Minimize cost of network reinforcement

2 DESIGN MANAGEMENT OF THE WORKS

According to the contractual requirement of this Design and Build Contract, the contractor has to appoint her own designer to carry out the design of the works. The designer shall be suitably experienced in the design of a tunnel of the required type and the design shall be verified by an Independent Checking Engineer (ICE). The role of the ICE is to ensure all temporary and permanent works is of sound design and to issue a Design Checkers' Certificate upon his satisfaction of the design submissions. To effectively manage the detailed design and construction stage of this project, a Responsible Officer Representative (ROR) was employed to oversee the contract administration and site supervision. His role and responsibility were delegated by the Responsible Officer at the onset of the project in accordance with Conditions of Contract. The Project Management Team is detailed in the following organization chart (Figure 2).

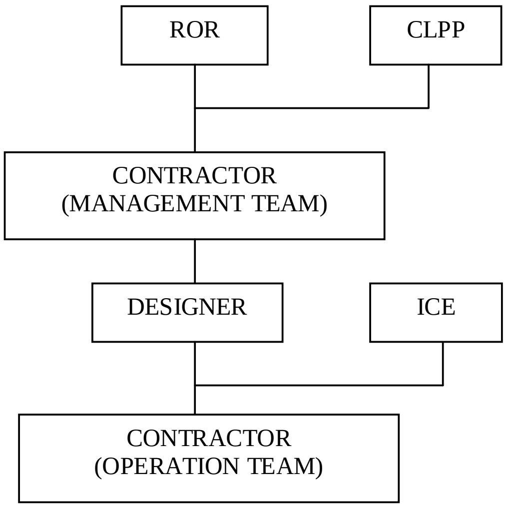

Figure 2: Organization chart of the KCT Cable Tunnel project

The designs performed were subjected to design checking requirements as stipulated in the contract. The objective is to ensure that the design is checked for compliance with the Employer's Requirements. In principal, the design checking process consists of two stages: 1) Approval in Principal (AIP); and 2) Detailed Design Approval (DDA). The two stage approval processes, as shown in Figure 3, was conducted to ensure effective design development and also to obtain governments approval timely.

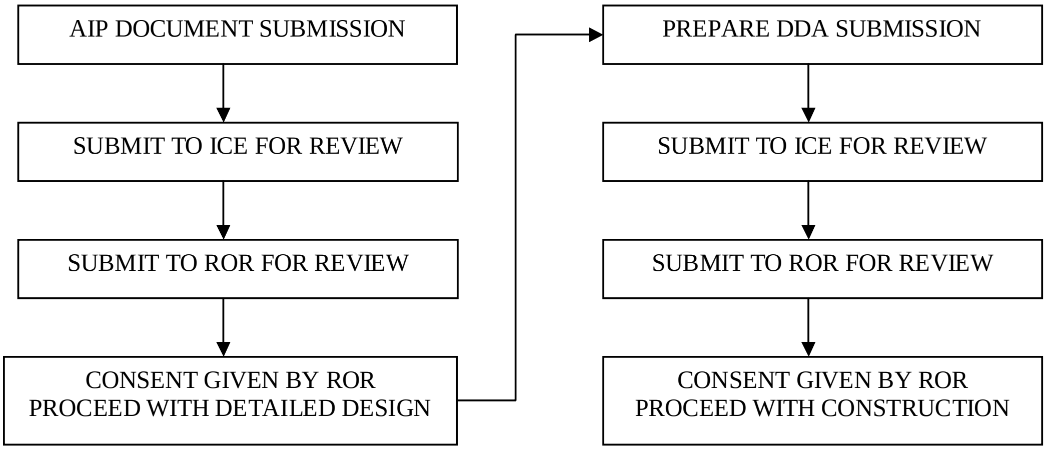

Figure 3: Flow chart of design review process

2.1 Risk Management

Risk management is a complete process involving risk identifications, risk assessments for likelihood and potential impacts, and strategies development to eliminate the risks or reduce its impacts at early stage. Risk will change throughout the life of the project and risk management will therefore be an ongoing activities. The ROR was required to take the lead to adopt a formal approach by conducting risk management workshop on this cable tunnel project before works commencement in order to identify risks as early as possible, and to manage them and reduce impact on site works, in terms of cost, time, safety, quality, environment and impacts on the public. In practice, this is an effective tool for decision making process.

A risk management system was set up during the planning stage with the following main characteristics:

- Project Risk Management Register outlined the plan and procedure for risk assessment and management of the KCT cable tunnel in order to identify, access, reduce and control the risks to ensure timely project completion within budget
- Risk Register was reviewed in the workshop with all related stakeholders including the employer, the ROR, main contractor and design consultant
- Risk response actions as described in the register were assigned to the risk owners in order to manage the risks
- Further risk workshops were held to review the validity of existing risks and also to identify any new risks during construction stage

2.2 Value Engineering

To encourage adaptation of best design practice with due consideration on minimal maintenance and optimum service life, a Value Engineering (VE) was bind in the contract as initiative to encourage the contractor to capture innovations in their design which fulfils functional requirement and at the same time achieve cost savings. The saving achieved would be shared between the employer and the contractor.

With effective implementation of Risk Management and Value Engineering, the design and construction works for the cable tunnel has been delivered with high quality in terms of overall cost-effectiveness, works quality and site safety.

3 DESIGN ASPECTS

3.1 Site Constraints

It was identified at the early stage of the project that the works area was restricted by the Short Term Tenancy (STT) land boundaries in respect of the nearby concurrent CEDD’s road and drainage works. Open cut excavation is therefore infeasible and excavation with lateral supports (ELS) was adopted as the conforming scheme.

As the site is classified as littoral zone which is surrounded by the sea, its groundwater level is subjected to tidal change. The design groundwater level was assumed at +3.6mPD in accordance with the Port Works Design Manual (CED, 2002). The high groundwater table and the effects of recharge from the sea becomes a major challenge for the design of both the permanent and temporary works.

Relatively low stiffness (mean SPT ‘N’ value below 10) of the existing fill within the excavation depth is also considered as another constraint. With such low SPT value, relatively large deformation of the ground and a heavy duty ELS structure were thus expected. The existing fill is predominately sand fill with a thickness of about 20m underlying by a layer of Alluvium.

3.2 Key Design Features of Temporary Works

In the selection of retaining structure as part of the required ELS, the use of sheet piles or pipe piles were considered as feasible options. From rough estimation, the use of pipepile wall would have doubled the cost when compared with the use of sheet piles. It was mainly due to the cost associated with installation of grout curtain for groundwater control and “left in place” pipepile wall. On the other hand, the site was located at open area of former Kai Tak runway where there is no other existing buildings or features nearby the works area that is likely to be affected by vibration and settlement problems due to deep excavation. As such, sheet piles were selected as the conforming scheme in view of lower construction cost and satisfactory performance on groundwater control. To effectively control ground movement, the proposed ELS system consists of FSP-III and FSP-IV Sheet piles with 3 layers of supporting steel struts and waling for the required 8m deep excavation. The sheet pile was designed to penetrate 13.5m deep and the required toe-in depth was 5.5m. Provisional measures such as the installation of grout curtains behind the embedded sheet piles were also allowed in dealing with problems such as high groundwater level and recharge effect from the sea. A layout plan of the site and the cable tunnel is given in Figure 4 and a typical cut section of the ELS system is shown in Figure 5.

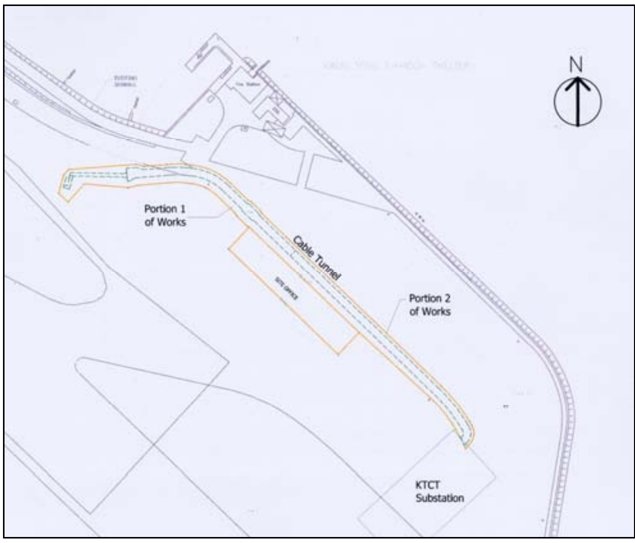

Figure 4: Site layout plan of KCT Cable Tunnel

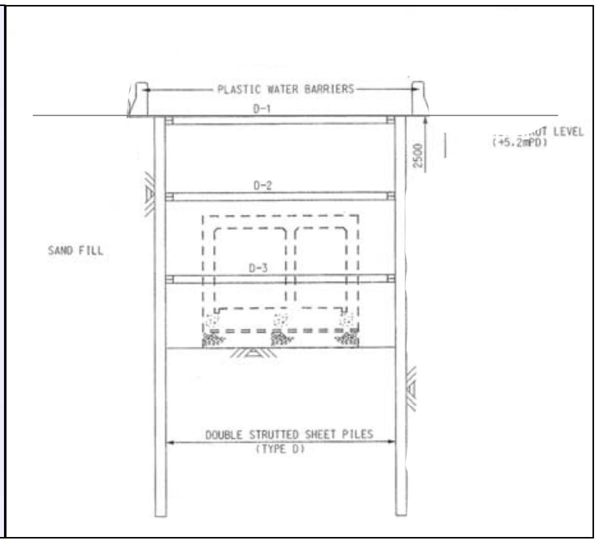

Figure 5: Typical section of the ELS system

The design of ELS system was based on Conventional limited state design, with reference to GCO Publication No.1/90 (GEO 1990) and Geoguide 1 (GEO 1993) to check the structural stability of the ELS system. Structural analysis programme by Finite-element modeling computer program “PLAXIS” was used to carry out detail design. Design parameters adopted were based on the results of previous ground investigations. In particular, for the settlement analysis, the Young’s modulus (E) of the fill material was derived based on empirical correlation with SPT ‘N’ values. Chan (2003) studied the case histories for a range of deep excavation in Hong Kong and suggested that an E value (MPa) equal to 1.5 times the ‘N’ value for fill material would give a reasonable estimation on calculated wall deflection. The use of this correlation was expected to yield an economical design which forms part of the common goal of the employer and the contractor in respect of the design constraints described in the previous section.

For the design against groundwater control, the designer performed seepage analysis to calculate rate of groundwater ingress to excavated trench and to ensure sufficient toe-in depth was being provided to prevent against hydraulic failure. From the analysis results, the designer would be able to determine the required number of temporary pumps required during the construction stage.

3.3 Key Design Features of Cable Tunnel

In accordance with the employer’s requirement to house the power cables, the KCT Cable Tunnel was designed as a twin box reinforced concrete (RC) structure (3.7m high x 5.0m wide) with a length of approx. 300m and to be founded at a depth about 8m below the existing ground. A typical section of the twin box cable tunnel is shown in Figure 6. The cable tunnel was designed to be submerged entirely in groundwater where the tunnel was designed as a RC water-retaining structure. Apart from the structural design against loads from overburdened soils, groundwater, E&M installations and other live loads, structural stability checking against floatation effects must also be considered. To maintain the tunnel dry without any drips or water inflow, the watertightness of the cable tunnel was achieved by adopting a special waterproofing system. This system was designed to enfold the tunnel by a layer of waterproof membrane with double water-stops at the construction joints as shown in Figure 7. The advantage of enfolding waterproof membrane is comparatively better than the use of sprayed or painted type waterproof layer because it offers fast installation and better durability since the membrane itself can be lapped by build-in adhesive without the need for welding together.

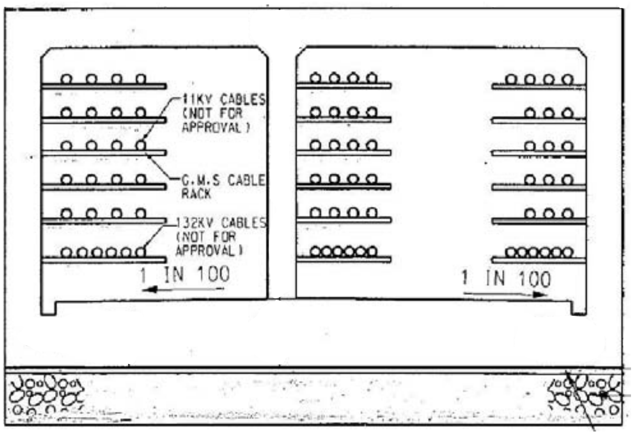

Figure 6: Typical section of cable tunnel box structure

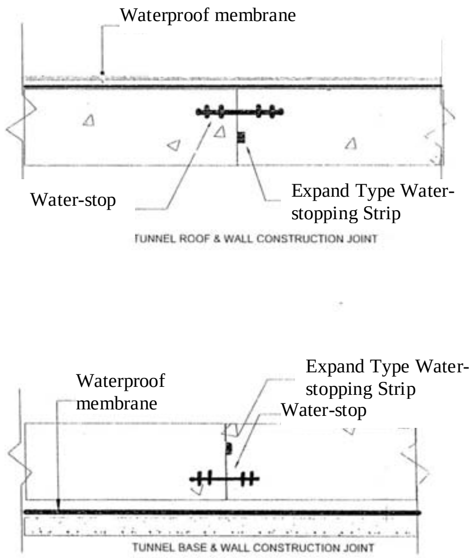

Figure 7: Typical waterproof details at construction joint

4 CONSTRUCTION ASPECTS

4.1 Diversion of Existing Utilities

Further to the risk assessment carried out at the initial stage of the project, it was noticed that a large diameter (1200mm) storm water drain and a 250mm diameter fresh water main were obstructing the construction of the Phase I cable tunnel and advance diversions works were necessary to facilitate cable tunnel construction. Other utilities were also found near to the edge of the proposed excavation. To protect the existing services, the contractor had conducted a comprehensive investigation to identify the location of existing utilities prior to the commencement of the works. The inspection comprised the non-destructive radar detection of services followed by the excavation of inspection pits at varies locations for verification. Utility diversion was thus carried out in a timely and safe manner.

4.2 Instrumentation and Monitoring

A comprehensive instrumentation and monitoring for the adjacent existing services and the existing ground is the key to ensure no undue settlement affected by the tunneling works. In terms of construction risk assessment, the Alert, Action & Alarm levels had been defined as 50%, 75% and 100% of the allowable value respectively for vertical settlement and angular distortion against different existing services and the existing ground. On top of the general practice to carry out daily monitoring during the course of the works, monitoring period was commenced 2 weeks prior to the commencement of works, and would cover a period of 2 months after works completion to ensure the ground and structural movements are stabilized.

Vibration monitoring to the existing utilities and the nearby pitched slope seawall was also carried out on a daily basis during the course of ELS installation to avoid monitor the impact caused by site activities.

4.3 ELS Works

As a result of the early involvement of the contractor under the design and build contract, the contractor take lead to achieve design refinement on ELS system during construction stage. The use of tailor made 14m long steel sheet piles.

Sheet piles are usually fabricated in 12m long and given that the required penetration depth of the sheet pile wall was 13.5m for this project. The use of tailor made 14m long sheet piles will eliminate intensive site welding works on site and eventually shorten the time of ELS installation by almost a week.

The use of “double struts” without “splay head” as shown in figure 8 also demonstrate the design refinement on ELS design. “Double struts” was designed to group in 2 successive struts together. With the increase in stiffness, the spacing between struts can be maximized. The said arrangement on strutting layout would maximize working spaces also provide fast installation due to reduction on site welding and associated compliance test. Moreover, double struts also reduce cutting of short steel where contractor could reuse the structural members for subsequent use in other project which is cost effective in long run. From quality point of view, double struts also reduce wall openings and hence a better waterproofing performance is achieved.

To reduce the use of steel, Contractor have use cement grout bags as the connection between sheet piles and waling as shown in figure 9. Conventional connection between the sheet pile and waling comprises inserting pieces of steel plate/channel welded to both the sheet pile and waling as for load transfer. The alternative detail using cement grout bags not only reduced the hot works on site but also fasten the structural installation significantly.

Figure 8: “Double strut” without “splay head”

Figure 9: Cement grout bags between sheet piles and waling

Considering the constraint on Works Area on Phases handover of Phase I works area, the contractor hence worked out the transition detail for the ELS as shown in Figure 10 to keep the tunnel dry without water ingress to Phase I area upon handover. Upon the completion of the cable tunnel structure for Phase I works, a temporary “bulk head” was installed at the cable tunnel opening at the interface of Phase I and Phase II to allow an “overlapping” region for future preparation of construction joints and waterproof membrane installation of Phase II. This bulk head comprised steel plates supported by brick/concrete packing and welded to sheet piles combined with mass concrete infill to prevent ingression of soil and groundwater at the interface. The simple design of this “bulk head” proposed by the contractor facilitated the phasing construction of the cable tunnel in a relatively low cost compared to open cut excavation where repeated excavation and backfilling may be required despite the fact that some of the construction material had to be left behind.

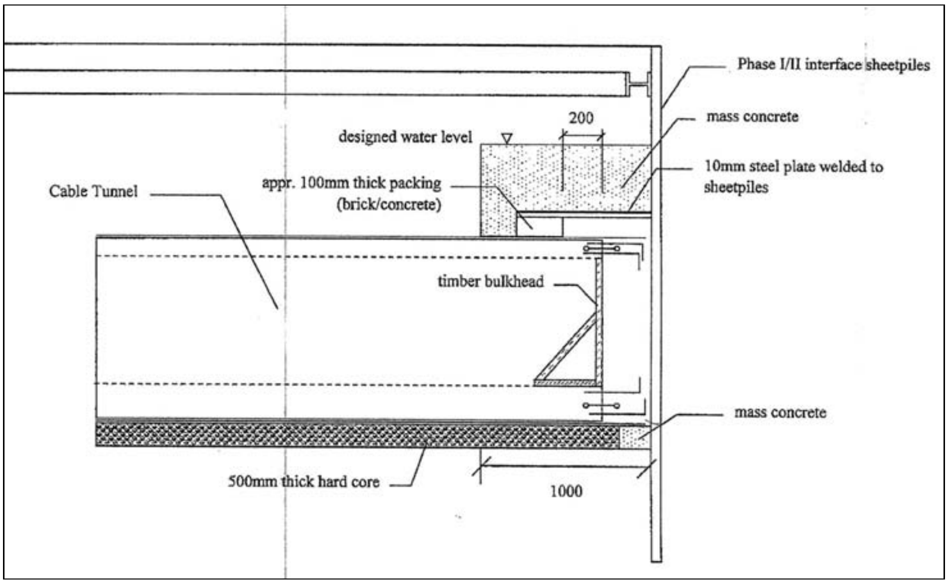

Figure 10: Details of temporary works at interface between phase I and phase II cable tunnel (longitudinal section)

4.4 Construction consideration of the Twin Box Cable Tunnel

The proposed cable tunnel is a RC twin box water-retaining structure. In respect of the employer's waterproofing requirements, the tunnel structure was casted in segments with a relatively shorter length of about 11m. Successive segments were connected by Construction joints were provided between segments with double water-stopping strips as described in Section 3.3. The short segment length helped to control the development of shrinkage cracks during concrete casting.

5 CONCLUSIONS

The design of the KCT cable tunnel had been successfully developed by the project team through adaptation of systematic management approach on approval process, risk management and value engineering exercise to cater for time, budget and physical site constraints. Construction of the works was also delivered in a safe, environmental friendly and effective manner as a result of good design management. More importantly, all stakeholders including the employer, the consultant and the contractor have worked together in a partnering approach and that is the essential element contributed to the success of the project.

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Comparison and Verification of Numerical Methods for Deep Excavation Design Adopting CIRIA Report C580

J. Yiu & M.C.Y. Ko
Ove Arup & Partners (H.K.) Ltd.

ABSTRACT

Since the release of CIRIA Report C580 on the design of deep excavation, the industry has been gradually moving towards the use of limit state design approach for excavation and lateral support (ELS) design. This involves using numerical methods to carry out both serviceability limit state (SLS) and ultimate limit state (ULS) analyses. In this paper, the authors compares and verifies two commonly used numerical methods, viz. boundary element method (BEM) and finite element method (FEM), on the analysis of ELS systems when adopting C580 approach. The result shows that both BEM and FEM were able to produce consistent and reliable estimate on horizontal soil pressure, wall deflection, bending moment, shear force, and strut forces in deep excavation. In addition, the paper discusses and explains the input parameters that are critical to the accuracy of both numerical methods. The predicted behaviour of the ELS systems in ULS condition towards failure is investigated and comments on the checking of toe stability are given.

1 INTRODUCTION

The use of numerical methods in deep excavation analysis is considered a common yet essential process for the design of an excavation and lateral support (ELS) system nowadays in Hong Kong. The conventional analysis, which is by large adopting the recommendation in GCO Publication 1/90 (GEO 1990), involves the use of numerical methods to estimate wall movement and structural forces. An additional hand calculation is carried out to check whether the retaining wall has sufficient toe penetration to achieve the required global factor of safety and guard against toe stability failure.

The release of CIRIA Report C580 (Gaba et al. 2003) provides an alternative design approach that builds on the concept of limit state design. This involves using numerical methods to carry out both serviceability limit state (SLS) and ultimate limit state (ULS) analyses. Structural design forces are determined by comparing the SLS and ULS results. The overall stability including toe stability of the ELS system is checked and demonstrated by the successful convergence of the numerical analysis in the ULS condition.

Over the last few years, the industry has been gradually moving towards the use of C580 approach for ELS design. As the approach heavily relies on the use of numerical methods in both SLS and ULS calculations, there have been concerns that whether consistent and reliable results can be obtained using different numerical methods.

In this paper, the authors compares and verifies two commonly used numerical methods, viz. boundary element method (BEM) and finite element method (FEM), on the prediction of wall deflection and structural forces of ELS systems when adopting C580 approach. Input parameters that are critical to the analyses are discussed and explained. The predicted behaviour of the ELS systems in ULS condition towards failure is investigated and comments on the checking of toe stability are given.

2 NUMERICAL METHODS FOR DEEP EXCAVATION DESIGN

2.1 General

An overview on different analytical methods for ELS design has been given in GEO (1990). The following paragraphs aims to provide a general description on the two numerical methods that are studied in this paper.

2.2 Finite Element Method (FEM)

The calculation using FEM involves discretizing the soil continuum and the structural components into manageable geometry, namely elements, that formed by sets of nodes. The initial ground condition, boundary conditions for the excavation, and the material strength and stiffness properties of different elements can then be specified. Based on the constitutive model adopted for different materials, the stiffness matrix is developed. An iteration technique is then adopted to solve the matrix and achieve equilibrium within an acceptable tolerance. Stresses, strains and displacements of each element at equilibrium are calculated. These results can be further evaluated to produce estimated wall movements and structural forces.

The continuous improvement of computing power of personal computers and the emergence of more user-friendly commercially available computer programs in recent years have made FEM more assessable to engineers and becomes a viable tool for daily ELS design in Hong Kong.

2.3 Boundary Element Method (BEM)

Although FEM provides a complete method for ELS analysis, demand on computational resource and time could increase considerably by the complexity of the model, such as deep excavation modelling that involve complex construction sequences. In view of this drawback some task specific computer program based on boundary element method (BEM), also known as pseudo finite element method, was developed for a quicker analysis that retains reasonable degree of accuracy.

One of the examples is presented in Pappin et al. (1985). The method involves modelling the retaining wall as a series of elastic beam elements. Rather than developing the stiffness matrix through discretizing the soil continuum, it makes use of two pre-calculated soil stiffness that obtained from previous finite element analyses of elastic soil blocks. Horizontal earth pressure coefficients (K_a, K_p) are used to control the soil strength limits, as well as to adjust the strength of the soil-wall interface. Additional equations and adjustments are introduced into the method to cater for the phenomenon of stress redistribution and the presence of surcharges. GEO (1990) commented that this method was found to be cheaper to apply with sufficient accuracy for most design problems and still be widely used in Hong Kong.

3 THE COMPARISON

3.1 Overview

The following commercially available computer programs, which are also commonly used for ELS design in Hong Kong, were employed for the comparison work:

i) FEM – Plaxis 2D
ii) BEM – *Oasys* FREW

Two sample cases were chosen for the study and both SLS and ULS analyses have been carried out in Plaxis 2D and *Oasys* FREW. Results such as wall deflection, bending moment, shear force and strut forces were compared. Although the design methodology as detailed in C580 was followed, partial factors as suggested in “Notes on Design of Excavation and Lateral Support Works Using the Limited State Partial Factor Method" (BD 2005) were adopted instead to reflect the application of limit state design for ELS in Hong Kong.

It should be noted that whilst the required inputs for each program are slightly different from each other, the authors have made their best effort to ensure all input parameters and construction sequences were consistent, such that their influence on the comparison result were kept to minimal.

3.2 The Sample Cases

The sample cases were chosen from the test examples adopted in J. Pappin & T. Tham (2005). They are graphically illustrated in Figure 1 and 2.

Whilst most of the information was provided, including geological condition, groundwater table and dewatering levels, applied surcharge, excavation levels and stiffness of structural members, the following data were left to the authors’ decision:

- The sequence of surcharge application, i.e. whether the surcharges are applied at the same time, and whether they are applied prior to the installation of the retaining wall;
- Initial horizontal soil stress coefficient (K_0);
- Seepage condition i.e. whether it is in hydrostatic or steady state seepage condition.

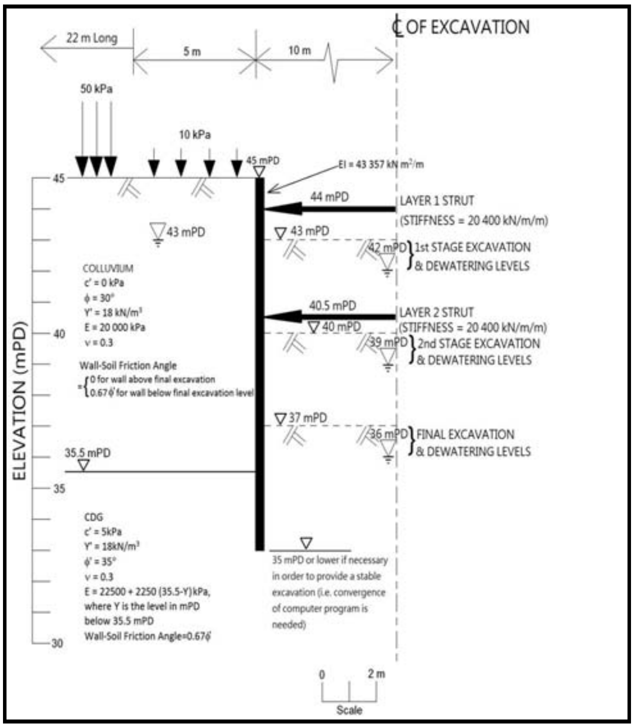

Figure 1: Sample ELS case 1

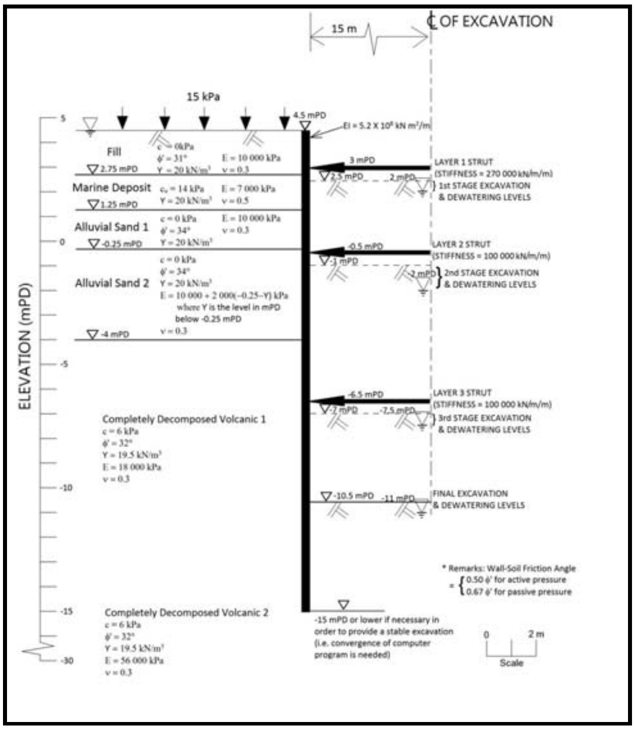

Figure 2: Sample ELS case 2

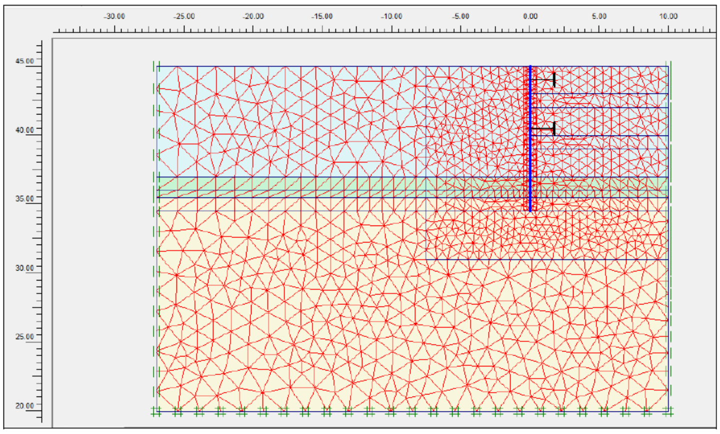

Figure 3: Plaxis 2D model adopted for sample case 1

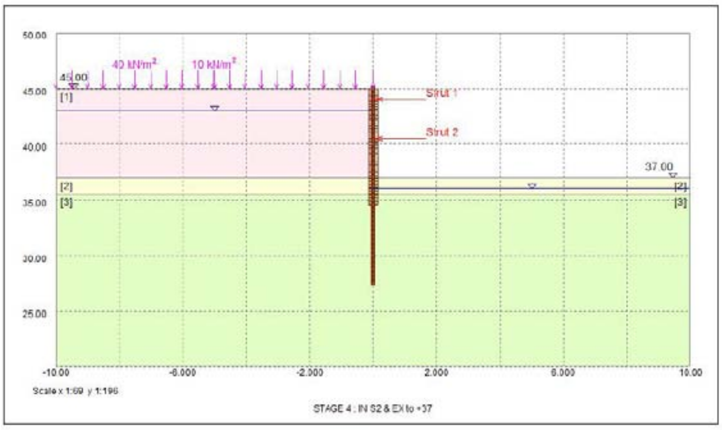

Figure 4: *Oasys* FREW model for sample case 1

3.3 The Model Setup

Figure 3 and 4 show the model setup in Plaxis 2D and *Oasys* FREW for sample case 1. Meshes created in Plaxis 2D for the sample problems were made sufficiently fine, especially in the area around the retaining wall, in order to eliminate unnecessary inaccuracy of the Plaxis 2D models due to insufficient node density. Similarly, sufficient nodes along the modelled retaining wall were created in the *Oasys* FREW model to ensure the accuracy of calculation.

3.4 The Input Parameters

The input parameters used in Plaxis 2D and *Oasys* FREW mainly followed the values shown in Figure 1 and 2. They were adopted directly in the SLS analysis case. For ULS analysis case, partial factors as suggested in BD (2005) were adopted.

Specific assumptions and adjustment on input parameters in Plaxis 2D and *Oasys* FREW are discussed below:

3.4.1 Sample Case 1

Oasys FREW –

In order to allow for a better estimate of horizontal soil stress due to the application of surcharge on the retained side ground level, the 5m wide 10kPa and 22m wide 50kPa surcharge were modelled as a 10kPa uniformly distributed load plus 22m wide 40kPa strip load. The total applied surcharge remains as specified in the sample case.

The horizontal earth pressure coefficients (K_a, K_p, K_{ac}, K_{pc}) of each soil type were derived using the equations presented in Annex G of Eurocode 7 (BSI 2004). K_0 for all soil materials were calculated by using the following equation:

$$K_0 = 1 - \sin(\varphi') \qquad (1)$$

Plaxis 2D –

It was noticed that the program became unstable when specifying a zero wall-soil friction angle, which was shown in the sample case. Therefore a very small wall-soil friction angle i.e. 0.1 of the parent soil material, was adopted instead for the colluviums layer above the final excavation level.

K_0 for all soil layers were calculated by using Eq. (1).

3.4.2 Sample Case 2

Oasys FREW –

Similar to sample case 1, BSI (2004) was adopted for determining horizontal earth pressure coefficients. K_0 for all soil materials except the undrained Marine Deposit were calculated using Eq. (1). K_0 for Marine Deposit was assumed to be 1.0.

Plaxis 2D –

The derivation of K_0 in the Plaxis 2D model for sample case 2 was the same as that of *Oasys* FREW.

3.4.3 Groundwater conditions

The groundwater level of both retained side and excavated at each excavation was specified in the sample cases. However, seepage condition was not clearly stated. Therefore the following assumptions were made for each sample case:

- Sample case 1 was assumed to be in steady state seepage condition. The steady state pore water pressure was calculated by adopting the method as described in figure 33 of Geoguide 1, 2nd Edition (GEO 1993).
- Hydrostatic condition was adopted throughout all construction stages for sample case 2.

3.5 Construction Sequence

The construction sequence for both sample cases generally follow the details as shown in Figure 1 and 2. The following additional assumptions were made:

- Any surcharge is applied after the retaining wall has been installed;
- 500mm unplanned excavation is allowed in every excavation stage in the ULS calculation.

3.6 Results

Table 1 to 4 summarise various maximum values estimated by two programs for the sample cases. Selected profiles of horizontal soil stress, wall bending moment, shear force and deflection are also shown in Figure 5 to 7 for reference.

Table 1: Result comparison for case 1 (SLS condition)

		Plaxis	FREW	Difference	% ratio
		a	b	abs(a-b)	a/b
Max. Shear Force	kN/m	95	89	6	106.7%
		-81	-76	5	106.6%
Max. Bending Moment	kN-m/m	156	155	1	100.6%
		-38	-47	9	80.9%
Max. Layer 1 Strut Force	kN/m	125	136	11	91.9%
Max. Layer 2 Strut Force	kN/m	130	141	11	92.2%
Max. Wall Deflection	mm	41.6	42.8	1.2	97.2%

Table 2: Result comparison for case 1 (ULS condition)

		Plaxis	FREW	Difference	% ratio
		a	b	abs(a-b)	a/b
Max. Shear Force	kN/m	197	195	2	101.0%
		-150	-155	5	96.8%
Max. Bending Moment	kN-m/m	287	280	7	102.5%
		-247	-297	50	83.2%
Max. Layer 1 Strut Force	kN/m	204	247	43	82.6%
Max. Layer 2 Strut Force	kN/m	292	330	38	88.5%
Max. Wall Deflection	mm	209.2	227	17.8	92.2%

Table 3: Result comparison for case 2 (SLS condition)

		Plaxis	FREW	Difference	% ratio
		a	b	abs(a-b)	a/b
Max. Shear Force	kN/m	624	567	57	110.1%
		-340	-361	21	94.2%
Max. Bending Moment	kN-m/m	1832	1942	110	94.3%
		-715	-607	108	117.8%
Max. Layer 1 Strut Force	kN/m	289	293	4	98.6%
Max. Layer 2 Strut Force	kN/m	606	565	41	107.3%
Max. Layer 3 Strut Force	kN/m	937	929	8	100.9%
Max. Wall Deflection	mm	70.8	65	5.8	108.9%

Table 4: Result comparison for case 2 (ULS condition)

		Plaxis	FREW	Difference	% ratio
		a	b	abs(a-b)	a/b
Max. Shear Force	kN/m	1259	1118	141	112.6%
		-948	-855	93	110.9%
Max. Bending Moment	kN-m/m	1989	1997	8	99.6%
		-4277	-4115	162	103.9%
Max. Layer 1 Strut Force	kN/m	565	557	8	101.4%
Max. Layer 2 Strut Force	kN/m	1080	943	137	114.5%
Max. Layer 3 Strut Force	kN/m	1937	1782	155	108.7%
Max. Wall Deflection	mm	178	187.4	9.4	95.0%

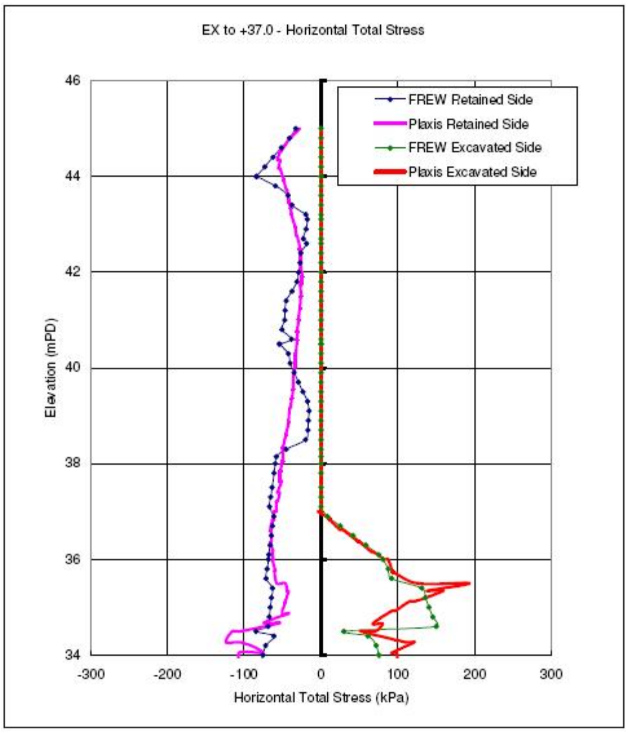

Figure 5: Calculated horizontal soil stress along retaining wall (case 1 SLS condition)

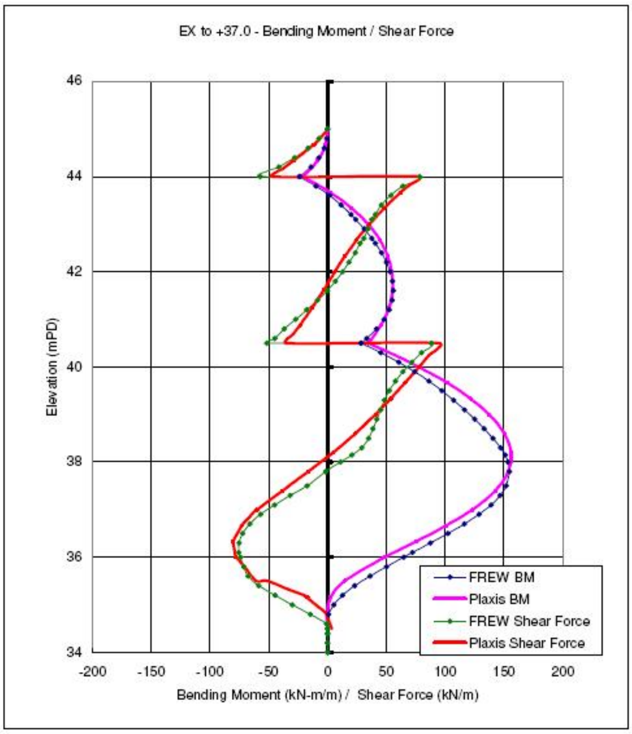

Figure 6: Calculated wall bending moment and shear force (case 1 SLS condition)

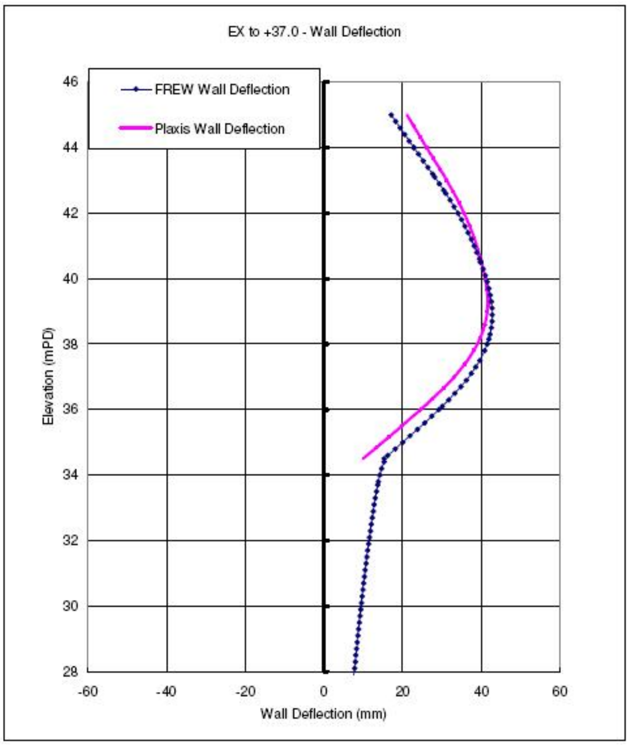

Figure 7: Calculated wall deflection (case 1 SLS condition)

4 DISCUSSION

4.1 General

As revealed from the above tables and figures, results of both Plaxis 2D and Oasys *FREW* demonstrated very similar estimate on horizontal soil stresses, retaining wall deflection and forces in modelled structural elements. In general, the difference in the results was within 10%. Given the natural variation of the actual ground condition, these results were considered satisfactory for daily engineering design for deep excavation. It also demonstrated that both FEM and BEM are the appropriate methods for ELS design.

4.2 Selection of Input Parameters

4.2.1 General

Although the comparison showed the two numerical methods give consistent results, the authors noted that the selection of the following input parameters may cause noticeable difference in predicted wall movement and structural forces:

4.2.2 Wall-Soil Interface

As mentioned in Section 2.3, BEM analysis takes the advantage of pre-calculated soil stiffness matrix obtained from previous finite element analysis in its calculation. For example in *Oasys* FREW, pre-calculated soil stiffness matrix based on fixed or free wall-soil interface solution can be chosen. In the fixed solution, the soil stiffness matrix is generated from a finite element model in which the soil blocks are constrained at the interface with no vertical displacement allowed. On the other hand, in the free solution vertical displacement at the interface is allowed and no shear can be transmitted.

In this study, it was noted that for most situations when the anticipated vertical movement of the wall relative soil is small the fixed solution would give results closer to that of Plaxis 2D. However, when analysing the behaviour of the ELS system which the wall is expected to move vertically relative to the soil and/or the result is close to non-convergence in the *Oasys* FREW analysis, e.g. sample case 1, the free solution would give a better estimate on wall deflection and structural forces.

It should be reminded that the choice of the wall/soil interface option is related to the modelling of the relative soil/wall movement in the vertical direction, and this should not be confused with the choice of K_a or K_p values that correspond to the wall friction available. The modeller should obtain the correct K_a and K_p values for *Oasys* FREW inputs by considering the available wall friction.

In FEM analysis which the retaining wall is modelled as a 1D element, the modeller should be aware of the fact that non-convergence may occur if zero wall-soil friction is specified. A parametric study was done by analysing the wall movement in the first stage excavation (2.0m from existing ground level) in sample case 1 with different wall-soil friction adopted in Plaxis 2D. The result is shown in Figure 8. It can be seen that excessive wall movement was predicted in Plaxis 2D when if wall-soil friction was taken less than 0.1. Therefore, it is suggested that a small wall-soil friction should be adopted even for the case of no wall-soil friction is available.

4.2.3 Lateral Earth Pressure Coefficients

From the result of the study, it can be seen that the lateral earth pressure coefficients calculated using Annex G of Eurocode 7 in *Oasys* FREW provides consistent results with Plaxis 2D analysis. It is therefore suggested, in addition to adopting figure 18 & 19 of Geoguide 1(GEO 1993), Annex G of Eurocode 7 can be adopted for estimating lateral earth pressure coefficients. For engineers that would like to follow the recommendations in Geoguide 1, they should be aware that the values obtained from figure 18 and 19 have to be resolved for their horizontal components, i.e. K_{ah} & K_{ph} before adopting in *Oasys* FREW analysis.

4.2.4 Surcharge Application and modelling

In both Plaxis 2D and *Osays* FREW analysis, when applying surcharge the engineer should decide whether it appears prior to or after the wall installation, as this will affect the prediction of wall deflection.

When the surcharge is expected to appear after the wall installation, the surcharge values should be applied in stage 1 instead of stage 0 of the *Oasys* FREW analysis. It should be noted that the purpose of stage 0 is to model the existing ground condition prior to any construction works. Surcharge value applied in stage 0 corresponds to the situation where the loading is present at the existing ground condition, and *Oasys* FREW will reset the wall deformation to zero prior to the stage 1 analysis. This is similar in Plaxis 2D, although wall deformation will not be reset to zero automatically.

As modelled in sample case 1, if the surcharge is widespread across the site, it is recommended to use uniformly distributed load (UDL) surcharge instead of strip load surcharge in *Oasys* FREW analysis. This gives a better estimate on the change of horizontal soil stresses due to surcharge application. It should also be noted that in *Oasys* FREW strip load surcharge will only modify the active pressure limit of the underlying soil; whereas the application of UDL surcharge will modify both active and passive pressure limits of the underlying soil.

4.3 Determination of Required Wall Toe Penetration

Apart from the comparison work, an additional parametric study was done using *Oasys* FREW on the ULS check for wall toe penetration requirement. The result showing the effect of the wall toe penetration on calculated wall bending moment, shear force and strut force for sample case 1 are illustrated in Figure 9 and 10.

The ULS analysis was successfully converged for retaining wall toe penetration as shallow as +34.5mPD. It can be seen that a saving could be achieved by reducing the wall toe penetration and at the same time maintaining the ULS stability requirement. However, at shorter wall toe, i.e. > 32.5mPD, the saving could only be realised with substantial increased structural forces as demonstrated in Figure 9 and 10. In other words, an ELS system with higher structural capacity will be required.

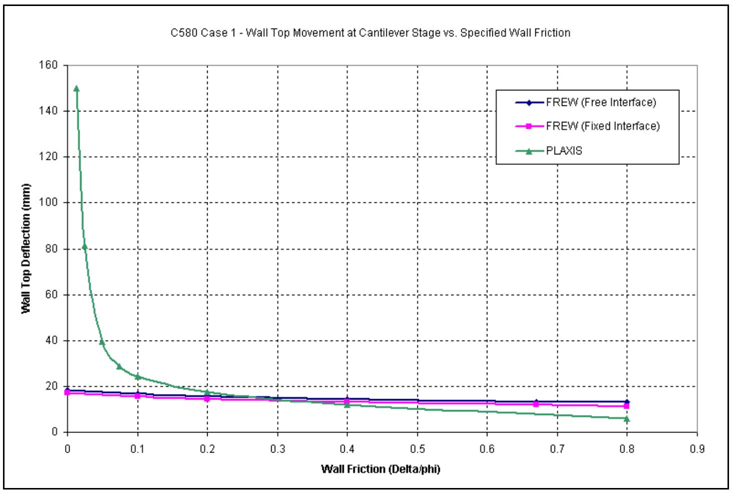

Figure 8: Effect of specified wall friction on wall deflections

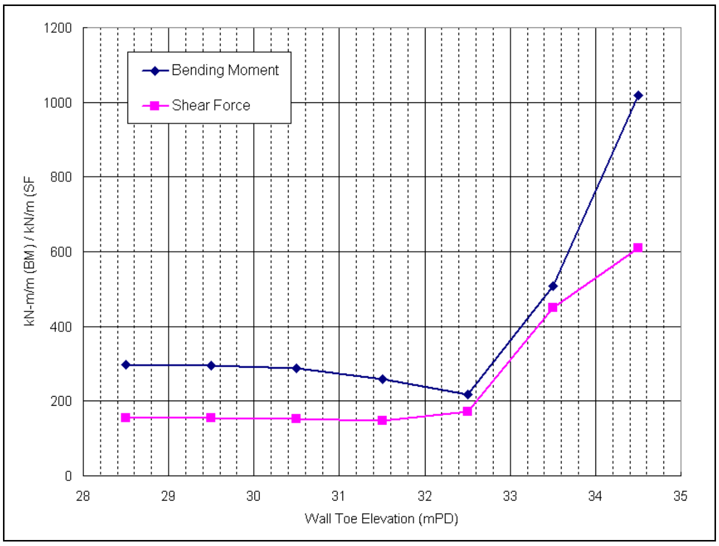

Figure 9: Effect of toe penetration on wall bending moment and shear force in ULS calculation

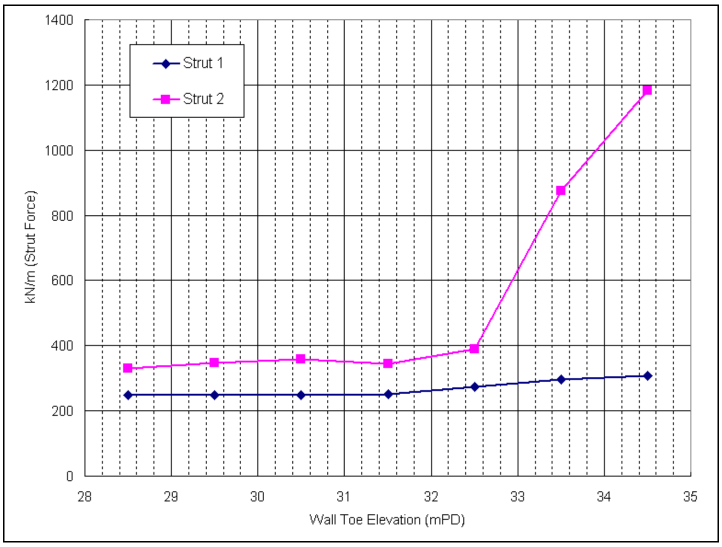

Figure 10: Effect of toe penetration on strut force in ULS calculation

Nonetheless, it must be stressed that by achieving the convergence in the numerical analysis, in this case the *Oasys* FREW, the specified wall toe penetration has already satisfied the requirement of toe stability in regardless to whether the convergence is achieved with substantially higher structural forces. It should also be noted that wall displacement in the ULS calculation should not be taken into account when determining the required wall toe penetration for stability.

5 CONCLUSION

The comparison study described in this paper demonstrated that both finite element method (Plaxis 2D) and boundary element method (*Oasys* FREW) were able to produce consistent and reliable estimates on retaining wall movement and structural forces using C580 approach. The accuracy of the prediction would depend on the choice of input parameters, and engineers should be aware of their effects as discussed in Section 4.2.

The parametric study on the determination of required wall toe penetration using ULS calculation in *Oasys* FREW analysis suggested that the saving in wall toe penetration may result in considerably higher structural forces estimated for the retaining wall and struts. This should be taken into account when determining the wall toe penetration in the ELS design. However, as long as the structural force has been designed for in the ELS system, the requirement on ULS stability is fulfilled, and the checking on wall deflection in ULS calculation is considered not necessary.

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An Analytical Review of Excavation and Lateral Support, Case History in Hong Kong

Paul K.F. Yau & Angus H.L. Sum
Meinhardt (C & S) Ltd.

ABSTRACT

Analysis of performance due to deep basement excavation is always not an easy task in geotechnical engineering. Accurate analysis of performance of the excavation and lateral support system is vital to the success of a project especially for the site locating in urban area. In Hong Kong, commercial computer programs using either finite element method or pseudo finite elements method is widely adopted for the analysis of the temporary excavation and lateral support works.

This paper aims to briefly report on the commonly used commercial computer programs applied for the analysis of the excavation and lateral support works in Hong Kong. Also, two case histories of deep basement excavation works in urban area using temporary steel pipe pile wall and diaphragm wall propped by steel shoring are presented to compare the results of analysis. Wall deflection, ground deformation and prop loads of the lateral support system computed by using different computer programs are examined. On-site monitoring records are also compared to calibrate the results of analysis.

1 INTRODUCTION

In Hong Kong, deep excavations are commonly constructed to maximize the land use for development in the dense urban area. The design of the excavation and lateral support works for deep basement should be controlled within the acceptable deformation limits of the nearby utilities, ground and buildings. Reliable design and predictions of excavation and lateral support performance including ground movements around deep excavations in urban areas are essential to assess their potential risk for causing damage to adjacent structures, road and utilities. Computer programs utilizing finite element method, finite different method as well as pseudo finite element method are widely used for the design analysis of excavation and lateral support work in Hong Kong. The computer programs are often employed to predict the performance of the lateral support system during excavation. The most suitable computer program much depends on the complexity of the case and quite often on the limitation of the computer program.

In this paper, two case histories of typical deep excavation and lateral support have been selected for the design analysis using three widely used computer programs namely "FREW", "FLAC" and "PLAXIS". The selected case histories represent two common types of the deep excavation and common geological conditions in Hong Kong. The walling types of the two case histories were temporary steel pipe pile wall and diaphragm wall accompanied by bottom up excavation method. The purpose of the design analysis is to provide observation, review and comparison of the excavation and lateral support design by using these computer programs. The on-site monitoring results have also been compared in particular on the wall deflection and ground movement.

2 OVERVIEW OF COMMERCIAL COMPUTER PROGRAMS FOR DEEP EXCAVATION DESIGN IN HONG KONG

2.1 Limit State Design

The limit state design with global factor of safety is adopted for the design of the excavation and lateral support works in Hong Kong. As defined, the limit state is the "state beyond which a structure no longer satisfies the design performance requirements". There are two main types of limit states, namely ultimate limit state and serviceability limit state. Ultimate limit state (ULS) refers to a state at which a failure mechanism can form in the ground or the structural damage. Serviceability limit state (SLS) refers to a state at which specified serviceability criteria are no longer met.

A lateral support system and complex construction sequences can be modeled by using the computer programs for the geotechnical design. Computer programs can calculate the induced bending moment and shear force as well as prop loads of the shoring system for structural design. Ground movements caused by the excavation can be either predicted directly or indirectly by using the computer program results.

2.2 Oasys FREW

Oasys “FREW” is a pseudo finite element method which models an excavation and lateral support system as an elastic continuum and is capable of considering soil and structure interaction of a flexible retaining wall. The wall is represented as a line of nodal points. The program analyses the behavior for each stage of the construction sequences. The force imbalance at each node imposed by each stage can be calculated. The displacement and soils stresses at each node can also be computed. The soil behavior is commonly modeled using the “SAFE” flexibility matrix method. However, the relevant ground deformation or subsurface soil movement due to excavation cannot be computed. The lateral ground movement obtained from the “FREW” analysis would be used to predict settlement at ground surface by some empirical correlations well published in HK.

2.3 FLAC

“FLAC” is a two-dimensional explicit finite difference program for engineering mechanics computation. The program simulates the behavior of structure built of soil, rock or other materials. Materials are represented by elements, or zones, which form a grid that is adjusted by the user to fit the shape of the object to be modeled. This computer program also allows different soil types to be modeled as a linear or a non-linear elasto-plastic soil with Mohr-Coulomb soil failure criterion.

2.4 PLAXIS

“PLAXIS” is a two-dimensional finite element computer program used to perform deformation and stability analysis for various types of geotechnical applications. Finite element methods are a class of methods for obtaining approximate solutions of differential equations, especially partial differential equations. This computer program is equipped with features to deal with various aspects of complex geotechnical structures. This program also allows different soil types to be modeled, along with structural & interface elements for realistic representation of soil-structure interaction effects.

More complex structures and construction sequences can be modeled in using finite difference or finite element computer programs such as “FLAC” and “PLAXIS” to predict the ground deformation around the deep excavation. The common adopted soil models in “FLAC” and “PLAXIS” are linear elastic perfect plastic Mohr-Coulomb model.

3 RE-ANALYSIS OF TWO CASE HISTORIES OF EXCAVATION AND LATERAL SUPPORT WORKS

Two recently completed case histories of deep excavation works in Hong Kong have been selected for the re-analysis using the above-mentioned computer programs.

3.1 Case History No. 1 - Deep Basement Excavation Works for a re-development project at Nathan Road

-Site Location and Ground Conditions

The site was about 85m x 35m in plan and is situated at Nathan Road. The site was previously occupied by a former hotel building. The redevelopment comprises a commercial retail building of some 30 storeys with 3-level basement of about 20m in depth. The site is surrounded by existing old buildings supported on shallow footings. The site is also located in close proximity to the MTR station and tunnel. The site geology is characterized by a stratigraphic sequence of Fill, Alluvium and decomposed granite of different weathering grades as illustrated in Figure 1. The parent rock materials underneath the site are described as medium grained granite of Jurassic to Cretaceous age. The rock head level is about 15m below ground level of around -10.0mPD. The groundwater level is located at about 2m below the existing ground level. A geological cross section is shown on Figure 2.

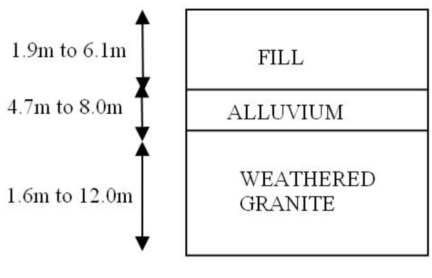

Figure 1: Soil stratum

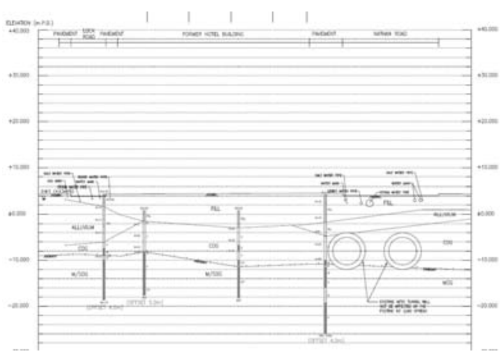

Figure 2: Geological cross section

-Geotechnical Design Parameters

Based on the results of ground investigation works, a set of geotechnical parameters were derived for the design of excavation and lateral support works. The parameters were assessed according to the results of field and laboratory testing as well as comparison with the typical values as commonly adopted in Hong Kong. Table 1 summarizes the adopted soil parameters for the design. The Young Modulus E value of the soil layer was expressed as a function of the SPT N value where E = 1N (MPa) for fill/alluvium layer and E = 1.5N (MPa) for decomposed granite soil.

Table 1: Summary of adopted soil parameters

Soil Stratum	Density (kN/m^3)	Cohesion c' (kPa)	Frictional Angle (degree)	E value (MPa)
Fill	18	0	35	9 to 15 (1N)
Alluvium	17	2	32	8 to 36 (1N)
CDG	19	6	36	18 to 200 (1.5N)
C/HDG	19	8	38	200 (1.5N)

-Excavation and Lateral Support Works

The excavation for the 3-level basement was about 20m in depth. Based on the option review, bottom-up construction method using steel pipe pile wall of 356mm diameter x 12.5mm thickness at 400mm c/c supported by 5 layers of strut at maximum 8.0m horizontal spacing were adopted for the excavation and lateral support works. The toe level of the pipe pile wall ranged from about 4m to 6m below the final excavation levels. Preloading of 200kNm/m to 250kNm/m on the temporary strut was adopted to minimize the wall deflection and ground movement induced by the excavation work to avoid any significant impact to the nearby MTR structures and adjacent old buildings. A grout curtain to provide an effective groundwater cut-off system was provided down to the bedrock behind the pipe pile wall. Pumping test was carried out to verify the water cut-off effectiveness of the grouting for the excavation and lateral support works system. The layout and section of the excavation and lateral support works are shown on Figures 3 and 4 respectively.

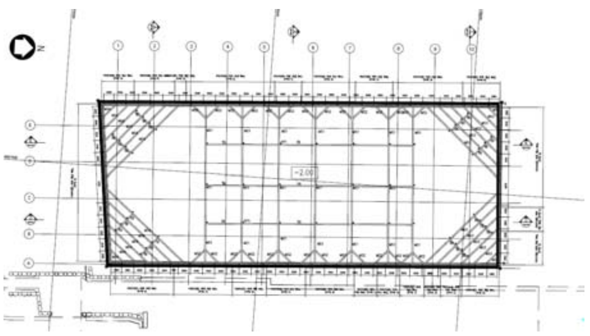

Figure 3: Layout plan of excavation and lateral support

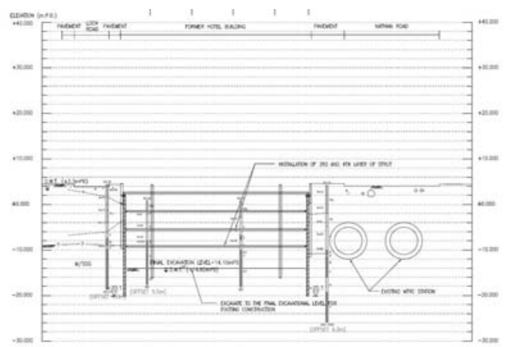

Figure 4: Section of excavation and lateral support

In the original design, a pseudo-finite element program "FREW" was used as an aid for the design of the temporary pipe pile wall and the shoring system. The computer program "FLAC" was used to assess the induced deformation on the nearby structures and MTR structure due to the excavation works. According to the assessment, the predicted lateral deformation of the pipe pile wall was about 30mm and the deformation of the adjacent ground due to wall deflection was about 15mm. The deep excavation works was successfully completed without any adverse effect on the nearby old buildings, ground, utilities and MTRC structures.

-Re-analysis of the Excavation and Lateral Support Design using Different Computer Programs

A selected section of the excavation design has been re-analyzed using the above-mentioned three computer programs on the basis of the same design parameters and groundwater condition. The re-analysis is aimed to compare the results of the ground deformation, strut loading and lateral deflection of the temporary wall. The computed results have also been reviewed with the actual on-site monitoring data. The following staged excavation sequences have been adopted in the re-analysis.

Stage 0: Initial Condition
Stage 1: Excavate to +2.0mPD and Dewater to +1.5mPD
Stage 2: Install strut at +2.5mPD
Stage 3: Excavate to -2.0mPD and Dewater to -2.5mPD
Stage 4: Install strut at -1.5mPD
Stage 5: Excavate to -6.0mPD and Dewater to -6.5mPD
Stage 6: Install strut at -5.5mPD
Stage 7: Excavate to -9.7mPD and Dewater to -10.2mPD
Stage 8: Install strut at -9.2mPD
Stage 9: Excavate to -14.1mPD and dewater to -14.6mPD

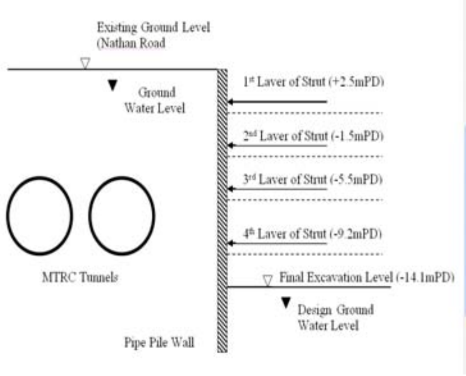

Figure 5: Design modeling for case history no. 1

The results of re-analysis are summarized in Table 2.

Table 2: Results of re-analysis using 3 different computer programs

	Results of Re-analysis using computer programs			Monitoring results
	FREW	FLAC	PLAXIS	
Maximum Wall Deflection (mm)	30	21	27	22
Maximum Strut Load (kN/m)	566	499	551	360
Maximum Ground Deformation (mm)	*15	10	11	8

Notes: * According to the empirical correlation assuming max ground settlement is 50% of wall deflection.

Figures 6 and 7 present the predicted wall lateral deflection and ground deformation at the final stage of excavation work with reference to the monitoring results. The estimated ground settlement using the empirical correlation from the wall deflection predicted by FREW was found in the upper ranges. The computed strut loads using three computer programs were different in the average loads by about 2 to 12% and within the uniform earth pressure distributed load diaphragm of 0.5γH as shown on Figure 8.

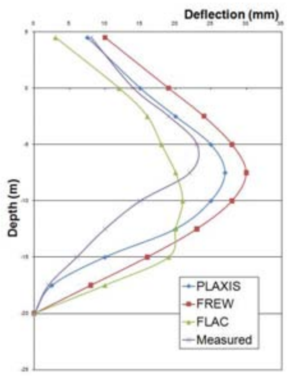

Figure 6: Calculated and measured wall deflection

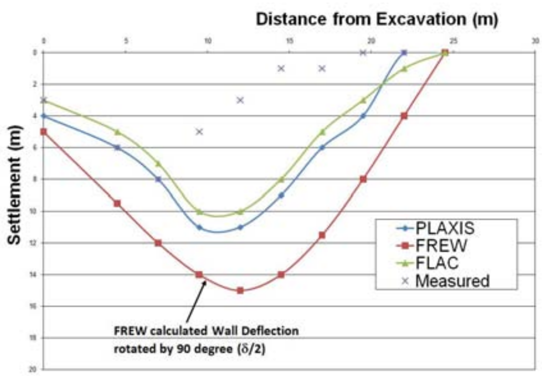

Figure 7: Calculated and measured ground deformation due to excavation

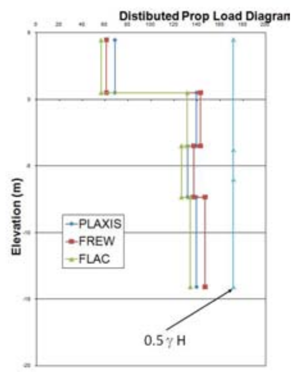

Figure 8: Calculated temporary props load

3.2 Case History No. 2 - Deep Basement Excavation Works for the Underground Development at West Kowloon

-Site Location and Ground Conditions

The development was located within West Kowloon area near Jordan Road. The site was about 300m x 50m in plan and the maximum excavation depth for the construction of the underground development was about 23m. The site geology is characterized by a stratigraphic sequence of Reclamation Fill, Marine Deposits, Alluvium and decomposed granite of different weathering grades as illustrated in Figure 9. The parent rock material underneath the site is described as medium to coarse grained granite of Jurassic to Cretaceous age. The lowest rock head level is about 40m below the ground level. The groundwater level is located at about 3m below the existing ground level. A typical geological cross section is shown on Figure 10.

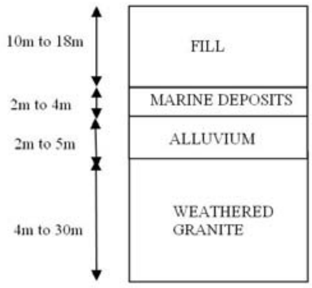

Figure 9: Soil stratum

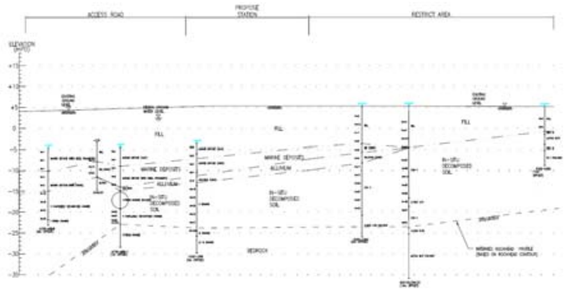

Figure 10: Geological cross section

-Geotechnical Design Parameters

Based on the results of ground investigation works, a set of geotechnical parameters were derived for the design of excavation and lateral support works. The parameters were assessed according to the results of field and laboratory testing. Table 3 summarizes the design parameters adopted for the design analysis. The Young Modulus E value of the soil layers were expressed as a function of the SPT N value where E = 1N (MPa) for fill/superficial layers and E = 1.5N to 2.0N (MPa) for decomposed granite soil.

Table 3: Summary of adopted soil parameters

Soil Stratum	Density (kN/m^3)	Cohesion c' (kPa)	Frictional Angle (degree)	E value (MPa)
Fill	18	0	35	8 to 30 (1N)
Marine Deposits	17	2	30	5 to 10 (1N)
Alluvium	17	2	33	10 to 30 (1N)
CDG	19	5	35	30 to 200 (1.5N to 2.0N)
C/HDG	19	8	38	200 (2.0N)

-Excavation and Lateral Support Works

The excavation for the development was around 23m in depth. After detailed study, 1.0m thick diaphragm wall with supported by 4 layers of temporary steel I-beam strut using bottom-up excavation method were adopted for the excavation and lateral support works. The designed toe level of diaphragm wall was at around -25mPD which was about 10m below the final excavation level. During the construction stage, the diaphragm wall acted as a temporary retaining structure to provide lateral support and groundwater cut-off to the excavation works and the diaphragm wall became a part of the permanent structure as well as structural load carrying members to the above ground structure. The layout and design modeling of the excavation and lateral support works are shown on Figures 11 and 12 respectively. In the original design, a pseudo-finite element program "FREW" was used as an aid for the analysis of the diaphragm wall and the shoring system. The predicted deformation of the diaphragm wall was about 69mm and the deformation of the adjacent ground due to wall deflection was about 35mm according to the empirical correlation.

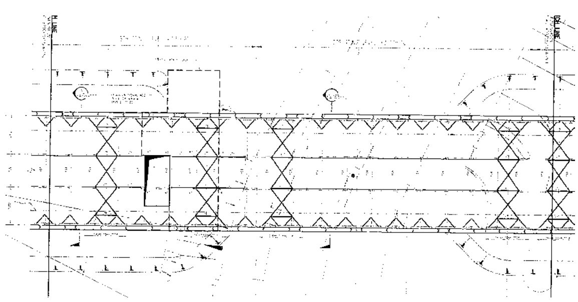

Figure 11: Layout plan of excavation and lateral support

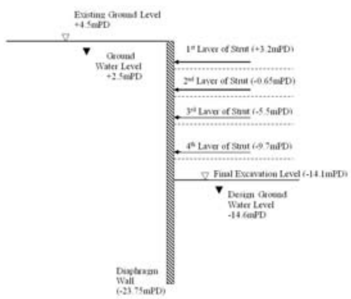

Figure 12: Section of excavation and lateral support

-Re-analysis of the Excavation and Lateral Support Design using Different Computer Programs

A selected section of the excavation design has been re-analyzed using the above-mentioned three computer programs. The results of re-analysis are summarized in Table 4.

Table 4: Results of re-analysis using 3 different computer programs

	Results of Re-analysis using computer programs			Monitoring results
	FREW	FLAC	PLAXIS	
Maximum Wall Deflection (mm)	69	58	68.	53
Maximum Strut Load (kN/m)	585	490	551	No monitoring data
Maximum Ground Deformation (mm)	*35	22	24	20

Notes: * According to the empirical correlation assuming max ground settlement is 50% of wall deflection.

Figures 13 and 14 present the predicted wall lateral deflection and ground deformation at the final stage of excavation work with relevant monitoring results. The similar observation as Case 1 for the empirical correlation of the ground settlements due to the wall deflection predicted by FREW was still in the upper ranges. The computed strut loads using the three computer programs generally were different in the average loads by about 6 to 17% and within the uniform earth pressure load distribution diaphragm of 0.65γH as shown on Figure 15.

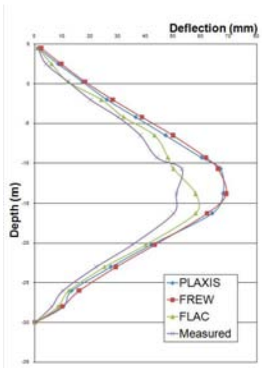

Figure 13: Calculated and actual ground deformation

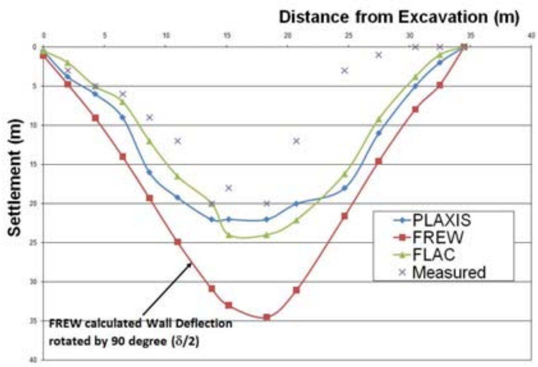

Figure 14: Calculated and measured ground deformation due to excavation

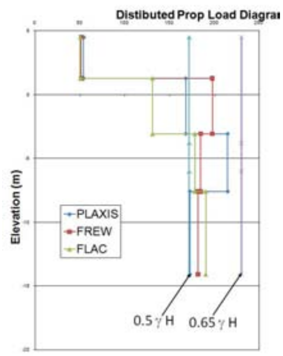

Figure 15: Calculated temporary props load

4 OBSERVATION REMARKS

This paper has presented an observation with using three geotechnical computer programs for the design analysis to the typical excavation and lateral support work involving over 20m deep for two case histories in Hong Kong. Based on the re-analysis results with reference to the available measurement records, the following observations are summarized.

- "FLAC" and "PLAXIS" have good approximation to the calculated ground deformation due to the excavation works.
- Using the empirical correlation to estimate the ground deformation obtained from calculated wall lateral deflection due to the excavation work was appeared on the higher side as compared with the full numerical analysis and monitoring results.
- The calculated prop loads from three computer programs are in similar order and the maximum prop loads line within the assumed uniform earth pressure load distribution diaphragm of adopting 0.5 to 0.65γH.

Computer program is a useful tool for the design analysis. However, the designer should never only rely on the calculation from the computer programs and they have been learned in the past by careful observing the good quality monitoring results of the deep excavation works. Adopting further back-analysis approach on the good quality results would have benefits of providing the acceptable risks for the continuous geotechnical design review. Empirical evidence combined with the ability using both complex full numerical models for the analysis is a good method to extend the current design practice in safe and optimal solutions.

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A Review of Current Practice for Design of Deep Excavations in Hong Kong

Victor Li
Victor Li & Associates Ltd. & Centre for Research & Professional Development, Hong Kong

ABSTRACT

The paper discusses some problems in the current practice for design of deep excavations in Hong Kong. They include over-conservatism, unreasonable conservatism, lack of a flexible approval system, unfair contract and designers not familiar with construction procedures, among others. The improvement of the current situation requires a joint effort by various parties of the engineering profession and the government authorities.

1 INTRODUCTION

The local practice for design of excavations has evolved with time. In the main, the practice tends to become progressively more conservative, despite short-lived relaxations from time to time. In this paper, the author attempts to present some aspects of the current practice of design of deep excavations in Hong Kong, focusing on the difficulties faced by designers and contractors in designing and implementing the works.

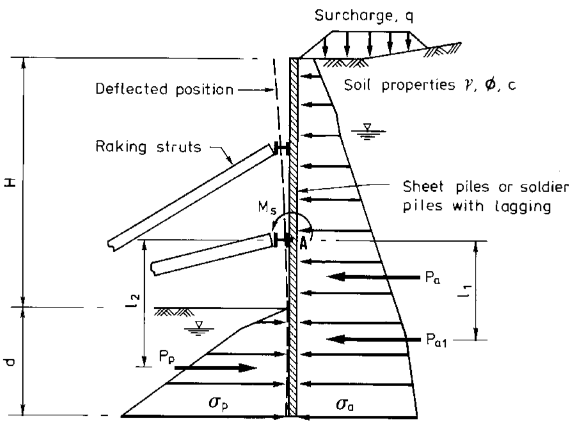
2 OVER-CONSERVATISM

Conservatism occurs in many forms such as high design factor of safety, conservative choice of design parameters and unreasonable acceptance criteria.

2.1 High factor of safety

There is no formally published design code for design of excavations in Hong Kong. The GCO Publication 1/90 “Review of Design Methods for Excavation” (GCO 1990) is commonly used as a reference document for design of shoring works. The design approach in CIRIA Report C580 (CIRIA 2003) has been adopted for some local projects in recent years.

When designing cofferdams, the embedment length of embedded wall (e.g. sheetpiles) is controlled by the design factor of safety for the so-called kick-out stability. Figure 1 shows the conventional approach used for checking of kick-out stability for strutted excavation. The embedded wall is assumed to fail by rotation about the lowest struts. The bending moment capacity of the embedded wall can be relied upon in enhancing the safety against kickout failure.



Required penetration of sheet pile or soldier pile is generally controlled by conditions at completion of excavation. Penetration required is determined from consideration of equilibrium of the free-ended span below point **A**, assuming fixity at **A**:

$$P_{a1}\, l_1 - \frac{P_p}{F_s}\, l_2 - M_s = 0$$

where F_s is the safety factor adopted in design.

Figure 1: Calculation of factor of safety against kickout failure (based on GCO 1990)

The requirement for minimum factor of safety for kickout stability has changed over the years. The GCO Publication No.1/90 suggests that a factor of safety of 1.5 may be adequate for temporary works. In the Building (Construction) Regulations (B(C)R), it is stipulated in clause 15(1) that “the resistance to the overturning moment acting thereon shall be not less than 1.5 times the overturning moment due to wind loads and 2 times the overturning moment due to loads other than wind loads”. This clause implies a factor of safety of 2 for overturning failure not due to wind loads. There have been different views on the applicability of this clause to checking of kickout failure. Some regard kickout stability as simply a problem related to bending failure of the embedded wall and it is therefore not considered as overturning failure. If so, a lower factor of safety may be acceptable. Some designers or government officials maintain the view that this clause in the B(C)R should also be applicable to kickout failure of embedded walls. Under this circumstance, a higher factor of safety of at least 2.0 will naturally be required. Both design factors of safety of 1.5 and 2 have been accepted by the Buildings Department (BD) in the past for deep excavations designed by the author in the past 20 years. The BD and GEO has recently come up with a consensus view that the required factor of safety for kickout failure should be 2.0, at least for private development projects controlled by the Buildings Ordinance for the time being.

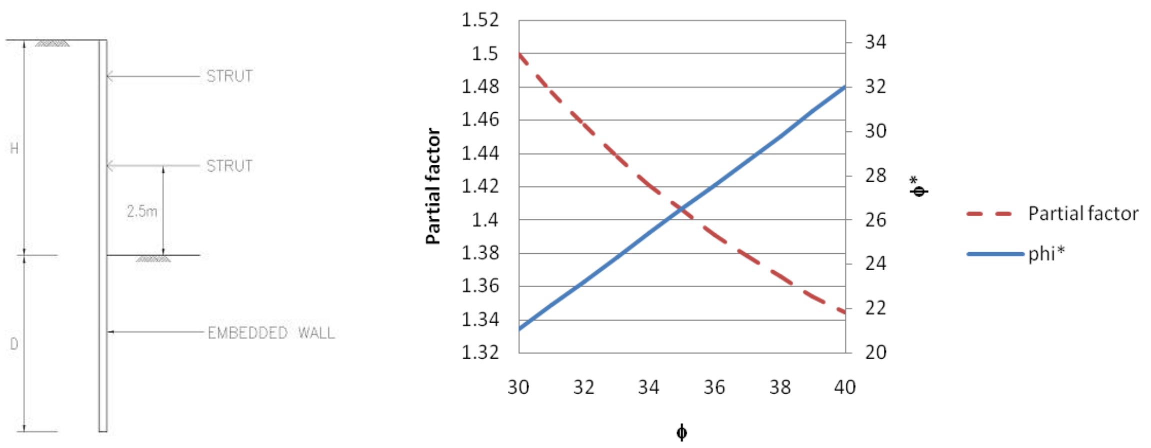


Figure 2: Illustrative example for kickout stability checking

A factor of safety of 2.0 implies a very stringent requirement for geotechnical design. Figure 2 shows an illustrative example of an embedded wall in a uniform soil with angle of shearing resistance ϕ. In the first step, the required embedment depth D is calculated based on an overall factor of safety of 2.0 for a cofferdam with the design retaining height H taken as 10m. The lowest strut is assumed to be 2.5m above the final excavation level. The coefficients of active and passive pressure are obtained based on Rankine theory. The bending moment of embedded wall is ignored for simplicity. In the second step, the “mobilized” angle of shearing resistance, denoted by ϕ^*, is calculated. It is defined as the value of ϕ that will cause limiting equilibrium of the embedded wall with the same embedment depth D obtained in the first step. That is to say, if the embedded wall is designed with a factor of safety of 2 based on ϕ, the magnitude of ϕ has to drop to $\phi=\phi^*$ before failure of the cofferdam will occur. The results of ϕ^* are presented in Figure 2. For the typical ranges of ϕ' for Hong Kong soils in Hong Kong (30^o to 40^o), the magnitude of ϕ^* varies between 22^o to 30^o. The angle of shearing resistance of sands will not drop to below its critical state value ϕ_{cv}. Geoguide 1 (GEO 1982) suggests that the lower bound values of ϕ_{cv} are about 34^o and 30^o for decomposed granites and volcanic soils respectively. The results in Figure 2 indicate that for completely decomposed granite (CDG) with a measured ϕ value of, say, 37^o, the magnitude of ϕ has to drop to about 29^o before kickout failure can be triggered. As $\phi^* = 29^o$ is lower than the lower bound value of ϕ_{cv} for CDG, kickout failure is virtually impossible from a soil mechanic point of view. A similar conclusion can be made for volcanic soils with a lower angle of shearing resistance.

There is another way of interpreting the physical meaning of ϕ^*. If we define $F_t = \tan\phi/\tan\phi^*$, F_t will become the partial factor applied to the soil strength, similar in concept to the partial factor of safety approach described in Geoguide 1 (GEO 1993). It is well recognized in the literature (e.g. Lumb 1966) that the variability of $\tan\phi$ is relatively small, typically less than 15%. According to Lumb (1970), a partial factor of about 1.15 for $\tan\phi'$ will be sufficient to cover the uncertainty associated with the innate variability of this strength parameter. This is in line with the recommendation of Geoguide 1 (GEO 1993). According to the result in Figure 2, an overall factor of safety of 2 will imply a partial factor well above the values recommended by Lumb (1970) and Geoguide 1 (GEO 1993).

The high factor of safety for kickout failure, coupled with other conservatisms to be discussed later, is overkill. The author has witnessed many damages to adjacent structures caused by excessive vibration when the contractor attempts to drive sheetpiles or channel plankings to attain the exceedingly long design embedment depth. In many occasions, a sufficiently safe design can be achieved with a much higher wall toe, thus averting many of the damages caused to surrounding facilities.

2.2 Unreasonable factor of safety

According to the structural codes for concrete and for steel published by the Buildings Department (BD 2004, 2005), a load factor of 1.4 is applied to the calculated bending moment for structural design due to earth or water pressure. In Hong Kong, many deep excavations are constructed in the urban area with groundwater table close to the ground surface. When a load factor of 1.4 is used, the calculated “ultimate load” may even be higher than the extreme scenario corresponding to water level at the ground surface. Unless the load factor can be relaxed or different load factors can be used for water and earth pressure for such design scenarios, money will continue to be wasted in over-designing the structural capacity of embedded wall.

Designers in Hong Kong are often required, forced or persuaded to adopt conservative design parameters for various reasons. In Hong Kong, the common practice is to adopt a design groundwater level outside the excavation to be at a level equal to 1/3 of the retaining height even if the existing groundwater level is well below the final excavation level. Worse still, the design groundwater level within the excavation is to be taken at the final excavation level. The former assumption over-estimates the disturbance force while the latter significantly reduces the passive resistance of the soils. It may be much less costly to provide some pumps outside or within the excavation to control the rise of groundwater level than to design for an extreme design event that will practically never occur during construction. Unfortunately, designers are usually not allowed to make this rational choice.

2.3 Unreasonable acceptance criteria

From the viewpoints of control and processing of design approvals, it is convenient for government authorities to adopt a uniform set of acceptance criteria for allowable settlements, angular distorting, vibration and etc for design of excavations. For the acceptance criteria to be general enough to cover most if not all excavations, such limits have to be conservative. Typically, the acceptable settlement limit for excavations in urban areas is 25mm. The limit may be further reduced to 10mm or smaller if old or more sensitive structures are present nearby.

While stability of excavations can be enhanced relatively easily by using more struts or a stronger embedded wall, settlements are often costly to reduce. The reduction of just a few millimetres of settlement may require closely-spaced struts, preloading of struts and a stronger embedded wall to reduce wall deflection and/or installation of grouting curtain to reduce drawdown settlement. Sometimes, the vertical spacing of struts may be so close that it is practically impossible for construction workers to carry out the excavation amongst the dense forest of struts. The imposed conditions are often too stringent. From experience, buried services and building structures supported by pile foundations can usually sustain higher ground settlements without damage. Money is often wasted to achieve the paper requirement of a small limit when there is no genuine need to be so stringent.

3 OBSERVATIONAL APPROACH

The advantages of the observation approach are well explained in his Rankine Lecture by Peck (1969). In Hong Kong, most geotechnical designs require pre-approval by one or more government authorities before construction. The time required for processing such pre-approvals makes the observational approach much less attractive or downright impossible to implement. For instance, for a deep excavation initially designed to be supported by 5 layers of struts, one may adopt an observational approach in deciding whether the 5th struts can be deleted depending on the performance of the excavation on reaching the excavation for the 3rd layer of struts. Of course, the level of the 4th layer of strut may need to be adjusted to cater for the deletion of the 5th layer of struts. The excavation from the 3rd to 4th layer of struts may take only a short time of a few weeks to complete. If the pre-approval process takes much longer than this duration, it is not practical to stop the construction on reaching the level for 4th layer of struts and wait for the approval for deleting the 5th layer of struts to be processed. For this reason, designers and private developers can seldom enjoy the benefit of the observational approach. Such benefits are usually reaped by the contractors who can decide on how much they can quietly over-excavate before they install the next struts depending on the performance of the excavation during construction.

4 PUMPING TEST

Full-scale pumping tests are often required by the GEO or the Engineer for verifying the water-tightness of the cofferdam before excavation is allowed to commence. A partial pumping test with test area restricted to a certain area of the cofferdam is sometimes permitted, but the water levels in the test area are still required to be lowered to the future final excavation level.

When water is lowered within the cofferdam, the differential water pressure across the embedded wall will be high for a deep excavation. The embedded wall is only supported by soils within the cofferdam during a pumping test. Soils usually have much lower lateral stiffness as compared with struts. Without installed struts, there will be no opportunity to reduce the wall deflection by preloading. Significant wall movements and hence ground settlements may occur during the pumping test before the bulk excavation is even started.

Lui & Yau (1995) reported a case study of pumping test of a diaphragm wall cofferdam. The water levels were lowered between 23.5m to 29.5m below ground within the site, resulting in a wall movement of over 80mm in some areas and a maximum ground settlement of 15mm outside the site. If the allowable settlement is 25mm as mentioned above, a settlement of 15mm will represent over 50% of the allowable limit.

There are other alternatives to full-scale pumping tests. It is feasible to install the struts above the groundwater level before starting the pumping test such that the embedded wall can be partially supported by stronger and stiffer struts before lowering the water level. From a theoretical point of view, it is not necessary to lower the water level completely to the final excavation level for testing the water-tightness of cofferdams. Piezometers can be installed outside the cofferdam at different levels and at different locations. Pump wells can then be installed within the cofferdam close to these piezometers. When a pump is lowered in the pump well, there will be a local drop in piezometer pressure around the pump. If there is a leakage in the embedded wall, the piezometer can register an unusual drop in piezometric pressure during passage of the pump.

With a well-planned monitoring system, leakage of the cofferdam can still be detected during excavation and remedial works such as grouting can then be implemented timely to stop the leakage. When deciding whether a pumping test should be conducted, one must carefully compare the risks of adverse effects caused by full-scale pumping test with the possible risks of not being able to implement the remedial works in time to control the leakage problem during excavation.

5 UNFAIR CONTRACT

In Hong Kong, the responsibility for designing excavations is commonly shifted to contractors under the foundation contracts. The contractor has to face high contractual and hence financial risk in undertaking such projects. The terms are often very unfair to the contractor. To name a few, they may include:

- not adequate space allocated to the contractor for installation of embedded wall,
- lack of ground investigation or laboratory information at the tender stage
- lack of information of adjacent utilities and adjacent structures at the tender stage
- short contract period, leaving no sufficient time for preparing a proper design submission and no flow time to cater for disapproval of design by government authorities and remedial works.

Unfortunately, the contractors have little power in selecting the preferred contract arrangement.

6 DESIGNERS NOT FAMILIAR WITH CONSTRUCTION

For a design to be practical, economical and constructable, it is important that the designer should be familiar with the construction procedures, the equipment used and the limitation of various types of embedded walls. If there are underground obstructions, sheetpiles are not preferred because they cannot penetrate through boulders or hard strata. Pipe piles are installed with a rotator. The size of the rotator is larger than the pipe pile, which implies that the pipe piles cannot be installed right next to an existing building. When constructing diaphragm walls, guide walls are required. Again, this will mean that the alignment of the diaphragm wall needs to be set back by a distance of 150 to 200mm from adjoining structures. The operation of grabs and cutter used for trench excavation required long vertical space. If there are objects protruding from the adjacent building, such as air conditioner, it is may not be feasible to construct a diaphragm wall close to the building.

The author have seen numerous excavation designs which are outright not feasible. There are examples of sheetpiles with design toe levels deep below the rockhead; contiguous pipe piles with near-zero clear spacing that are impossible to construct and many others. A good understanding of all these construction related constraints is equally important as if not so than soil mechanics theory in developing a good design scheme for excavations.

7 DISCUSSIONS

This paper discusses some aspects of the current practice for design of deep excavations in Hong Kong. Some of the problems are associated with over-conservatism in one way or the other. As remarked by Peck (1977) in his interesting article, over-conservatism can sometime cause more harm than good in geotechnical projects. The example of damage caused by excessive vibration in installing over-designed sheetpiles mentioned earlier in this paper is a classic one. One should take a holistic view and perhaps a new mindset in dealing with both the design risks and construction risks. Improvements of the current practice require joint efforts by various parties of the profession, including consultants who should acquire better knowledge related to construction, contractors who need to be more vocal in voicing out their objections to unfair contracts, developers who should implement fairer contracts and government authorities who need to be more flexible in the approval process.

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The Use of Hat-Type Sheetpiles in Hong Kong

Joley Lam
Formerly Victor Li & Associates Ltd.

ABSTRACT

Sheetpiles are commonly used in Hong Kong as embedded wall for cofferdams. The bending strength of sheetpiles depends on the position of interlock in the sheetpile and the effectiveness of the sheetpile interlocks in transmitting shear forces.

U-type sheetpile is the most common type of sheetpile used in Hong Kong. Conventionally, the design of sheetpile cofferdams in Hong Kong is based on the assumption that the interlocks for U-type sheetpiles are fully effective in transmission of shear force without slippage. This assumption has recently been a matter of debate. Unless justifications are provided, designers have to treat interlocked U-type sheetpiles as being non-interlocking or partially interlocked when assessing the bending moment capacity or wall stiffness.

To achieve more economic sheetpiling design, Hat-type sheetpiles which are more effective in transmitting shear forces have recently been introduced to Hong Kong from Japan. This paper aims to present a review of design practice for dealing with the effectiveness of sheetpile interlocks and the experiences gained from the recent use of Hat-type sheetpiles in Hong Kong.

1 INTRODUCTION

Sheetpiles are widely used in Hong Kong as embedded walls for supporting excavation. Figure 1 shows the common types of sheetpiles. They include U-type, Z-type and the newly developed Hat-type sheetpiles manufactured in Japan. U-type sheetpile is by far the most common type of sheetpile used in Hong Kong because of the ready availability in market and ease of installation relative to other types of sheetpiles. Individual sheetpiles are connected together by threading into interlocks during installation.

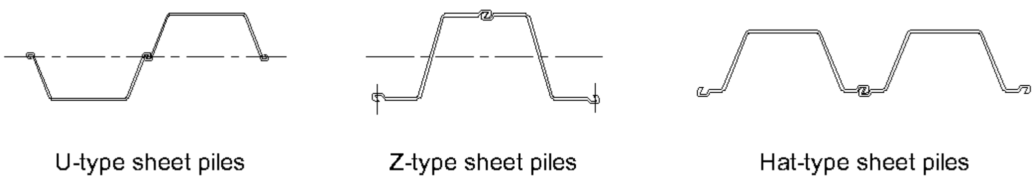

Figure 1(a): Common types of sheetpiles

Figure 1(b): Photo of Hat-type sheetpile

Conventionally, the design of sheetpile cofferdams in Hong Kong is based on the assumption that the interlocks for U-type sheetpiles are fully effective in transmission of shear force without slippage, i.e. the interlocked sheetpiles can be regarded as a fully composite section. This assumption has recently been a matter of debate. Unless justifications are provided, designers have to treat interlocked U-type sheetpiles as being non-interlocking or partially interlocked when assessing the bending moment capacity or wall stiffness.

For Z-type and Hat-type sheetpiles, the interlocks can still be regarded as fully effective in shear transmission and such interlocked sheetpiles can be regarded as fully composite sections.

Hat-type sheetpiles were only introduced recently to Hong Kong. This paper aims to discuss the advantages and limitations of Hat-type sheetpiles and present two case studies in which Hat-type sheetpiles were used for supporting deep excavations in Hong Kong.

2 SOME BASIC CONCEPTS

The advantage of Hat-type or Z-type sheetpiles relative to U-type sheetpiles can be explained by reference to the simple concept of the classical beam theory.

Figure 2 shows the variation of bending stress and transverse shear stress when an elastic rectangular beam is bent. The transverse shear stress is highest at the neutral axis and decreases to zero at edges of the beam.

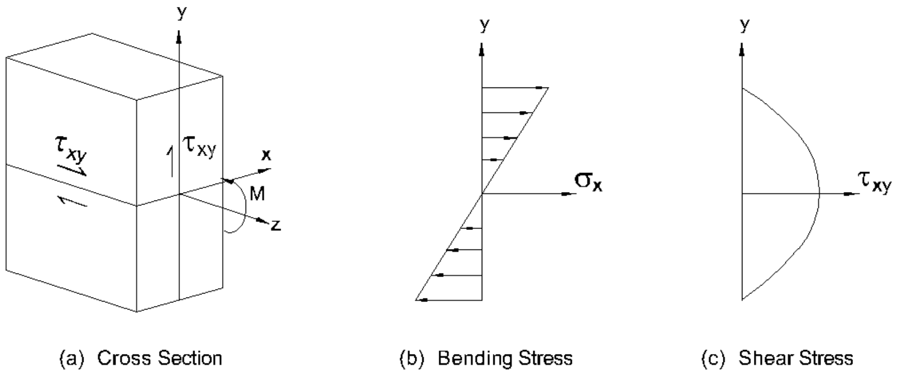

Figure 2: Stress distribution in a beam subjected to pure bending

Similarly, when a sheetpile wall is bent, the shear stress is highest at the neutral axis and lowest at the extreme faces of the sheetpile. If U-type sheetpiles are used, the interlocks will be at the point of maximum shear stress. If slippage occurs at the interlock, the development of bending stress will be affected and the stress distribution as illustrated in Figure 2(b) will not occur. This will result in a smaller bending moment capacity for the interlocked sheetpiles. In the extreme case of a purely smooth interlock, the development of bending stress will be similar to that shown in Figure 3, while the bending moment capacity of the interlocked sheetpiles will be identical to that of a single sheetpile on a per-metre-run basis.

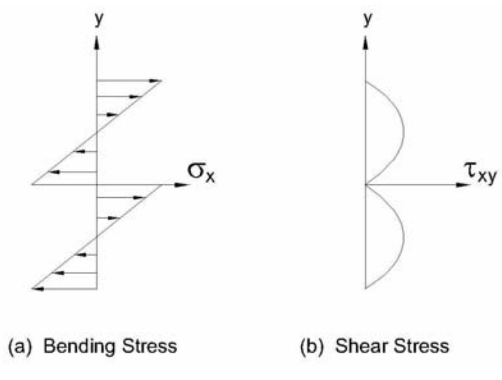

Figure 3: Stress distribution of a rectangular beam with smooth plane at mid-section

For Z-type and Hat-type sheetpiles, the interlocks are located at the outerface of the sheetpiles where the transverse shear stress is smallest. Therefore, whether the interlock is smooth or fully rigid will not affect the bending moment capacity of the sheetpile as a fully composite section.

In summary, the wall stiffness and bending moment capacity of interlocked sheetpiles depend on the position of the interlock and its effectiveness in transmitting shear forces.

Research on the effectiveness of sheetpile interlock has been very limited (Kort 2002; Schmieg & Vielsack 2002; Byfield & Crawford 2002). It is difficult to quantify the friction that can be developed at the sheetpile interlock and hence the effectiveness of shear force transmission. As discussed by Kort (2002), sheetpile interlocks will never be perfectly straightly aligned. Rotation at clutches, slight bending of sheetpiles, tolerances in sheetpile geometry, soil intrusion into the interlocks during pile installation, and installation of struts and walings will all contribute to significant increase in friction in the interlock against slippage. The assumption of smooth interlock and complete loss of shear resistance of the interlock is considered unlikely and unnecessarily conservative.

The reduction in moment of inertia and section modulus of the three common types of sheetpiles by assuming smooth interlock is compared in Table 1.

Table 1: Comparison of percentage reduction in sectional resistance of U-type sheetpiles with smooth interlocks

U-type Sheetpile	% reduction in moment of inertia	% reduction in section modulus
FSP II	65	57
FSP III	67	58
FSP IV	70	60

It can be seen from the above table that if the sheetpile interlock is designed as smooth, the moment of inertia will be reduced by over 65% and the section modulus by over 57% as compared with the maximum sectional resistance for a fully composite section. Such reduction also increases slightly with the size of the sheetpile.

3 DESIGN OF SHEETPILE IN HONG KONG AND OVERSEAS

Eurocode 3: Part 5 (BS EN 1993-5:2007) and the National Annex to Eurocode 3 classify sheetpiles into 3 categories and apply different reduction factors on the bending strength as follows:

$$M_{c,Rd} = \beta_B W_{pl} f_y / \gamma_{MO} \qquad \text{for Class 1 and 2 cross sections}$$

$$M_{c,Rd} = \beta_B W_{el} f_y / \gamma_{MO} \qquad \text{for Class 3 cross section}$$

where $M_{c,Rd}$ = design moment resistance of the sheetpile; β_B = reduction factor; W_{pl} and W_{el} are the plastic and elastic section modulus respectively for a continuous wall; f_y = yield stress; and γ_{MO} = partial safety factor.

The reduction factor ranges from 0.4 to 1.0 depending on the treatment to the interlocks, number of structural support levels and ground conditions. No reduction is needed for Z-type sheetpiles and triple U-piles and the same should apply to Hat-type sheetpiles.

A reduction factor of 0.6 is adopted for bending as a norm in Japan (Metropolitan Expressway Co. Ltd. 2003). If sufficient restraints in form of welding, concrete capping or sufficient embedment depth in soil are provided, the factor can be increased to 0.8.

The design of sheetpile cofferdams in Hong Kong conventionally assumes that U-type sheetpiles can develop full composite bending strength in design. This assumption has recently been questioned by some government authorities, although the bending strength of sheetpiles designed without any reduction factor has been well proven by a wealth of past experiences in Hong Kong. Unless justifications are provided, the bending moment capacity and wall stiffness of U-type sheetpiles have to be reduced by either applying a reduction factor or adopting the section modulus for a single U-pile instead of a continuous wall in design. This has resulted in a significant increase in material cost as compared with the conventional design.

There are methods for improving the interlocking property of U-type sheetpiles. They include crimping three U-piles together before installation and provision of sufficient restraints against lateral movement. However, all these options are generally not favoured for site operation. There is thus a pressing need to search for alternative sheetpiles that are either more effective in transmitting shear force, or much stiffer to produce a higher moment of inertia and section modulus per pile.

4 USE OF HAT-TYPE SHEETPILE IN HONG KONG

Hat-type sheetpiles have been in use in Japan since 2005 and is now being widely used in Japan (Harata *et al*. 2008; Kawabata 2008). The sheetpile was jointly developed by Nippon Steel Corporation, JFE Steel and Sumitomo Metal Industries.

The main advantage of Hat-type sheetpile over the conventional U-type sheetpile is that the interlocks are located at the point of minimum shear stress. Hat-type sheetpiles are therefore more efficient than the U-type sheetpiles in providing a higher section modulus for a sheetpile wall. Z-type sheetpile also has this advantage but there is an additional interlock in the middle of its flange, which creates greater difficulty in handling the sheetpiles during driving, whether they are interlocked or not.

In addition, the wide and thin section of Hat-type sheetpile means that less steel material can be used to achieve the same moment capacity and stiffness of the conventional sheetpiles, leading to saving in construction cost. The number of sheetpiles used would also be reduced with a wider section and pile installation can be expedited.

Similar to other types of sheetpiles, driving of the Hat-type sheetpiles can be effected by fork-shaped drop hammers, vibratory hammers, hydraulic hammers or hydraulic jacking. Fork-shaped drop hammers are lighter in weight and therefore suitable for installing sheetpiles in soft soils. Vibratory hammers with different weight can generally be used for all types of ground conditions. If hydraulic hammers are used, a steel driving cap is needed to be placed on the top of the sheetpiles. Hat-type sheetpiles are commonly installed by the Silent Piler developed by Giken Seisakusho Co., Ltd. in Japan.

The Hat-type sheetpiles are wider than the conventional U-type sheetpiles. In terms of driveability, the Hat-type sheetpiles may not be as agile as the U-type sheetpiles in passing around small boulders due to the larger sheetpile size.

As discussed in Section 1, Hat-type sheetpiles were recently introduced into Hong Kong to achieve more economic sheetpile design. The two recent projects where Hat-type sheetpiles were installed are described below.

4.1 Site A

The first project in which Hat-type sheetpiles were installed was a residential development located at Des Voeux Road West, Hong Kong. Figure 4 shows the geological profile and the variation of SPT 'N' with depth of the site.

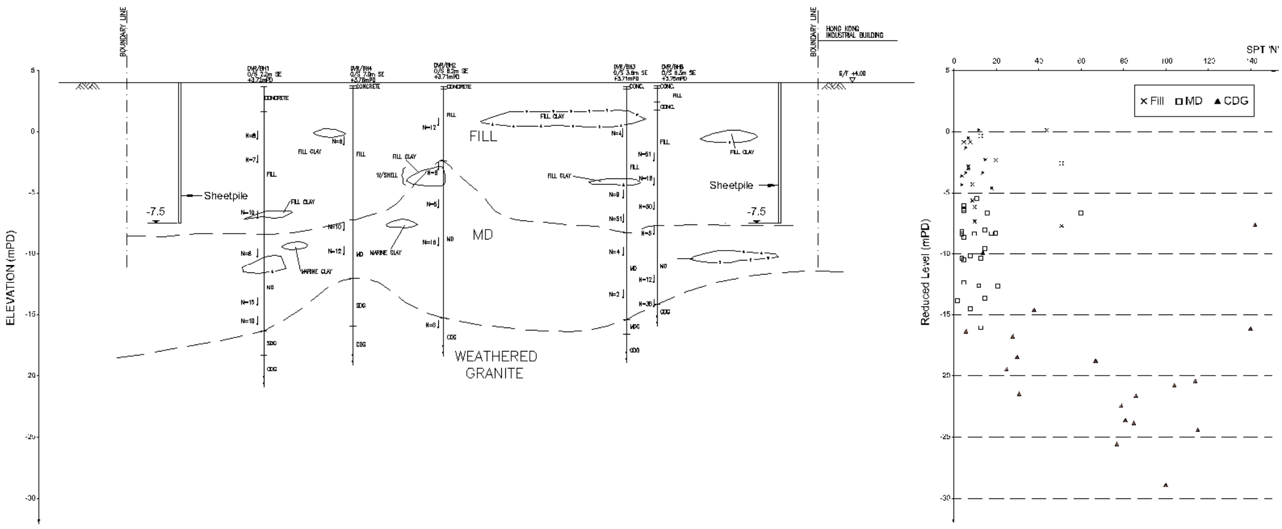

Figure 4: Geological profile and variation of SPT 'N' vs. depth of Site A

The Hat-type sheetpiles used were Type NSP-10H Hat-type sheetpiles manufactured by Nippon Steel Corporation with section properties listed in Table 2. During installation, they were driven by vibratory hammer to a depth of about 12m in the fill layer, with SPT 'N' mostly below 20. No large boulder was encountered during sheetpile driving.

Table 2: Sectional properties of type NSP-10H Hat-type sheetpile

Dimension			Properties per metre of wall			
Effective width (mm)	Effective height (mm)	Thickness (mm)	Sectional area (cm^2)	Moment of inertia (cm^4)	Section Modulus (cm^3)	Unit weight (kg/m)
900	230	10.8	122.2	10,500	902	96.0

The maximum ground settlement measured during the sheetpiling works was very small, only about 4mm. The maximum ground vibration recorded during the sheetpiling works was only about 5mm/s, which was well below the limit of 15mm/s as specified for the project. The settlement and vibration generated by the installation of Hat-type sheetpiles were similar to those induced by other types of sheetpiles despite the larger sheetpile width per pile and presumably larger penetration resistance of the sheetpile.

4.2 Site B

The second site using Hat-type sheetpiles as cofferdam wall was located at Po Wu Lane, Tai Po, N.T. There were settlement sensitive structures located near the site, which included a clinical building on shallow foundations and an existing box culvert and water mains. Figure 5 presents the geological profile and the variation of SPT 'N' with depth of the site.

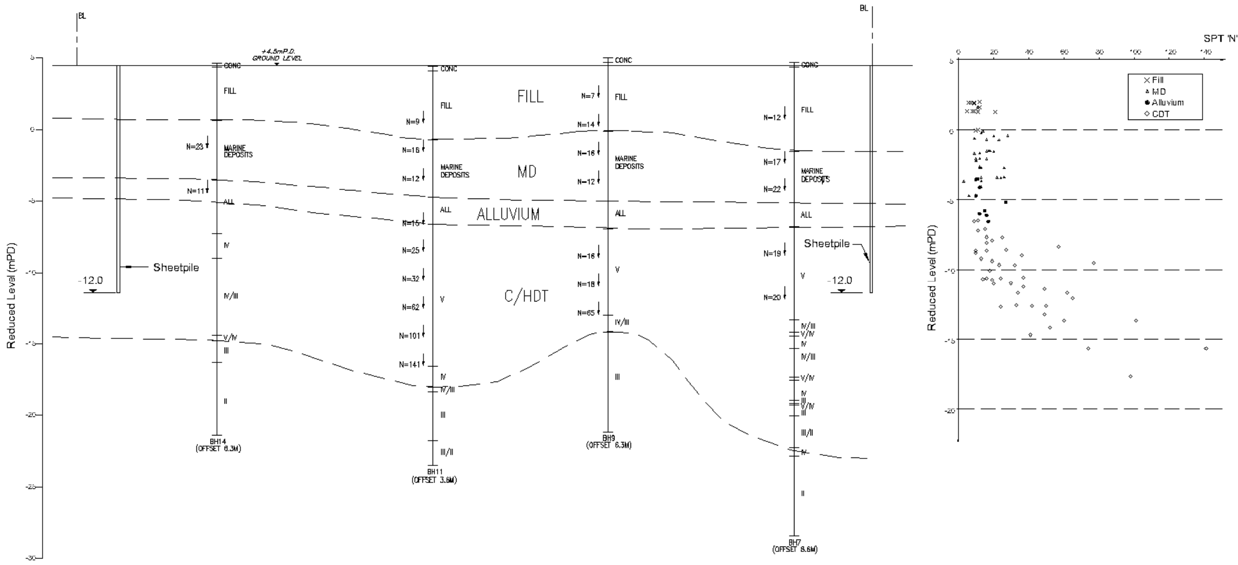

Figure 5: Geological profile and variation of SPT 'N' vs. depth of Site B

Type NSP-10H Hat-type sheetpiles were also used for this site. The sheetpiles were driven by vibratory hammer, except for the first trial sheetpile, to a depth of about 17m in completely decomposed tuff, with SPT 'N' value of approximately 50. The first trial sheetpile was installed by hydraulic hammer. Figure 6 shows the Hat-type sheetpiles being installed by vibratory driving and hydraulic hammer at this site.

Figure 6: Installation of Hat-type sheetpile by vibratory hammer (left) and hydraulic hammer (right) at Site B

The vibration level was monitored during installation of sheetpile by vibratory hammer. The maximum ground-borne vibration measured at 2m from the sheetpile was only about 5mm/s and dropped to about 2mm/s when measured at 4m from the sheetpile. This demonstrates that the vibration effect of driving the larger Hat-type sheetpile on adjacent ground is generally small.

Figure 7 shows a typical interlock of the Hat-type sheetpile. The interlocking joints for Hat-type sheetpile are generally tighter than those of U-type sheetpiles to achieve better water-proofing property. However, the interlock may be overheated due to abrasion along the joints during installation. To improve the driveability, water may need to be sprayed on the sheetpile interlock to reduce the heat generated.

Figure 7: Tight interlock of Hat-type sheetpile

5 CONCLUSION

Hat-type sheetpiles were recently introduced in Hong Kong to achieve more economic sheetpile design. The benefits and limitations in using Hat-type sheetpiles are discussed in the paper. Hat-type sheetpiles are more efficient than the conventional U-type sheetpiles in transmitting shear force in the interlock. The interlocking Hat-type sheetpiles can be regarded as a fully effective composite section in design and no reduction in bending strength is needed, resulting in a more economic design.

Two case studies in Hong Kong in which Hat-type sheetpiles were successfully installed in urban areas with low level of vibration and small ground movement have been presented. The sheetpiles showed good driveability despite the larger sheetpile size as compared with conventional sheetpiles.

ACKNOWLEDGEMENTS

The information on Hat-type sheetpiles provided by Nippon Steel Corporation Ltd. and Victor Li & Associates Ltd. is gratefully acknowledged.

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Geotechnical Aspects of a Deep Excavation in Doha

L. Tony Chen
Hyder Consulting Limited, Hong Kong

ABSTRACT

The Doha high-rise tower complex consists of a four level basement which measures approximately 15,600 m^2 on plan and requires an excavation depth of approximately 16m. The temporary excavation retaining system initially consisted of anchored secant pile walls with toe grouting, along the entire site perimeter. After excessive lateral wall movement was detected on one section of the southern wall, an additional strutting system was installed as a remedial measure to provide additional lateral support to the moving wall section. This paper discusses the geotechnical aspects of the design and construction of the excavation and its retaining system, the analysis approach employed to evaluate the condition of the moving wall, and the remedial measures taken to prevent further wall movement.

1 INTRODUCTION

The Doha high-rise office tower complex consists of a 45 story tower, a 4 story multi-function building and a 4 level underground basement. The tower is cylindrical in shape, measuring approximately 45m in diameter and 231.5m in height above ground. The basement area is approximately 15,600 m^2, with an excavation depth of 15.65m along the site perimeter.

Figure 1: Site location

Located at the east of Doha, Qatar, the site is approximately pentagonal in shape with its southern side being located next to Al Corniche Street which is a major road in Doha and runs along the Arabian Gulf shoreline, see Figure 1. There are also other office buildings located adjacent to the project site.

Minimization of effects of excavation-induced ground movements on the adjacent structures, Al Corniche Street and the various utilities underneath it was one of the major issues needed to be considered in the design and construction of the excavation and its temporary retaining system.

The temporary retaining system initially consisted of anchored secant pile walls with toe grouting, along the entire site perimeter. After excessive lateral movement was detected on one section of the southern wall, inclined bracing steel beams were installed inside the excavation as a remedial measure to provide additional lateral support to the wall. Published design charts were used to evaluate the condition of the wall based on the measured wall movement.

This paper discusses the geotechnical aspects of the design and construction of the excavation and its retaining system, the analysis approach employed to evaluate the condition of the moving wall, and the remedial measures taken to prevent further wall movement.

2 SITE CONDITIONS

2.1 Surface and sub-surface conditions

Topographically the site is relatively flat. The Qatar Peninsula geologically represents a part of the Arabian Gulf Basin. This basin is mainly composed of extensive carbonate sediments with different ages overlying the basement rocks and may reach up to 10 km in thickness. The outcropping rocks in Qatar are mainly referred to Quaternary and Tertiary Ages. The geology of the study area is mainly of the recent and Tertiary sediments Simsima Limestone and Rus Formation.

According to the results of ground investigations carried out for this project, the ground materials consist of, in sequence from top, Hydraulic Fill, Marine Sand/Caprock, Simsima Limestone, Dukhan Alvelina, Midra Shale and Rus formation.

The Midra Shale layer was encountered in all boreholes and was about 4.7m thick. This layer was known as impermeable.

2.2 Groundwater

Groundwater was encountered in all boreholes at different depths ranging between 2.25m and 4m below the existing ground level, which might vary due to the proximity of the sea shoreline to the site and the tidal fluctuation effect.

2.3 Permeability of ground

Since excavation was to terminate in the Simsima limestone, several packer tests were carried out in this layer to determine its permeability. The packer test results indicated that the permeability of this layer was in the order of $5x10^{-5}$ m/s.

A pumping test was also carried out to assess the permeability of the Simsima limestone layer and the materials overlying it, the results of which could be used to determine the effect of dewatering on adjacent areas and to assess dewatering requirements during excavation.

The test included one 25m deep pumping well, 300mm in diameter, and eight 125mm diameter observation wells, four of which being 20m deep and the other four being 18m deep. The observation wells were installed at specified distances from the test well in four directions set at 90º to the test well (two observation wells at each direction).

The pumping rate selected for the test was approximately $64m^3$/hr. The discharge rate and the ground water levels in all the wells were recorded.

The test results indicated that the permeability of the test depth was in the order of about 5.5×10^{-5}m/s, very permeable.

It should be noted that, although only relatively small size cavities were identified during the site investigations, large cavities which are connected to the adjacent sea water, were expected to exist at the site based on local experience. This was subsequently proved to be the case during construction of bored piles.

3 EXCAVATION RETAINING SYSTEM

3.1 Design considerations

The excavation depth of the basement area is 15.65m close to the basement outer boundaries. At the tower area and a few lift pit locations which are located away from the basement outer walls, the excavation is a few meters deeper.

The following points were considered in the selection of an excavation retaining system:

- The ground above the impermeable Midra Shale layer is very permeable and recharge of groundwater from the adjacent sea is expected to be rapid;
- Excavation induced ground movements need to be tightly controlled to minimize adverse impact on the adjacent structures, underground utilities and roads especially Al Corniche Street.

After evaluation of several options, an anchored secant pile wall option was considered appropriate and was adopted along the entire excavation perimeter. Due to piling rig limitations, the depth of the secant piles was limited to 20m which was, as will be discussed later in this paper, found to be inadequate to cut off groundwater inflow. Consequently, a grout curtain below the toe of the secant piles was adopted as a groundwater cut off barrier. Details of the anchored secant piles with toe grouting are shown in Figure 2.

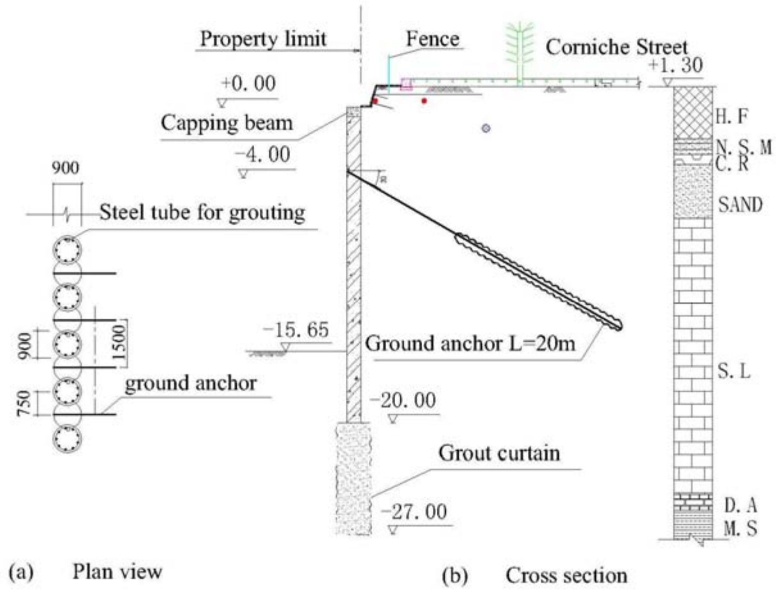

Figure 2: Anchored secant pile wall with toe grouting

3.2 *Anchored secant pile walls*

The anchored secant pile wall consisted of primary and secondary bored piles connected by a capping beam on the top. The primary bored piles were constructed of plain concrete, while the secondary bored piles were constructed of reinforced concrete. Both the primary and secondary piles were 900mm in diameter and 20m long. The neighbouring piles overlapped by a maximum of 150mm. The secant pile wall was supported laterally by one row of ground anchors connected by a wailer beam. The level of the anchor heads was at -4m, about 0.5m above the groundwater level expected at the time of anchor installation.

The geotechnical stability of the anchored secant pile wall was analyzed using Plaxis 2D. The ground model was simplified as a loose sand layer overlying a Simsima limestone layer, with the adopted parameters being presented in Table 1.

Table 1: Adopted parameters for analysis of secant pile wall

Soil type	Layer thickness (m)	c' (kPa)	ϕ' (deg)	E_s (MPa)
Loose sand	8	0	30	20
Simsima limestone	17	100	38	40

The calculated maximum working bending moment and shear force of each secondary pile (i.e. reinforced pile) are presented in Table 2, together with the calculated maximum wall deflection. These forces were used for the structural design of the reinforced piles.

Table 2: Calculated maximum pile responses

Bending moment (kN.m)	Shear force (kN)	Deflection (mm)
1,035	615	24

Correspondingly each reinforced pile was designed to have an ultimate bending moment capacity of about 1,500 kN.m.

The calculated maximum anchor force was 697 kN per anchor and each anchor was designed to have an ultimate load capacity of about 1,465 kN.

3.3 *Toe grouting*

Seepage analysis results, later confirmed by results of trial dewatering after installation of the secant pile walls, indicated that the 20m deep pile walls were not deep enough to cut off expected groundwater ingress into the excavation. Therefore, it was decided to install a grout curtain to form a groundwater cut off barrier between the toe of the secant piles and the underlying impermeable Midra Shale. The grout was to be injected through the vertical tubes pre-installed inside the secondary bored piles. The spacing of the grouting tubes was 1.5m.

Grout injection was carried out in three steps at different spacings, 6m at first step, reducing to 3m at second step and 1.5m at the final step. This sequence was adopted to ensure that sufficient overlap between successive grout bodies could be achieved.

The amount of cement consumption was used to judge the grouting effect. A decrease in cement consumption with decreasing grouting spacing would indicate proper overlap between successive gout bodies. Generally, if the amount of cement consumption at the current grouting step is less than 75% of that at the preceding step, the overlap is considered to be adequate.

Subsequent dewatering proved that this system of secant piles with toe grouting was effective in cutting off groundwater inflow.

3.4 Excavation sequence

The excavation was carried out in the following sequence:

- construct secant piles and capping beam;
- carry out toe grouting;
- dewater and excavate to -4.5m;
- install wailer beam and ground anchors at -4m; and
- dewater and excavate to final formation level.

Lateral and vertical ground/wall movements were monitored by inclinometers and settlement markers, respectively.

Plate 1: Excavation to level of -12m

Plate 1 shows the site condition when the excavation was carried out to the level of about -12m.

4 REMEDIAL MEASURES

4.1 Excessive wall movement

When the excavation reached the level of about -13.5m, excessive lateral wall movement was detected on part of the southern secant pile wall. Within a couple of days, the measured wall deflection had reached about 80mm which was significantly greater than the design limit of 25mm. Plate 2 shows the ground depression behind the wall.

A subsequent investigation found that the excessive wall movement had been caused by some loosened ground anchors, installed by a local sub-contractor who also installed the secant piles.

Plate 2: Ground depression behind secant pile wall

In order to check the structural condition of the moving piles, it was necessary to estimate the maximum pile responses induced by the measured wall movement. Such a check may be undertaken using either numerical methods or published design charts as discussed in Chen & Poulos (1997, 1999, 2001) and Poulos & Chen (1996a, 1996b).

As a first step reacting to the emergency situation, the simple design charts proposed by Chen & Poulos (1997) were used to back calculate the maximum bending moment induced in the pile due to the measured wall movement. The procedure is illustrated below.

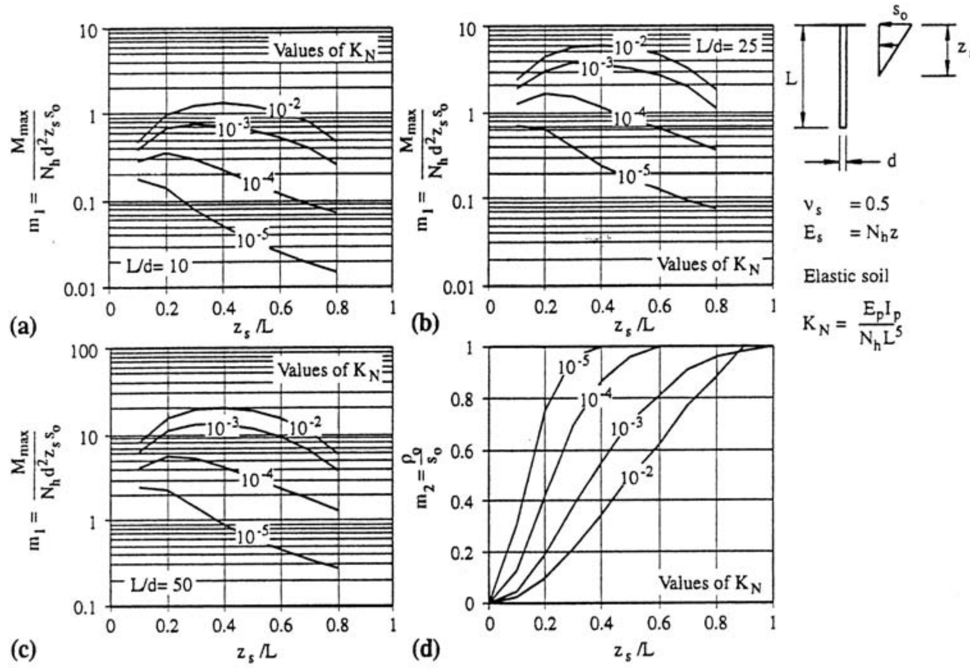

Figure 3: Elastic solutions for unrestrained free-head pile in uniform soil (linear soil movement profile) (after Chen & Poulos, 1997)

The input pile and soil parameters required for these design charts include pile diameter (d), pile bending rigidity (E_pI_p), pile length (L), soil movement at surface level (s_o), thickness of moving soil layer (z_s) and soil Young's modulus ($E_s = N_hz$ for Gibson soil).

Based on the measured inclinometer data, the lateral soil movement profile was closely simplified as a linear profile, reducing from 80mm at the pile top level to zero at the level of -8m. A Gibson soil was assumed with the Young's modulus increasing with depth at a rate of 2,500 kN/m^3.

For each reinforced pile, d = 0.9m, L = 20m and $E_pI_p = 1.9 \times 10^6$kN.m^2. For L/d = 20/0.9 = 22.2, $K_R = E_pI_p/N_hL^5 = 2.3 \times 10^{-4}$ and $z_s/L = 8/20 = 0.4$, the maximum bending moment can be calculated as $M_{max} = 1.1 \times 2{,}500 \times 0.9^2 \times 8 \times 0.08 = 1426$ kN.m which is approaching its ultimate capacity of 1,500 kN.m, indicating that the concerned piles had reached a critical condition. This simple estimation was subsequently verified by a more detailed numerical analysis.

The excavation was immediately stopped and the concerned wall section backfilled to the level of about -10m as an emergency measure to prevent wall collapse.

The loosened ground anchors were subsequently re-stressed to their design load level. Furthermore, it was considered necessary to strengthen the concerned wall section by additional lateral support prior to resumption of excavation works, in order to prevent further wall movement.

4.2 Remedial works

After option evaluation, it was decided to adopt a strutting system consisting of inclined steel tubular pipes having an external diameter of 600mm and a wall thickness of 10mm, installed at an inclination angle of about 30^o to the horizontal and at a spacing of 4.3m, see Figure 4 and Plate 3. The top of these bracing pipes were connected by a steel wailer beam fixed to the wall at the level of -10m. The bottom of the bracing pipes was supported by a foundation consisting of inclined steel sections and the 750mm diameter bored piles which had been installed as tension piles to support the basement.

In the design of this strutting system, one of the key issues was the potential obstruction of its footings to the subsequent construction of the basement slab and its waterproofing layer. This issue was tackled successfully by adopting the footing details shown in Figure 4.

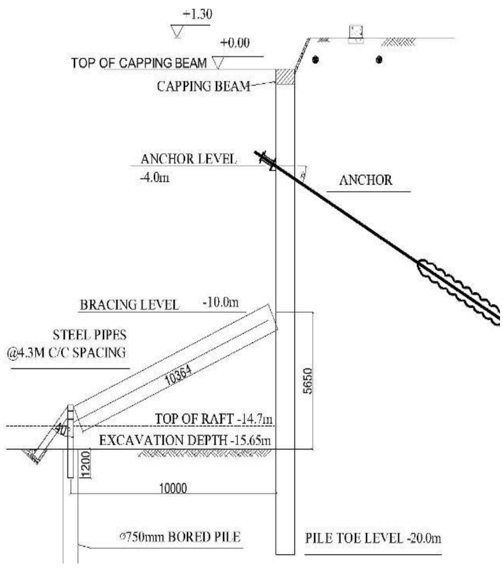

Figure 4: Secant pile wall with additional bracing pipes

Plate 3: Installed strutting system

Although this strutting system had caused inconvenience to the subsequent excavation and construction of the basement slab, it played a critical role in safeguarding the retaining system, hence the important Corniche street behind the secant pile wall.

Following installation of the additional strutting system, the excavation was continued successfully to completion.

5 CONCLUSION

The temporary retaining system consisting of anchored secant pile walls with toe grouting was proved to be effective in not only retaining the excavated ground but also cutting off groundwater inflow.

However, due to careless installation of ground anchors, excessive wall movement was detected on one of the wall sections. This incident shows that it is imperative to implement proper site supervision and monitoring for excavation works.

The design charts proposed by Chen & Poulos (1997) were found to be very useful in providing a prompt and accurate evaluation of the condition of the moving wall, especially considering the emergency situation whereby a detailed numerical analysis was not immediately available due to time constraints.

An additional strutting system was adopted to provide additional lateral support to the moving wall prior to resumption of excavation works, in order to minimize further wall movement. Subsequently, the excavation was continued successfully to completion.

ACKNOWLEDGEMENTS

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A Case Study of Design of Diaphragm Wall Cofferdam Based on the Design Approach of CIRIA Report C580

Alan Liu & Leo Lee
Intrafor Hong Kong Ltd., Hong Kong
Victor Li & Alan Tam
Victor Li & Associates Ltd., Hong Kong

ABSTRACT

The design approach of CIRIA Report C580 has recently been introduced to Hong Kong for the design of cofferdams. This Report recommends that progressive failure of struts should be considered in the design of the lateral support system. Alternatively, a risk management approach may be used to risk of damage to the shoring systems to acceptably small level. This paper presents the details of a risk management approach adopted for a recently completed deep excavation project in the urban area of Hong Kong.

1 INTRODUCTION

The Construction Industry Research and Information Association (CIRIA) published the report C580 – Embedded Retaining Walls – Guidelines for Economic Design in 2003 (CIRIA 2003). In January 2004, the Geotechnical Engineering Office (GEO) set up a Review Group, comprising members from GEO, Buildings Department (BD), consultants and contractors, to review the design approach outlined in the CIRIA Report No. C580 (CEDD 2004). In a letter issued in 2005, the Buildings Department (BD 2005) promulgated that design approach is CIRIA Report C580 may be accepted for design of shoring works for private development projects in Hong Kong.

The authors have adopted the design approach of CIRIA Report C580 for the design of a diaphragm wall cofferdam for construction of a deep basement in urban area in Hong Kong. This paper presents a particular aspect of this project related to the use of risk management approach in dealing with the risk of progressive failure of struts. Other details of the project are presented in Li et al. (2010).

2 SITE DESCRIPTION AND GROUND CONDITIONS

The site is located within the Mass Transit Railway (MTR) protection zone. It is relatively flat with existing ground level at about +5.1mPD. It is bounded to the north by an existing road, to the east by an existing commercial building supported by bored piles and diaphragm wall, to the south and west by industrial buildings supported by pile foundations. The site is located within the Mass Transit Railway (MTR) protection zone. There is a MTR Station and a MTR viaduct structure located about 85m and 15m respectively away from the site. There are existing services and utilities located along Road and the service lane outside the site, including water mains, sewer and storm water drains and some cables.

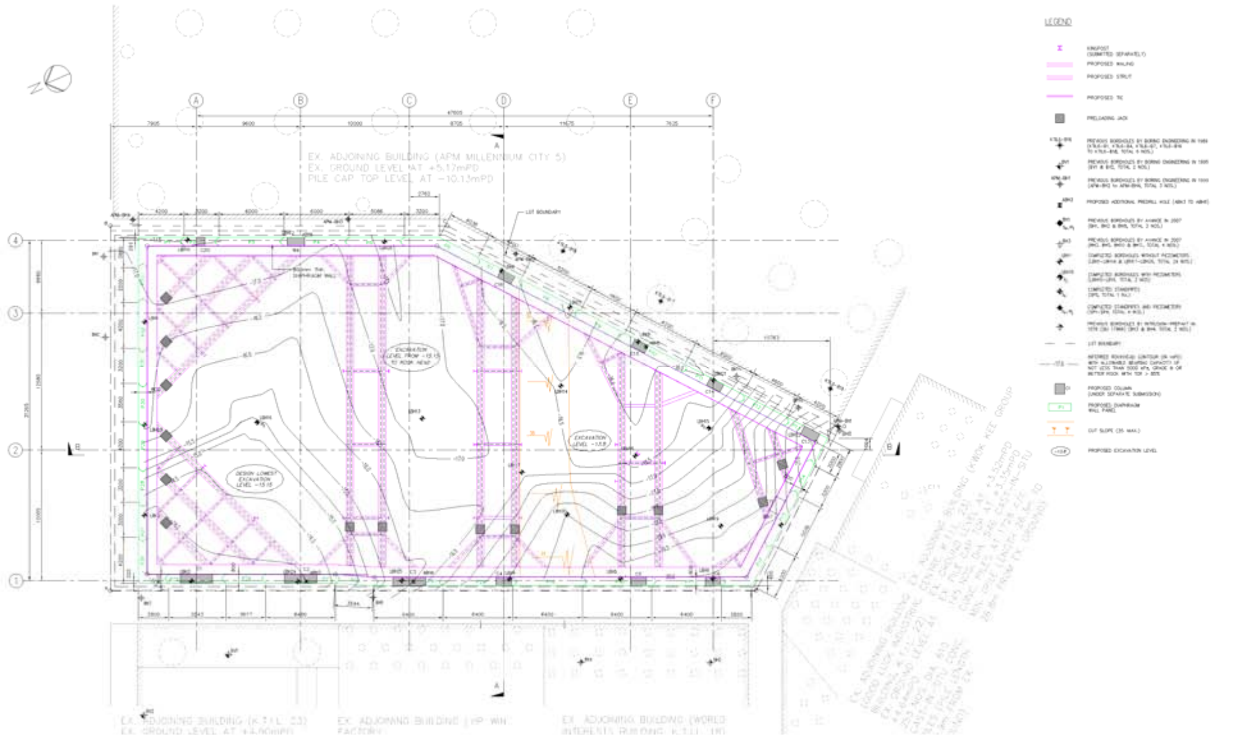

Figure 1: Layout of cofferdam

The site is a very old reclamation site formed over 40 years ago. Based on the ground investigation (GI) records, the site is generally covered in sequence of fill, marine deposit, alluvium and decomposed granite. Bedrock of Grade III or better rock with total core recovery > 85% is encountered at −13.3mPD to −23.3mPD. The groundwater monitoring records indicate that the groundwater level varies from −1.0mPD to +4.2mPD.

There was an existing basement at the site. The proposed new diaphragm walls were constructed within the confine of the existing basement. The screen wall of the old basement will be left in place outside the new diaphragm wall, but all basement slabs and pile cap of the old basement will be completely removed during the construction of the new basement.

3 DETAILS OF DIAPRHAGM WALL COFFERDAM

The layout out plan for the proposed cofferdam is shown in Figure 1. Two cross sections across the site are presented in Figures 2 and 3. The shoring works involve a maximum excavation depth of about 25m to reach the bedrock for construction of raft footings or isolated footings. The diaphragm wall were supported by a maximum 5 layers of struts. At the deepest location of excavation, secondary vertical struts connected to the 5th layer of struts as shown in Figure 2 were installed vertically to provide additional lateral support to the diaphragm wall. In effect, the diaphragm wall cofferdam can be regarded as being supported by “5½” layers of struts. The diaphragm wall was founded on the bedrock and shear pins have been provided beneath the toes of diaphragm wall panel to prevent kick-out instability.

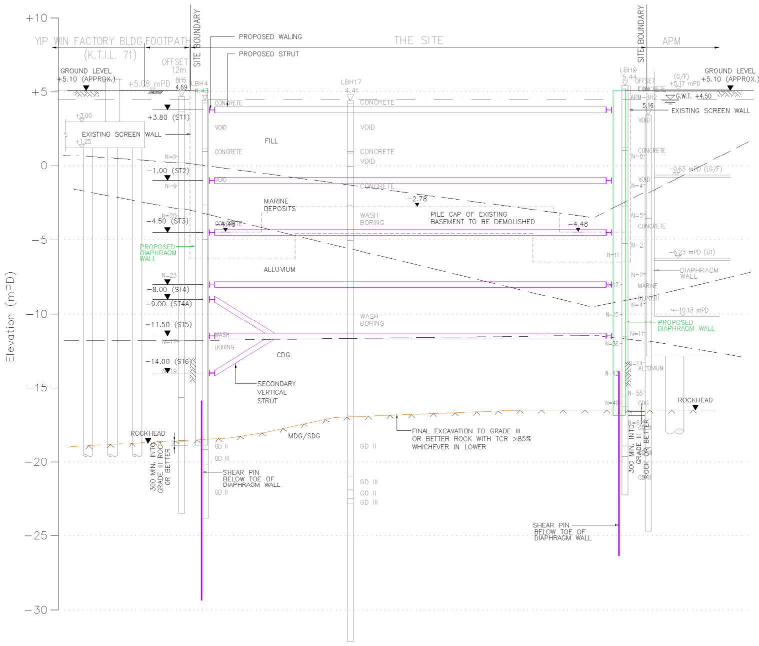

Figure 2: Section A-A across the site

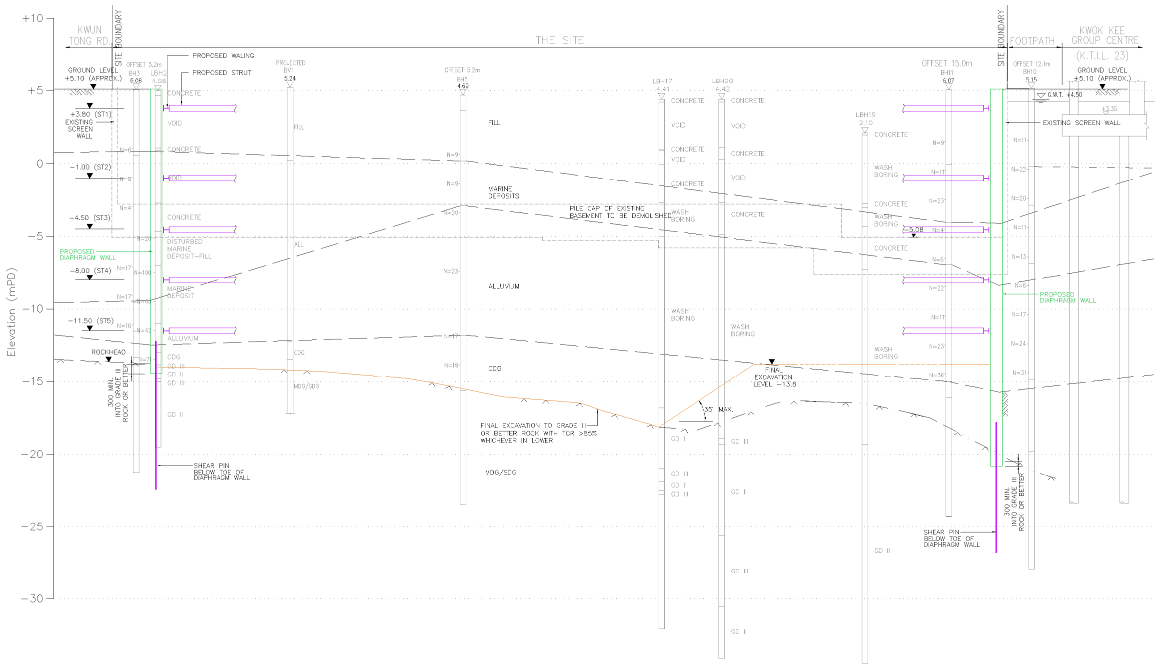

Figure 3: Section B-B across the site

4 THE RISK MANAGEMENT APPROACH

According to the C580 approach, due consideration should be given to progressive failure of struts. The shoring system has to be stable when one of the struts is removed from the support system. Alternatively, a risk management may be used to control the risk of progressive failure.

The project described in this paper was the first private development project of its kind approved by the Buildings Department/GEO in which a risk management approach had been adopted within the framework of the C580 approach for mitigating the risk of progressive failure of the lateral support system. Full details of the risk assessment and mitigation measures proposed to reduce the risk of accidental damage of struts to acceptably low level are presented in Li et al. (2010). The salient features are:

(a) Construction planning - The excavation procedures were well planned in advance. Vertical transportation of muck-out materials and delivery of materials and plant were confined to designated access areas with larger open space to minimize the risk of hoists or other objects hitting the struts. Figure 4 shows the bird's eye view of the cofferdam. The struts around the designated access areas are coated with yellow paint to alert construction workers.

Figure 4: Bird's eye view of cofferdam

Locking pins were provided in the crawler crane to control allowable horizontal swing of the boom when the crane was performing vertical transportation of muck-out materials or other equipments (see Figure 5).

Figure 5: Locking pins in crawler crane

(b) Precautionary measures - Several measures were implemented to minimize of risk of damage to steel members of the cofferdam. Figure 6 shows small rebars welded onto king posts, struts and waling around the access areas. The small diameter rebars served the functions of (i) absorbing the energy of hitting objects; (ii) preventing the object from directly hitting the struts, walings or kingpost; and (iii) keeping the workers in the alert when construction equipments hit the rebars.

Figure 6: Strut and waling (left) and king post (right) protected by anti-collision rebars

The damaged rebars were regularly repaired / replaced. The struts/waling/king posts around the access areas were painted to alert the workers (see Figure 6). Distance sensors are installed at the arms and rear of backhoes (see Figure 7). When the backhoes are too close to surrounding objects, audio and visual signals will be generated to alert backhoe drivers.

Figure 7: Distance sensors mounted at the rear (left) and at the arms (right) of backhoe

(c) Increased site supervision - more supervisory staff, such as banksman and lifting supervisors, were deployed by the contractor to supervise all lifting and excavation operations

5 DISCUSSIONS

Although the designer can deal with the problem of progressive failure by allowing one strut to be removed without affecting the safety of the shoring system, such an approach will often be difficult to handle in practice for the following reasons:

(a) A proper numerical modeling of progressive failure requires a 3-dimensional (3D) model. When a strut is removed, the loading will be transferred to its adjacent struts, both vertically and horizontally. At present, there is no pre-approved program approved by the BD for 3D analysis of shoring system. If a program for two-dimensional (2D) analysis of shoring system such as 2-D Plaxis, 2-D FLAC or FREW is used, removal of a strut will be equivalent to complete removal of one layer of struts. In Hong Kong, most shoring works are undertaken by contractor as part of the design-and-build contract. This will be far too conservative to be affordable by the contractor to model progressive failure as complete loss of one layer of struts.

(b) Sometimes, a simplified approach of progressive failure is dealt with by assuming that the adjacent struts in the same layer will fully take up the strut loads of the removed struts. Even if this approach is adopted, the member sizes of all struts have to be increased to cater for the additional loading transferred from the removed strut. When a strut is removed, the span of waling between adjacent struts may be doubled and the bending moment quad-tripled. The member size of the waling has to be increased significantly to allow for such an increase in bending moment arising from loss of one strut.

(c) Even when a 3-D program is available for analysis of shoring system, such analyses are often numerically unstable. Obtaining a correct and convergent 3-D analysis of a shoring system may be time-consuming and not warranted for design of small excavations.

The risk management approach provides a simpler and more economical alternative to the approach of over-designing the struts and walings to allow for removal of one strut in the shoring system. The application of the risk management approach is not difficult, but it requires a dedicated effort or perhaps a new culture in construction in Hong Kong for its enforcement.

6 CONCLUSION

This paper presents the risk management approach as an alternative based on the CIRIA Report C580 approach for dealing with progressive failure of struts.

The risk management measures adopted for preventing damage of struts, walings and king posts were found to be very effective and much more economical than using heavier struts and walings to deal with the risk of accidental loss of struts.

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Deep Excavations Supported by Soil Nailed Walls

J.S.H. Kwan, M.L. Ho, C.K. Wong & C.H.C. Chan
Geotechnical Engineering Office, Civil Engineering and Development Department, Government of the Hong Kong SAR

ABSTRACT

Soil nailed walls have been adopted to support deep excavations in local construction. Nevertheless, the interactions between the soil nails, the wall element and the retained earth are complex, the engineering design of the soil nailed wall may involve certain level of simplification or idealisation of the complex interactions. A holistic approach comprising a comprehensive load testing, monitoring and design review programme should be adopted in order to arrive at a robust design. The present paper provides a brief introduction on the design principles of soil nailed wall as an excavation support system. Salient points that may be overlooked in planning and designing soil nailed walls are discussed. The necessary verifications of the parameters adopted in the stability and numerical analyses are recommended, and details of a case history of using soil nailed walls to support deep excavation are reported.

1 INTRODUCTION

One of the most common types of support to deep excavations in Hong Kong is strutted wall. However, when the construction site is congested and if land constraint is not a major hurdle, soil nailed wall may become an attractive option.

Excavations in a developed urban area, like Hong Kong, are normally surrounded by sensitive facilities, for instances, buried pipelines, buildings, structures, etc. Limiting the induced settlement to within a tolerable value would be a prime target in the excavation design. Therefore, the soil nailed walls that were adopted to support excavations in Hong Kong usually had a relatively high stiffness with toe embedment, rather than flimsy, flexible wall facings. This paper aims to review on the design principles of the soil nailed wall, and discuss, based on the observations of some case histories, the necessary considerations to be made when planning and designing soil nailed wall as an excavation support system.

2 DESIGN OF SOIL NAILED WALLS

2.1 Load Transfer Concept

The earth loading and the deflection development processes of a soil nailed wall and a strutted wall are different. For a strutted wall, the struts are loaded due to the wall deflections; there is no contact between the strut and the retained earth. However, in the case of a soil nailed wall, interactions between the soil nails, the wall element and the retained earth come into play, and such interactions could be complex. Detailed study of the behaviour of soil nails can be found in Clouterre (1993). The loading exerted onto the soil nail is resulted, primarily, from the outward wall movement during excavation, which induces load transfer between the retaining soil and the soil nail direct. Since the soil nail is grouted throughout its entire length, the load transfer between the retained soil and the soil nail takes place with varying grout / soil bond stress along the soil nail upon movement. In addition, the distribution of the soil nail load along its length differs as the excavation progresses (CIRIA 2005).

Figure 1 depicts the conceptual model of the development of soil nail load during excavation suggested by FHWA (2003). The transfer of load of the soil nail which provides support to the wall can be broadly delineated into two portions based on the loading profile along its length. The first portion starts from the connection with the wall, in which the soil nail is gradually loaded up, due to the outward soil and wall movements, towards a peak, and this portion of the nail is within the active zone of the retained soil. Beyond the load maxima, the shear stress acting on the soil nail reverses and the soil around the nail within this second portion containing its distal end holds the nail in place, providing pullout resistance against the load exerted onto the nail in the first portion.

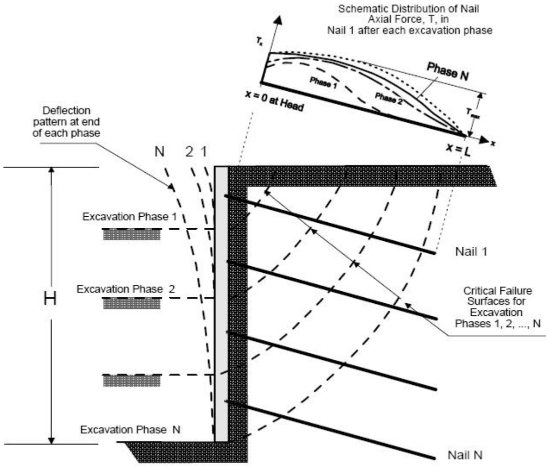

Figure 1: Load transfer along the soil nail (after FHWA 2003)

2.2 Design Approach

The design should consider ultimate limit states as well as the serviceability limit states.

2.2.1 Ultimate Limit States

Concerning external and internal instabilities, appropriate failure modes akin to those shown in Figures 5.1 and 5.2 of the Geoguide 7 – Guide to Soil Nail Design and Construction, GEO (2008), should be considered.

The length of the soil nail should be properly designed such that it should be long enough to protrude beyond the critical failure slip surface i.e. beyond the active zone. Nowadays, computer programs that model soil-structure interactions are able to predict the load distribution along soil nails. This prediction renders a guide for the designer to determine the possible extent of the active zone in the retained soil. Nevertheless, as the interactions between the wall, the nail and the soil could be complex, the algorithm of the computer program inevitably involves idealisation of the interactions, the designer should take cognisance of the modelling strategy adopted by the program and carry out necessary sensitivity analyses to demonstrate the appropriateness of the predicted results.

It is noted that some practitioners have also used limit equilibrium analysis to determine the location of the critical slip surface at various stages of the excavation. FHWA (2003) indicates limit equilibrium analysis may over-estimate the extent of the active zone.

Attention should be paid as to whether persistent and weak layer exists within the retained soil mass. The presence of such an adverse geological feature will dictate the extent and geometry of the active zone.

It is worthwhile to note that the maximum load of the upper soil nails may not occur at the final stage of the excavation in accordance with FHWA (2003), also see Figure 1. The load of the upper soil nails may decrease due to load redistribution following the wall deflection profile which changes at different stages of the excavation. Thus, the soil nail load at each stage of excavation should be determined and the maximum should be adopted for checking against the pullout capacity.

Besides, the kickout, hydraulic and foundation failures should also be considered as discussed in GCO (1990).

2.2.2 Serviceability Limit States

In local practice, serviceability of the design is normally checked by deformation analysis using computer program. However, some of the required inputs to the computer program may not be easily determined or may not have been calibrated by case histories or field observations.

For instance, the stiffness of soil nail is an input required by the “beam-on-elastic-foundation” type or “pseudo-finite-element” type computer program, in which soil nails are basically treated as struts without taking consideration of the interaction between soil nails and the retained soil. It is noted that the many practitioners have proposed to adopt the Young’s modulus of the steel re-bar of the nail for calculation of the nail stiffness i.e. stiffness equals to E_sA_s/L, where E_s is the Young’s modulus of the steel, A_s is the cross-sectional area of the re-bar in the proposed soil nail and L is the total length of the proposed nail. However, it should be cautioned that the load-deformation characteristic of nail is not merely controlled by the stiffness of the steel re-bar but also the movement between soil and grout to mobilise the necessary pullout resistance. This was evident in some excavation sites that the movement between soil and grout could be a controlling factor in determining the stiffness of tie-backs.

It is, therefore, advisable to verify the assumed stiffness of soil nails by comparing it with the actual stiffness determined from pullout tests. The pullout tests should be carried out at the design soil-grout bond stress, and should follow the procedures outlined in GEO (2008). Figure 2 illustrates one of the rational ways of the comparison. The solid line in the figure shows the load-deformation relationship of a steel re-bar in the same length of the soil nail subject to pullout. Broken-line A is the load-deformation relationship of the nail deduced from the test result, which is obtained by subtracting the elastic extension of the free-length from the elongation measured in the test. If the stiffness of the nail exceeds the stiffness of the steel re-bar as depicted by Broken-line A, the assumption to adopt the Young's modulus of steel for estimation of the stiffness of the soil nail would be valid with a certain degree of conservation. However, if the stiffness is found lesser as depicted by Broken-line B, the assumed stiffness adopted in the deformation analysis should be factored down appropriately based on the actual stiffness observed.

Some finite element programs allow users to carry out numerical simulations without taking into account the relative movements between nail and the surrounding soil. This may not be compatible with the actual behaviour of the nail when it is subject to tension. A better modelling strategy using interface elements along the soil nails in the analysis should be adopted. The parameters of the interface element should be adjusted to simulate the load-deformation behaviours found from pullout tests.

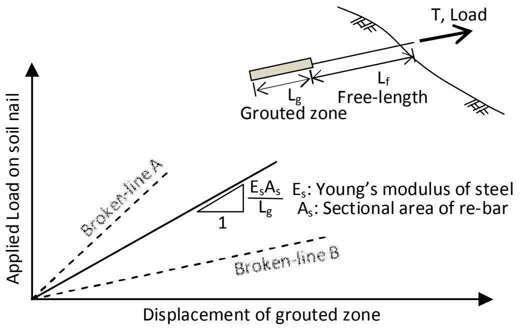

Figure 2: Verification of the assumed soil nail stiffness

Since the nails used in the excavation support system carry sustained load, creep test should be carried out as per GEO (2008), if the nails are bonded in soil, in particular, fine-grained soils. The creep rate revealed by the test should satisfy the criteria suggested in GEO (2008). In addition, the additional deformation due to creep, if any, should not be neglected, in particular, if there are important facilities, sensitive to settlement, nearby or the excavation will not be backfilled in a short period of time. The extra deformation due to the nail creep can be extrapolated from the creep test results based on the expected excavation period. The effect of the creep should be incorporated into the deformation analysis by considering that there is a loss in the stiffness of the nail. The strategy of modelling the creep behaviour of struts recommended in the manual of FREW (2001) is relevant in this context.

3 OTHER SALIENT POINTS

3.1 Corrosion protection

Although most of the excavations are temporary, appropriate recommendations on the provision of corrosion protection measures to soil nails outlined in GEO (2008) should not be overlooked.

3.2 Constructability

The local construction industry has a lot of experience in installing soil nails. Construction difficulties have encountered sometimes as reported by Ng et al. (2004). Other construction problems like significant water outflow from drillholes are also reported by Chang (2006).

During the planning stage of the excavation, if the option of soil nailed wall is considered, the constructability of the proposed nails should be carefully examined, in particular, when long soil nails are required.

Some construction difficulties of soil nails such as collapse of drillholes in loose materials or buried drainage lines could result in disturbance to the vicinity of the site. The cumulative effect of the collapse of drillholes could result in undue settlements of the adjacent ground.

In addition, grouting under high groundwater tables or within regimes of significant groundwater flow should be avoided since the integrity of the grout may be affected.

Schemes which comprise soil nails installed in close vicinity of sensitive facilities, for example, foundations of buildings, should be critically assessed, given the vulnerability of damaging the sensitive facilities as the drill rod could be deflected due to obstructions within the soil layer. Adequate clearance between the proposed nails and the sensitive facilities should be allowed.

If installation of soil nails in difficult ground conditions is needed, constructability should be demonstrated by trials at the early stage of the excavation.

Besides, the use of steep soil nails should be avoided, since significant deformations of soil nails would be needed in mobilising stabilisation force (GEO 2008).

3.3 Monitoring

Monitoring using at least ground settlement points, movement and tilting markers on buildings and piezometers / standpipes should be provided similar to other excavation works.

In addition, the monitoring scheme should also be planned with an objective to provide forewarning of any malfunctioning of the soil nail system. For instances, installation of inclinometers on wall, strain gauges along nails at different elevations and load cells at nails should be allowed for in the construction contract. This renders a possibility to examine the extent of the active zone of the retained soil assumed in the design and to counter check whether the soil nail load has exceeded the design pullout capacity.

4 CASE HISTORY

A detailed account of a case history using soil nailed wall to support a 12 m deep excavation, next to an access road, is presented below.

4.1 Background information

The excavation was carried out on a gently sloping ground, comprising a 5 m thick Fill layer of medium dense slightly silty / clayey SAND, underlain by a layer of Completely Decomposed Tuff (CDT) of firm clayey SILT. The SPT-‘N’ values of the Fill ranged from 5 to 20, while the SPT-‘N’ values of the CDT varied from 20 to over 70. Monitoring data showed that the groundwater level of the site was 1m below the final excavation level.

The proposed soil nailed wall system comprised a vertical pipe pile wall made of 273 mm dia. steel circular hollow section at 600 mm c/c with 10 mm thick steel lagging plates. The toe level of the wall was 9 m below the final excavation level. The excavation was carried out in stages, 6 rows of 12 m long soil nails (T32 in 150 mm dia. drillhole @ 1.2 m c/c) were provided. A section of the proposed excavation is shown in Figure 3.

4.2 Soil nail design

Soil nail forces calculated in the deformation analysis were generally adopted in the soil nail design. The deformation analysis was carried out with a finite element method programme, PLAXIS Version 8. The design shear strength parameters of Fill and CDT were $\phi' = 33°$, $c' = 0$ kPa and $\phi' = 36°$, $c' = 5$ kPa respectively. In view of the rather limited groundwater monitoring data available when the design was prepared, the design groundwater level was taken as 1/3 of excavation height to cater for rise in the groundwater level. Surcharge due to construction plant was allowed in the design calculations. The axial stiffness and flexural rigidity of pipe piles followed those of the steel circular hollow section. The predicted maximum wall deflection and the predicted ground settlement immediately behind the wall were 34 and 14 mm respectively.

The results of the deformation analysis were consistent with some of the observations outlined in FHWA (2003). For instance, the maximum forces of some soil nails calculated in the deformation analysis, particularly the upper soil nails, did not occur at the final stage of excavation.

In checking pullout resistance, the portions of soil nails beyond the active zone determined by the deformation analysis were taken as the effective anchorage lengths.

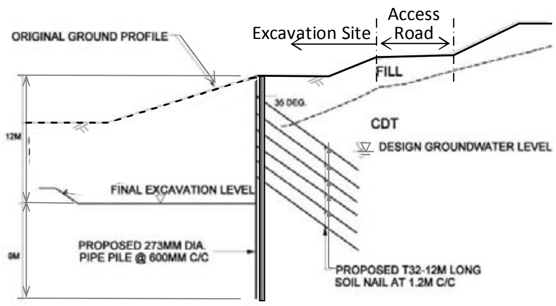

Figure 3: A section of excavation in the case reviewed

4.3 Monitoring

The deflection of the wall top and the ground settlement of the access road were monitored. The available monitoring records showed that the wall top deflection and the ground settlement of the access road were 30 mm and 5 mm respectively. However, no measurements were taken regarding the deflection profile of the pipe pile wall.

5 CONCLUSIONS

Soil nailed wall could offer a viable option to support deep excavations. There have been successful cases using this kind of excavation support system in Hong Kong. Nevertheless, some salient aspects that should be considered during the planning and design of the soil nailed walls should not be overlooked, such as the constructability of soil nails and the effects of the soil nail construction on the facilities in the vicinity of the excavation.

Since the interactions between the soil nails, the wall element and the retained earth are complex, for engineering design purpose, the stability and numerical analyses inevitably requires simplification or idealisation of the complex interactions. In order to secure a robust design, a holistic approach comprising a comprehensive load testing, monitoring and design review programme should be adopted. These include verification of the assumed soil nail stiffness based on pullout test results and revision of the settlement calculation by incorporating the effect of creep, if any, revealed by the creep test.

In addition, monitoring scheme which aims at providing forewarning of any malfunctioning of the excavation support system should be planned.

ACKNOWLEDGEMENTS

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A Special Cofferdam in a Reclaimed Site at West Kowloon

James W.C. Sze & Henry C.F. Lau
Ove Arup and Partners Hong Kong Limited

ABSTRACT

Due to the proximity of the adjacent properties in a built environment, stiff embedded reinforced concrete retaining walls, such as diaphragm wall, are usually required to facilitate the construction of deep basements.

A residential development situated in West Kowloon contains a deep basement with a maximum excavation depth of 14m below the street level. Despite from the constraints from the sensitive utilities and the properties in the vicinity, with careful planning, a relatively flexible temporary steel sheet pile wall with pre-loading onto the struts was adopted to facilitate the basement excavation in the medium dense reclamation fill. With adjustment to the cut-off levels of some piles, unconventional semi top-down construction sequence had been derived to enable an early commencement of the superstructure construction prior to the completion of basement. The basement wall was to be erected in a bottom-up manner. The design not only resulted in the programme benefit but also provided cost savings for the shoring works. This project is also an illustration of foundation and excavation & lateral support works to be designed in a holistic manner to achieve common goals amongst the Team.

1 INTRODUCTION

The site, measured about 140m by 80m, is located at Hoi Fai Road, West Kowloon. The site was reclaimed in early 1990s using a dredged method to the existing ground level of about +5.5mPD. The proposed development comprises the construction of five high-rise residential buildings, a five-level podium and a three-level car-park basement structure.

In terms of constraints, the site is bounded by the surrounding adjacent development structures, as illustrated in Figure 1, and various underground utilities. The Long Beach development adjoins the northern site boundary. The podium structures of the Long Beach development are supported by driven steel H-piles whereas the towers are founded on large diameter bored piles. One Silversea development adjoins the southern site boundary and the structures are supported by driven H-piles. To the east of the site, various utilities comprising a fresh water main, a gas main and various communication cables are running along Hoi Fai Road. To the west, the proposed development has a 26m set back zone named “Yellow Area” from the existing seawall facing Victoria Harbour.

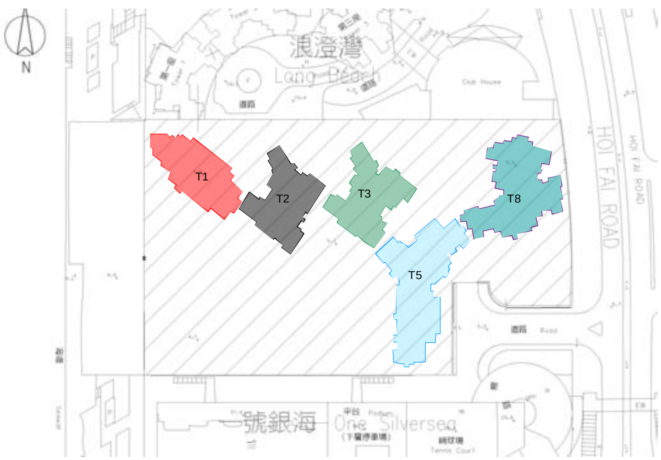

Figure 1: Site Layout with adjacent constraints

2 GROUND CONDITIONS

Based on the existing ground investigation results for the site area, the soil stratigraphy comprised fill, lenses of marine deposits left in-place, alluvium, completely to highly decomposed granite and then granitic bedrock. The fill was generally a greyish brown, silty fine and medium sand. The SPT 'N' values ranged from 9 to 57 which indicated a loose to dense state, with the majority being medium dense sand. Owing to the dredged reclamation, the fill layer had a thickness of about 20m and 28m. The lenses of marine clay was generally of soft to firm, grey to dark grey, silty clay with occasional shell fragments while the marine sand/silt was medium dense, grey to dark grey, slightly clayey silty sand. The alluvial clay was firm, yellowish brown and pink, silty clay. The decomposed granite generally varied from medium dense completely decomposed granite at the top of stratum to very dense highly decomposed granite. The SPT 'N' values of decomposed granitic soil ranged from about 20 to greater than 200. The rockhead levels as defined as "Grade III or better rock" was found to be around 25m at the eastern end and deeper than 85m at the western end with a deep weathering zone revealed at north-western corner of the site. The groundwater table had direct connection with the sea and fluctuated within tidal range. A typical geological section along the west-east direction is presented in Figure 2.

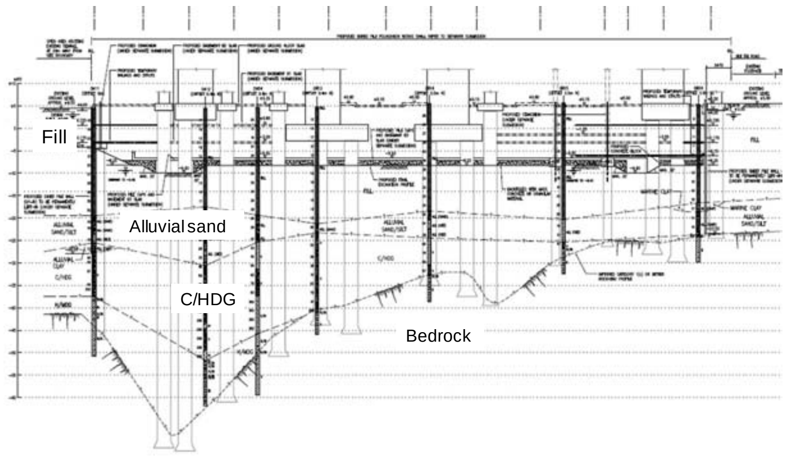

Figure 2: Typical geological section

3 DEVELOPMENT OF LATERAL SUPPORT SYSTEM

At the early stage of the project, a one-level basement structure was planned. A temporary sheet pile wall surrounding the basement box structure was considered to be appropriate and cost effective. In view of the proximity of sensitive utilities underneath Hoi Fai Road, a 5m offset of the basement was adopted along the eastern side. The sheet pile wall had been terminated into medium to dense completely decomposed granite in order to provide adequate toe stability and more importantly to provide effective water seepage cut-off during the basement construction. The sheet pile wall is also left-in permanently as it was also taken as the seepage cut-off wall for the basement.

During the course of the sheet pile installation, the building scheme had been modified and demanded a three-level basement structure, which required an excavation depth of around 14m, over the majority area of the site. With a temporary steel sheet pile wall of type SP-IV adopted as the embedded retaining wall, conventional bottom-up construction of the basement was first considered and four layers of long-span heavy steel shoring were proposed for the excavation within the medium dense reclaimed soil. Pre-loading was applied to the second and third layers of the shoring to minimize the potential lateral deflection of the wall and hence the incurred settlement at the area beyond the wall.

Most importantly, the project demanded a fast-track construction programme in order to enable an early start of the superstructure. The overall construction programme could be significantly affected by the probable unforeseen construction difficulties/delays during the bottom-up construction method. Furthermore, due to the fluctuation of the market price of steel members at the time of design, the construction cost with the steel shoring option for the bottom-up excavation was very high. Instead, the feasibility of a semi top-down construction method was carefully examined in respect to the project need. Such construction sequence could facilitate an earlier commencement of the podium and tower construction while the basement excavation was carried out concurrently. In addition, the tonnage of the steel shoring members required could be greatly reduced.

4 CONSIDERATIONS OF THE COFFERDAM DESIGN

4.1 Proposed Sequence of Work

After installation of the pile foundation and the peripheral temporary sheet pile wall, the ground level was reduced to +3.2mPD to enable the entire ground floor slab to be cast. The pile caps for two towers near the site boundary could be also built with a soffit level of around +2mPD, as indicated in Figure 3, to facilitate the podium construction in due course. In order to cast the pile caps for the inner towers, local excavation to B1 level would be carried out as illustrated in Figure 4.

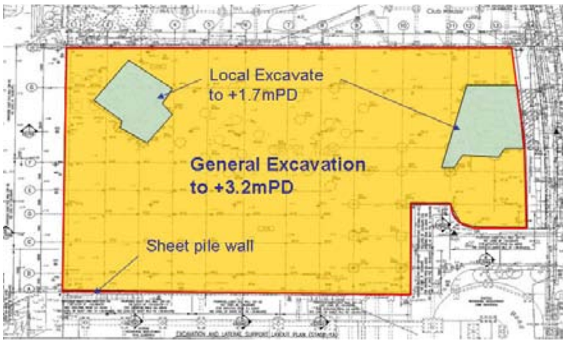

Figure 3: General excavation near ground level

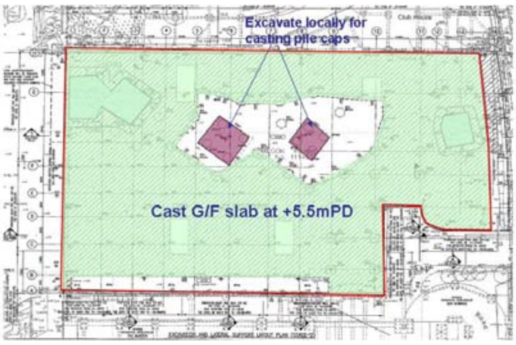

Figure 4: Local excavation for pile cap construction

This allowed the podium and the tower structure to be erected prior to the on-going basement excavation. A typical cross-section is shown in Figure 5. A snap shot during the basement excavation underneath the ground floor slab is illustrated in Plate 1.

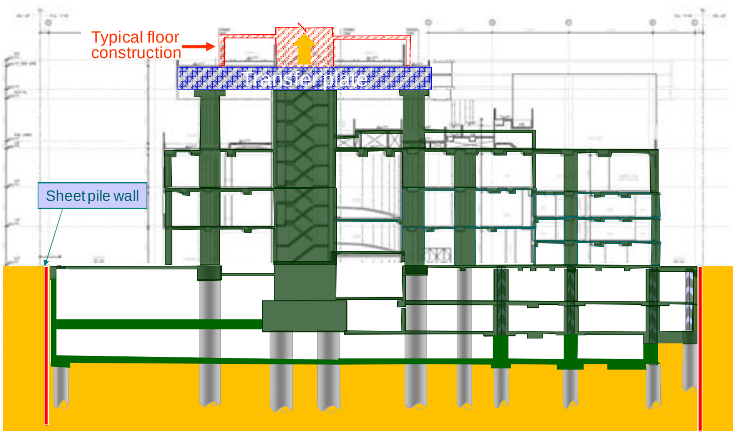

Figure 5 : Typical cross-section along N-S direction

Plate 2: Basement excavation underneath G/F slab

4.2 Foundation Works

A total of 117 nos. large-diameter end-bearing bored piles, 1.8m to 3.0m, were adopted as the foundation to carry the vertical and lateral loads from the proposed tower and podium structures. Initially, the pile caps were located at B3 level as per conventional design and these caps would be cast upon reaching the final excavation level. In order to facilitate a semi top-down construction and to commence the tower construction earlier, the cut-off levels of the bored pile within the tower zone were adjusted to the higher elevations.

The bored piles underneath the towers within the basement zone were also adopted as permanent columns. Special steel collars were reserved in the bored piles to allow coupler connection with the beams and slabs of the basement, as shown in Plate 2.

Plate 2: Steel collar in the bored pile

Plate 3: Stanchion installation into bored pile

At the podium zone, the bored piles had a cut-off level at B3 level. Steel stanchions typically comprising four H-steel members of S460 305x305x223kg/m UBP, combined into a squared bundle, was plunged into bored piles to support the semi top-down construction of the basement. The stanchions were encased by concrete to form square-sized composite columns in a later stage of the bottom-up excavation. A general view of the stanchion is shown in Plate 3.

4.3 Design of the Semi Top-down Basement Construction

Being the first layer of shoring to provide lateral support, the permanent concrete ground floor slab at +5.5mPD was cast against the sheet pile wall. After this a further excavation was carried out to a level of -0.2mPD and the second prop level at the B1 basement level at +0.9mPD was carried out as shown in Figure 6.

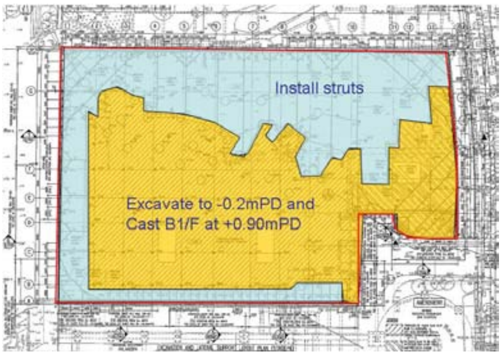

Figure 6: B1/F slab and temporary strut extent

Plate 4: General view of peripheral struts

The central portion of the permanent basement slab was cast whereas steel struts were installed at the peripheral zone to allow pre-loading at the main struts to be applied in order to minimize the wall movement and hence the induced settlement behind the temporary wall. The pre-load also resulted in more balanced sagging and hogging moments along the steel sheet piles which resulted in a more economic member size design. The prop level had been arranged such that the future basement slab can be cast prior to removal of the temporary struts at the corresponding layer. Similar process was repeated for the B2 level at -3.0mPD. The magnitude of pre-loading ranged from 300kN/m to 450kN/m. A general view of the peripheral struts installed at the basement slabs is shown in Plate 4.

After the completion of the slab and temporary struts at B2 level, the excavation was further proceeded to allow the installation of fourth layer of struts at -6.15mPD which were inclined struts against the completed central portion of B2 slab. This allowed the excavation to final excavation level of -8.25mPD to facilitate the B3 basement slab construction.

At the final excavation level, a 550mm thick underground drainage system was laid prior to construction of the permanent basement slab. The underground drainage system consisted of the sub-soil drains and filters to dissipate and control the uplifting water pressure acting onto the lowest basement slab and hence the basement box. The basement would then be constructed upward starting from the B3 level and the permanent peripheral basement wall. The peripheral late cast concrete slabs at B1 and B2 levels within the basement would be cast in bottom-up manner with traditional falseworks. The general view of the superstructure construction at the time of preparing this Paper is shown in Plate 4.

Plate 5: Podium construction concurrently with basement excavation

4.3 Analysis of the Semi Top-down Cofferdam

Pseudo finite element program OASYS FREW was used to conduct various sensitivity checks of the steel sheet pile wall using working stress and global factor approaches. The displacement, bending moments and shear forces in a flexible retaining wall and earth pressure on each side of the wall at the different stages of the construction were analysed from the program. The toe stability of the excavation and lateral support system was assessed by comparing the active and passive resistances. The credibility of the findings was further verified using the FEM program PLAXIS 2-D.

In order to prevent hydraulic failure during excavation, the sufficient penetration depth for the sheet pile wall below the final excavation level was provided. The hydraulic gradient and the potential groundwater drawdown was assessed using the OASYS program Seep.

The maximum ground settlement was estimated to be 30mm to 50mm along different areas beyond the site. This was mainly attributed to volumetric deformation induced by the wall deflection during the course of excavation. Since the two developments, to the north and to the south of the site, were largely suspended structures on deep foundations such deformations had an insignificant impact onto these structures. The maximum settlement of the underground utilities along Hoi Fai Road was estimated to be 25mm with an angular rotation of less than 1 in 500. A typical ground settlement profile against time at Hoi Fai Road is presented in Figure 7.

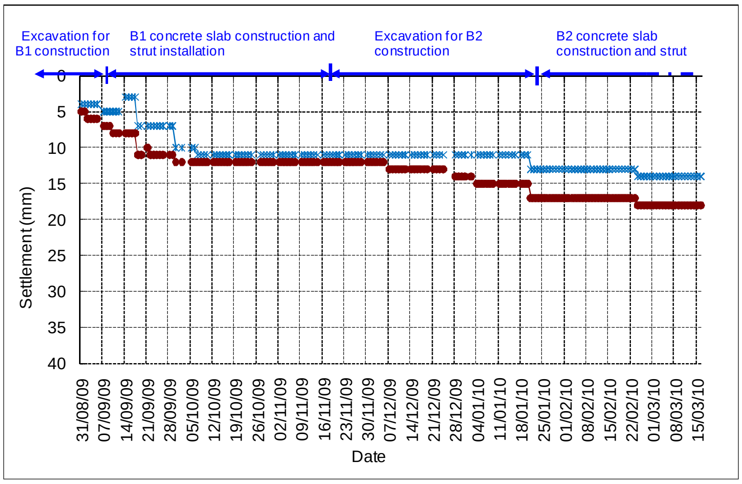

Figure 7: Typical settlement plot at Hoi Fai Road

5 OBSERVATIONS

5.1 Benefits of this Special Cofferdam

Traditional bottom-up construction allowed the development to be built in sequential manner of excavation work, basement box construction, podium construction and then the tower construction. The basement excavation and construction fall into the critical path and control the completion of the entire project.

If a top-down construction is adopted, diaphragm wall or secant bored pile wall is commonly adopted to serve as the supports, both laterally and vertically, during the excavation as well as forming part of the permanent basement wall. However, a diaphragm wall or a secant bored pile wall is costly and usually demanded a longer construction time. The cohesionless reclamation fill may also cause severe overbreak to the reinforced concrete walls which required extra efforts in trimming the wall face upon exposure.

In this cofferdam design, a steel sheet pile wall is adopted which is cost effective to serve the purpose of a temporary wall. The sheet pile wall is also to be left in place to provide seepage barrier for the under-slab drainage system. With the semi top-down construction method, the cofferdam has facilitated a fast-track construction programme for the superstructures as well as balanced the induced movement and the cost for the shoring works.

5.2 Vibration induced Settlement

Cohesionless sandfill is vulnerable to ground vibration causing settlement, the magnitude of which is difficult to accurately quantify. Various installation methods, including hydraulic hammers with different drop heights, different frequencies of eccentric vibrators had been tried at site in order to derive the least disturbing method to install the sheet pile sections. Nevertheless the induced ground settlement was found to be excessive. In the sensitive areas, the sheet pile wall design was subsequent changed to prevent hard driving at the very last portion of penetration. Toe grouting was used behind the wall from 2m above the toe level to a level of CDG with SPT ‘N’ > 60 to enhance seepage cut-off below the wall toe. The groundwater cut-off ability of the sheet pile wall was also demonstrated by a pumping test after the completion of the sheet pile wall installation and the toe grouting work.

5.3 Inspection of Sheet Pile Wall Condition during Excavation Stage

Similar to other types of walls, local defects may occur at the sheet pile wall. The most common defect of the sheet pile wall is de-clutching at the interlock of individual sections due to percussive or vibratory installation method, in particular in the event of underground obstructions being encountered. This usually resulted in an excessive ground water inflow through leakage of the gap.

With such potential risk, it is essential in the excavation sequence planning to allow the condition of the sheet pile wall to be exposed and inspected in stages such that mitigation measures to these local defects can be implemented in time before further excavation. The site supervision from the Contractor as well as the Engineer also proved to be important to early identification of potential problems and to derive appropriate mitigation solutions prior to the situation deteriorating further.

5.4 Holistic Planning, Structural & Geotechnical Designs & Construction Team Input

This project is demonstration of a holistic approach adopted amongst the Client, the Architect, the Designer as well as the Contractor in deriving the optimised design as well as construction method and sequence in achieving the common goals.

This project is also an illustration of the collaborative and interactive efforts amongst the designers in achieving a balance between the planning and engineering requirements with respect to the public safety consideration, in particular the serviceability requirements in ground and structural deformations in associated with a deep excavation. Furthermore, the experienced planning of construction activities by the main contractor is also proven.

At the time of preparing this Paper, the construction progress is ahead of the tentative programme and the basement has largely been completed.

6 CONCLUSIONS

Through the careful and detailed planning, a special semi top-down construction method of the basement has been derived to enable an early commencement of the superstructure construction prior to the completion of basement with the use of a steel sheet pile wall to serve as a temporary embedded retaining wall. The design not only resulted in the overall construction programme benefit but also attained cost savings in comparing to the sequence and shoring works required for a conventional bottom-up construction of the basement.

Design for a Deep Shaft under Unbalanced Loads in Mid-levels Scheduled Area of Hong Kong

Seven K.M. Yau, H.W. Tam & Edwin Y.F. Lee
Ove Arup & Partners Hong Kong Limited

ABSTRACT

This paper presents the geotechnical concerns and design methodology of an 80m deep and 16m diameter circular shaft excavation within the Mid-levels Scheduled Area, Hong Kong Island. The design process involves assessment of potential groundwater damming effects of the permanent shaft structure, their impact on the regional groundwater regime and hence the regional slope stability. An unbalanced load condition, attributed to the ground level difference of about 11m and unbalanced groundwater pressure along the greatest gradient, has been considered in the design.

A 2-D PLAXIS model has been first set up to assess the overall lateral stability of the shaft under the unbalanced loads. Having obtained satisfactory assessment results of the 2-D analysis, a more rigorous 3-D GSA model is then adopted to assess the detailed structural behaviour of the circular shaft and its structural adequacy. Comparisons of the analysis results obtained from the 2-D PLAXIS and 3-D GSA models are made. While 2-D plane strain model for a circular shaft subject to unbalanced loads is considered adequate to assess the ground movements and the overall stability of the lateral support system, a 3-D model is warranted for the detailed design of excavation works in a circular shaft under unbalanced loads.

1 INTRODUCTION

1.1 Project Overview

The West Island Line (WIL) is a 3.3km long extension of the existing MTR Island Line from Sheung Wan to Kennedy Town, with two intermediate stations at Sai Ying Pun (SYP) and the University of Hong Kong. A project key plan is shown in Figure 1.

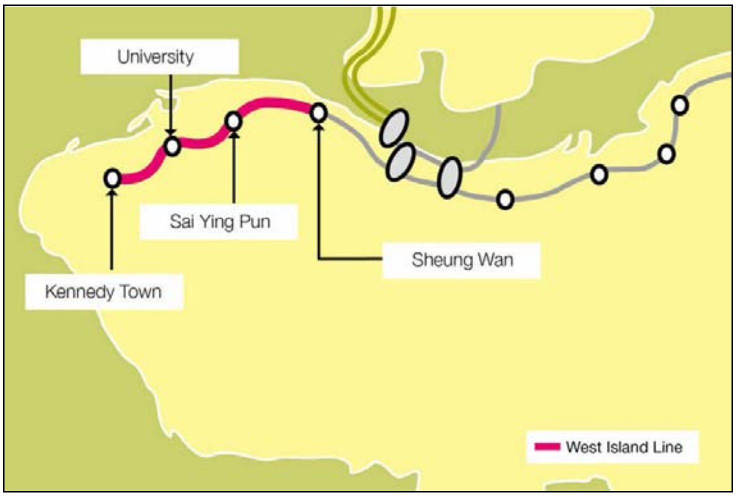

Figure 1: WIL project key plan

To serve both uphill and downhill areas of the old and historic district of Western, the WIL alignment has been strategically shifted inland and deeper underground, with some entrances being located within the Mid-levels Scheduled Area, see Figure 2. The construction work of WIL, with an estimated cost of HK$15.4 billion, began in July 2009 for completion in late 2014.

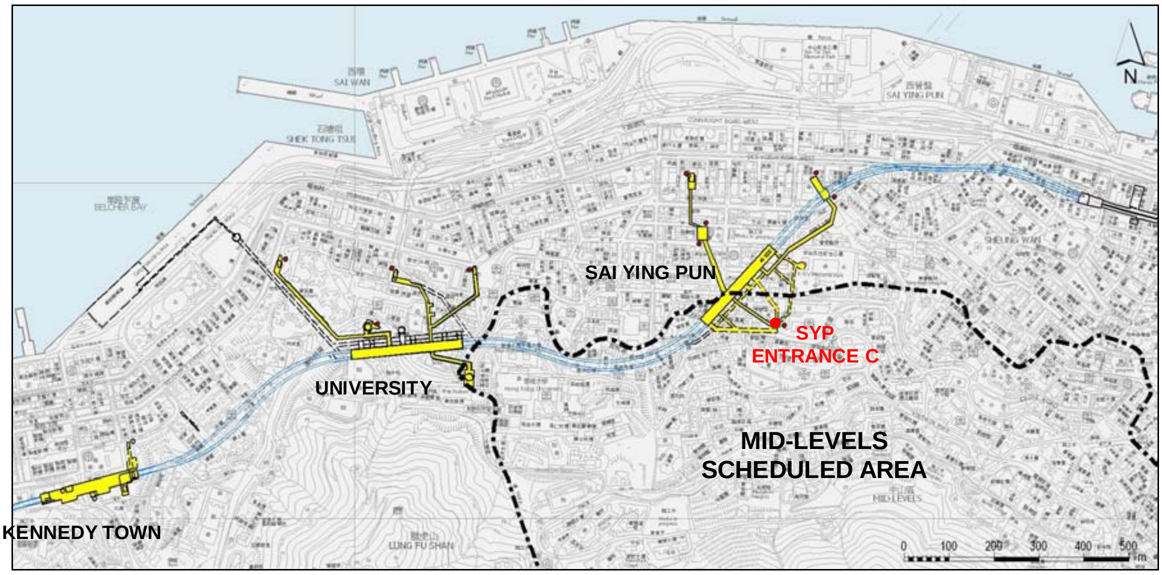

Figure 2: WIL project layout plan

1.2 Location of SYP Entrance C

SYP Entrance C (hereafter called “the Entrance”) is located to the southeast of the SYP station. A location plan of the Entrance is shown in Figure 3. The site area is currently occupied by the existing David Trench Rehabilitation Centre which will be demolished to the existing building platform at +50mPD. Bonham Road runs along the southern site boundary at about +61mPD and is retained by an existing structural wall of the Centre. Eastern Street Methadone Clinic (a Grade II Historic Building) is located at appropriately +44mPD to its north, with a 5.5m high retaining wall supporting the level difference. Another retaining wall (maximum 4.7m high) runs along the eastern boundary of the site on Eastern Street. To the west, there are existing two high-rise residential buildings namely, Bon-Point Building and Kam Ning Mansion. The site and these two buildings are separated by an existing staircase, named West End Path. An access road separates the site and Bonham Road Government Primary School (another historic building) in the east.

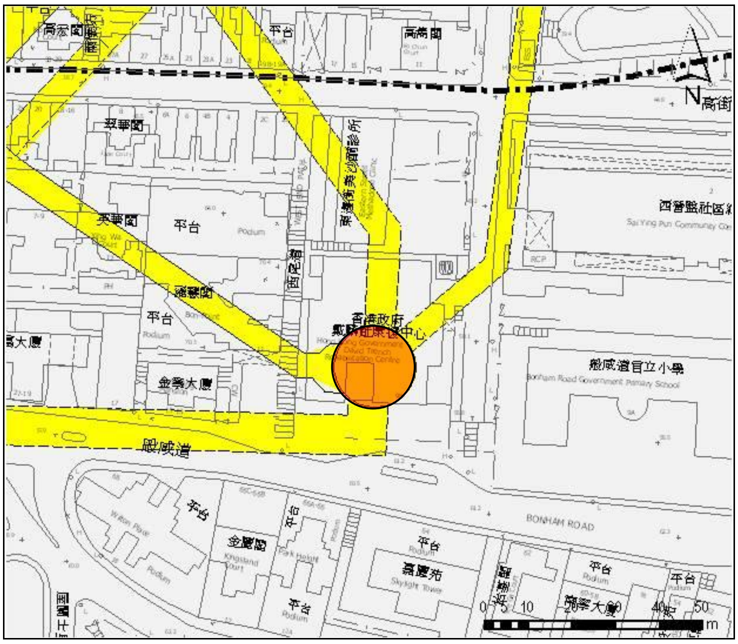

Figure 3: SYP entrance C layout plan

1.3 Proposed Structure for SYP Entrance C

The Entrance provides a pedestrian access connecting Bonham Road area at +61mPD and the concourse level of SYP Station at -24.85mPD. This leads to an approximately 80m deep shaft coupled with an adit to overcome the marked level difference between the station concourse and the street in Mid-levels Scheduled Area, see Figure 4.

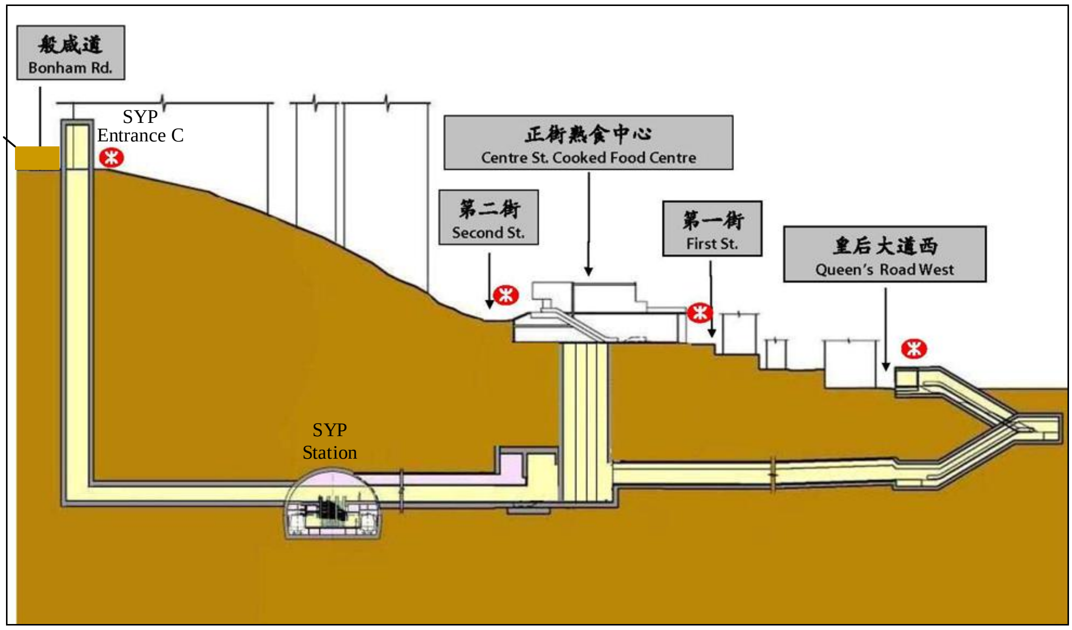

Figure 4: Cross section of SYP station and network of pedestrian subways

The proposed entrance structure consists of a three-storey reinforced concrete superstructure above +50mPD and an underground circular shaft within the footprint of the superstructure. The 80m deep circular shaft has an internal diameter of 18.5m to accommodate four lifts. The shaft is linked with a pedestrian adit and three other ventilation adits leading to SYP station, see Figure 5.

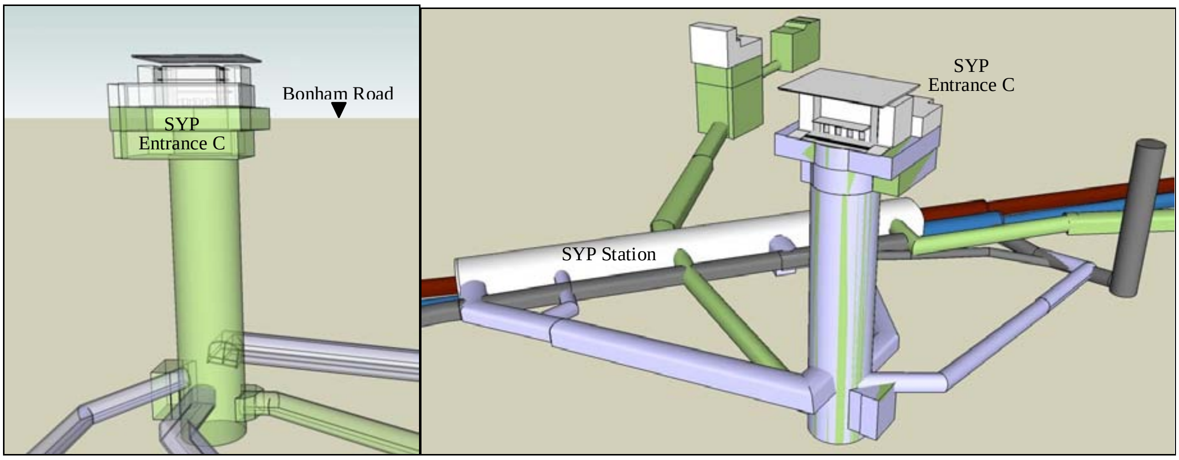

Figure 5: Locations of SYP entrance shafts relative to SYP station in 3-D space

The proposed circular shaft is located in front of an existing 11m high retaining wall below Bonham Road. The minimum clearance between the existing retaining wall (after temporary modification works) and the proposed shaft is about 2m, see Figure 6.

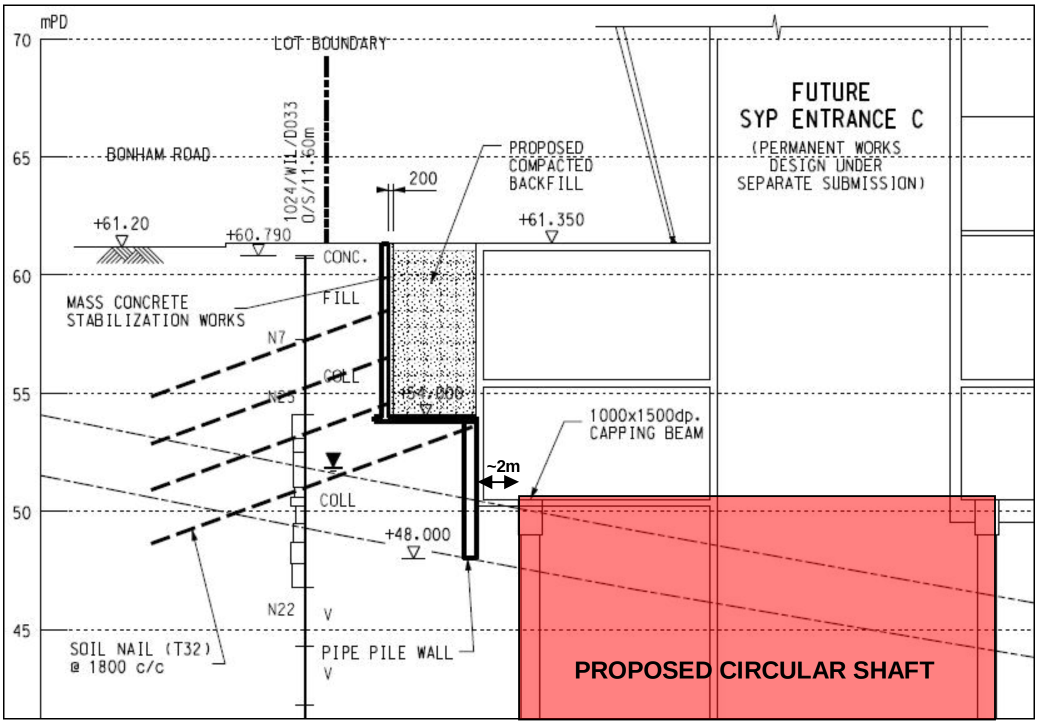

Figure 6: Cross section of the entrance

1.4 Geological and Groundwater Conditions

The site is generally underlain by 10-20m thick colluviums, with localized fill behind the existing walls, followed by in-situ weathered rock. Grade III or better rock head is encountered at +22mPD. The groundwater table is generally located at +53.5mPD on uphill side of the entrance, and is dipping following the general gradient of the Mid-levels Scheduled Area to +47.0mPD on the downhill side of the entrance. A geological section across the site is shown in Figure 7.

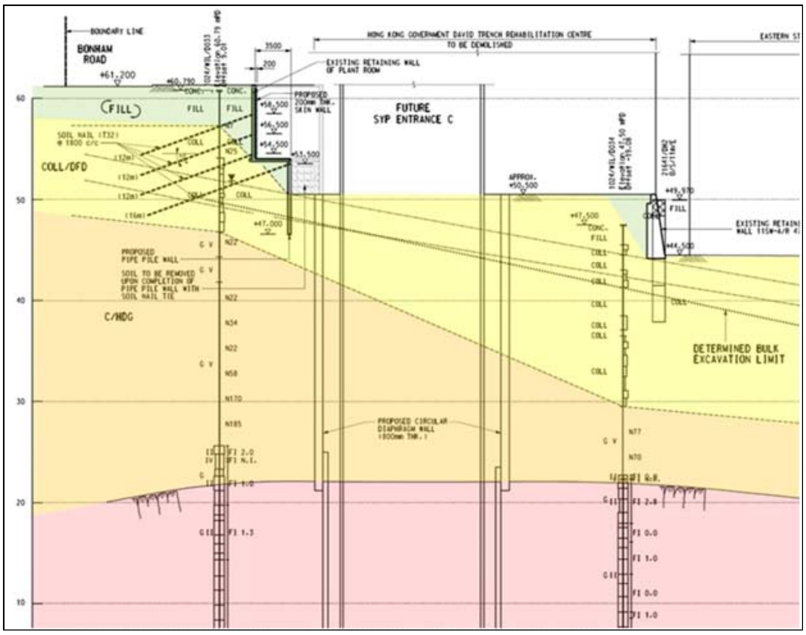

Figure 7: SYP entrance C geological cross section

1.5 Proposed Works for SYP Entrance C

Construction of the shaft will involve approximately 28m deep excavation in soil followed by 52m deep rock excavation below a platform at +50mPD after demolition of the existing David Trench Rehabilitation Centre and temporary site formation work. The final excavation level for the shaft is -32mPD.

An 800mm thick diaphragm wall is proposed to form a circular cofferdam with an internal diameter of 18.5m to support the excavation in soil. The diaphragm wall will toe into the Grade III rockhead by 300mm at approximately +22mPD. Rock fissure grouting below the toe of the diaphragm wall will be carried out along the perimeter of the excavation area to avoid excessive groundwater drawdown outside the circular cofferdam during bulk excavation. After installation of diaphragm wall and necessary toe grouting, a full-scale pumping test will be conducted to check the effectiveness of the water cutoff and make sure that no excessive groundwater drawdown exceeding 1m will occur during bulk excavation stage.

Bulk excavation will then commence at +50mPD until the required excavation level of -32mPD is reached. The toe of the diaphragm wall will be supported by temporary ring beam if necessary before rock excavation proceeds. Rock joint mapping will be conducted as rock excavation proceeds downward. Temporary rock support measures, such as temporary rock dowels and sprayed concrete cover, will be implemented in 2m vertical intervals in accordance with the Q method of rock classification, using methods outlined by Grimstad & Barton (1993). Upon reaching the final excavation level, a circular concrete lining wall will be constructed bottom up with a base raft founding directly on Grade III or better rock to form the lower shaft portion.

2 DESIGN SYNOPSIS

2.1 Drained against Undrained

It has been assessed that the top 20m Grade III rock cover is capable of providing a good barrier against groundwater drawdown above the rockhead. To achieve a cost-effective design, the shaft portion 20m below Grade III rockhead and the associated adits are designed as drained underground structures, and a drainage layer will be provided over the drained portion of the shaft and underneath the base raft. Though no build-up of water pressure is anticipated over the drained shaft portion, a minimum hydrostatic groundwater pressure of 20kPa is considered in the design as a contingency in the event of groundwater build-up, see Figure 8.

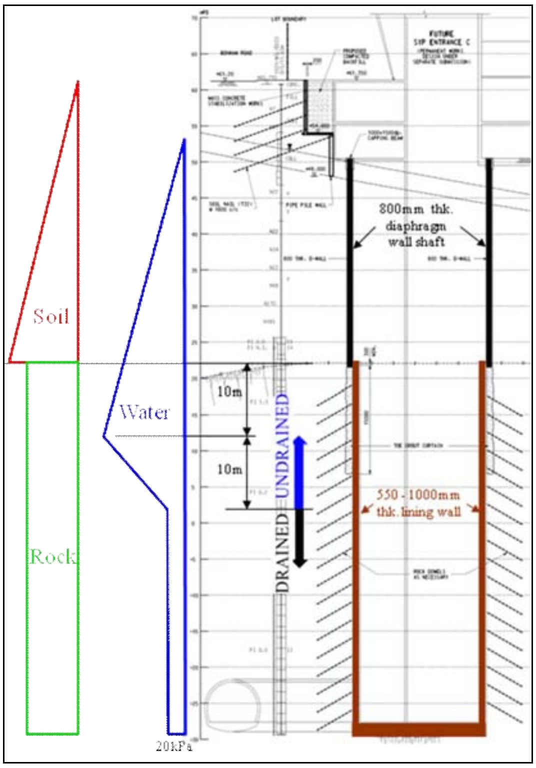

Figure 8: Design water, soil and rock pressures for the shaft

2.2 Design Considerations

Deep shaft excavation below the “Bulk Excavation Limit”, as imposed by the Building Authority to control cumulative adverse effect of the bulk excavation in this built-up terrain area, is required. Geotechnical design of this 80m deep shaft excavation in Mid-levels Scheduled Area demands the following special considerations and assessment.

- Damming of groundwater flow due to the presence of the shaft and the adverse impact on the subsequent bulk excavation works;
- Adverse effect on the regional slope stability due to the excavation works;
- Overall stability of the excavation works due to the unbalanced groundwater pressure and the unbalanced earth load arising from the ground level difference between Bonham Road on the uphill side (+61mPD) and the platform level of +50mPD on the downhill side of the shaft;
- Assessment of ground movements due to the excavation works and associated dewatering within the cofferdam;
- Effect on nearby existing geotechnical features and buildings, some of which are graded historical buildings.

2.3 Effects on Damming of Groundwater Flow

The shaft has an outer diameter of 20m against the groundwater flow. An assessment of the potential damming effect on the groundwater regime due to the presence of the undrained shaft structure has been carried out. The assessment was conducted using the Visual MODFLOW software package (Schlumberger 2006), which is a well-established computer program developed by the US Geological Survey in the early 1980’s and is widely used for simulating groundwater flow.

The assessment involves the following key steps:

- Review of all existing available geological and hydrogeological data;
- Development of the conceptual groundwater model covering a plan area of 100m x 100m around the shaft;
- 3D finite-difference analysis of steady state groundwater flow analysis of the existing pre-construction site condition in order to simulate/calibrate the existing piezometric heads and groundwater flow conditions;
- Steady state analysis of the post-construction site condition, incorporating an impermeable wall at the location of the proposed shaft, see Figure 9.

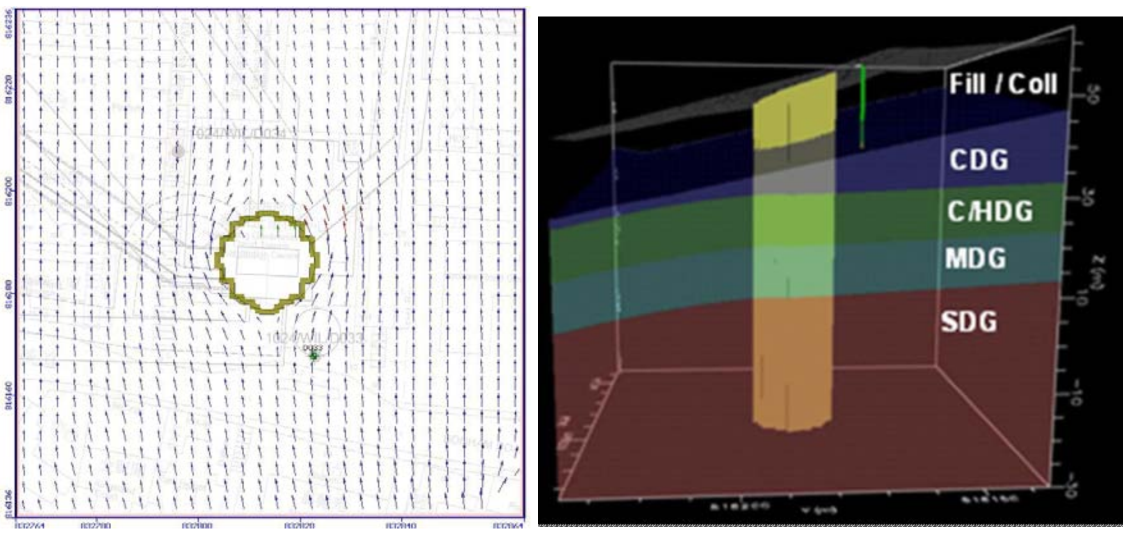

Figure 9: Simulation of an impermeable shaft within the MODFLOW model domain

The results of the analysis indicate a localized rise in groundwater levels of maximum 1.6m on the uphill side of the shaft and a localized groundwater depression (down to -1.2m) immediately on the downhill side of the shaft. Any damming effect is found to be highly localized and negligible impact is observed at distances greater than 20m from the shaft, as illustrated in Figure 10.

The design groundwater levels on the uphill and downhill sides of the entrance are taken to be +53.5mPD and +47.0mPD respectively. These design groundwater levels have catered for seasonal fluctuation above the highest recorded groundwater level and localized damming effect associated with full construction of entrance structure. The unbalanced groundwater pressure on the shaft would be considered in the excavation works design.

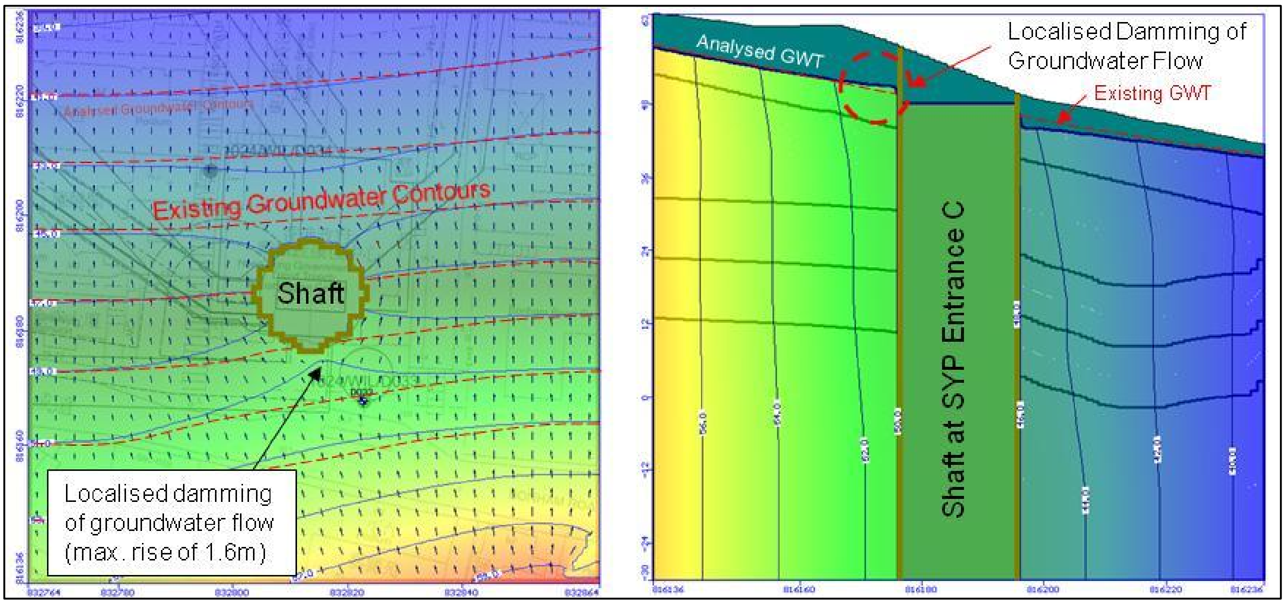

Figure 10: Comparison of pre- and post-construction piezometric head around the shaft

2.4 Methodology for Shaft Excavation Works Design

The circular diaphragm wall cofferdam is subject to unbalanced soil and hydrostatic loads. The total lateral load acting on the shaft, in this case, is a superimposition of an all-round equal lateral load and a net unbalanced lateral load. The all-round equal lateral load is obviously resisted by the hoop action of the circular diaphragm wall cofferdam. The net unbalanced lateral load would be resisted by virtue of the lateral / bending stiffness of the circular shaft core in a similar manner to a wind core of a tall building resisting lateral wind load.

A 2-D PLAXIS model, as shown in Figure 11, was established to assess the overall stability and cumulative ground movements due to the proposed circular shaft excavation work in a sequential manner. To simulate the hoop action of the circular shaft in 2-dimentional space, equivalent horizontal hoop springs at 1m vertical interval are assigned. The equivalent hoop stiffness (k) in the 2-D model is derived based on the following equation, ignoring the effect due to closure of joints between adjoining panels under loading.

$$k = \frac{E\,t}{R^2} \tag{1}$$

where E = Young's Modulus of Concrete, t = thickness of the circular wall and R = radius of the circular shaft.

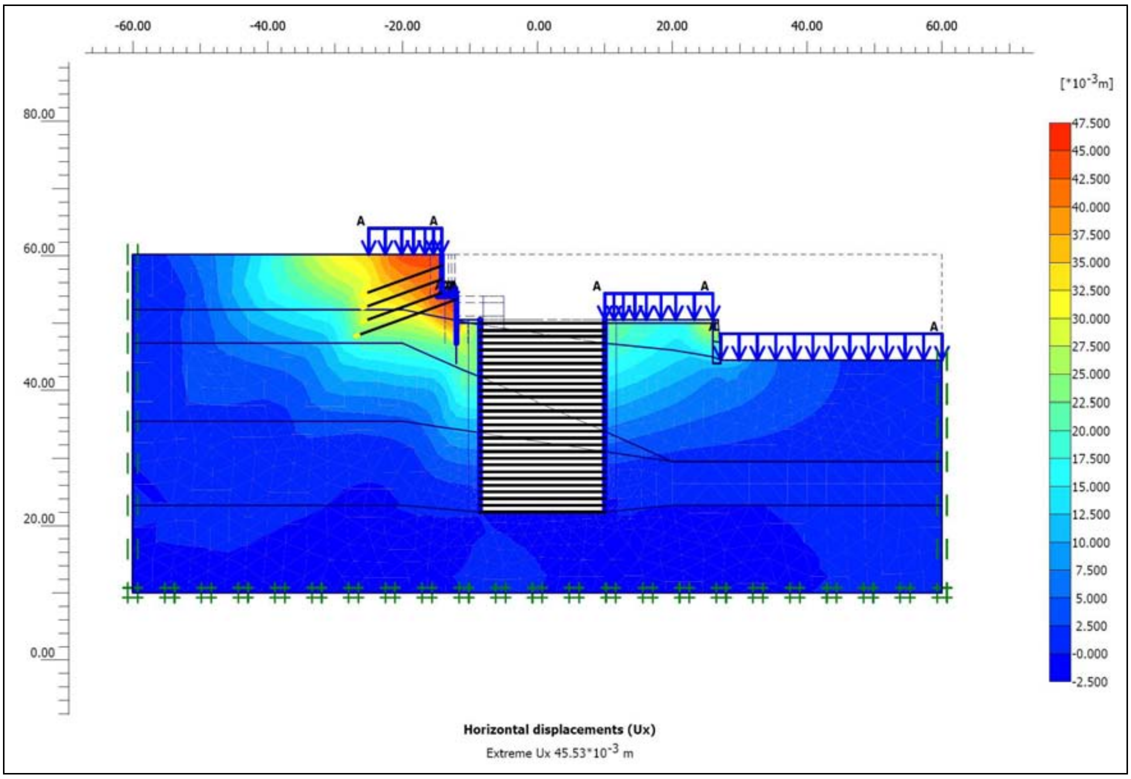

Figure 11: 2-D PLAXIS results (horizontal movement) due to the shaft excavation

In addition to the 2-D PLAXIS model, an equivalent 3-D GSA model (see Figure 12) has been established to analyze the full structural behaviour of the circular cofferdam subject to unbalanced loads. The equivalent hoop stiffness for the circular shaft is not required for this 3-D mode as the GSA analysis will account for the hoop actions based on the 3D structural arrangement.

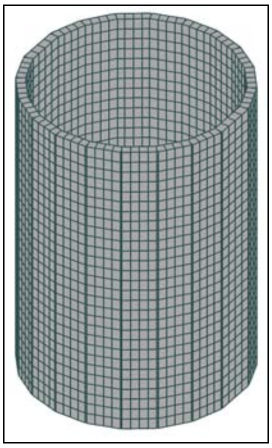

Figure 12: 3-D GSA model for the circular diaphragm wall shaft above rockhead

The circular diaphragm wall shaft is divided into the "east", "south", "west" and "north" quadrants vertically. The "south" quadrant faces Bonham Road (on the uphill side) while the "north" quadrant faces the Methadone Clinic. Different lateral soil pressures are applied to each quadrant of the diaphragm wall shaft. The applied lateral pressures for the "East" and "West" sides are identical though.

The diaphragm wall panels, each having a typical panel width of 2.8m, are divided into rectangular grids of 0.7m (horizontal) x 1.0m (vertical). Each of the grid points is attached with a non-linear no-tension soil spring in the radial direction to provide passive soil resistance to the lateral loadings. The soil spring stiffness is derived by a unit horizontal pressure divided by the corresponding lateral wall movements obtained from a 2-D PLAXIS model of the wall. The joints, with a vertical shear key between adjoining diaphragm wall panels are assumed to be pinned in horizontal planes so that horizontal shear forces can be transmitted between panels. The panels are however free to slide along the joints in vertical direction, assuming the worst scenario of having a skim of bentonite slurry being trapped between the joints, but the panels are monolithically connected at the cutoff level by a capping beam. The base of the diaphragm wall panels are resisted by 3 types of springs, namely (i) vertical base bearing springs representing the bearing resistance provided by the founding rock; (ii) horizontal socket friction springs acting tangentially to the faces of the diaphragm wall panels which are in contact with rock; and (iii) horizontal base friction springs acting in the radial direction at the base of the diaphragm wall panels. The adopted spring values are as follows:

Base bearing spring:	2,000,000kN/m/m^2	based on the Young's Modulus of Grade III or better rock.
Socket friction spring:	20,000kN/m/m^2	based on Hill et. al. (2000) which suggests that a socket friction of about 800kPa is achieved at a movement of about 30mm.
Base friction spring:	20,000kN/m/m^2	

The resulting bearing and friction spring forces derived from the GSA models have been checked and are found to be less than the allowable resistances of 5,000kPa (for bearing) and 350kPa for friction. In addition, the soil spring forces are checked against the allowable soil passive resistance.

The 80m deep excavation is generally supported by a circular diaphragm wall shaft in soil and a relatively smaller diameter lining wall in rock, see Figure 8. A capping ring beam will be constructed at the top of the circular lining wall. There is no structural connection between the toe of the diaphragm wall shaft and the capping beam of the lining wall. The circular lining wall however is designed to cater for the loads exerted from the base on the diaphragm wall and nearby pre-bored H-piles for the superstructure.

Due to the complexity of the adit connections with the circular lining wall, a separate 3-D GSA model is also set up for the design of the lining wall shaft structure and the connection collars with the adits, see Figure 13 for illustration. Its details are not presented in this paper.

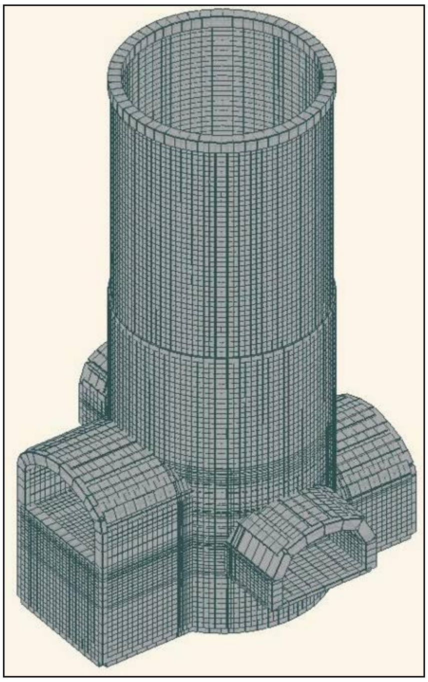

Figure 13: 3-D GSA model for the lining wall structure below rockhead

2.5 Comparisons between 2-D PLAXIS and 3-D GSA Results

The whole circular diaphragm wall cofferdam will lean towards the downhill side under the action of unbalanced loads. The deformed shape of the circular shaft is undoubtedly uneven at any levels on plan, as shown in Figure 14, and the validity of the circular hoop action can be verified by the 3-D GSA analysis. It not only provides a full structural design of the shaft structure, but also serves as a validation of the 2-D PLAXIS analysis.

Based on the GSA model results, no tension force would be developed at the interface of any adjoining panels, and the circular cofferdam is under compression in circumferential sense at any elevation. Therefore, the hoop action of the circular cofferdam is still in effect though it is subjected to an unbalanced lateral load.

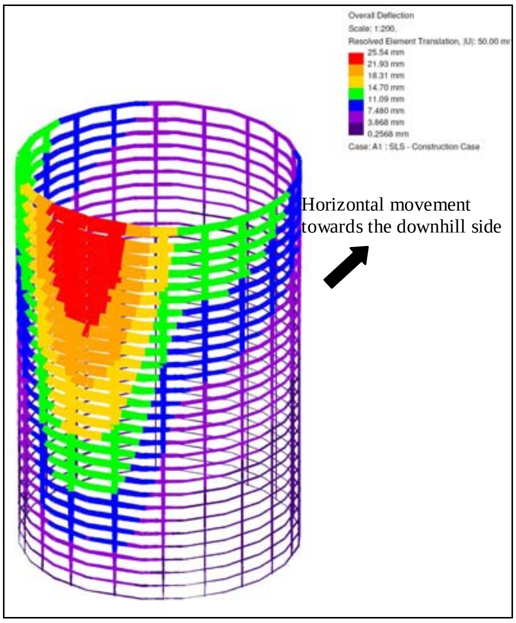

Figure 14: 3-D GSA model for the circular diaphragm wall shaft

As shown in Figure 15, the lateral wall movement profiles obtained from both the 2-D PLAXIS and 3-D GSA models are consistent for the wall portion on the uphill side. However, this is no longer consistent for wall deflection profiles on the downhill side. This may be attributed to the ignorance of the bending / complimentary shear actions of the circular shaft in the 2-D model as the circular hoop action is simulated solely by equivalent horizontal struts. The internal forces (i.e. bending moment and shear forces) of the shaft so obtained from the 2-D analysis do not tie in with those from the 3-D analysis, see the results presented in Figure 16.

Though the 2-D analysis results are not fit for structural design purpose, it provides relatively simpler and quicker assessments on the overall stability of the circular shaft and the induced ground movements at every stage of the shaft excavation works under unbalanced loading conditions.

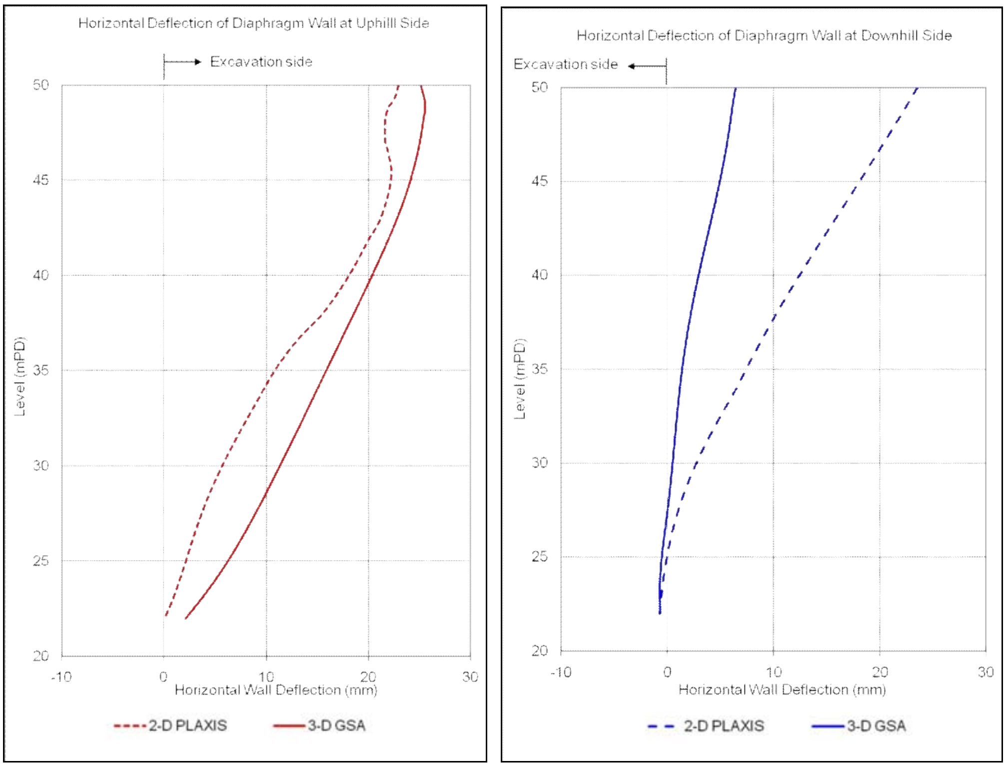

Figure 15: Horizontal deflection of the shaft

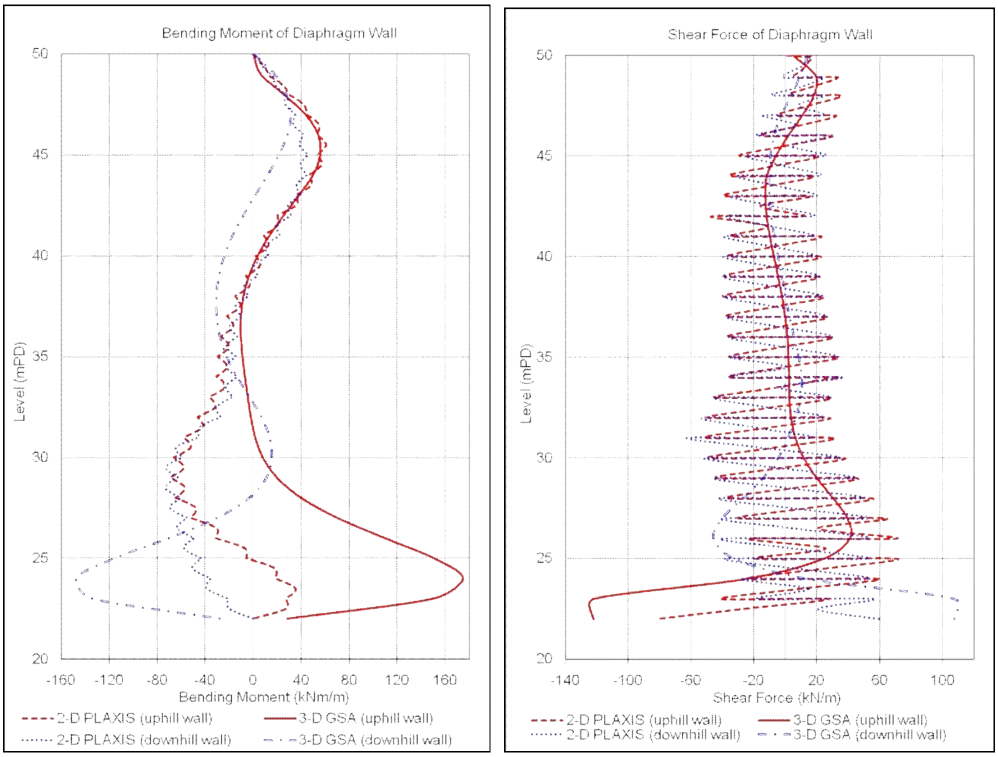

Figure 16: Bending moment and shear force of the shaft

3 CONCLUSIONS

For the design of a deep shaft under unbalanced loads (or in a steep terrain), a 2-D plain strain model could help assess the ground movements and the overall stability of the lateral support system during excavation. However, as illustrated in this paper, its 2-D analysis results are inappropriate for detailed structural design purpose. It is considered that a 3-D model is required to analyze the structural behaviour of the circular cofferdam under unbalanced earth loads.

ACKNOWLEDGEMENTS

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Design Considerations for the Excavation of the Dry Dock for Guangzhou Zhoutouzui Immersed Tube Tunnel

Joseph Y.C. Lo, W.W. Yang, K.F. Wong & C.K. Tsang
AECOM Asia Co. Ltd., Hong Kong

ABSTRACT

In order to alleviate the traffic congestion in the downtown area of Guangzhou, P.R. China, a new immersed tube tunnel will be built beneath Pearl River to connect Haizhu District and Fangcun District. The 340m long immersed tube tunnel, with a rectangular cross section of 31.4m wide and 9.68m high, will be divided into 4 tunnel units connecting with flexible joints under water. The immersed tunnel units will be divided into two batches for fabrication in the dry dock which is located at the alignment of approach tunnel along the river bank of Pearl River. To accommodate the fabrication of two tunnel units in one time, a dry dock with dimensions of approximately 220m long, 50m wide and 13.5m deep in a trench is required to be constructed in a densely populated residential area. Besides, the dry dock needs to be opened twice for the tunnel units to be floated out. This paper discusses the design considerations for the excavation of the dry dock under these special conditions.

1 INTRODUCTION

Several techniques are available for the construction of river crossing tunnels. Among them, immersed tube tunnels are composed of segments, constructed and floated to the site to be sunk into place and then connected together, which have unique features. Since 1900s, immersed tube tunnels have been widely adopted in cities around the world. In order to alleviate the traffic congestion in the downtown of Guangzhou, P.R. China, a new immersed tube tunnel will be built beneath Pearl River to connect Haizhu District and Fangcun District of the city. AECOM has been assigned as the chief designer of this key infrastructure project in Southern China.

One of the most important considerations in using immersed tube tunnel technique for river crossing tunnel is the availability of a dry dock for fabrications of the tunnel units. This dry dock should be built with sufficient space for the fabrication of several tunnel units in one batch. In addition, the design of a dry dock is governed by several factors such as available space, geological conditions, tidal level of the river and height of immersed tunnel units. The dry dock has to facilitate casting of immersed tunnel unit in the dry with sufficient draught to facilitate subsequent float up to transport out to the tunnel location.

Generally, the purpose built dry dock for the fabrication of immersed tunnel unit is located away from the tunnel site such that the construction works in the dry dock and the tunnel site can be carried out concurrently. Since Zhoutouzui Tunnel is located on a very constrained area with highly developed waterfront property adjacent to the tunnel site, it is impossible to find a suitable location for such dry dock. Consideration was given to construct the dry dock and immersed tunnel units remote from the site.However, due to the shallow river bed, this would require extensive dredging of Pearl River to allow transport of the tunnel element to its final destination. Therefore the better option is to construct the dry dock in line with the tunnel approach at the edge of the river bank. The advantages are that the transportation route for the tunnel units will be very short and the amount of dredging of Pearl River will be limited. However it entails the impact on the overall construction programme as the tunnel units need to be installed within the river and the dry dock drained before the construction of the cut and cover tunnel located inside the dry dock could commence.

This paper discusses the challenges encountered during the design of a dry dock for fabrication of the immersed tube tunnel units.

2 PROJECT BACKGROUND

Guangzhou Zhoutouzui Tunnel will form a main trunk road connecting Fungcun District and Haizhu District in Guangzhou City by passing beneath Pearl River. The location and layout plans of Guangzhou Zhoutouzui Tunnel are shown in Figures 1 and 2 respectively.

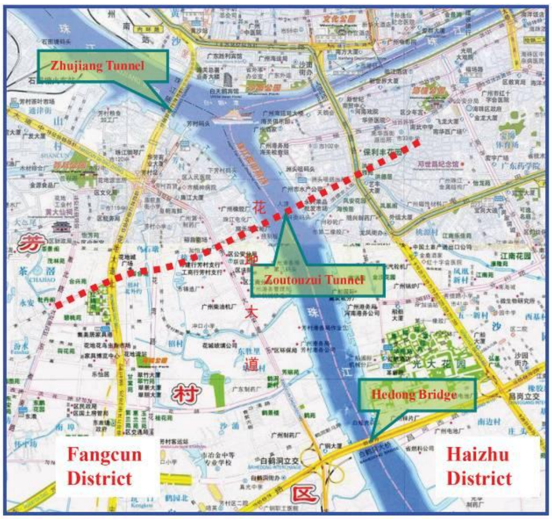

Figure 1: Location plan of Zoutouzui Tunnel

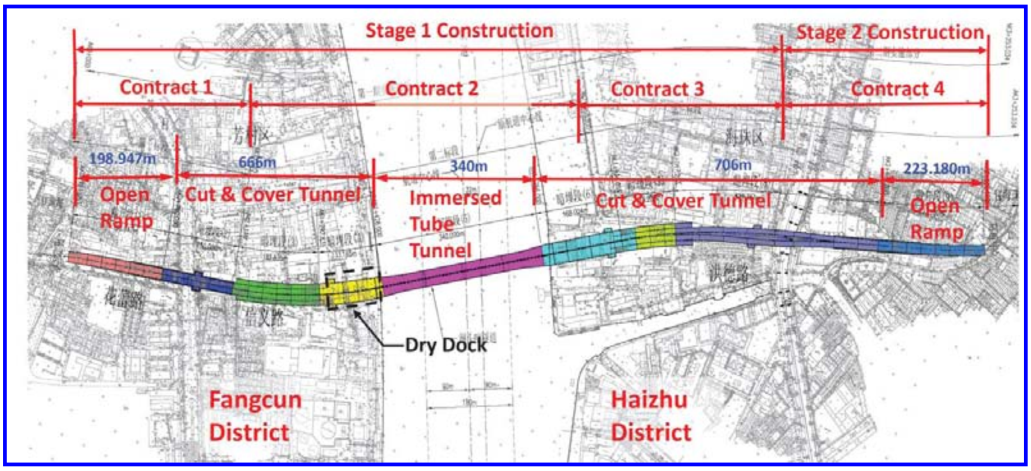

Figure 2: Layout plan of Zoutouzui Tunnel

The 340m long immersed tube tunnel, with a rectangular cross section of 31.4m wide and 9.68m high, will be divided into 4 tunnel units (E1 to E4) connecting with flexible joints. The lengths of E1 and E2 are both 85m while the length of E3 is 79.5m. Total length of E4 is 90.5m which consists of E4-1 (3.5m), E4-2 (85m) and final joint (2m) between E4-1 and E4-2. The layout plan and longitudinal section of the immersed tunnel are shown in Figures 3 and 4 respectively.

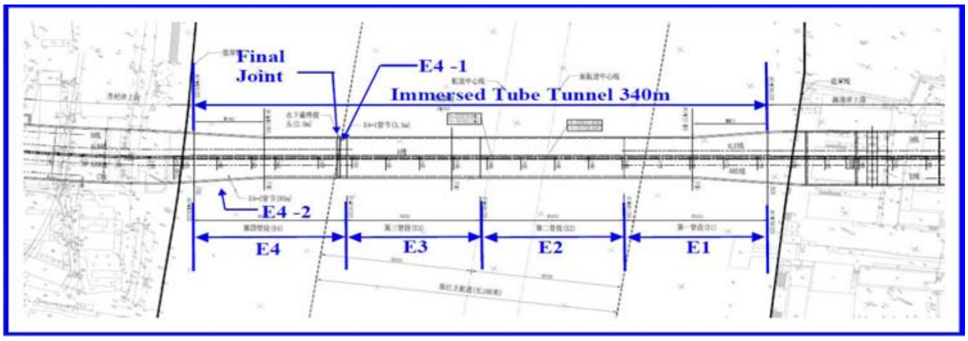

Figure 3: Layout plan of immersed tunnel

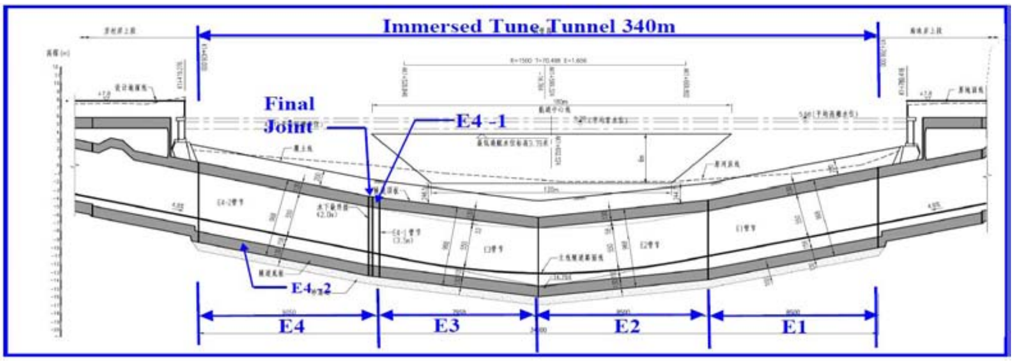

Figure 4: Longitudinal section of immersed tunnel

3 SITE DESCRIPTIONS OF THE DRY DOCK

The dry dock is located in line with the cut and cover tunnel on Fangcun side as shown in Figure 2. The fabrication of the four immersed tunnel units will be carried out in two batches in the dry dock. In order to accommodate sufficient space for the construction of two tunnel units at the same time, a dry dock with plan area of approximately 220m by 50m is required.

The location of the dry dock is within a densely populated residential area with relatively low rise buildings, generally of the order of one to three storeys; where the buildings within the footprint of the dry dock need to be demolished. The general layout plan of the dry dock is shown in Figure 5. Several important buildings in the vicinity of the dry dock include the followings:

(1) Zhujiang Electrical and Chemical Factory located at about 25m to the north western side of the dry dock;
(2) Historical German Church 1 located inside the dry dock that needs to be underpinned, and transport outside the dry dock during construction stage. It will transport back to its original location after construction completed.
(3) Historical German Church 2 located at about 13m to the south eastern side of the dry dock;
(4) A new church located at about 12m to the southern side of the dry dock;
(5) Fangcun Jianshe Building located at about 10m to the southern of the dry dock is to have piled foundations.

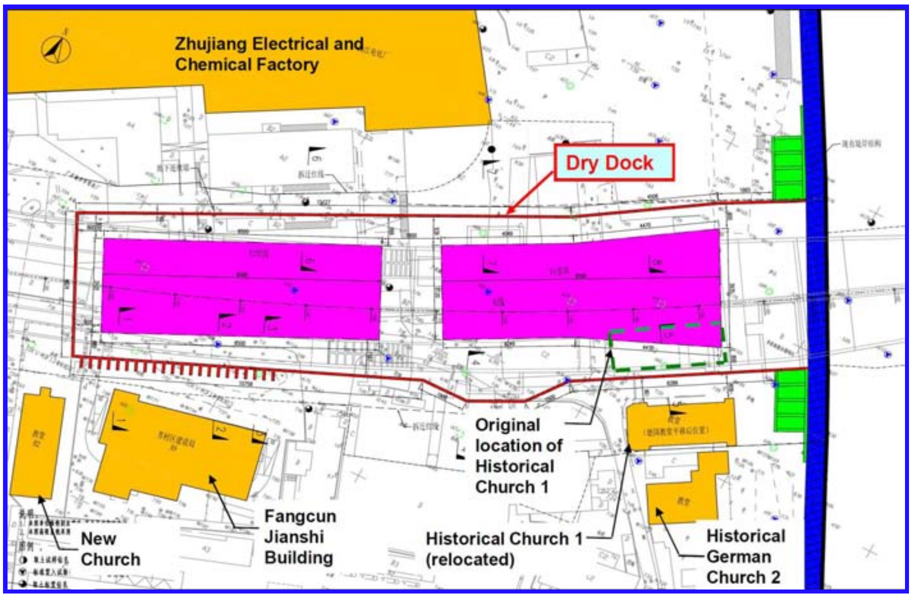

Figure 5: General layout plan of dry dock

4 GROUND CONDITIONS AND ENGINEERING PROPERTIES OF SOIL

Based on the ground investigation, the ground level at the dry dock is about +7.5m and the geological sequence anticipated in the dry dock consists of Fill, Alluvial Sand, Alluvial Silt / Clay and subsequently Decomposed Sandstone. The soil stratigraphies and their engineering properties are summarized in Table 1.

Table 1: Summary of ground condition and engineering properties of soil

Soil Layer	Thickness (m)	Base Level (m)	E (MPa)	c (kPa)	ϕ (degree)
Fill	3.0 to 7.2	4.1 to 1.0	12	5	15
Alluvial Sand	2.7 to 5.8	1.5 to -1.8	9	0	23
Alluvial Silt/Clay	0.0 to 11.9	-2.2 to -4.5	6	15	13
Completely Decomposed Sandstone	3.0 to 10.7	-5.3 to -13.0	85	28	20
Moderately Decomposed Sandstone	4.0 to 17.4	-12.0 - -25.0	500	200	28

5 DESIGN RIVER LEVEL FOR THE TUNNEL UNIT BEING FLOATED OUT OF THE DRY DOCK

According to the Zhoutouzui Tunnel Project River Monitoring Analysis Report produced by Guangdong Hydropower Research Institute, the mean lower tidal level is 4.41m. Based on the tidal monitoring record between January 2006 to December 2006 as shown in Figure 6, this tidal level has a confidence level of 85% (85% of tidal level is higher than 4.41m throughout the year) and is therefore selected as the design river level for the tunnel units being floated out of the dry dock.

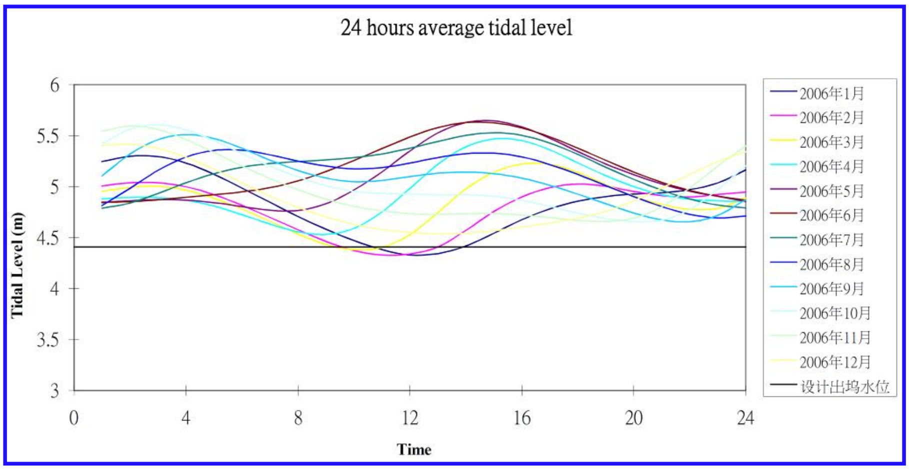

Figure 6: Tidal monitoring record

6 EXCAVATION LEVEL OF THE DRY DOCK

In order to ensure the immersed tunnel unit can float safely without hitting the bottom of the dry dock, the determination of excavation level, h, of the dry dock is presented in Figure 7.

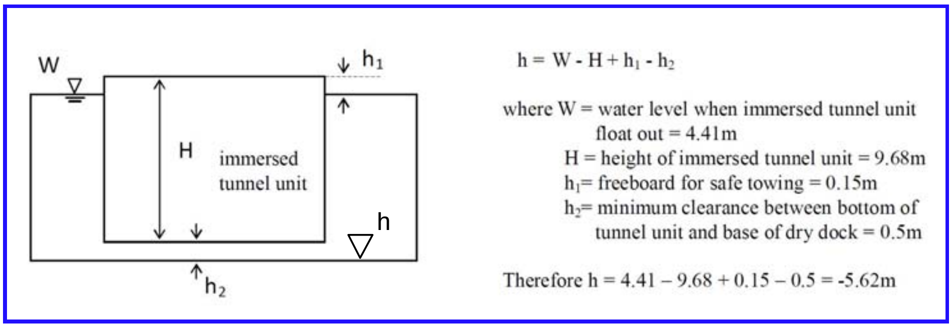

Figure 7: Determination of dry dock excavation level

In order to ensure that an even hydrostatic pressure is obtained on the base of the units during flooding to assist in the flotation process, a layer of 150mm no-fines concrete and 150mm gravel bed is proposed at the base slab of the dry dock below the tunnel unit positions. Therefore additional 300mm need to be excavated and making the design base level of dry dock to be -6.0m , with a total excavation depth of 13.5m.

7 EXCAVATION AND LATERAL SUPPORT SYSTEM

The dry dock is bracketed on both sides by heavily used buildings leaving a limited space for excavation. Therefore 800mm thick diaphragm walls along the perimeter of dry dock are proposed as the temporary retaining structure. In addition, struts are not allowed as they will obstruct the floating of tunnel units. Tie-back anchors are therefore proposed to be installed in the diaphragm wall for staged excavations. Once the tunnel units are constructed and floated out of the dry dock, the retaining walls along the river side of the dry dock need to be removed. Therefore strutted steel pipe pile wall with jet grout is proposed for the side of the dry dock facing the river.

Having constructed the tunnel units in a dewatered dry dock, it will therefore be necessary to flood the dock to allow floating out the tunnel units, then repeating the cycle of dewatering and flooding again for the construction and floating of second batch of tunnel units. The dewatering will be carried out again for the construction of the approach cut and cover tunnel. This special loading condition needs to be considered in the design of excavation and lateral support system. Besides, the soil deformations and wall displacements induced by the excavation should be controlled and limited to accepted values. The layout of excavation and lateral system is shown in Figure 8.

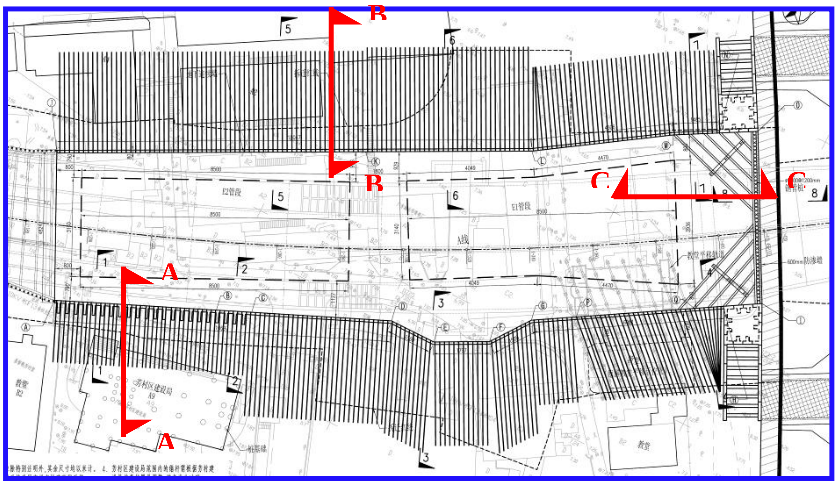

Figure 8: Layout of excavation and lateral support system

In order to prevent the anchors hitting the foundation of Fangcun Jianshe Building, T-shaped diaphragm wall (2.8m wide and 3.2m deep) with two layers of anchor with lengths of 27m installed at 40 degree to horizontal is proposed as shown in Figure 9. The general view of steel cage fabrication of T-shaped diaphragm wall is shown in Plate 1.

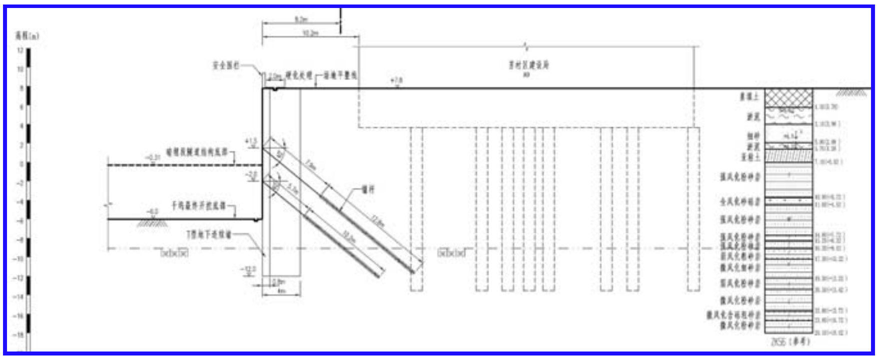

Figure 9: Section A-A

Plate 1: Steel cage fabrication of T-shaped D-wall

Plate 2: Installation of anchor

On the northern side and the southern side except the area of Fangcun Jianshe Building, diaphragm wall with 3 layers of anchor with lengths ranging from 16m to 40m are proposed. The general view of installation of anchors is shown in Plate 2. Typical section of support system is shown in Figure 10 and the general view of diaphragm wall and anchors is shown in Plate 3.

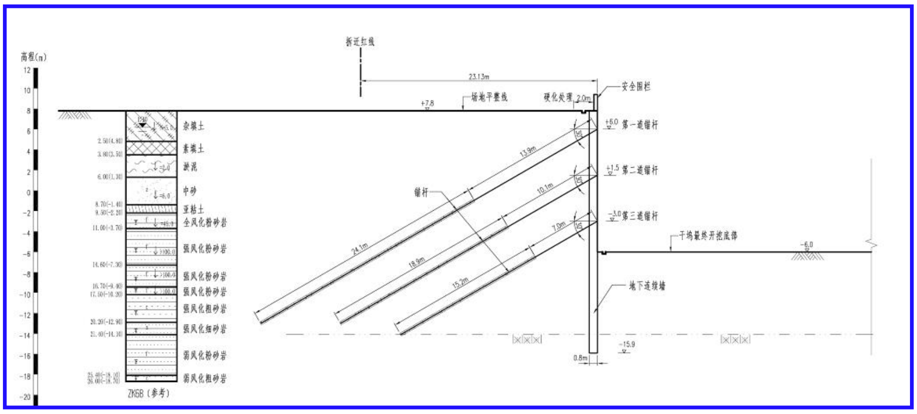

Figure 10: Section B-B

Along the river side of the dry dock, 100mm diameter steel pipe pile wall with 2 layers of strut is proposed. Jet grout is provided at the back of pipe pile wall to prevent the ingress of water into the excavation. The cross section of the support is shown in Figure 11 and the general view of support is shown in Plate 4.

Plate 3: General view of D-wall and anchors

Plate 4: General view of lateral support at river side

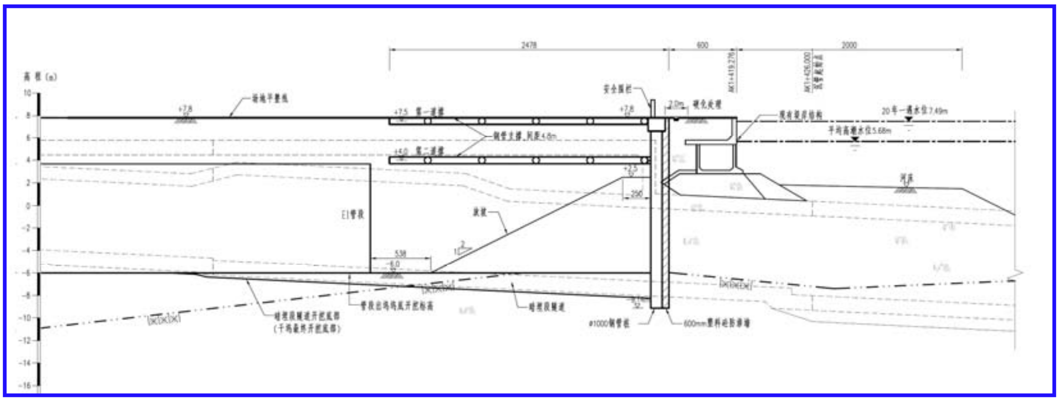

Figure 11: Section C-C

The various stages of the excavation of the dry dock are simulated using PLAXIS 2D. The modeling sequences are compatible with the actual stages associated with excavation, dewatering, installation of anchors, flooding and dewatering of dry dock. The design of lateral support system is carried out according to the design codes and specifications of PRC and Guangdong Province.

Installation of instrumentation and monitoring are carried out to ensure the wall deflections, ground movements and forces in the anchors are all within the allowable limits. The excavation of the dry dock is in progress and the general view of the dry dock after excavation is shown in Plates 5 and 6.

Plate 5: General view of dry dock

Plate 6: Fabrication of tunnel unit inside dry dock

8 CONCLUSIONS

Because of the heavy weight of immersed tunnel units, it is impossible to lift and place them. Therefore a dry dock is constructed for using immersed tube tunnel technique to facilitate the tunnel units being fabricated, floated, sunk and connected in the final location. The arrangement, size and depth of dry dock need to be carefully considered accounting for site geological conditions, amount and size of tunnel units, and river tidal level. The excavation and lateral support system should be designed with due consideration for effects of the subsequent dewatering and flooding. This paper skims over the major deign considerations for the construction of a dry dock.

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Development of 1881 Heritage

Chris Cheung, Alan Lai & P.L. Leung
AECOM Asia Co. Ltd., Hong Kong

ABSTRACT

The design encompasses innovative approaches to meet the project requirements of creating a leisure high end retail environment and merging into an historical building complex.

The innovative design of the integrated building, retaining structure and substructure provided the open space of the piazza. The pioneering of the application of the Directional Drilling method for the pile wall construction provided unyielding support to the historical main building without any adverse effect, resolved the excavation design for the very tight site boundary along Kowloon Park Drive and Canton Road. The different underpinning and lateral support systems also contributed to the preservation of the valuable trees and existing signal tower.

1 INTRODUCTION

The site is located in the heart of the high-end retail and cultural centre of Tsim Sha Tsui, Hong Kong. The heritage complex consists of the Former Marine Police Headquarters, the Signal Tower, the Stable House, the Fire Station and the Accommodation Block. These buildings were constructed between 1881 and 1920, and declared as monuments under the Hong Kong Antiquities and Monuments Ordinance in 1994.

The aim of the developer and government planning department was through restoration, preservation, and conversion to develop the site into a sustainable commercial and tourism-themed development. The project involves tree preservation, alteration and addition to existing heritage buildings in order to create a boutique hotel with high-end retail and restaurants.

These buildings are some of Hong Kong's oldest examples of buildings constructed during the early stage of the British Colony. They have now been revitalized and have re-emerged as a major landmark in Tsim Sha Tsui – "1881 Heritage". It is the largest completed private section conservation and revitalization project in Hong Kong.

There are a lot of constraints including requirement of government on the preservation of the monument, trees, KCRC protective zone and adjoining features. The redevelopment has to take into account the site constraints and achieve the aim of the developer.

1.1 Site Constraints

All the monument structures, except the Fire Station and the Accommodation Block, were located at an elevated platform supported by slopes and retaining walls with an elevation 10 m above the adjoining ground (see Plate 1). To the north of the site is a commercial building No. 1 Peking Road. It is bounded by Kowloon Park Drive, Salisbury Road and Canton Road at the eastern side, southern side and western side respectively which are all heavy traffic road. A lot of trees were located on the original elevated platform and on the slopes. Most of the trees were transplanted and few of them were fell at the beginning of the project. Important trees, such as tree no. T10, T54 and T96, had to be preserved in-situ by means of underpinning or lateral support system to suite the design of the architectural design (see Figure 1). Moreover, percussive piling was not permitted as it would affect the operation of Hong Kong Cultural Centre.

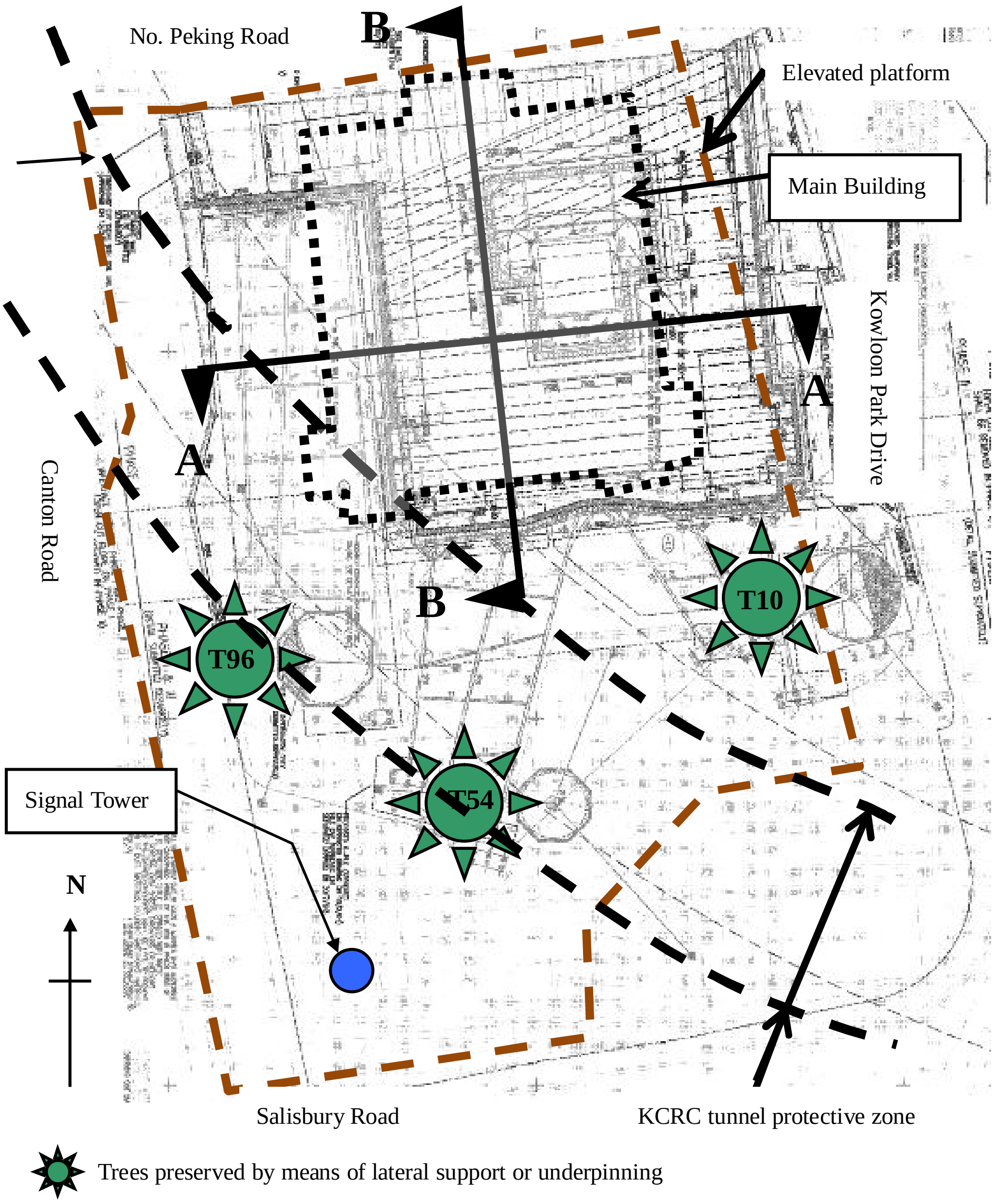


Figure 1: Key features preserved by means of lateral support or underpinning structure

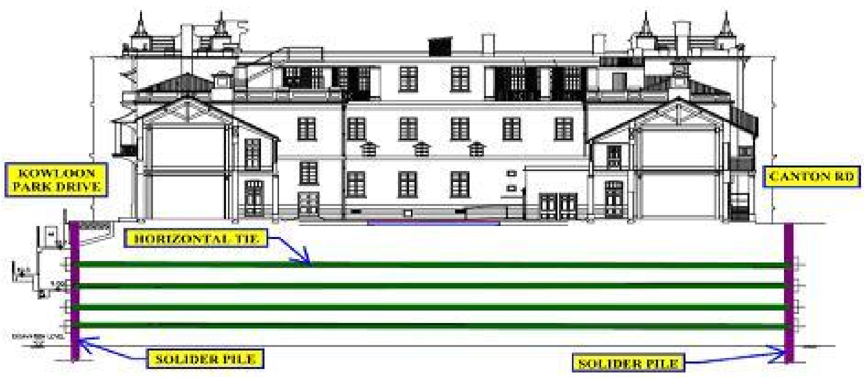


Figure 2: Section A-A of Figure 1

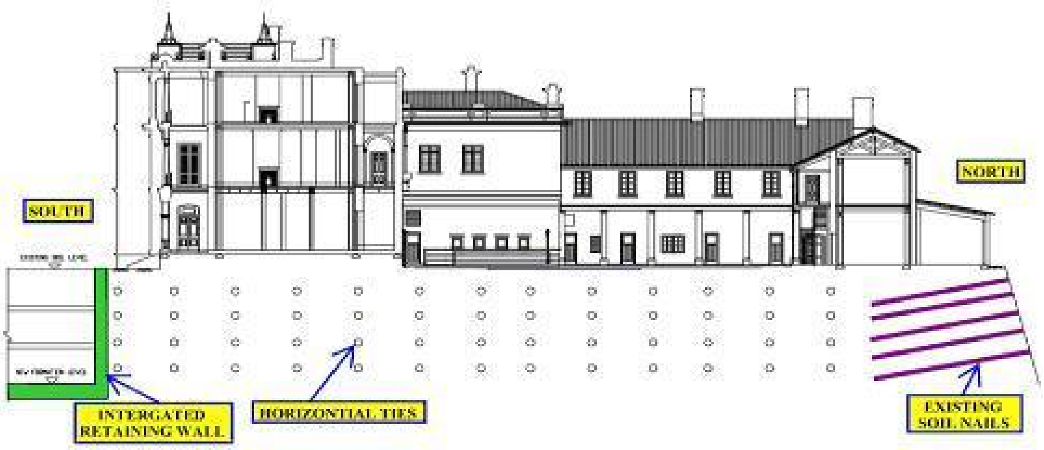


Figure 3: Section B-B of Figure 1

Plate 1: General view of the site before construction works

A new 3-storey reinforced concrete building was constructed around and under the Main Building. To the east of the Main Building, there was only about 4m to 8m between the Kowloon Park Drive and the Edge of the Main Building. To the west of the Main Building, there was KRCR tunnel protection zone where no structure was allowed to be lower than +2.0mPD within the KCRC tunnel protection zone. Excavation and lateral support system had to take into account such site constraints.

1.2 Ground Condition Underneath Main Building and Signal Tower

The ground condition was quite stable within this site. The Main Building and Signal Tower are being supported by shallow foundation resting on Completely Decomposed Granite (CDG) with SPT'N' ranged from 8 to 48. The founding level is about 1.2m below ground. Above the CDG/founding level of footing is a thin layer of fill which was the backfill after the construction of the shallow foundation. Below the CDG is the Completely to Highly Decomposed Granite (C/HDG) with SPT'N' value over 100. Underneath the C/HDG is the Moderately Decomposed Granite (MDG).

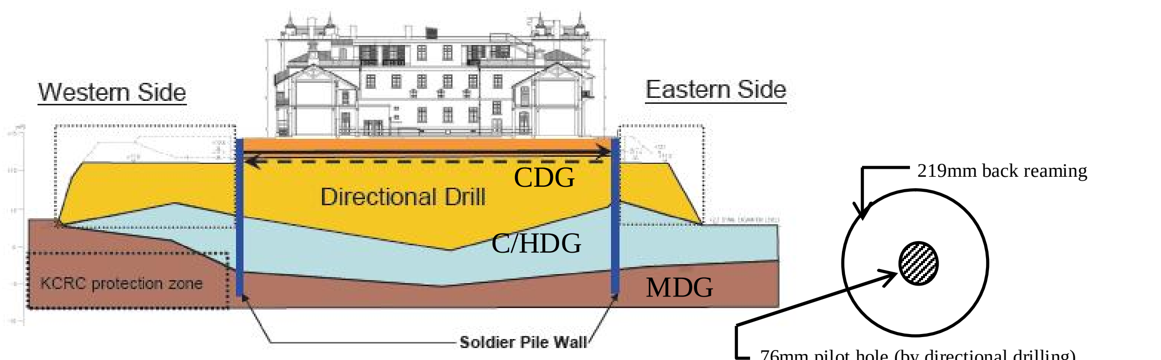
1.3 Horizontal Tie System installed by means of Directional Drilling

Permanent Solider Pile Walls, at spacing 0.7m to 1.3m (internal diameters were 550mm and 730mm respectively) with maximum clear spacing between soldier piles of 400mm and 900mm respectively, with about 55m long horizontal tie back system (diameter was 219mm) were proposed as the lateral support system. The horizontal tie were installed by an innovative method - “Horizontal Directional Drilling (HDD) Method” underneath the Main Building from the west to the east without affecting the building and the underground Heritage Water Tanks at the open courtyard of the building. Directional drilling method was commonly adopted for the underground utility laying. It was adopted to avoid any depression of the drillhole alignment due to the self weight of the drill rod and long distance drilling. It also monitored any deviation in direction of drilling due to the interface between soft and hard materials so that the drilling direction could be adjusted to achieve the target. The mechanism of the directional drilling could limit the deviation of the drill tip to less than 1:292 or even more precise for the whole drilling length so that the horizontal tie could be come out within the limited spacing between soldier piles.

At the beginning, the drilling machines were located at the west of the site (Canton Road side). The opening frame for the hole was installed on both sides of the pile walls. A transducer was installed in front of the drilling rod. Guided pilot hole drilling with a diameter 76mm was installed by Horizontal Directional Drilling (HDD) method. The horizontal tie hole positions was set out on both sides of the pile walls as well as their respective alignment on the ground surface. Bentonite slurry was used as the flushing medium and air/foam for specific horizontal tie holes in order to minimize the impact to the Main Building above. A walk-over drill head tracking system was used to track the position of the drill head during the whole course of the pilot hole drilling. The driller should adjust the direction of the drilling according to the drill head tracker information provided via a remote display so that real time hole steering was possible.

After the pilot hole was formed, the pilot bit was replaced by a back reamer of 235mm diameter on the other side of the cofferdam. 219mm diameter steel casing was fixed and attached to the back reamer via a swivel. The hole was then back reamed to sufficient diameter for the 219mm diameter casing to pass through.

After the drilling reached the destination, back reaming with a diameter of 219mm casing (outer diameter) and a guided wire was then carried out from east to west. Then the steel bars (4T50) were then installed by pulling by the guided wire fixed to the steel bar from west to east and pushing from the eastern side simultaneously until the whole tie was installed. After the steel fixing, grouting was then carried out. Grouting was carried out by means of pumped grouting into the drilled hole with a vent hole higher (about 500mm) than the ties until the grout was come out from the vent hole. The following figure shows the general procedures of the horizontal tie installation.



Legend:

→ 1 Direction of 76mm pilot directional drilling

← - - 2 Direction of 219mm back reaming with

<u>Cross section of Installation of Horizontal Tie</u>

Figure 4: Horizontal tie installation

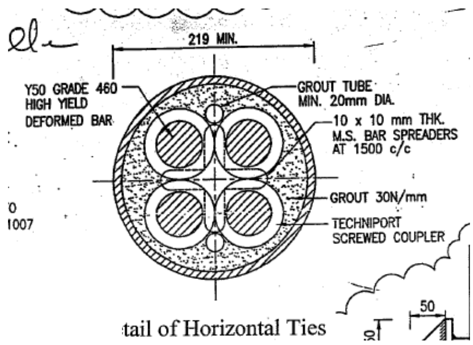


Figure 5: Typical cross section of horizontal tie

For installation of 1st and 2nd top layers, a specialist carried a walk-over guiding equipment transmitting an electromagnetic wave to the transducer in order to monitor the alignment of the drill-rod at the west to east (Kowloon Park Drive side). Skill labours with proper calibration to eliminate the ambient noise to the signal were required in the electromagnetic wave method. For the lower two layers, optical (laser) equipment was installed at the head of drilling rod to monitor the alignment of the rod for each 5m drilling. The reason of adopting the optical method for the lower layers was that the electromagnetic wave would be disturbed by the steel bars of upper layers of horizontal ties. The optical method required a precise setting out of the drilling equipments. Counter check setting out points at the optical drilling equipments were provided and the equipments were carefully protected from any disturbance to ensure the setting out of the drilling equipments were drilled with the right alignment.

Plate 2: Pilot drill (photo taken from western side)

Plate 3: Electromagnetic detector at ground level

The application of horizontal ties was successful, as there was no any sign of distress at the Main Building during the ties installation and excavation. The settlement induced by the excavation at the Main Building was less than 10mm. The most difficult part in the whole drilling process was the drilling met the interface between the soft and hard material (i.e. soil and rock interface). That made the adjustment of the alignment more difficult which required more time to adjust the alignment.

3 nos. of trial tie installations, namely P1, P8 and P18, were carried out prior to the installation of remaining working ties in order to verify the practicality of directional drill, piloting system and whether there would be any adverse effect to the Main Building. The results are summarized as follows.

Table 1: Summary of trial tie installation

	Tie Length (m)	Tolerance and Levels of Start Points (western side)	Tolerance and Levels of End Points (eastern side)	Tolerance	Performance
P1	52.47m	40mm 12.85 mPD	60mm 12.82 mPD	Hor. 1/2624 Vert. 1/1749	Satisfactory
P8	54.44m	60mm 12.7 mPD	50mm 12.86 mPD	Hor. 1/495 Vert. 1/340	Satisfactory
P18	55.45m	10mm 11.48 mPD	10mm 11.67 mPD	Hor. 1/2773 Vert. 1/292	Satisfactory

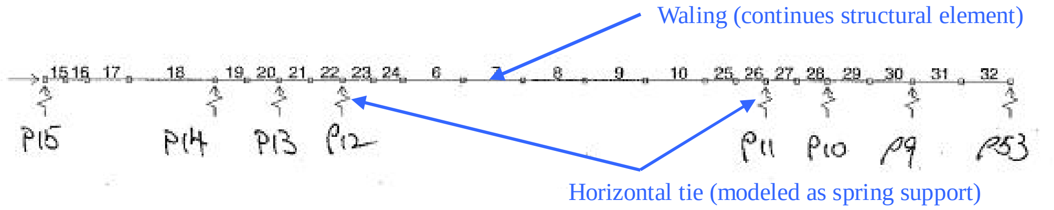
Remarks: The platform ground level is +14.5mPD

No settlement at the building and the ground was observed and measured due to the installation of the horizontal tie.

This system provides a reliable locking system as compare with traditional grouting between soil/rock and grout. It provided a reliable lateral support to this Main Building which was sensitive to settlement. There was not any adverse effect to the Main Building due to the installation of the horizontal tie i.e. installation of horizontal tie by the method as described above in CDG or better was satisfactory.

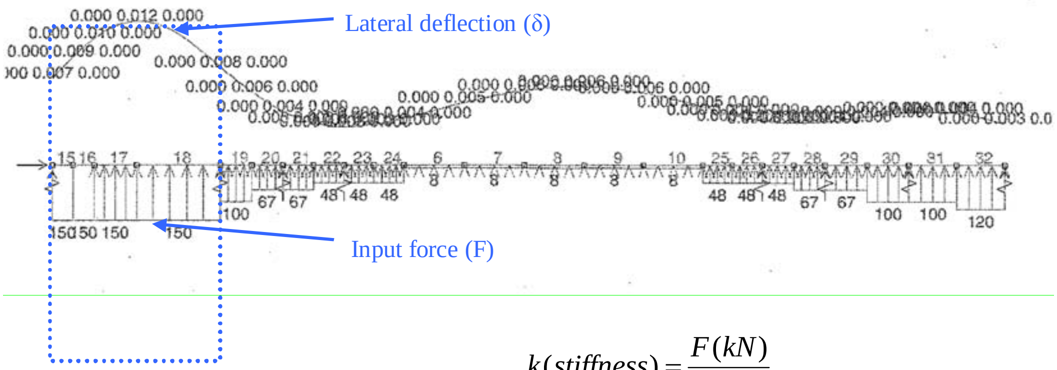
2 ANALYSIS

The prediction of the ground settlement at the monument is crucial, as advised by the structural engineer, the differential settlement (tolerable movement) of the Main Building should not be larger than 1:1,000. The horizontal tie was spaced at about 4.9m c/c. Robust and rigorous analysis was adopted to precisely predict the possible impact to the building such as settlement etc. A strucural model was set up at first with the preliminary size of the waling to obtain the apparent stiffness of the waling beam around the soldier pile wall. This can be achieved by applying a virtual load (F kPa) on the waling and the corresponding lateral deflection (δ) was then obtained. The apparent stiffness (k) of the waling was then obtained by using the formula of k = F/ δ. An example below illustrates the process to obtain the corresponding stiffness.



Remarks:
Spring of the horizontal tie is determined by k=EA/l
L is the effective length of the tie (i.e. half of the overall length of the horizontal tie)
E is the Young's Modulus of steel bars of horizontal tie
A is the cross sectional area of steel bars of horizontal tie

Figure 6: Input for structural analysis (plan view)



$$k(stiffness) = \frac{F(kN)}{\delta(m)}$$

Figure 7: Results of structural analysis

The k value was then input to the 2-Dimension soil-structure interaction model, a finite element model, as the lateral support to the soldier pile wall. Then a more realistic loading on the soldier pile wall and the waling could be obtained. The preliminary size of the waling was then reviewed. Structural model was then updated by using the revised waling and the corresponding stiffness was then obtained. Subsequent analysis was then carried out again until the results were converged.

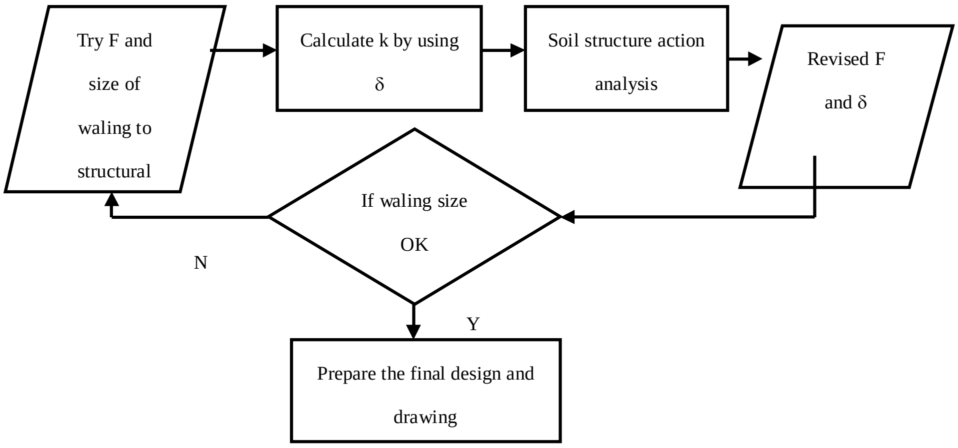


Figure 8: Flow chart for calculating the apparent stiffness

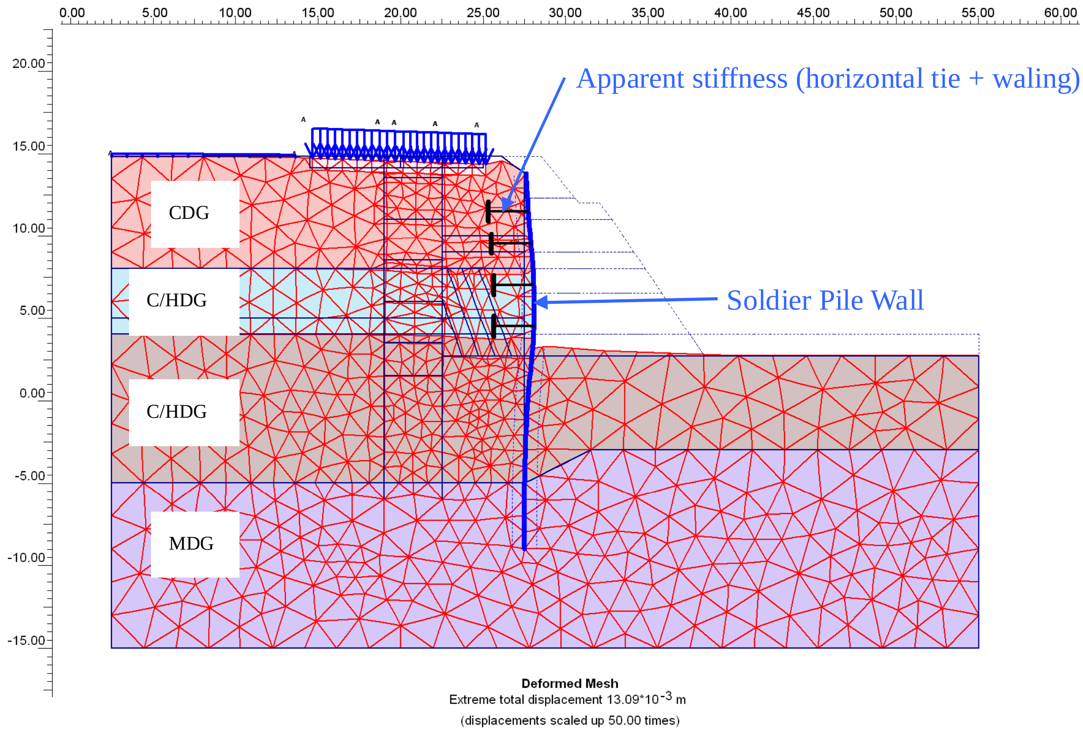
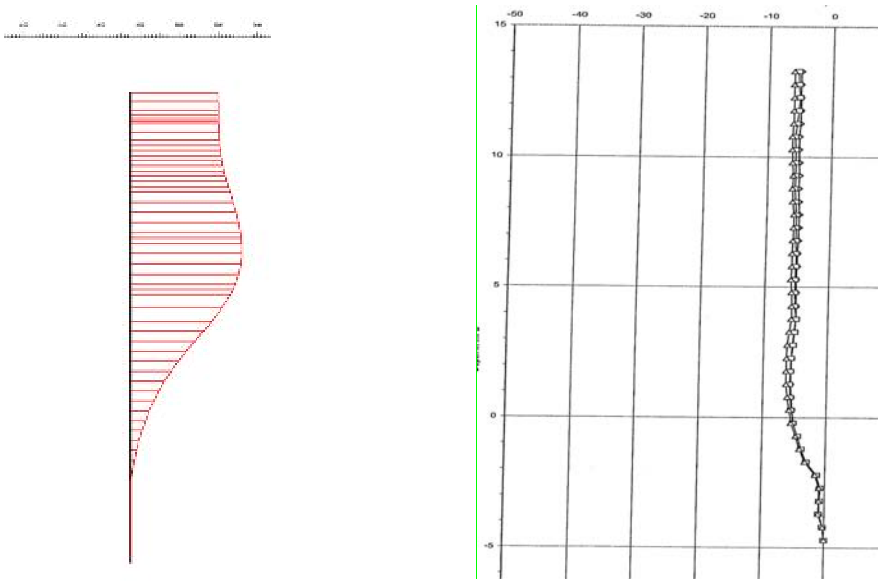


Figure 9: Soil-structural model analysis (by Plaxis)

The predicted and measured soldier pile wall lateral movements are approximate 11.4mm and 7.6mm respectively. It is observed that the lateral movement is within our prediction and the deflected curvature is similar to the prediction.



Predicted lateral movement is 11.4mm   Monitored movement is 7.6mm

Figure 10: Predicted vs monitored movement of soldier pile wall

Table 2: Summary table of using directional drilling against non-directional drilling

	Directional Drilling	Non-Directional Drilling
Construction tolerance	According to the trial installation of the tie, the precision could be more precise than 1:292 in vertical and 1:495 in horizontal.	1:75
Require pilot hole	Yes. 75mm pilot hole with a piloting system is required. There are different piloting systems which should be determined due to different site constraints.	No.
No. of drilling per horizontal tie	Two. First is for the pilot hole (75mm). Second is for the reaming to form the hole with designed diameter	One.
Alignment can be controlled	Yes.	No.
Cost	Higher	Cheaper
Time	Longer. However, it minimizes the time of re-drill the hole due to misalignment	Shorter. However, there would be a high rise of having a hole with mis-alignment. Re-drill the hole may be required.
Application	To be used in area with a precise alignment. E.g. Underground utilities installation, drilling with sensitive feature nearby which is very easy to be disturbed and the alignment does not need to be in straight line.	To be used in area without any strengthen site constraints and there is not any major consequence due to misalignment of the drillhole. Alignment is in straight line (for short distance). Long distance drilling would cause depression of the drilling alignment due to self weight of the casing and drilling rod.
Specialist	Specialist is required.	Only normal skill labour is required.

Table 3: Summary table of horizontal tie against traditional tie back system

	Horizontal Tie	Traditional Tie Back e.g. soil nail, ground anchor
Crash of tie back from east and west side	Very low risk.	Risk is high as alignment is difficult to control for long distance drilling.
Interact with adjacent tie back.	Very low risk	Low risk for short tie. High risk for long tie.
Substantial change of direction due to soft/hard material interface	Deviation can be controlled and mitigated	Cannot be mitigated and misalignment cannot identified.
Performance Verification	Simple system. Only Mechanical fixings or welding at both ends of horizontal tie.	Performance depends on grouting interface between soil, rock and the steel which tests are required to verify the performance of the bonding.

2.1 Supporting system for the Trees and the Signal Tower

Two supporting methods were adopted for preservation of the tree and Signal Tower.

- Underpinning method for T10 and T96
- Steel Cage method for T54 and Signal Tower

2.2 Underpinning method for T10 and T96

Underpinning by means of socket H-piles with horizontal pipe piles and steel girders were used to support the trees T10 and T96. The diameters of them were both 12m. The thickness of the soil mass needed to be supported was 3m. Due to the large span and the heavy weight of soil mass, steel girders with depth of beam over 900mm were adopted.

Soldier piles were installed at first. The installation of the them required high headroom which was about 6m to 8m. The branches of the trees should be carefully protected. Local excavation with lateral support was carried out to allow sufficient headroom for the piling rigs to be operated. 1.5m depth Concrete ring with steel reinforcement was constructed segment by segment by local excavation. Subsequent 1.5m concrete was then constructed after the 1st ring completed by the same method as well until the whole 3m depth concrete ring completed. Reinforcement inside the ring was used to take up the tension load. Driving pit formed by means of local excavation deeper than the ring at one side of the tree ring was then carried out for the installation of the horizontal pipe pile. Hand mining method was adopted for the installation of each steel girder which steel frames were used to temporary support the hand mined tunnel. The headroom of it should be sufficient for the labours to work inside. The girder was then installed and fixed on top of the socket H-piles i.e. trees were supported by the pile. Then excavation was carried out underneath the girder. Lateral bracing to the socket H-piles was then provided to prevent them failure from buckling.

Plate 4: T96

Plate 5: T10

Plate 6: Installation of girder by tunneling method

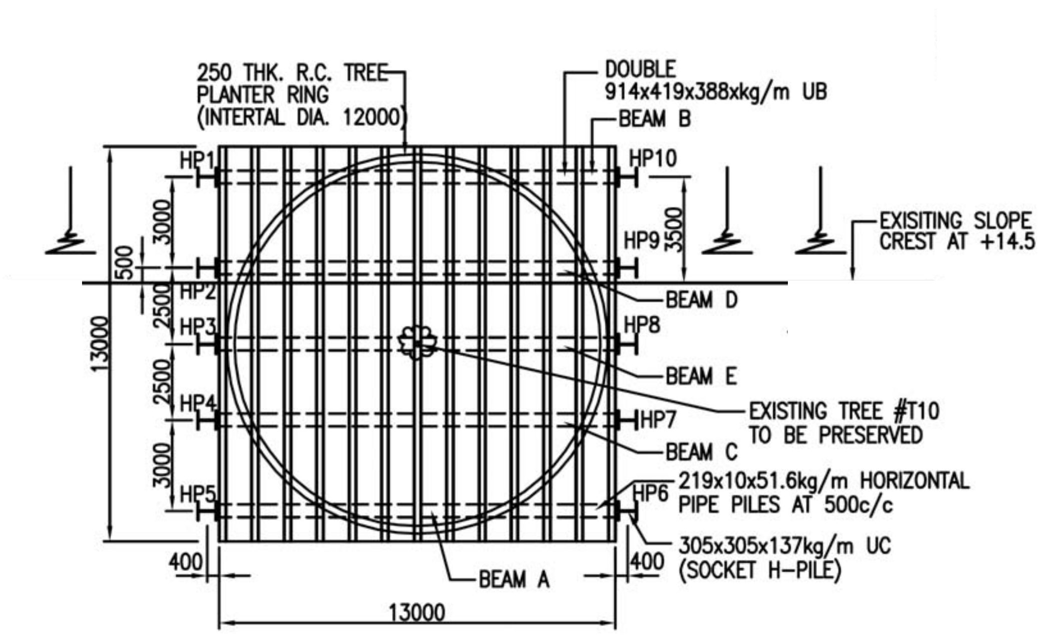


Figure 11: Plan view of underpinning structure

In the design consideration, the dead load of the tree, the soil and the wind load acting on the tree should be considered. Different failure mechanisms were considered as illustrated in the following section. Should the weight of the tree be insufficient to provide the frictional force, there should be shear connector at the bottom of concrete ring to connect with the horizontal pipe pile. Buckling of the socket H-pile should be checked under the maximum horizontal load and the maximum compression load.

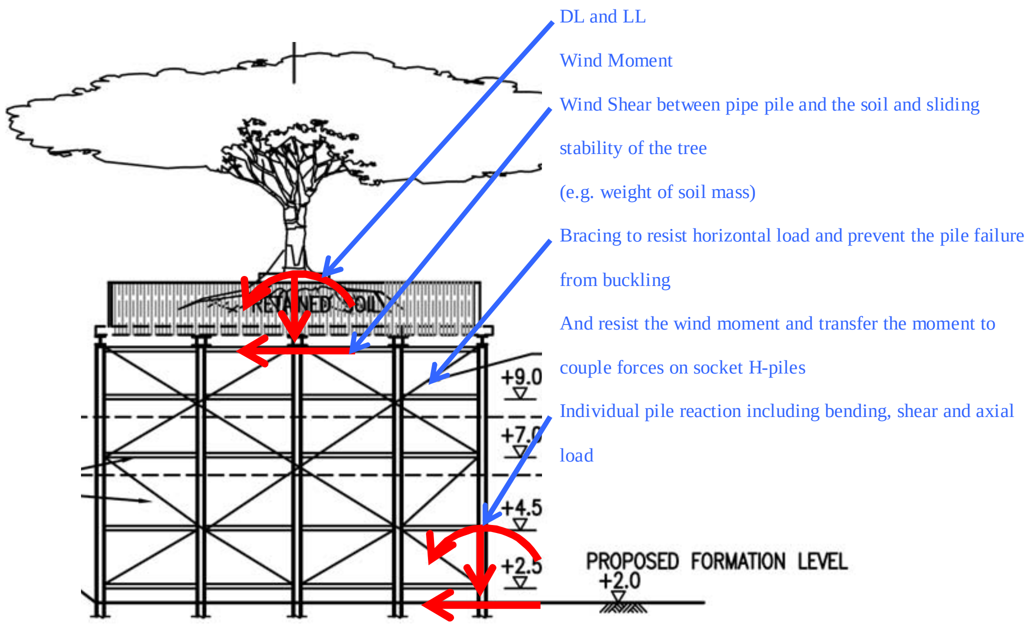


Figure 12: Section showing the design consideration of underpinning

2.3 Steel Cage Method for T54 and Signal Tower

The diameter of the soil mass needed to be supported was 9m which was advised by the Tree Expert. Tree punning around the root ball of the tree was carried out in stages (separated into three circles) which took about 4 months. For tree no. T54, it was located directly above the KCRC tunnel protection zone, therefore, no part of the structure could be lower than +2.0mPD (see Figure 1, Plate 7 and Figure 13). Pipe pile wall with steel lagging plate and waling to form a circular shape steel cage to contain the soil supporting the tree was adopted as the lateral support system. The final excavation was very near to the KCRC protection zone, therefore, special considerations were made.

(a) Waling at the portion near to the excavation level was very closely spaced in order to maintain kick out stability.
(b) The excavation near to the lower portion required grouting prior to excavation. The excavation around the circular column was separated into several portions (see Plate 7 and Figure 13). Footing for the permanent structure was also constructed at stages, i.e. separated into panels, with respect to the excavation sequence at the footing level.
(c) Horizontal load on the tree due to wind load was particularly important which included both overturning, sliding and the complementary shear contributed to the lagging plate and the welding of the steel cage which was to resist the deformation of the steel cage. The condition of soil at the base of the excavation was nearly to Highly Decomposed Granite (HDG). The bearing capacity was sufficient.

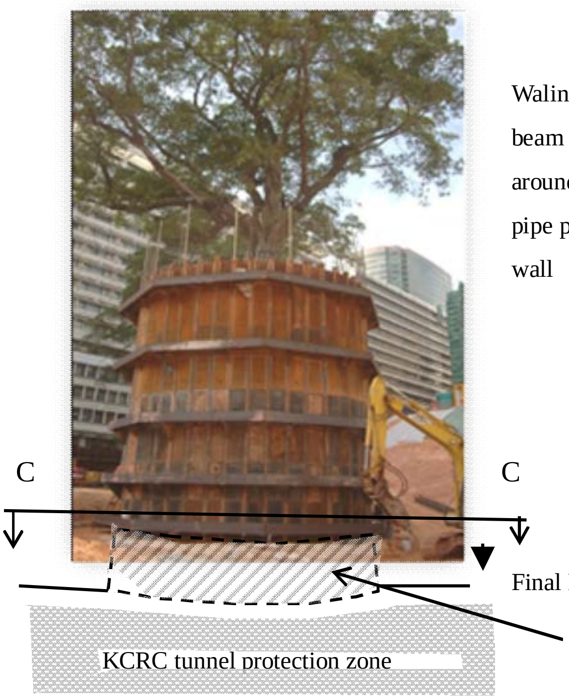


Plate 7: T54 rested on the soil and above the KCRC tunnel protection zone

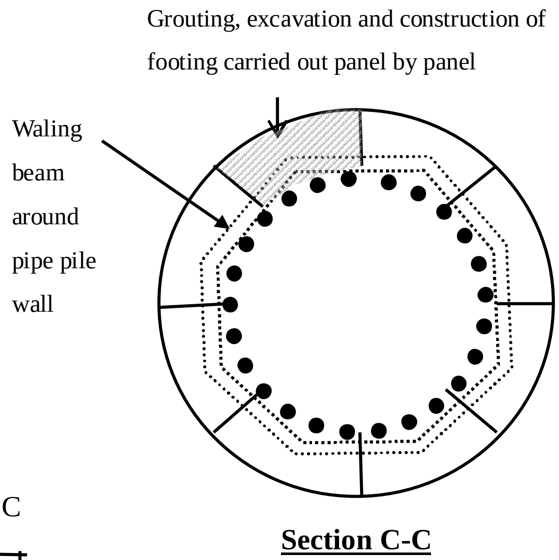


Figure 13: Section C-C of Plate 7

The stiffness of the lateral support of waling beam around the pipe pile wall was obtained by a setting up a structural model. Similar to the horizontal tie analysis, preliminary size of the waling was adopted to obtain the apparent stiffness of the waling beam around the pipe pile wall. This can be achieved by applying a virtual load (H kPa) on the ring and the corresponding lateral deflection (δ) was then obtained. The apparent stiffness (k) of the waling was then obtained by using the formula of $k = H/\delta$. The k value was then input to the 2-Dimension soil structure interaction model, a finite element model, as the lateral support to the pipe pile wall. Then a realistic loading on the pipe pile wall and the waling could be obtained. The preliminary size of the waling was then reviewed. Structural model was then updated by using the revised waling and the corresponding stiffness was then obtained. Subsequent analysis was then carried out again until the results were converged.

2.4 Signal Tower

Similar to tree T54, pipe pile to form a circular support to the Signal Tower was adopted. Signal Tower was much sensitive to settlement than the tree. It was located outside the KCRC tunnel protection zone therefore, the pipe pile could be embedded deeper to provide the kick out stability and minimize the settlement as well. The weight of the Tower and the corresponding wind load acting on the lateral support system was considered. Prior to the installation of pipe pile, a comprehensive ground investigation such as boreholes and trial pits etc. was carried out to investigate the underground conditions and the foundation condition and the extent of foundation as well.

Plate 8: ELS for signal tower

Plate 9: Final preserved signal tower

3 CONCLUSION

Different ELS systems should be adopted by considering the following issues.

1. The site constraints such as working space, any site specific requirement e.g. tunnel reserve zone etc;
2. Ground geology such as soil properties and ground water table;
3. The foundation system and the conditions of nearby structure;
4. The tolerance of structure/utilities nearby against movement, settlement and vibration;
5. Suitability of the equipment to suit site constraints e.g. headroom, access etc.;
6. Particularly issue for the preservation of tree such as the preserved root ball size, can grouting be carried out nearby? etc. should consult with the tree expert;
7. Any special analysis technique is required;
8. Trial installation tie may be required subjected to the conditions of structure/utilities nearby and the geological conditions;
9. Any other special requirement such as environmental requirement;
10. Innovative idea could resolve some site constraints but the availability of the plants/equipment/technique for the execution of ELS works should be considered;

Horizontal Direction Drilling (HDD) method was found satisfactory in drilling through the CDG. The level of accuracy in the alignment was substantially increased by this method. The performance of this method in other ground condition such as fill layer may need to be further investigated.

Further application of the HDD for deep excavation could be as follows:

- Drill any hole for soil nail or tie back support for retaining wall which is very near to the sensitive receiver such as WSD tunnel, utilities, DSD tunnel, MTR structure etc.
- Install the horizontal pipe pile for tunnel construction.

A combination of different ELS systems may be required in some situation such as the tree underpinning structure in this project which hand-mining tunnel, tree ring and lateral support were adopted to support the tree.

Finally, usage of different computer programs (geotechnical and structural) is essential in the analysis in order to have a more realistic model for the excavation and lateral support system. Calibration is particular important. Iteration, if found necessary, will be required. Therefore, the designer should have both structural and geotechnical knowledge in designing any excavation and lateral system.

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Some Design and Construction Aspects of a Deep Excavation Supported by Anchored Diaphragm Wall

A.K.M. Lam, J.W.C. Sze, J.W. Pappin & J.A. Davies
Ove Arup and Partners Hong Kong Limited

ABSTRACT

In Hong Kong, for various reasons it is uncommon to adopt earth anchors as the lateral support to facilitate deep excavations. However, earth anchors have been routinely applied in Asian countries, such as South Korea, where the geological conditions are somewhat similar to Hong Kong. A development project in Seoul required an excavation with a maximum depth of 38m to provide a 7-level basement for car parking and shopping arcade. A 1m thick diaphragm wall with multi-layer retrievable single bore multi-point (SBMP) earth anchors was adopted to provide an unconstrained working space to accelerate the excavation rate which was the critical activity in the construction programme. The majority of the basement was to be built bottom-up with the development founded on competent rock. With the observations from the instrumentation data, this paper discusses the design and construction issues of the anchors in particular the limitations of Pesudo-Finite Element (FE) programs in simulating the earth anchor supports. The anchor installation practices and some associated issues are also be highlighted.

1 INTRODUCTION

A 38m deep 7-level basement has recently been excavated in Yeouido, Seoul, South Korea using diaphragm wall supported by retrievable earth anchors over a site area of approximately 40,000m^2, as shown in Figure 1. The site is to be developed into a multi-complex with commercial office towers, a hotel, a podium and a basement for car-parking and shopping. The site is located towards the north-eastern side of Yeouido Island, which is a reclaimed area from its original flood plain and is bounded by roads on four sides. LG twin towers and Gong Jak Apartments are located to the northeast of the site. The Seoul Subway Line 5 runs along the southeast boundary of the site. Subway ventilation shafts are located at the southern and eastern corners of the site.

2 GEOLOGICAL CONDITIONS

Yeouido Island lays in the valley of the Han River which is incised into the Gneiss and Granite, the predominant bedrock in Seoul. The valley has been over deepened and is now infilled with superficial Alluvial Deposits. The geology is fairly consistent across the island. The typical geological profile is around 8-10m Fill overlying a thick layer of Alluvium sand and gravel, which is then underlain by Weathered Gneiss Rock. Solid Gneiss Rock is encountered at a depth of between 22m below Ground Level (bGL) at the south-eastern side of the site and 38m bGL towards the north western edge of the site. The existing ground level is at Elevation Level (EL)+13m. The bedrock contains weaker zones resulting from faulting and folding. Weak graphite beds were noted in the excavation within the Gneiss appears to be associated with the faults and shear zones.

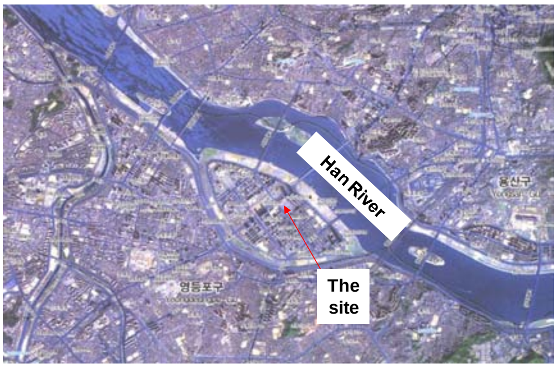

Figure 1: Site location plan

The normal groundwater level within the site is generally at around EL+2 to EL-1.5m, which is close to the typical Han River's level at EL+1m. Under the construction stage, a design groundwater table was taken as EL+2m to cater for possible risk of flooding of the Han River and a design groundwater table of EL+7m was adopted for the permanent basement design.

3 EXCAVATION DESIGN

A 1m thick diaphragm wall with panel plan length varying from 2.6m to 7m was used as the temporary retaining structure as well as the permanent basement wall. A combination of both top-down and bottom-up construction methods was adopted. About 80% of site area was constructed by bottom-up construction method using retrievable earth anchors as lateral support. A maximum of 16 rows of anchors were proposed at typical spacing of 2 to 3m vertically and 1.5 to 1.8m horizontally. The earth anchors consisted of 6 and 8 strands and had working capacities of 66 and 88 tons respectively. Pre-stressing to about 75% of the working capacity was applied.

Since the required final formation level over most of the site area was in good quality rock, the majority of the diaphragm wall panels were designed to be terminated at the top of soft rock (i.e. Unconfined Compressive Strength ≥ 10MPa). A near vertical rock cut was then formed beneath the wall toe to the final formation level.

Where the diaphragm wall terminated above formation level, shear pins in the form of steel H-piles at typical spacing of 1.5m to 1.8m centers were installed to maintain the toe stability of the diaphragm wall prior to the installation of the lowest level of anchors. The shear pins were also designed to share the vertical force component resulting from the ground anchors. A typical section is shown in Figure 2.

Towards the south-eastern end of the site, the existing subway tunnel is aligned 9m beyond the site boundary. The use of earth anchor was prohibited in this area and a top-down construction method with permanent basement slab as lateral support to the diaphragm wall was adopted. This also allowed the hotel structure to be built concurrently with the basement excavation. Temporary steel stanchions supported by reverse circular drilling (RCD) pile were adopted to support the structure in this area. Due to different construction sequence with the bottom-up excavation, a temporary soldier pile wall supported with non-retrievable (fixed) anchors was constructed as a separator as shown in Figure 3.

The diaphragm wall supported by earth anchors was modelled by pseudo-FE program Oasys FREW to determine the bending moment, shear force and deflection of the diaphragm wall under the staged excavation.

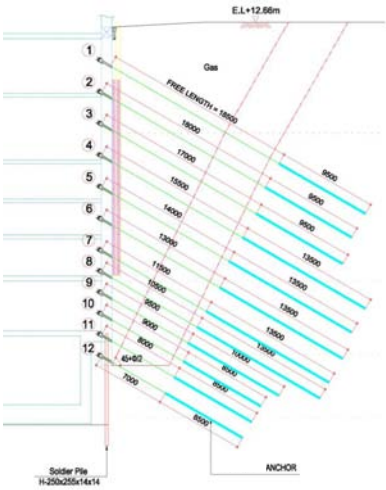

Figure 2: Typical section of anchored diaphragm wall

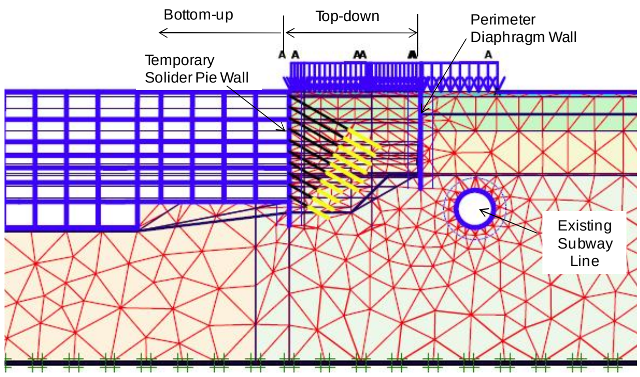

Figure 3: Temporary wall and example of PLAXIS model

The 2D FE program PLAXIS and Oasys SAFE were adopted to study the effects of excavation on existing subway tunnel and ground deformations. Figure 3 illustrates an example of a PLAXIS model to simulate the excavation effects on the adjacent subway tunnel.

4 CONSTRUCTION WORKS

The excavation works commenced in January 2008. The rock surface was exposed towards the northwestern end in May 2008 and the final excavation level was first reached in November 2008. The excavation for the bottom-up area was completed in June 2009 while the excavation at the top-down area is still in progress at the time of preparation of this Paper. Plates 1 and 2 show the overview of the site and a close up at the top-down area respectively.

Plate 1: General view of the site (Oct 2008)

Plate 2: Top-down area (July 2009)

4.1 Diaphragm Wall Construction

The diaphragm wall panels consisted of primary and secondary panels. Predrill holes were sunk to 5m below the tentative toe of diaphragm wall for each primary panel to determine the founding level of individual panel. The wall panel was excavated by mechanical grab and followed by a trench cutter. The stability of the open trench was maintained by bentonite slurry which was re-circulated through the high power mud pump mounted near the cutter. The cuttings were brought to the ground together with the slurry and filtered out via desander unit. The cutter was equipped with built-in inclinometers and was able to monitor the biaxial verticality. Adjustment could be made through the steering plates at different levels of the cutter frame. The observation on the wall surface had confirmed a good workmanship on the verticality control. Cutter joints were formed between primary and secondary panels to ensure water tightness. All panels were founded on soft rock with a Total Core Recovery (TCR) of not less than 85% and Rock Quality Designation (RQD) of not less than 10. Finally, the shear pins were then installed through the 400mm diameter reservation tubes in diaphragm wall panels followed by tremie grouting.

4.2 Earth Anchor Installation

Temporary earth anchors are widely used in South Korea to provide support for basement excavation walls. The anchors are generally classified into tensile type anchor or compressive type anchor. The pre-stress in the tensile anchor may generate tension cracks in the fixed length grout and creep due to load concentration near the front end of the fixed body, which greatly reduce the load in anchor. The project adopted compressive type anchor, which consisted of polyethylene (PE) coated strands generating compressive force on the grout by fastening the strands to 3 or 4 fixed bodies. The loss of pre-stress due to creep is small compared to that of the tensile type anchor. The type of anchor adopted in this project uses multi-fixed bodies which allow the anchor load to be distributed more uniformly over the full length of the grouted body.

The strands of the retrievable anchor could be removed after releasing the anchor head. The SPEED anchor was used by the Contractor, which could be removed by striking the strands with a tailor-made hammer to displace 2 out of the 7 steel wires such that the thrust spring in the load-bearing body could be pushed backward to release the wedge gripping the remaining wires. Figure 4 illustrates the removal mechanism of a SPEED anchor.

Anchors were installed in a 125mm diameter drill holes with full length temporary casing support. On reaching the specified length, the grouting was conducted in 2 stages. Stage 1 grouting was carried out to fill up the casing with cement grout through the tremie hose followed by tendon installation. After the tendon installation, the casing along the fixed length was withdrawn and Stage 2 grouting was performed by injecting cement grout through the casing with grout pressure up to 5 bars. The remaining casing was then withdrawn and a sleeve hole was plugged by a packer filled with grout to stop the ingress of groundwater. The installation process takes about 2 hours. Following installation, acceptance test was carried out on each anchor and the anchor load was locked off following the design requirement.

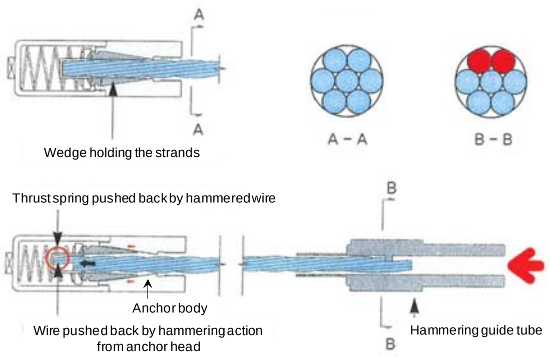

Figure 4: Mechanism of strands removal from anchor body

Plate 3: Ingress of groundwater during anchor installation

The Contractor started with conventional anchor drilling method which caused significant seepage through the reservation pipe at the diaphragm wall, see Plate 3. The rate of groundwater inflow was roughly measured to be 4 to 5 litre/s with water depth of about 3 to 4m outside the site. This would inevitably draw the groundwater down outside the site. Improvement using rubber rings around the casing was introduced to limit the amount of seepage. Nevertheless, the groundwater drawdown still occurred at nearby buildings but was measured to be within an acceptable limit.

4.3 Rock Excavation

Open blasting with surface protective mat was applied to facilitate the rock excavation. Blasting zones were established based on the amount of explosive and the distance from the blasting point. The adopted charge per delay ranged from 0.18kg to 3.0kg.

Trial blasts with different blasting patterns and explosive charges were carried out and the maximum vibration, measured at a distance of approximately 10m from the blasting point and immediately in front of the diaphragm wall, was about 10mm/s Peak Particle Velocity (PPV), which was considered acceptable. A blasting method namely New Pre-spitting (NPS) with line drilling and charged with small amount of explosive near the diaphragm wall was introduced to further limit the propagation of vibration when blasting was carried out closer to the wall.

As the blasting was only carried out during lunch hours, the nuisance to the surrounding buildings compared with using day-long hydraulic breakers was substantially reduced.

4.4 Rock Face Mapping

Based on the borehole data, several high risk areas were identified with potential persistent daylighting discontinuities or geological weakness zones such as faulting, which were considered to be the most critical failure mode for the overall stability of the retaining wall system. Well planned inspection and mapping of the rock faces at and beyond the high risk areas by a competent engineering geologist at the construction stage was therefore required. Any adverse feature was treated before proceeding with excavation works at the next panel of rock face. This approach looked familiar in Hong Kong but rarely implemented in building projects in Korea.

A guideline as shown in the table below was established using Slope Mass Rating (SMR) and Rock Mass Rating (RMR) system to estimate the scale of the mitigation measures required for the vertical rock cut. An example of rock face mapping and the associated stabilization works are illustrated in Figure 5.

Table 1: Table of SMR and RMR values with different types of stabilization works

SMR	*RMR*	*Type of Stabilization Works*
81 - 100	-	Rock dowels are not required with no sign of adverse geological feature nearby
61 - 80	-	Local rock dowels to be required
41 - 60	-	Patterned rock dowels to be required
11 - 40	≥41 with low persistent adverse joint (< 3-4m)	Patterned rock dowels to be required (Pattern rock anchors to be required if persistence of adverse joint is in doubt)
	< 41 or ≥41 with high persistent adverse joint (> 3-4m)	Patterned rock anchors to be required

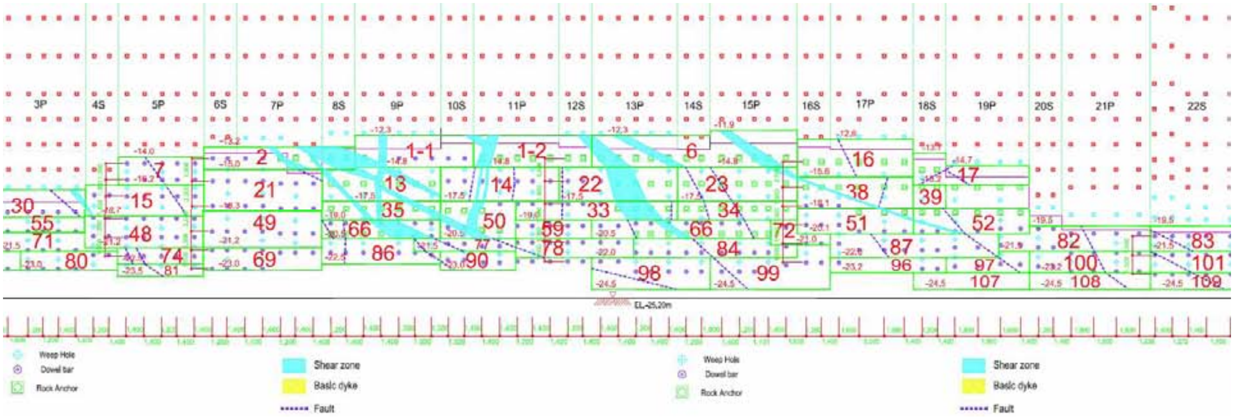

Figure 5: Example of rock face mapping and associated stabilization works.

5 GEOTECHNICAL MONITORING

An extensive monitoring scheme was implemented to monitor the ground movement, wall displacement, building settlement and tilting, vibration, noise, groundwater table and anchor loads during the construction. The proposed monitoring system included inclinometers both inside and outside the diaphragm wall, ground settlement markers, building settlement markers, tiltmeters, load cells at anchor head and standpipes/piezometers. Some of the inclinometers and piezometers were equipped with automatic sensors to give real-time monitoring data.

It is worth noting that the deflection of anchored wall was much lower than other forms of lateral support, such as steel shoring. The main reason is due to the pre-stressing force built into the closely spaced anchors which is able to control the wall deformation in staged excavation effectively. The maximum wall deflection recorded when the final excavation was reached was in the order of 30 to 35mm, which is about 0.1% of the overall retained height, or 0.15% of the retaining height in soil.

6 OBSERVATIONS

6.1 Creep of Earth Anchors

The installation of anchors commenced in early 2008. Most of the anchors have been installed for more than 18 months at the time of preparation of this Paper. The anchor loads as measured by the load cells were generally stable (i.e. greater than 90% of the initial pre-stress) over the past year. Creeping of anchors was not obvious but re-distribution of loads could be observed at some panels, as indicated from different load cells installed at different levels.

6.2 Earth Anchor Modelling

The wall deflection predicted by PLAXIS appears to be close to the actual site monitoring records while a smaller deflection is predicted by FREW. However, the anchor loads predicted by PLAXIS and FREW are comparable with the actual site measurement. The FREW model showed that the action of pre-stressing moves the wall back more than in PLAXIS, which is probably because of the forward movement at the fixed anchor length in PLAXIS. The stiffnesses of the anchor in the two programs are similar, and in FREW further displacement is proportional to change in force (i.e. elastic spring). But this is not the same in PLAXIS, because of displacement of the fixed anchor length. It is also noted that greater load re-distribution is observed in FREW than in PLAXIS. This may be explained by the fact that the fixed anchor lengths do not move in FREW effectively makes the anchors stiffer leading to greater load re-distribution. Another possible reason for more load re-distribution in FREW is that the behaviour is fully elastic until the limiting earth pressures are reached. In finite element, although each element is elastic until it reaches its strength limit, the soil body as a whole tends to become plastic more gradually, effectively giving a softer behaviour when approaching the limiting earth pressures. It is concluded that for relatively short anchors, 2D finite element models such as PLAXIS are more reliable than simplified programs such as FREW.

6.3 Failure of Earth Anchor

Occasional incidents of failed strands shooting out from the anchor head plate were observed on site, see Plate 4. Based on the site observation, it was suspected that the wedge at the head of the anchor failed allowing a sudden release of the strain energy in the strands. The energy wave would then travel down the strand to the anchorage at the tip of the anchor. The energy wave is effectively replicating the methodology for releasing the strands on de-stressing whereby the strand is "hammered" to release the strand for removal.

Plate 4: Close-up view of shooting out of individual strands

In releasing the strand the residual energy in the system would then have allowed the failed strand to move out of the anchor hole as the stress wave reverses along the strand. The fact that the central wire in the strand normally appears to have moved out further than the main body of the strand is due to the central wire being untwisted and therefore easier to displace. What promotes the wedge failure at the head of the anchor is not clear but strand failure suggests a failure rate of about 4 per month to give a rate of about 0.1% per month (there are approximately 4,000 anchors). It should be noted that some failure of anchors is to be expected and allowance for this was made in the design and the failure rate as observed was not critical.

6.4 Construction Supervision

It is a normal practice in South Korea that the site supervision team who is statutory responsible for the excavation is employed by the Client. The drawback of this arrangement is that the supervision team has limited knowledge about the design and may follow their normal basic rules that are commonly applied to other projects to carry out the supervision work. In fact, for such a project with high complexity in the design, Arup carried out periodic site visits during the construction stage to act as a third party review on the quality, documentation and geotechnical monitoring and to ensure that the design assumptions and considerations were well understood by the construction and supervision teams.

7 CONCLUSION

This is a successful example of using a 1m thick diaphragm wall supported by multi-layer retrievable anchors to facilitate a 38m deep excavation. Controlled blasting is demonstrated to be productive for rock excavation inside the basement with acceptable vibration and minimum nuisance to the surroundings. The approach of rock face mapping of the temporary rock cut was successfully implemented to secure the stability of the temporary rock cut while providing cost-effective stabilization solutions. In addition, the periodic visits by the design team have minimized the construction risk due to lack of understanding of design principles by the construction and supervision teams.

ACKNOWLEDGEMENT

Gratitude is given to the Client for their permission in publishing this Paper.

Constructability and Safety Perspectives in Design of a Deep Basement Excavation in the Urban District of Tsim Sha Tsui

Gavin S.H. Toh
Lambeth Associates Limited

ABSTRACT

This paper presents the design of a challenging deep excavation project along Nathan Road in the urban district of Tsim Sha Tsui. The approval of deep excavation design has often focused on the stability of the temporary cofferdam with lateral supporting system and on the ground movement impact to adjacent existing ground and structure to ensure public safety. In this project the design adopted an approach where constructability and safety are addressed together with the geotechnical challenges such as ground condition, design parameters and numerical modeling. The important of building damage control mechanism from instrumentation, design verification to works control and risk assessment management system are emphasized.

1 INTRODUCTION

1.1 Site Location and Adjacent Sensitive Structures / Utilities

The old Tung Ying Building located in the urban and busy district of Tsim Sha Tsui, was redeveloped and the new commercial building is known as "The ONE." The redevelopment includes a 5-level basement that required excavation up to 30m deep.

The site is approximately 110m in length and 26m wide. To the east of the site is the existing Carnarvon Road, to the south is Granville Road and to the West is Nathan Road. At the North are the existing Mira Hong Kong hotel (formerly known as Hotel Miramar) where its building line lies along the site boundary, and the existing Champagne Court that is about 3m from the site boundary. Site layout plan is shown in Figure 1.

Similar to the common scenario in Hong Kong, utilities such as fresh water mains, storm water main and sewer drain are located along the roads surrounding the site. Furthermore, the site lies partially within the MTRC protection zone. The existing operating MTRC tunnel about 20m below ground beneath Nathan Road has a minimum clear distance of only 3.8m from the site boundary. The MTR track level is at approximately –13.0mPD (i.e. about 23m below ground).

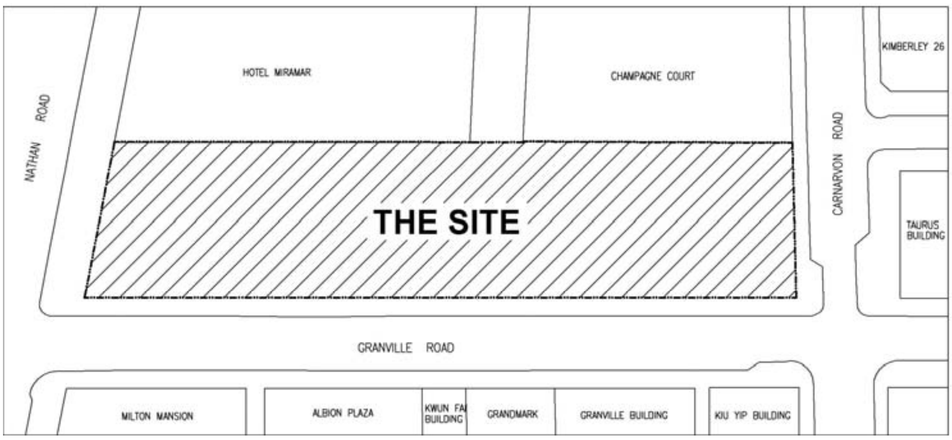

Figure 1: Site location plan

1.2 Initial Design of the Excavation and Lateral Support

An initial Excavation and Lateral Support (ELS) design was submitted to the Buildings Department (BD) and approval was obtained in order to expedite the consent application of the excavation works when the construction work was awarded. The design used a pipe pile wall to retain the surrounding ground during excavation. The pipe pile wall comprised 610mm diameter pipes at 800mm centre-to-centre with an internal UC steel section. The toe level of the wall required at least 5.2m below final excavation level. At the location where rockhead is high the maximum rock drilling required for installing the pipes was 20m.

Twelve layers of strutting were allowed in the ELS design to provide lateral support to the pipe pile wall cofferdam. The spacing between strut levels ranged from 1.75m to 3.0m. The strutting design consisted of single Universal Column (UC) steel section with diagonal struts and king posts. Short steel section between pipes and waling at each strut layer was designed to allow construction of the basement wall prior to the removal of the struts.

The sequence of construction included staged excavation with struts installation until the final excavation level, and staged removal of struts where at each stage the struts would be removed after the basement floor slab at a higher level was to be completed.

2 CONSTRUCTABILITY AND SAFETY

In an urban society such as Hong Kong, it is not uncommon to see that underground structures such as building basements and tunnels for mass transportation are going deeper into the ground. In design of deep excavation, apart from addressing the geotechnical challenges, public and construction safety are important considerations. Ground movement induced from deep excavation is assessed in design to minimize any adverse impact to the surrounding condition to ensure public safety. However, construction safety is often left to the contractor to deal with according to their method of construction.

Safety by design has been an important reposition the construction industry is taking. Safety in construction is often presumed to be the Personal Protective Equipment (PPE) that workers is needed to put on and is personally responsible. However, it should be noted that PPE is the last line of defends concerning safety in construction. Safety in construction begins at engineer's "drawing board". Hazards in deep excavation work can be designed out rather than left to the contractor to deal with in their risk assessment and management.

In the Tung Ying redevelopment project, the deep excavation design was re-examined with the perspective of constructability and construction safety without compromising public safety. A few key aspects were addressed in the re-design:

- Headroom for excavator between struts layer. The number of struts layers were reduced to five layers providing 4m to 5m of gap between struts levels.
- Strutting layout to provide sufficient area for lifting of struts and spoil removal. Lifting impact loads are considered and twin Universal Beam (UB) steel section were used in the design to provide robustness of the strutting system. Sufficient bracing members and a fully welded connection were incorporated.
- Pre-fabrication of strutting system. Excavation footprint covers almost the entire site and storage of steel sections was limited. A semi-truss portion of the struts was designed and pre-fabricated prior to delivery to the site. Welding works were therefore reduced.
- Revised sequence of struts removal. The re-design allowed for removal of struts after completion of the basement slab at the lower levels. Short steel sections between waling and the pipe piles were therefore not necessary. This greatly reduced the amount of welding and steel installation works under the deep excavation environment.
- Access and egress in the deep excavation to eliminate hazards of falling from height. Waling and strut members were often used for workers access and egress. Detailed design for safety steel barrier, toe board and stairs were carried out and installation of these features was properly supervised. Special access to dewatering well locations and pre-loading point was designed and provided.

In view of constructability and safety, the ELS were re-designed starting with ground condition review to design parameters, design analyses and suitable strutting system. It was also necessary to enhance the building damage control system to ensure safety both to the public and workers.

The layout plan showing the strutting system arrangement and section showing the strutting levels and geological conditions are shown in Figure 2 and 3 respectively.

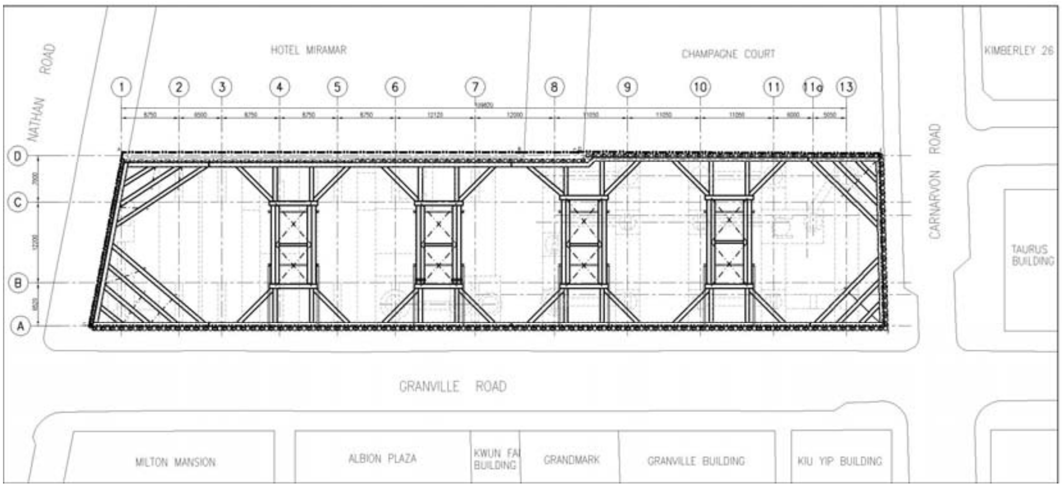

Figure 2: Strutting arrangement layout plan

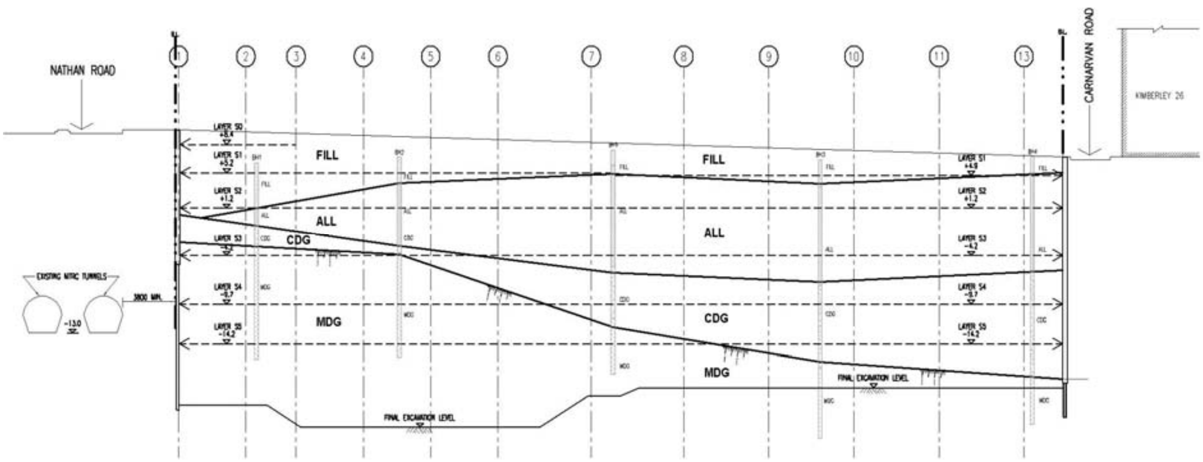

Figure 3: Geological section and strutting levels

3 DESIGN CONSIDERATIONS

3.1 Ground Condition

There were 10 numbers of vertical boreholes sunk around the site, typically to a depth of 20-35m below ground level. Ground strata comprising approximately 2 to 5m of sand fill, underlain by 2 to 11m of Alluvium. Beneath the Alluvium layer is the Grade V completed decomposed granite (CDG). A zone of Grade V/IV granite with SPT N value greater than 200 was presence above the bedrock. Bedrock of moderately strong to strong granite was encountered at approximately 10m below ground at the Nathan Road side and dipped to about 30m below ground at the Carnarvon Road side. Ground level is around +9.5mPD at Nathan Road and sloping down to +6.5mPD at Carnarvon Road.

Ground water level (GWL) measured from standpipe / piezometer was around 3.0m to 3.5m below ground. A 2.0m rise of GWL was assumed in the ELS design. In the seepage analysis to determine settlement due to groundwater drawdown, the lowest GWL was considered taking into account of the effective stress history of the soil.

3.2 Design Parameters

The design soil parameters for the Fill, Alluvium and CDG were determined from the laboratory tri-axial tests. In the revised ELS design, the design parameters followed the assumptions in the initial approved ELS design. However, for the CDG it was divided into two classes, one with SPT N<200 and the other with SPT N>200. From the tri-axial tests, the parameters of CDG with N<200 generally follow the assumptions in the initial design. A higher friction angle and cohesion parameters were deduced for the CDG with N>200.

Chan (2003) has published, based on back analyses of deep excavation works of past projects, the values of Young

Modulus (E) that can be directly correlated to SPT N-values. The reported results are summarized in Table 1 below.

Table 1: Correlation of E and SPT N-value (Chan 2003)

Project	Material	E = factor x N
Chater Station, Central	Fill & Marine Deposit CDG CDG/HDG	E = 1.5N E = 2.0N E = 4.0N
HSBC Headquarter, Central	Fill & Alluvium CDG	E = 1.5N E = 2.0N
Evergreen Hotel, Wanchai	Fill & Alluvial Deposit CDG	E = 1.5N E = 3.0N
Dragon Centre, Sham Shui Po	Fill & Alluvium CDG	E = 1.0N E = 1.5 – 2.0N
Festival Walk, Kowloon Tong	CDG	E = 2.0N
Hong Kong Station, Central	Fill/Alluvium/CDG	E = 4.0N

Chan (2003) reported that an E/N value of 1.5 (MPa) for Fill, Alluvium and Marine Deposit and an E/N value of 2 (MPa) for CDG are considered a reasonable and conservative estimation of lateral displacement of deep excavation. In the ELS design for this project, the recommendation by Chan (2003) was adopted for CDG where E/N value is 2.0 (MPa). For Fill/Alluvium the E/N value of 1.0 (MPa) was considered.

3.3 Design Analyses

In the ELS design, the temporary cofferdam wall consisted of 610mm diameter pipe pile with internal steel core UC section similar to the initial design, except the spacing was revised to 710mm centre-to-centre. At the location where rock head is high, 406mm diameter pipe pile was used. In addition, a higher steel grade compared to the initial design was adopted. To minimize over-breaking the rock during installation of the pipes with the ODEX system, the pipe pile was designed to socket 500mm into rock. The internal steel core that required a smaller pre-bored hole extended into the rock (as shear pin) to sufficient socket length to provide additional shear for toe stability. In area where rock head was higher than the final excavation level, the shear pin would extend to at least 500mm below the final excavation level. Along the Mira Hong Kong Hotel and Champagne Court the pipe piles were installed using a water-flushed technique (a closed loop system) instead of the conventional air-flushing technique to further reduce ground disturbance.

Grout curtain was formed to provide water cut-off to the cofferdam wall. The grout curtain extended 1m into rock to improve water cut-off at top portion of the bedrock that would be highly fractures. Prior to the commencement of excavation, pumping test was carried out to verify the design assumption and the integrity of the grout curtain.

In the design, five layers of struts were adopted to provide the required lateral support and stability to the excavation cofferdam. At the Nathan Road side where ground level is higher, an additional layer of strut was needed. Higher strut stiffness, compared to the initial design, was used and preloading to predetermined forces was incorporated in the design to control ground movement.

Excavation was carried out in stages with installation of the five layers of struts. At the strut removal stage, the design allow struts removal after the completion of the basement slab at the lower level. Hence, the short steel section between the pipes and waling, which was adopted in the initial design, was therefore not required. Without the short steel section, construction of the basement wall and waterproofing was easier without obstruction of the struts. Furthermore, the method of struts removal became easier and simple that greatly enhanced construction safety.

In the design analyses of the ELS, six critical design sections were chosen based on variations in the ground strata and excavation levels around the perimeter of the cofferdam wall. Details of the adjacent buildings foundation such as the Mira Hong Kong Hotel that is founded on pad footings and caissons, and the Champagne Court founded on 500mm diameter Vibro-piles were incorporated in the numerical analyses. Foundations of the other buildings across the Granville Road such as the Grandmark building and the Albion Plaza were taken into account.

The numerical analyses used the OASYS FREW computer program, which assessed the wall bending moment, shear force, deflection and strut forces and take accounts of the wall stiffness, the soil stiffness and the stress history incurred over the staged excavation process.

Ground settlement due to both the effects of groundwater drawdown and deflection of the pipe pile wall during excavation were assessed. Settlement due to the installation of the pipe piles was negligible where deformation and yielding of the surrounding soil was highly localized. Seepage analyses were carried out to assess the water drawdown and settlement. Movement due to the lateral deflection of the pipe pile wall was based on the work of Nicholson (1987) and Miligan (1983). A maximum total ground settlement of 25mm was predicted at the location of the cofferdam wall and reduced to 10mm at a distance 10m from the wall. Detailed movement assessment on the adjacent structures and utilities was completed including a rigorous finite element PLAXIS analysis assessing the effect of the existing MTRC tunnels.

4 BUILDINGS DAMAGE CONTROL

To verify the design assumptions and better manage and control buildings damage, a comprehensive system of instrumentation monitoring system was set up on the surrounding pavement, road and nearby existing buildings and utilities. Instrumentation installed included ground settlement checkpoints, building settlement markers, utility settlement points, standpipies / piezometers and inclinometers. Vibration levels were monitored specifically during the pipe pile installation.

To control deflection of the pipe pile cofferdam and ground movement preloading to predetermined forces were applied to the strutting design. Preloading arrangement and sequence were clearly defined on the construction plans. Loading on the struts was essential for the excavation works in the cofferdam. It was therefore prudent to install strain gauges to monitor the struts forces during excavation works. A control system was set up where struts reloading would be carried out if ground movement exceeded an undue magnitude and struts force decreased in an unprecedented magnitude. Flow chart of the control system to monitor and reloading sequence of the struts is included in Figure 4. Loading jacks were standby on site during the excavation works.

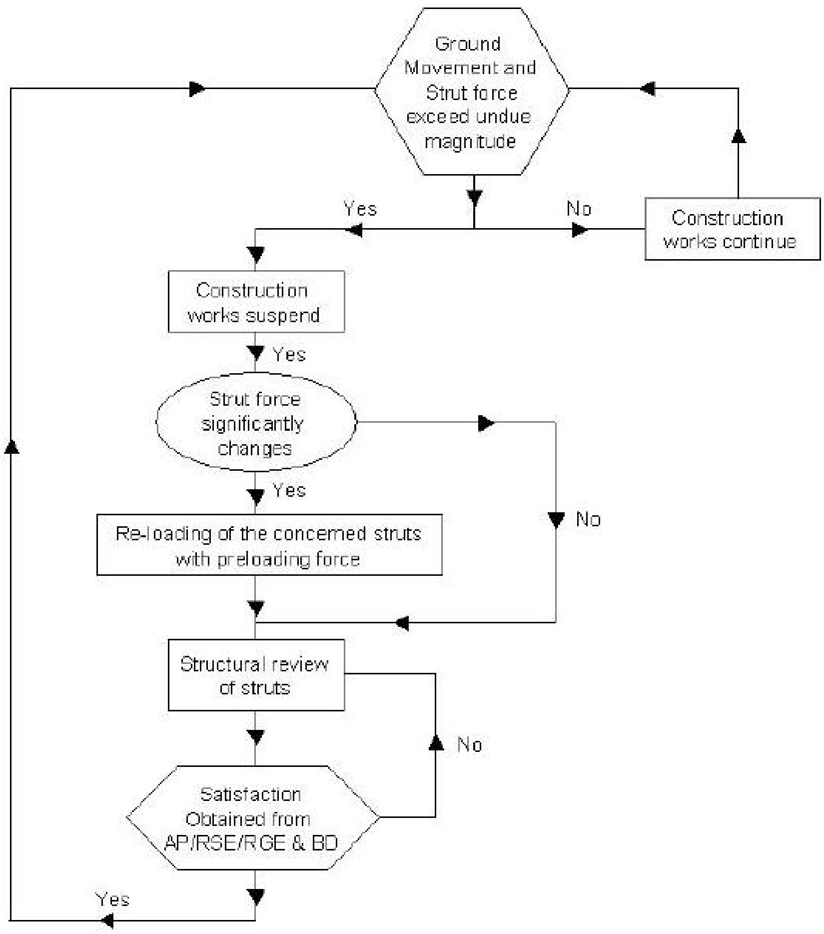

Figure 4: Flow chart of the strutting reloading control system

In view of the important of monitoring data specifically the strain gauges reading, a ‘real-time’ monitoring GEOMON system developed in-house by the Contractor was used. GEOMON provides a centralized database for all monitoring data including survey type monitoring instruments such as settlement checkpoints, inclinometers and standpipe / piezometer, and instrument (in this case the strain gauge) monitored by data-loggers in which the GEOMON enables automated uploading to the database that effectively provides “real-time” monitoring. The information is accessible at any time to approved users over the Internet, and the system allows setting trigger levels in which alarm reports can be automated and emailed to targeted list of addresses. Screenshots of strain gauges and inclinometers of this project are shown in Figure 5.

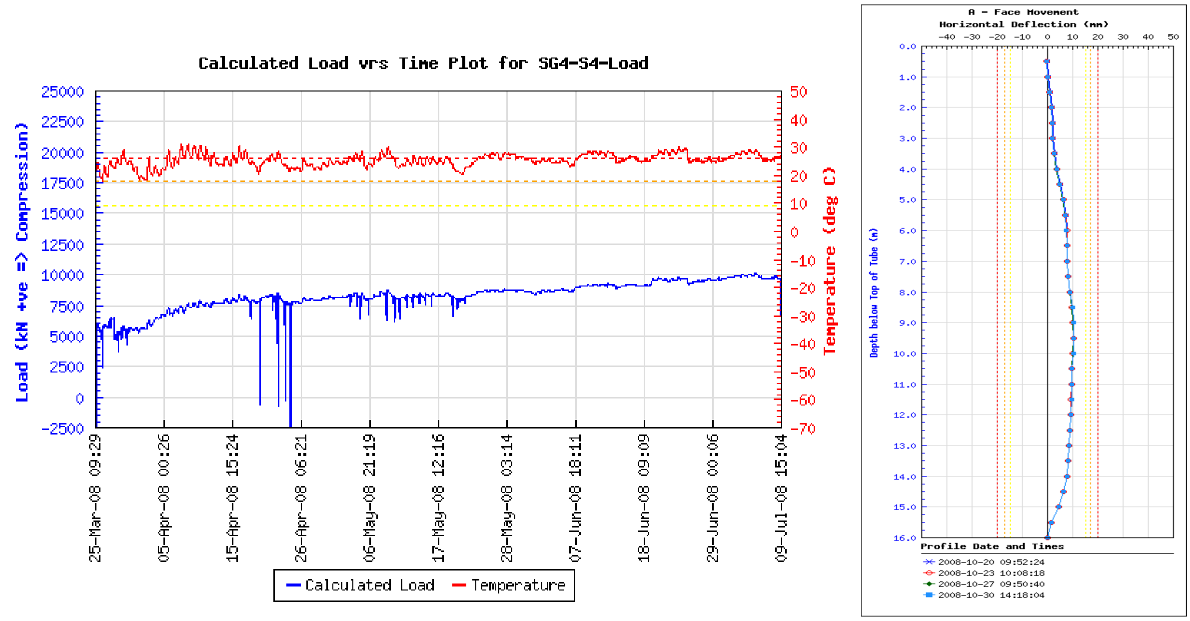

Figure 5: GEOMON screenshots of strut forces and wall deflection

To further enhance building damage control, a Temporary Works Control (TWC) management system was set up as part of the safety policy of the Contractor to achieve zero harm in construction. The system enabled all elements of the temporary works such as the pipe pile wall, struts and working platform to be predefined, design delivered and certified, built according to certified details, endorsed for loading or removal, and regularly checked and inspected.

In addition to the TWC management system, a Risk Assessment management system identifying construction hazards and design risks, assessing the consequence of the hazards, setting up mitigation measures to reduce the impact of the hazards if occurred, and identifying responsible competent person to manage during construction was set up. Monthly risk assessment review meeting was carried including participants from project mangers, site engineer, site agents, supervisors and designers of the temporary works.

In addition to the above damage control mechanism, the statutory requirement of the RGE document was submitted monthly reporting the works progress, verification of design assumption specifically the exposed ground condition, and analyses of the monitoring data.

5 DISCUSSIONS AND CONCLUSIONS

Comparison of the predicted and measured maximum wall deflection is summarized in Table 2. The result indicates measured deflection between 15 to 22% of the predicted magnitude. This was likely contributed to the different actual ground condition compared with the numerical model, and the corner 3-Dimensional effect compared to the 2-Dimensional numerical model. In the numerical model the rockhead was taken at level approximately 22m below ground. The rockhead at the inclinometer I1 and I2 was 12.5m and 16.5m, respectively, below ground.

Table 2: Summary of maximum deflection wall along Nathan Road

Prediction	Measured	Location of Inclinometer
46mm	10mm	I2 – at middle portion of the cofferdam wall & rockhead at 16.5m below ground
	7mm	I1 – at the corner of the cofferdam wall & rockhead at 12.5m below ground

Comparison of wall deflection profile between predicted and measured is shown in Figure 6. In view of the different rockhead levels considered in the predicted model and actual inclinometers location, normalized deflection (with maximum value) and normalized depth (with respective rockhead depth) are plotted in the figure. The deflection profile in I2 matches the predicted curve at the lower portion, whereas in the upper portion the different is due to the lower measured deflection during initial cantilever stage compared with the predicted magnitude. From the profile of I1 the corner effect is apparent where maximum deflection occur at the top of the wall, during the initial cantilever stage, and further movement during excavation is minimal.

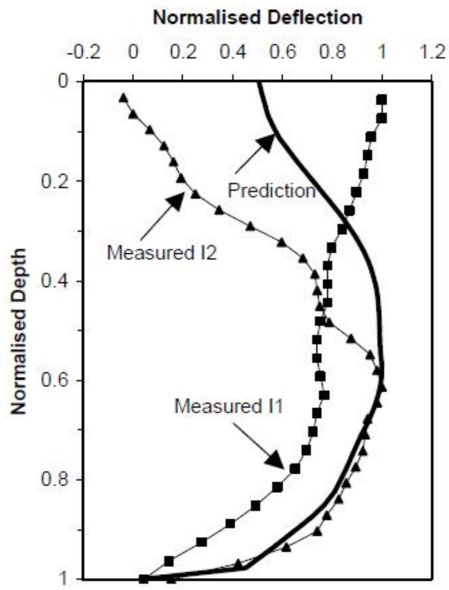

Figure 6: Predicted and measured wall deflection

Table 3 summarises the comparison of predicted and measured strut forces during the deep excavation stages. The measured strut forces are obtained from strain gauges at the location adjacent Mira Hong Kong Hotel. The result indicates actual strut forces up to 80% of the prediction at the upper portion of the deep excavation and reduces at the struts at the lower portion. This is likely due to the conservative assumption of the lower portion of the CDG layer in the numerical model.

Table 3: Summary of maximum strut forces

Strut Layer	Predicted Force (kN/m)	Measured Force (kN/m)	Measured / Predicted Ratio
S2	600	455	76%
S3	1150	930	81%
S4	1850	930	50%
S5	1350	560	41%

In the experience of deep excavation works in Hong Kong, it has been recognized that the wall deflection and struts forces measured are often well below the predicted values. Pan et al. (2001) reported the measured deflection is about 50% of the FREW predicted values for a diaphragm wall used in the excavation for the Tseung Kwan O station and tunnel. The ground consisted of 15 – 18m of reclaimed fill with soft marine clay of 8m thick. In a deep basement design for a project in Central where the ground comprises 8 – 10m of Fill and 8m of Alluvium / Marine Deposit, the measured deflection is about 53% of the value predicted by FREW for a diaphragm wall (Sze & Young 2003). In an East Rail tunnel project designed and constructed by the same Contractor as Tung Ying redevelopment, the measured deflection was about 74% of the FREW predicted values for a pipe pile wall and the measured strut forces were 40% – 70% of the predicted values.

In conclusion, the experience of this project demonstrated that in deep excavation design apart from the geotechnical challenges constructability and safety in the excavation works could be achieved. Potential hazards in the excavation works such as restricted headroom for construction plant, complicated sequence of works and type of strutting system were designed out in the design stage. The important of building damage control mechanism from instrumentation and design verification to works control and risk assessment management system must be emphasized and strictly implemented in the entire deep excavation works.

Plate 1 & 2: Excavation works in progress

ACKNOWLEDGEMENTS

This paper is published with the permission of the Chinese Estates (Tung Ying Building) Limited and Gammon Construction Limited.

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Deep Excavation for the Construction of Joint between IMT Tunnel and C&C Tunnel in River for Guangzhou Zhoutouzui Immersed Tube Tunnel

Joseph Y.C. Lo, W.W. Yang, K.F. Wong, C.K. Tsang & S.F. Chau
AECOM Asia Co. Ltd., Hong Kong

ABSTRACT

In order to facilitate the construction of the cut and cover tunnel in the river to connect with the first immersed tunnel unit underwater for Guangzhou Zhoutouzui immersed tunnel project, a temporary reclamation by environmental friendly sand bags with watertight interlocking pipe pile wall system is proposed for about 20m deep excavation in the river. Struts are installed within the cofferdam during the excavation for the cut and cover tunnel. After the cut and cover tunnel is completed, water is recharged into the cofferdam to balance the water pressure from the river for the removal of struts and temporary reclamation. This paper discusses the design considerations as well as the numerical models performed for the study of these complicated loading conditions on the retaining wall system.

1 INTRODUCTION

The Guangzhou Zhoutouzui immersed tunnel project, consisting of 422m open ramp and 1372m Cut and Cover Tunnel in both Haizhu District and Fangcun District, and 340m Immersed Tunnel underneath the Pearl River, is proposed to alleviate the traffic congestion in the downtown of Guangzhou, P.R. China. AECOM has been assigned as the chief designer of this key infrastructure project in Southern China.

A layout plan of the Zhoutouzui tunnel is shown in Figure 1. The immersed tunnel units will be constructed in a dry dock located in line with the cut and cover tunnel in the Fangcun side. Having constructed the tunnel units inside the dry dock, the dry dock will be flooded and the tunnel units will be floated out from the dry dock for joining the cut and cover tunnel in Haizhu side underwater. Since the flexible joint between the first immersed tunnel unit and cut and cover tunnel in Haizhu side is located at about 22m from the existing shoreline, therefore temporary reclamation is required for the construction of cut and cover tunnel in the river before joining the immersed tunnel. Figure 2 shows the general arrangement of the interface between the immersed tunnel and cut and cover tunnel in the Haizhu side.

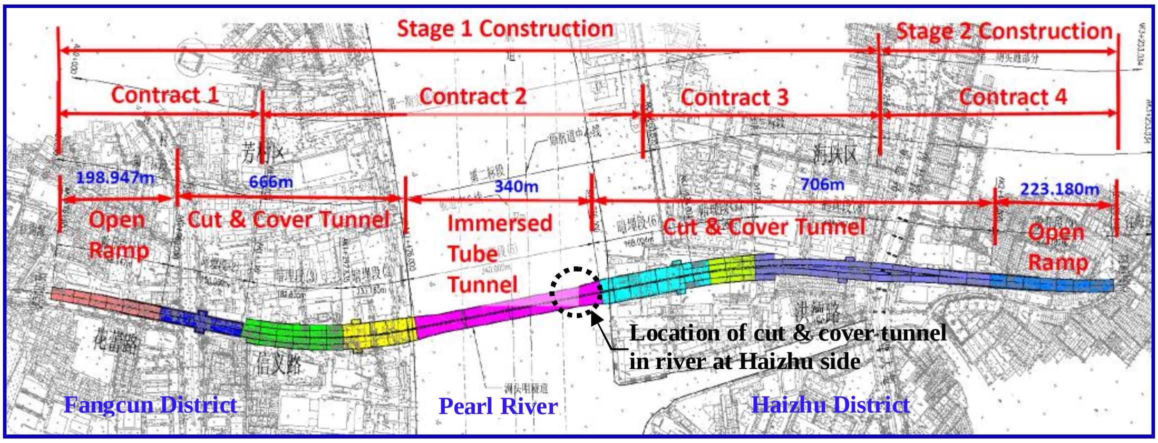

Figure 1: Layout plan of Zhoutouzui Tunnel

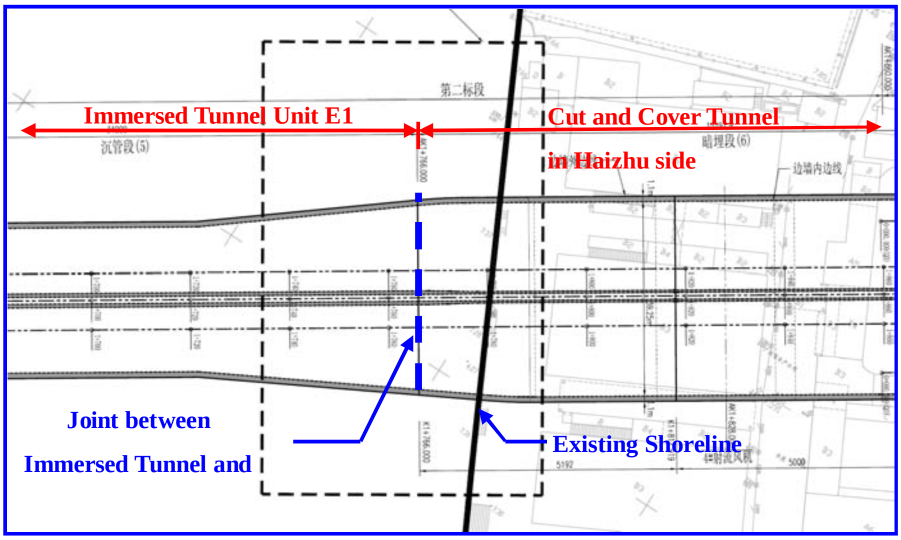

Figure 2: Arrangement of immersed tunnel and cut and cover tunnel in Haizhu side

During the design stages, the selection of the support system for deep excavation in the river is based on site condition, geological condition, shape and size of the excavation, limit of wall deformation, and construction sequence, etc. The existing fairway of Pearl River is approximately 100m from the existing shoreline of Haizhu District and the fairway will need to be maintained during construction. In order to facilitate the construction of cut and cover tunnels in the river, a temporary cofferdam is required. The option of forming a dike with slopes from the top of dike to the existing river bed on both sides will require large extent of reclamation area in river, having a significant impact to the existing fairway. Alternatively, a temporary cofferdam formed of environmental friendly sand bags reclamation with watertight interlocking pipe pile wall system is proposed for approximately 20m deep excavation in the river. The paper will focus on the design considerations, construction sequences of the proposed excavation and lateral support system, numerical model performed to study the complicated loading conditions and presentation of analysis results.

2 GEOLOGICAL AND HYDROGEOLOGICAL CONDITIONS

Total eleven boreholes have been drilled within the area of Haizhu cofferdam to confirm the geological stratification. The geological profile in the vicinity of the interface of IMT and C&C tunnels near Haizhu river bank is illustrated in Figure 3. It is revealed from the site investigation data that the geology of the site comprises four major geological strata: (i) Fill, (ii) Alluvium, (iii) Completely/Highly Decomposed Sandstone (C/HDS) and (iv) Moderately/Slightly Decomposed Sandstone (M/SDS).

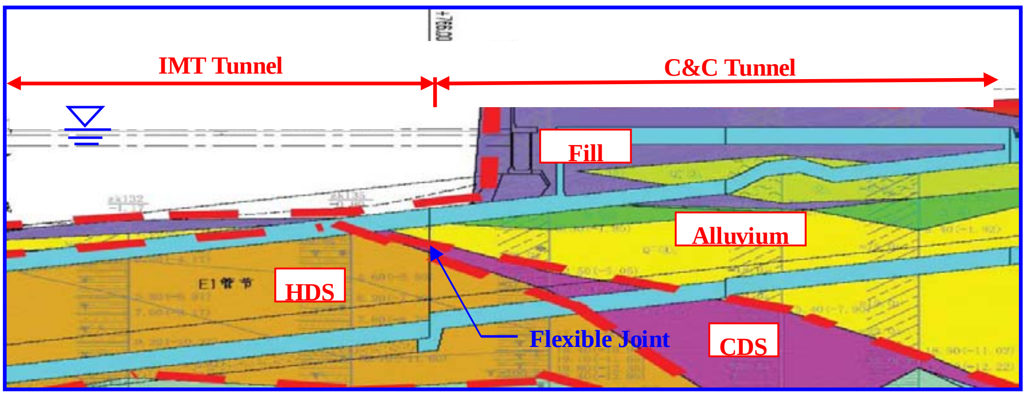

Figure 3: Geological longitudinal section along tunnel near Haizhu reclamation

Based on the in-situ and laboratory tests for soil and rock, together with the local experiences and codes of practices, the soil and rock design parameters used in analysis are listed in Table 1.

Table 1: Soil and rock parameters

Materials	Specified Gravity (kN/m3)	Elastic modulus (MPa)	Poisson's ratio	Cohesion (kPa)	Friction angle (Degree)
Fill	18.0	12	0.3	5	15
Alluvium	19.0	15	0.45	15	13
CDS	20.0	85	0.30	28	20
HDS	20.0	180	0.30	30	25
Bedrock	26.0	500	0.27	200	28

The level of the riverbed is -2.0 m, the average high tidal level is +5.68m, and the historical highest tidal level is +7.48m. The average high tidal level will be adopted as the design water level, whilst the highest tidal level will be used for sensitivity analysis.

3 CONSTRUCTION SEQUENCES OF EXCAVATION OF THE TEMPORARY COFFERDAM AND THE ASSOCIATED SUPPORT SYSTEM

The first connection for immersed tunnel will be carried out at the Haizhu side and the flexible joint of the immersed tunnel unit will be connected to the cut and cover tunnel in the river. Prior to the sinking and joining of immersed tunnel underwater, the cut and cover tunnel must be constructed first. To allow construction of the cut and cover tunnel and reinstatement of the seawall, a temporary cofferdam with support system (Refer Figures 4 and 5 for the layout and longitudinal section respectively) is required. By considering the site specific condition and programme requirement, construction sequences of the cofferdam are described as follows:

i) Construct the onshore excavation and support system, install grout curtain and construct temporary seawall (design of these elements is not discussed in this paper);
ii) Install sand bags layer by layer with side slopes of gradient of 2(V):3(H) to form the cofferdam (the construction of environmental sand bag cofferdam is shown in Plate 1 and Plate 2). Temporary reclamation is then carried out by sand jet method within the sand bag cofferdam (The general view of completed temporary reclamation is shown in Plant 3.);
iii) Install interlocking pipe pile wall and jet grout piles (hydraulic cut-off wall) and install king posts within the temporary cofferdam (Plates 4 and 5 show the installation of pipe pile wall on site);
iv) Excavate and install waling and strut layer by layer to the final excavation level within the temporary cofferdam (general view of the lateral support system is shown in Plate 6);
(as at the date of this paper issued, the works were completed up to this stage);
v) Construct cut and cover tunnel and water tight bulkhead wall at the end of cut and cove tunnel;
vi) Construct the permanent seawall on top of cut and cover tunnel;
vii) Backfill the cut and cover tunnel onshore behind the permanent seawall to final design ground level;
viii) Remove the temporary sand bag cofferdam layer by layer and carry out underwater trench excavation for IMT tunnel;
ix) Recharge water into the cofferdam at 1.5m below lowest layer of strut to increase the water pressure inside the cofferdam and remove the lowest layer of strut;
x) Repeat Stage ix) until all the struts inside the cofferdam are removed;
xi) Remove all pipe pile wall and jet grout piles in the river.

Plate 1: Installation of sand bags for temporary reclamation

Plate 2: General view of sand bags filled with sand

Plate 3: General view of completed reclamation

Plate 4: Installation of interlocking pipe pile wall

Plate 5: Interlocking of pipe pile wall

Plate 6: General view of excavation in river

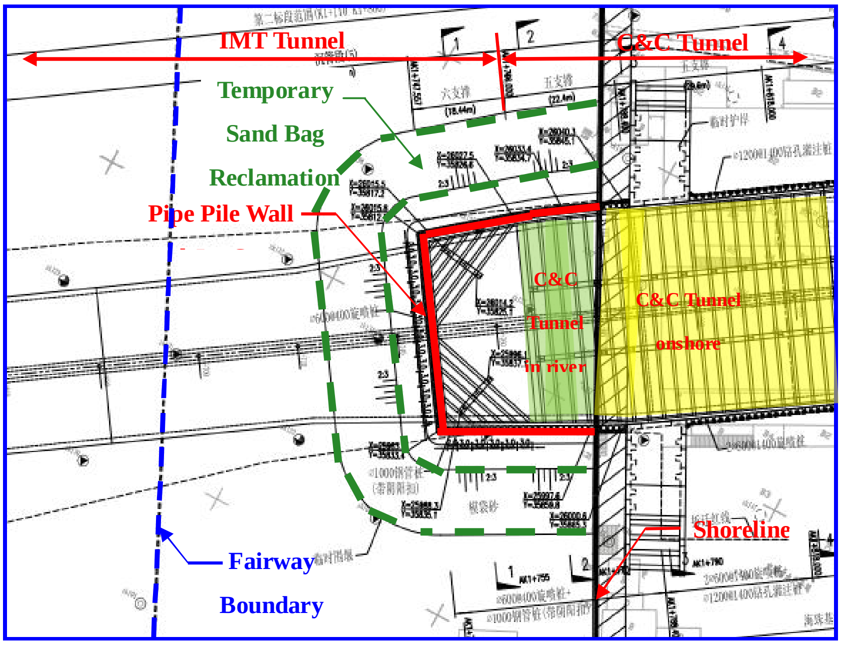

Figure 4: Layout of temporary reclamation and excavation and support system

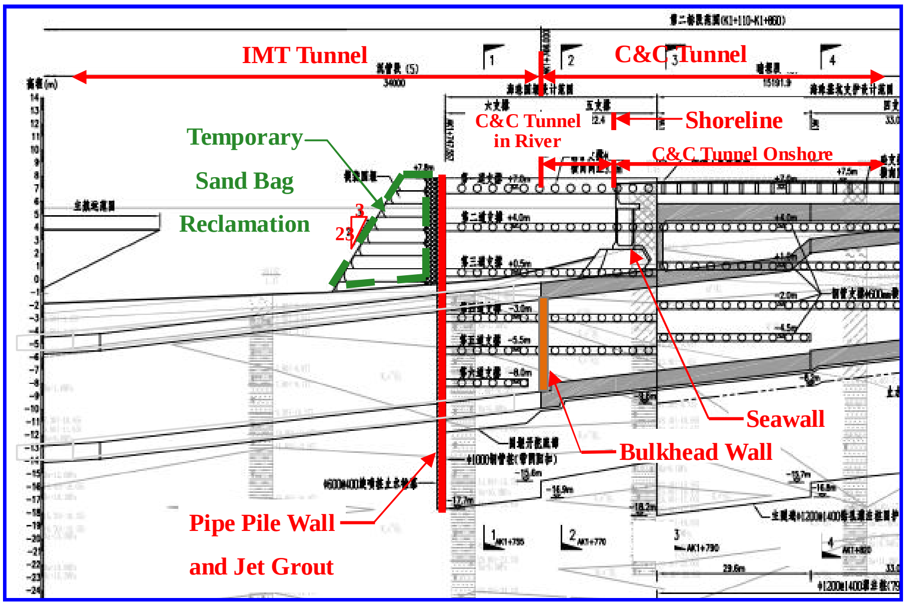

Figure 5: Longitudinal section of temporary reclamation and excavation and support system

4 DESIGN OF DEEP EXCAVATION IN THE RIVER

4.1 Safety Class of the Excavation and Support System

With reference to the "Code of Practice for Excavation and Support System in Guagdong Province (DBJ/T15-20-97)" there are 3 classes of excavation and support system dependent on the importance of the surrounding structures. Even though there is no sensitive structure in the vicinity of the cut and cover tunnel in the river, however it is considered that excessive deformation of shoring system will cause water inflow from the river and will cause catastrophic impact on the safety and economic loss of the construction. Therefore the safety class of Class 1 (most important) is adopted for the design of excavation and lateral support system with an Importance Factor of 1.1 and horizontal movement limit of 30mm.

4.2 Summary of the Excavation and Support System

In order to prevent the inflow of water from the river to the excavation, continuous interlocking pipe piles wall with diameter of 1000mm and a thickness of 12mm has been selected for the lateral retaining wall for excavation of about 20m. However after the expert review panel, a hydraulic cut off wall by continuous jet grout columns are also provided at the back of the proposed pipe pile wall as an additional measures to prevent water inflow. The minimum embedded depth is determined in compliance with the "Code of Practice for Excavation and Support System in Guagdong Province (DBJ/T15-20-97)". Based on the geological conditions of the site, the pipe pile wall will be embedded in moderately decomposed sandstone. The design of multi-strut system provided for the staged excavation was carried out according the Design Code for Structure Use of Steel (GB50017-2003) and Design Code for Structural Use of Concrete (GB50017-2003). The overall stability of shoring system is checked by employing computer software SLOPE/W.

In consideration of the geological conditions and the excavation depths, 6 layers of diagonal struts are proposed for the area with excavation depths of 18.0m to 19.6m (Section 1 in Figure 6) whilst 5 layers of struts are proposed for the area with excavation depths of 16.2m to 18.0m (Section 2 in Figure 7). The layout plan and longitudinal section of the shoring system is shown in Figure 4 and 5 respectively. The struts are general pipe pile of 600mm diameter in 3m spacing and the top strut is 0.6m x 0.8m RC beam.

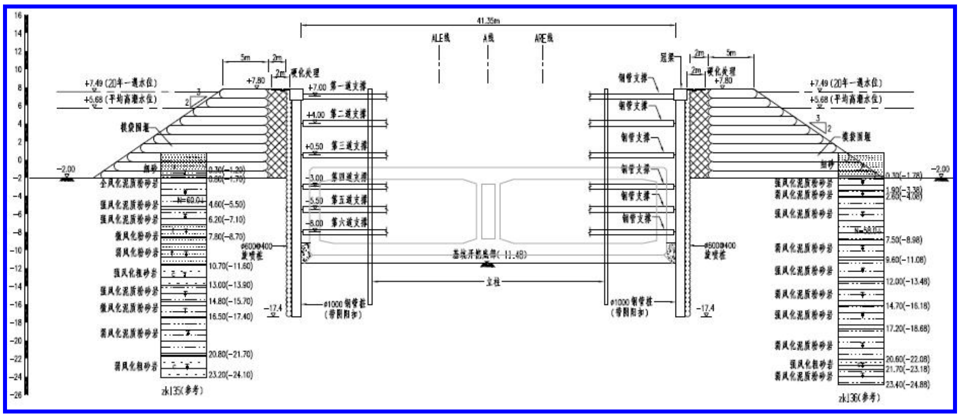

Figure 6: Cross section 1 for excavation and support system (6 struts)

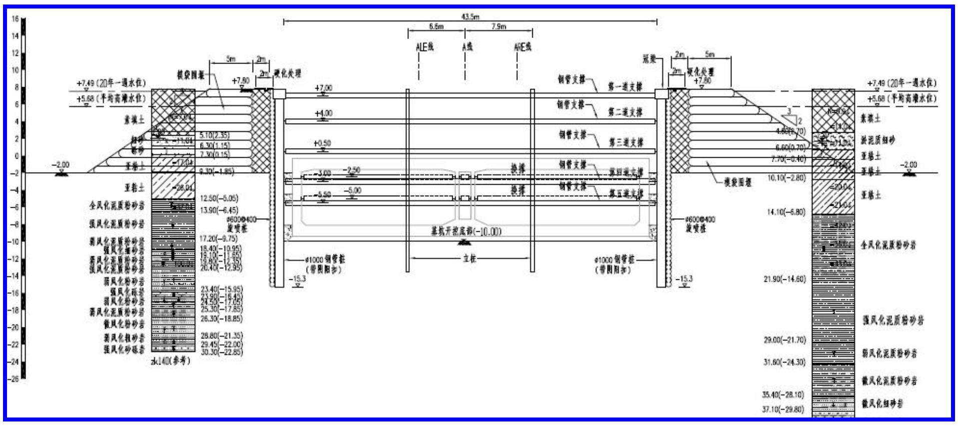

Figure 7: Cross section 2 for excavation and support system (5 struts)

4.3 Numerical Analysis for the Deep Excavation and Support System

In the design of deep excavation and support system, 20kPa surcharge is applied on the unexcavated sides (ground level is +7.8mPD). To model the load conditions of retaining and strut structure and the soil/structure interaction in different stages of excavation, Finite Element software PLAXIS is employed for analysis. The construction sequence as described in Section 3 is modeled with due consideration of the loading of lateral support system for sequence of dewatering and recharging inside the cofferdam.

Figures 8 and 9 show the PLAXIS model and analysis results (i.e. moment and shear force envelop for pipe pile wall) for the critical Cross Section 1. The estimated maximum horizontal displacement is 14.5mm, which is well within the allowable limit of 30mm. Axial forces exerting on the struts at different stages are also obtained from the PLAXIS models with the maximum axial force in each layer of strut used for structural design of struts and waling. The design bending moment, shear force of pipe pile wall and design axial load of struts need to be multiplied by the Importance Factor as described in Section 4.1 according to the following equations:

Design Bending Moment, $M = 1.25\gamma_o M_k$
Design Shear Force, $V = 1.25\gamma_o V_k$
Design Axial Force, $T = 1.25\gamma_o T_k$

where γ_o = Importance Factor = 1.1 and M_k, V_k and T_k are bending moment, shear force and axial force from the PLAXIS model.

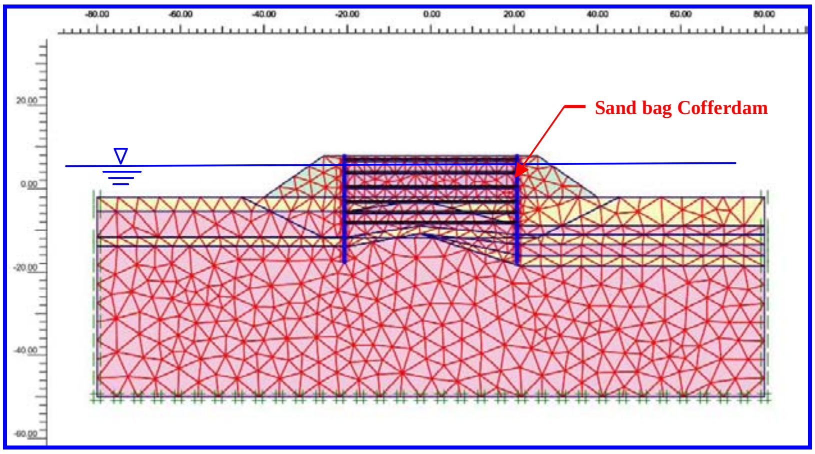

Figure 8: PLAXIS model for critical cross section 1 (6 struts)

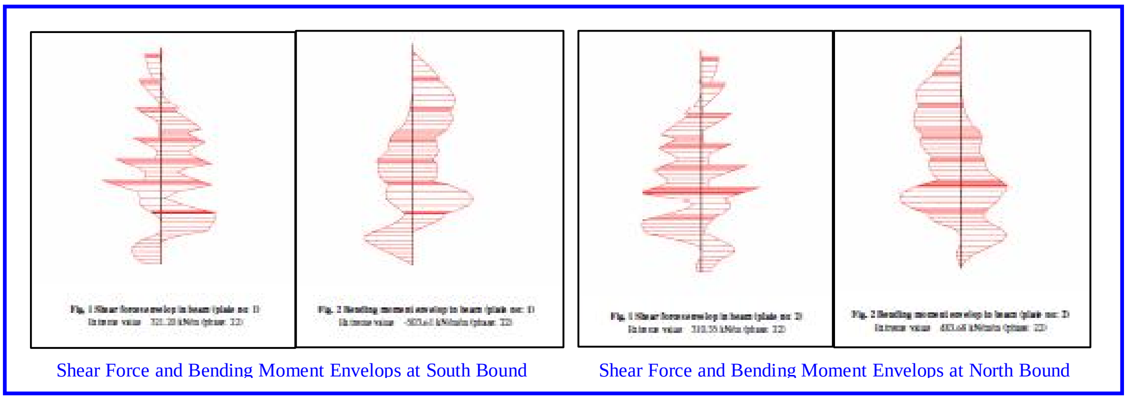

Figure 9: Analysis results for pipe pile wall (critical cross section 1)

5 CONCLUSIONS

Due to the unique site condition and progromme requirement of the Guangzhou Zhoutouzui IMT tunnel project, the immersed joint between the first immersed tunnel unit and the cut and cover tunnel at Haizui side is located at about 22m from the existing shoreline. Therefore, a temporary cofferdam by environmental friendly sand bags with watertight interlocking pipe pile wall system is proposed for the construction of cut and cover in the river. The paper discusses the design considerations for the excavation and lateral support system for excavation depth of about 20m in the temporary reclamation.

As a remark of the construction, the temporary reclamation within the cofferdam constructed by the construction of environmental sand bag were completed. The interlocking pipe pile wall is firstly adopted in Guangzhou City and water-tightness performance of interlocking is satisfactory. The excavation of cut and cover tunnel is on-going.

The use of sand bags method for temporary reclamation provides a fast track construction with minimum environmental impact. In addition, insignificant water leakage was observed during excavation by the use of stiff interlocking pipe pile wall. The combination use of the sand bag reclamation and interlocking pipe pile wall demonstrates a fast and economical engineering solution for deep excavation of cut and cover tunnel construction in the river.

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Ground Response in Deep Excavation in Soft Soil in Shanghai

C.S. Lau
Shanghai Wheelock Square Development Limited, Shanghai
S.L. Chiu, K.L. Lo & Kent K.N. Chu
AECOM Asia Co. Ltd., Hong Kong

ABSTRACT

This paper presents a case of well instrumented deep excavation in soft ground located in the central business district of Shanghai, China. The proposed development comprised a 55 storey office tower of 270m high, two low-rise annex buildings and a 3-level basement. Depth of excavation was 17m to 25m. Challenge in project is to protect the twin operating metro tunnels located at 5.4m outside along the northern site boundary, from any damage or disturbance caused by the excavation. Ground improvement and precise control of time and sequence of excavation and strut installation are implemented to control ground movement to the desired limits. Besides the metro tunnels, other significant structures adjacent to the excavation include two major business commuter roads, a viaduct, and a new high-rise building. Comprehensive geotechnical instrumentation and monitoring was implemented to monitor the behavior of the temporary retaining structure, ground movement and groundwater drawdown. The monitoring records of wall deformations, variation of strut force, movements of king posts and changes in ground water level were used to back analyze the soil behavior in response to the excavation.

1 INTRODUCTION

Since 1990's comprehensive redevelopment at the downtown of Shanghai has commenced to cope with the economic growth of the city. Many tall buildings of modern design and transportation infrastructure such as underground metro and viaducts have been constructed in the CBD. By the end of April 2010, 11 metro lines of total 410km comprising 266 stations are in service.

Deep excavation in close proximity of existing building, highway structure and metro tunnels are common in the urban area of Shanghai. Soil response to excavation in the soft alluvuial deposits in Shanghai have been studied thoroughly during construction of the Shanghai metro lines no. 1 and 2 and reported by Liu (1999). This paper presents a case of well-instrumented deep excavation in the soft soil in the CBD of Shanghai. The monitoring records of wall deformations, variation of strut force, movements of king posts and changes in ground water level were used to back analyze the soil behavior in response to excavation.

2 PROJECT DESCRIPTIONS

The development site is located at the downtown of Shanghai and the scope of development comprise a 55-storey office tower of 270m high, two low-rise annex buildings and a 3-level basement covering the whole site. The plan geometry of the excavation was L-shaped with maximum dimensions of 98m x 123m and plan area of 9812m^2. The excavation consisted of two portions namely the south pit and the north pit, which are divided by an intermediate diaphragm wall.

The office tower is located at the centre of the southern excavation and the two annex buildings are located at the northern and southern portion of the site respectively. Depth of excavation at the tower area is generally 20m and is increased to 25m at the two lift pits. Outside the tower area, depth of excavation is about 17.5m (see Figure 1).

Twin operating metro tunnels are located at 5.4m outside along the northern site boundary and are the most sensitive receiver to the excavation. The metro tunnels, with outer diameter of 6.2m, are comprised of segmental lining and founded at about 15m below ground. At the eastern and southern side of the excavation are pavements and roads with busy traffic. A 20-storey building with pile foundation is located at the western side of the excavation (See Figure 1).

This paper will concentrate on the excavation in the south pit which occupy substantial portion of the site. Excavation in the north pit has been reported by Lau (2008).

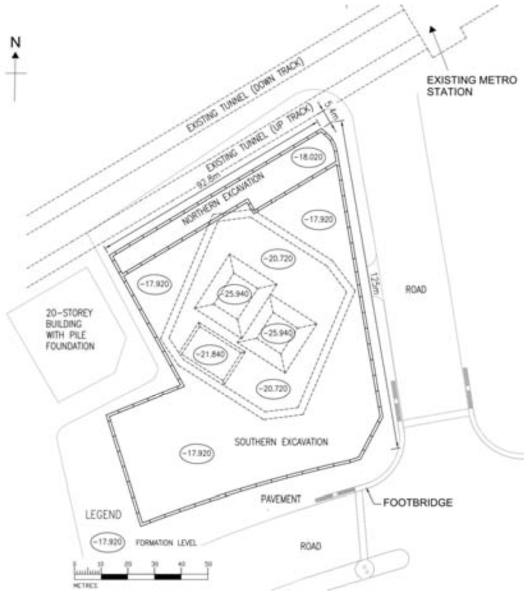


Figure 1: General layout of excavation

3 SITE CHARATERISTICS

Shanghai is located at the delta alluvial plain of Yangtze River. Typical subsoil geology can be divided into 12 layers (Shenyuan 2002). Depth and description of the top 7 soil strata encountered at the site are summarized in Table 1. The Atterberg limits and undrained shear strength at various depth are shown in Figure 2.

Table 1: Summary of soil strata encountered at the site

Soil Layer	Soil Classification	Thickness (m)	*Bottom Level of Soil Strata (m)	SPT N value	CPT Cone Resistance (MPa)
1	Fill – composed of construction debris, such as crushed concrete blocks and brick crumbles	1.5	-2.5	-	-
2	Brownish yellow, very soft, silty CLAY	1.5	-4	4.5	0.79
3	Grey, very soft, silty CLAY	3	-7	2	0.48
4	Grey, very soft to soft silty CLAY	9	-16	1.1	0.49
5-1	Grey soft CLAY	4	-20	2	0.76
5-2	Grey soft silty CLAY	8	-28	5	1.06
5-3	Grey soft to firm, silty CLAY with sandy silt	8	-36	10	2.14
6	Green, firm, silty CLAY	3	-39	14.4	3.26
7-1	Greenish yellow, dense, silty SAND	7	-46	34.2	17.88
7-2	Yellowish grey silty, dense to very dense SAND	5	-51	52.8	30.75

*- Relative level is adopted with ±0.0m defined at the finish level of the G/F of the proposed development.
Existing ground level is at -0.5m approximate.

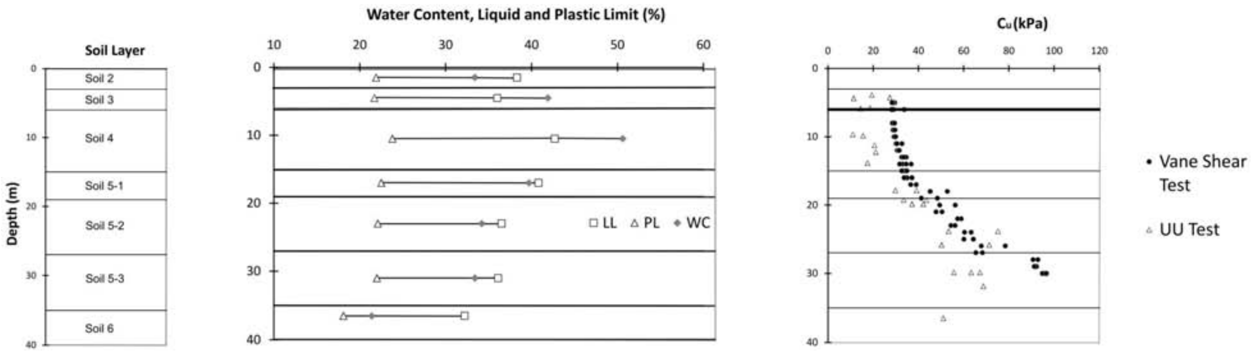


Figure 2: Physical properties of various soil layers within excavation

Groundwater table is encountered at 0.6m to 1.7m below ground and is mainly recharged by rainfall. Confined aquifer exists within soil layer 7 which is composed of silty sand and is 14m below the bottom of excavation. The measured piezometric head in soil layer no. 7 is at -4.5m approximate and is above the lowest point of excavation at -25m.

4 DESIGN CONSIDERATIONS

4.1 Requirement of Shanghai Metro

Shanghai Code for Protection of Metro Tunnel and Structures specifies the following limits that should not be exceeded in regard to construction works taking place within the protection zone:

- The induced vertical and horizontal movement of tunnel shall not exceed 20mm;
- Change of the radius of curvature of tunnel shall not exceed 15000m;
- Differential settlement shall not exceed 1in 2500;
- Additional pressure on the tunnel shall not exceed 20kPa.

4.2 Design of Excavation

The basement structure was constructed by bottom-up method of construction. Pile foundation of the tower and the basement structure comprised of 950 nos. of 800mm~850mm diameter bored piles with length of 55m to 86m and was carried out before excavation works commencing. The excavation was divided into two portions, namely the northern and the southern excavations, with an intermediate diaphragm wall as shown in Figure 1. All of the diaphragm walls were 1m thick, with typical panel width of 6m, and was installed to 36m to 41m below ground. In order to minimize the movement of the metro tunnels, the northern excavation was carried out after completion of the basement structure in the southern excavation.

Some of the soil within the excavation was strengthened by means of jet grouting (JG) and deep cement mixing (DCM) as shown in Figure 3a thus enhancing the soil stiffness. Design strength of JG and DCM in 28 days is 1.3MPa and 1.5MPa respectively. Soil at both sides of the diaphragm wall around the northern excavation was treated by DCM before trenching as precautionary measures to avoid impact on metro tunnels due to trench instability. All of the soil with the northern excavation was strengthened by DCM (see Figure 3c). At the southern excavation, soil within 6m from the wall at the excavation side was treated by JC (see Figure 3d).

Each lift of excavation was sub-divided into smaller clusters as shown in Figure 3b. Excavation of every cluster was carried out in such a sequence and manner that duration and length of unsupported span shall be minimized thus minimizing wall deflection. 4 levels of reinforced concrete struts of dimensions ranging from 0.8mx0.8m to 1mx1m were provided for the southern pit while reinforced concrete strut at the top and 4 levels of steel tubular struts were provided in the northern pit. The struts were such arranged that large openings were provided to facilitate excavation and mucking out. In order to mitigate the down drag effect of future tower settlement on the adjacent metro tunnels, a row of separation piles (41 nos.) of 1m diameter at 1.5m spacing were installed in the north pit below the basement structure down to -106m.

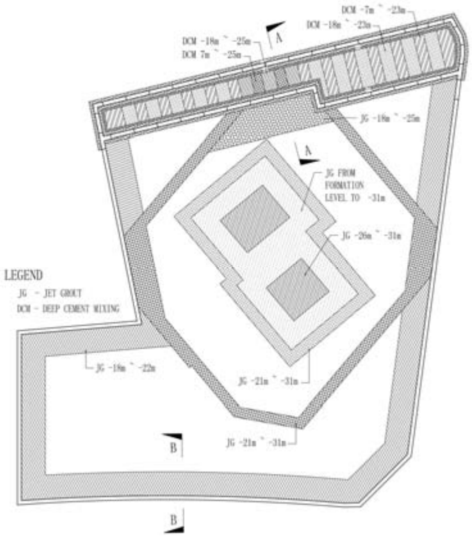


Figure 3a: Ground treatment layout

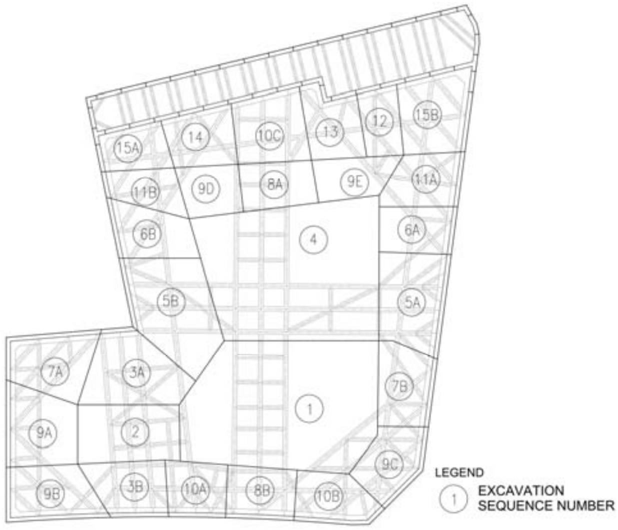


Figure 3b: Reinforced concrete strut layout

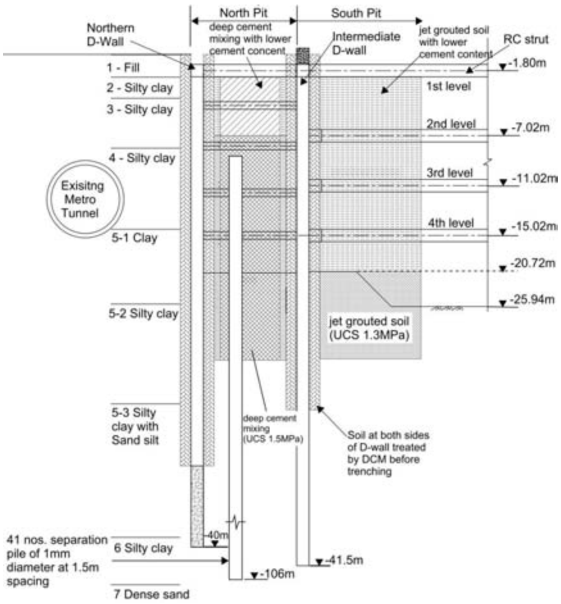


Figure 3c: Section A-A

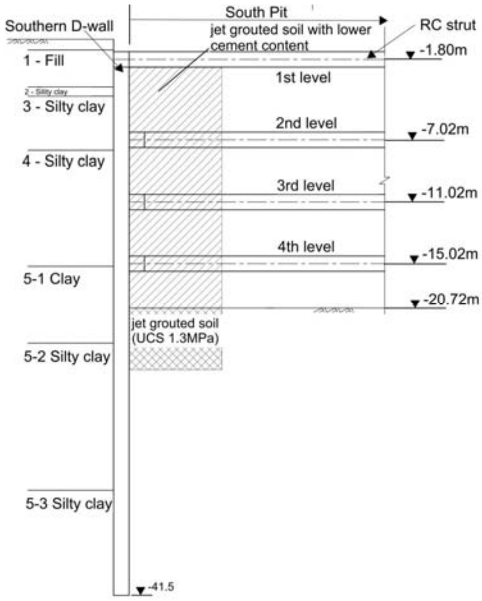


Figure 3d: Section B-B

Plate 1: General view of southern excavation (looking south)

Plate 2: General view of northern excavation (looking south)

4.3 Geotechnical Instrumentation and Monitoring

Scope of geotechnical instrumentation and monitoring are summarized in Table 2 below and the locations of the geotechnical instruments are showed in Figure 4. Measurement of the instruments was carried out at frequency of twice per week to daily dependent on the trend of readings and significance of the data needed for continuous review of the overall performance of the excavation.

Table 2: Scope of geotechnical instrumentation for excavation

Instruments	Purpose	Remarks
Inclinometers	Lateral movement of diaphragm walls. Lateral movement of ground outside excavation.	Inclinometer casings are installed in the diaphragm wall reinforcement cages and in boreholes in soil. The casing serves as an access tube to guide a servo accelerometer based probe in the two orthogonal directions of measurement. Lateral deformation of the access tube is measured with reference to the top of the access tube. Lateral movement at the top of diaphragm wall is measured for adjustment of inclinometer readings.
Survey nails at the top of diaphragm walls	Vertical and horizontal movement at the top of diaphragm wall	-
Survey markers at the top of king posts	Vertical movement at top of king posts	-
Strain gauges (vibrating wire type)	Strut forces	Four strain gauges, with one at each side of the rectangular reinforced concrete struts for determination of the average strain at particular transverse section of a strut.
Observation wells	Groundwater level outside excavation	-
Settlement markers	Settlement of adjacent ground, utility valves, viaduct and footbridge	-

Monitoring of the operating metro tunnels was conducted by Shanghai Metro. The section of tunnels adjacent to the excavation and 60m beyond was monitored. Settlement, lateral movement and tunnel convergence were measured at 5m to 10m intervals. Tunnel settlement was measured by a high sensitivity settlement system consisting of a series of vessels containing liquid level sensors interconnected by a liquid filled tube. The reference vessel was positioned at the metro station located at 60m away from the site, with additional vessels positioned within the tunnel. Differential settlement or heave between any vessel and the reference vessel resulted in an apparent rise or fall of the liquid level in that vessel. This system is particularly suitable for critical situations where high resolutions are required. Elevation changes of as little as ±0.02 mm are detectable. Regular inspections of the tunnel structure were conducted regularly by Shanghai Metro, and occasionally together with the developer, the consultants, the supervising engineer and the contractor.

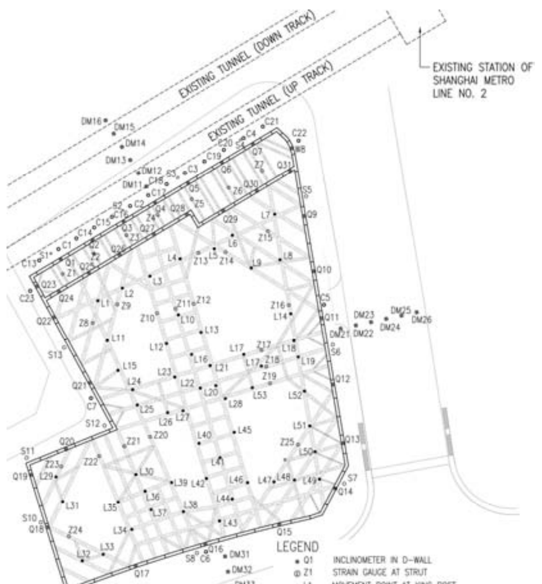


Figure 4: Geotechnical instrumentation layout

5 OBSERVED BEHEVIOR OF EXCAVATION AND ADJACENT TUNNELS

5.1 Lateral Deflection of Diaphragm Wall and Ground Settlement

Maximum deflection of diaphragm walls at the northern, eastern and southern sides and the associated ground settlement at various stages of excavation are shown in Figure 5 below. When depth of excavation reached 20m, the maximum deflection of diaphragm walls at the eastern and the southern side were 84 and 80mm respectively, equivalent to about 0.40% of the depth of excavation, while maximum ground settlement was 29mm and 32mm respectively, equivalent to about 37% of the maximum wall deflection.

As soil within the northern excavation was strengthened by DCM, and the intermediate and the northern diaphragm walls are close to each other, deflection at these two walls was much less than that at the other sides, and were 26mm and 6mm respectively.

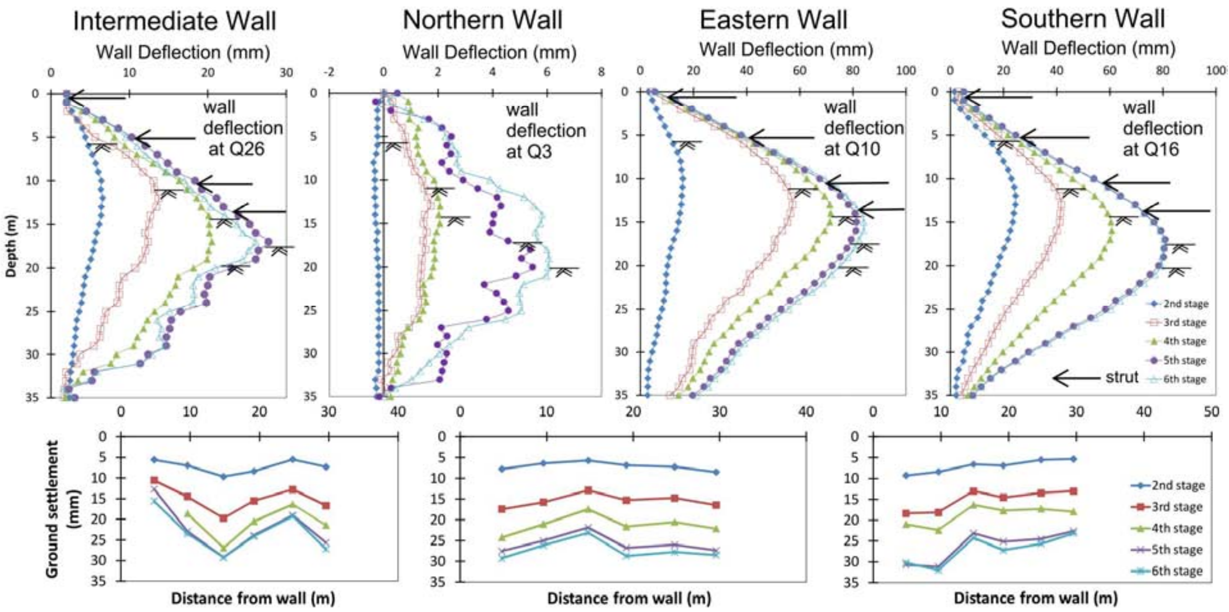


Figure 5: Lateral deflection of diaphragm walls and settlement at the ground behind

At each side of the excavation, deflection at the mid span was greater than that near the corner due to corner effect. Maximum lateral deflection at various locations at the end of excavation is shown in Figure 6 below. Figure 7 shows the trend of increase of maximum deflection at the mid span of the northern, intermediate, eastern and southern diaphragm walls, throughout excavation and demolition of struts. Wall deflection tended to cease at the end of excavation and increased again slowly by about 10% during demolition of the struts and construction of the basement structure.

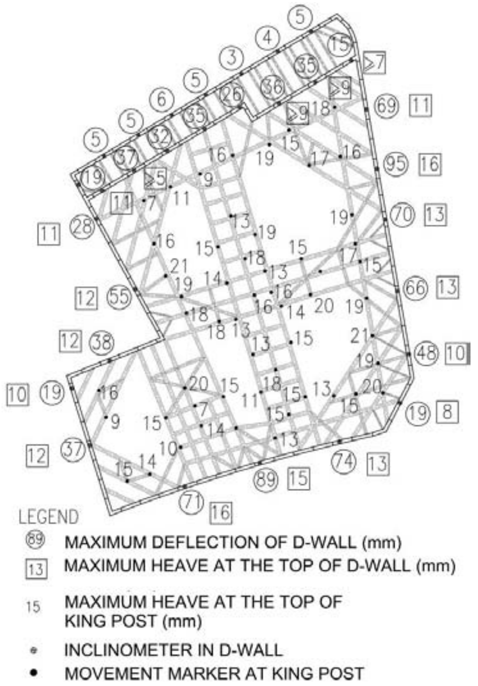


Figure 6: Maximum wall deflection and upward movement of diaphragm walls and king posts

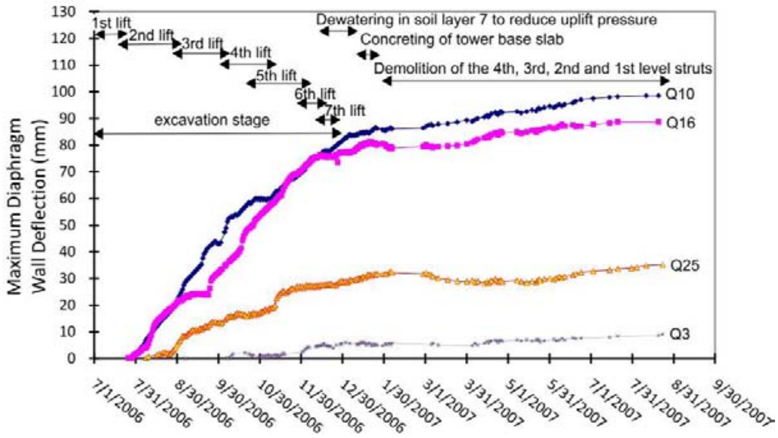


Figure 7: Trend of maximum diaphragm wall deflection throughout excavation and basement construction

5.2 *Vertical Movement of Diaphragm Walls and King Posts*

Excavation induced unloading effect to the soil below and resulted in upward movement of the soil mass together with the king posts and diaphragm walls. Figures 8 and 9 show the typical trend of vertical movement of diaphragm walls and king posts respectively. Maximum heave of the diaphragm walls and king posts at various locations occurred at the end of excavation is shown in Figure 6. During construction of the base slabs of the annex buildings and the tower, diaphragm walls settled by about 7mm while king posts only settled by 2mm~3mm. King posts were installed to 55m to 86m below ground and founded on the dense to very dense sand layers (soil layers no. 7 and 9), while diaphragm walls were installed to 35m to 40m depth and generally founded on the clay layers (soil layers no. 5 and 6). It explains why the diaphragm wall settled more during construction of base slabs. No significant settlement occurred after concreting of the base slab as any loadings on the slab will be transferred to the pile foundation.

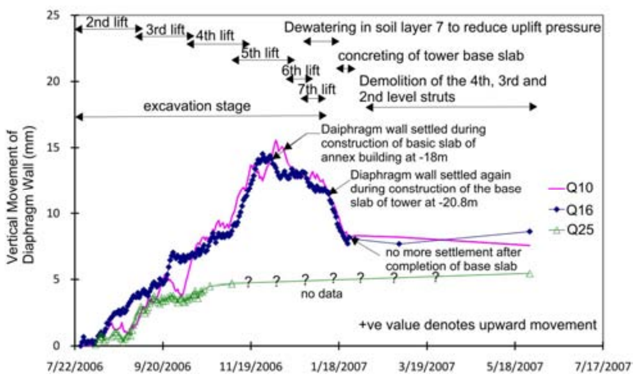


Figure 8: Vertical movement of diaphragm

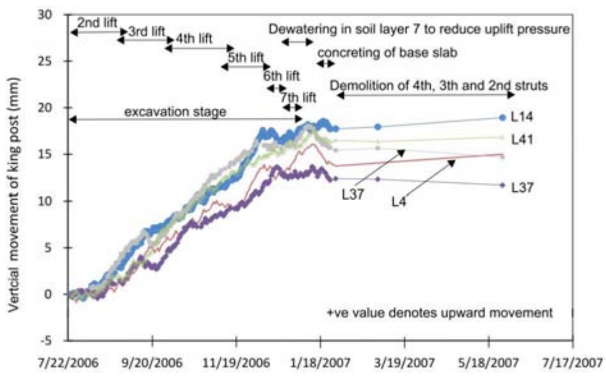


Figure 9: Vertical movement of king posts

5.3 *Strut Forces*

Typical variation of measured strut forces is shown in Figure 10 below. During the course of excavation, wall deflection near the excavated level was greater and resulted in gradual transfer of strut forces from higher level to lower level. During construction of the basement structure, removal of the struts at lower resulted in transfer of strut force to the remaining struts above. As shown in Figure 10, forces in the 2nd and 3rd level struts increased very much after removal of the 4th level strut. It is also due to additional wall deflection after strut demolition (see Figure 7).

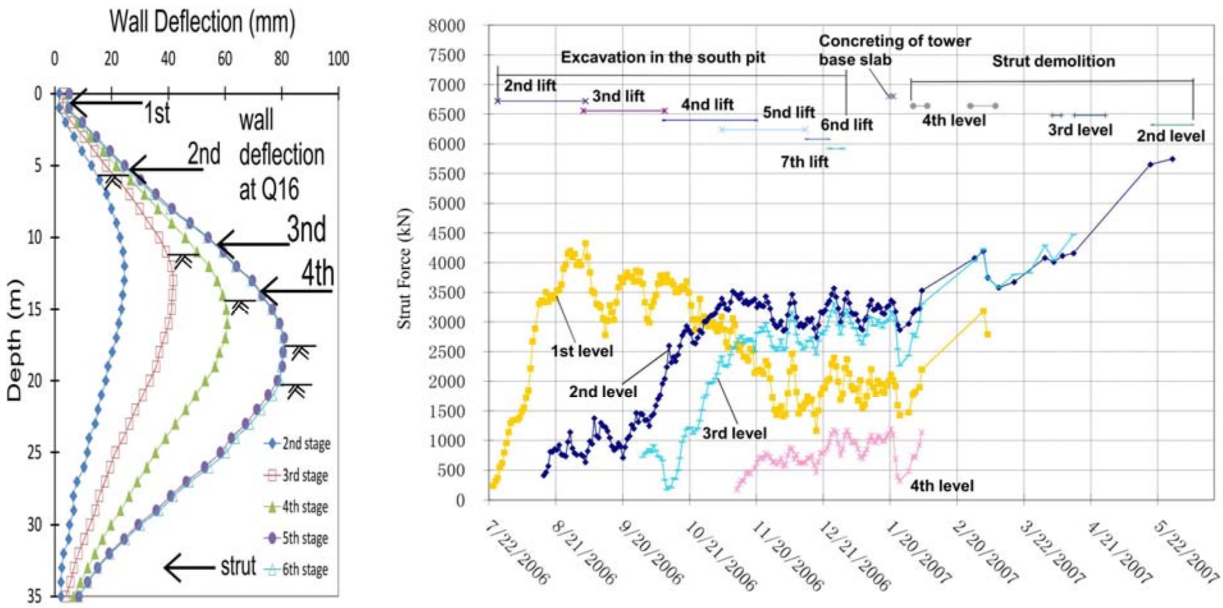


Figure 10: Strut force measured at strain gauge location Z20

5.4 Groundwater Drawdown

Groundwater table is located at about 0.7m to 1.5 below ground and dewatering was required within site prior to and during excavation. 36 nos. of dewatering well of 22m to 25m deep were installed over the site for dewatering. In the peak dewatering period, daily total pumping rate was about $35m^3$ to $45m^3$ per day. During the course of excavation works, groundwater drawdown of 2m~3m was observed at observation well S12 and S13 at the western side due to leakage of adjacent diaphragm wall. Counter measures including groundwater recharge and sealing of the leaking joint by grout injection were required. No significant groundwater drawdown was observed in other locations.

5.5 Movement of adjacent Metro Tunnels

The metro tunnels are located within soil layer no. 4 which is composed of very soft clay with undrained shear strength of about 20kPa.The tunnel structure was found very sensitive to any loading or unloading effect induced by construction activities like demolition of existing structure and ground treatment works at the north end of the site. The cumulative settlement of the up track and down track tunnels after completion of the basement structure at the south pit was 14mm and 4mm respectively. Figure 11a shows the vertical and horizontal movement of the up track tunnel till the end of basement construction at the south pit. Construction of the DCM and JC within the north pit caused soil displacement of the adjacent soil in undrained condition and resulted in upward and lateral movement of the adjacent tunnels. The tunnels would move back as excess pore water pressure gradually dissipated (see Figure 11a). The net settlement caused by excavation in the south pit was about 15mm (see Figure 11b).

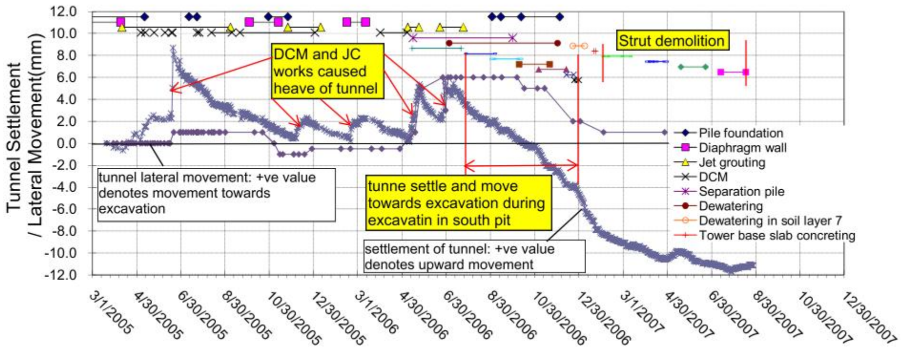
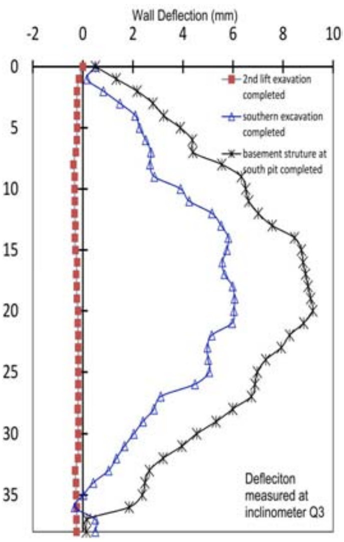
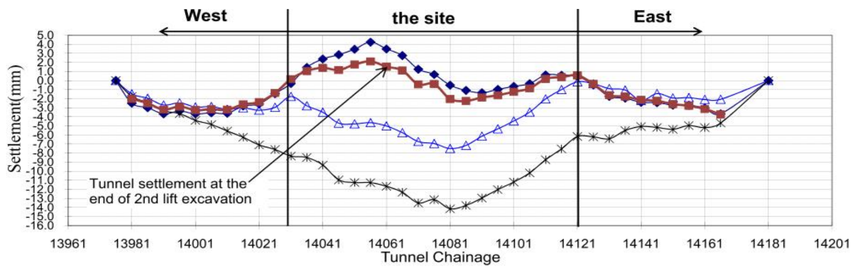


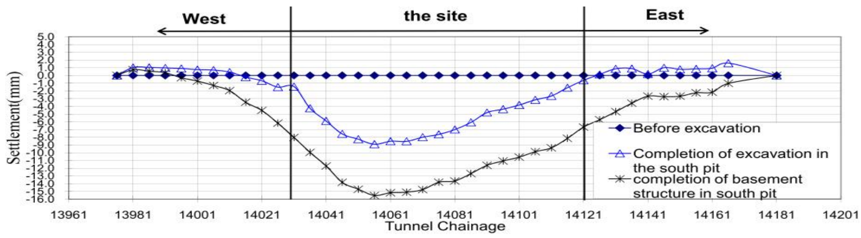
Figure 11a: settlement and lateral movement of up track tunnel (near Q26)



Cumulative deflection of northern diaphragm wall since completion of 2nd lift excavation



Cumulative settlement since construction of diaphragm wall



Cumulative settlement since southern excavation commencing

Figure 11b: Settlement of the up track metro tunnel in relation to deflection of the northern diaphragm wall

6 BACK ANALYSES

6.1 Model Geometry and Parameters

Back analyses were carried out by using commercial software, PLAXIS ver. 8, which is a two-dimensional finite element code developed by PLAXIS b.v. for analysis of geotechnical problem associated with soil-structure interaction, seepage and consolidation. The two cross-sections as shown in Figures 3c and 3d are analysed. The finite element meshes are shown in Figure 12a and 12b. Geotechnical parameters adopted for analysis are summarized in Table 3 and Figure 13. Mohr-Coulomb model is adopted for linear elastic-perfectly plastic analysis.

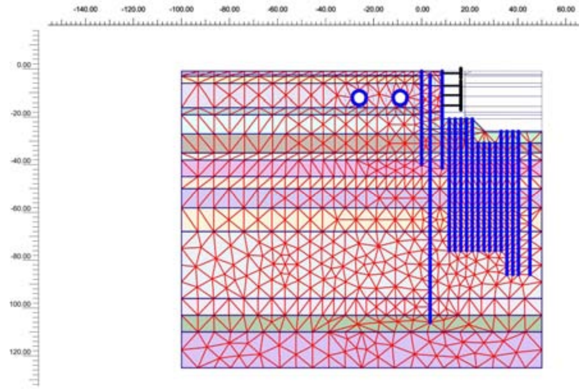


Figure 12a: Finite element mesh of cross section A-A

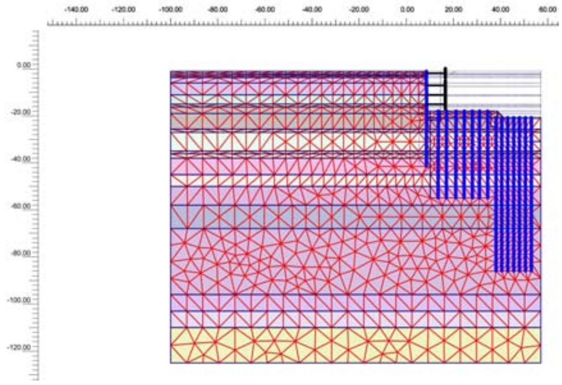


Figure 12b: Finite element mesh of cross section B-B

Table 3: Summary of soil parameters used for numerical modelling

Soil layer	Description	Bulk density	Saturated Density	Undrained shear strength, C_u/ $_v$'	Young's Modulus (undrained), E_u	Poisson ratio (drained), v'
		(kN/m^3)	(kN/m^3)	(kPa)	(kPa)	
2	Brownish yellow silty CLAY	18.3	18.5	0.40	~450C_u	0.35
3	Grey silty CLAY	17.4	17.6	0.35	~400C_u	0.35
4	Grey silty CLAY	16.7	16.8	0.20	~450C_u	0.4
5-1	Grey CLAY	17.5	17.7	0.25	~330C_u	0.35
5-2	Grey silty CLAY	17.9	18.3	0.30	~330C_u	0.35
5-3	Grey silty CLAY with sandy silt	17.9	18.3	0.30	~330C_u	0.30

$_v$' – vertical effective stress

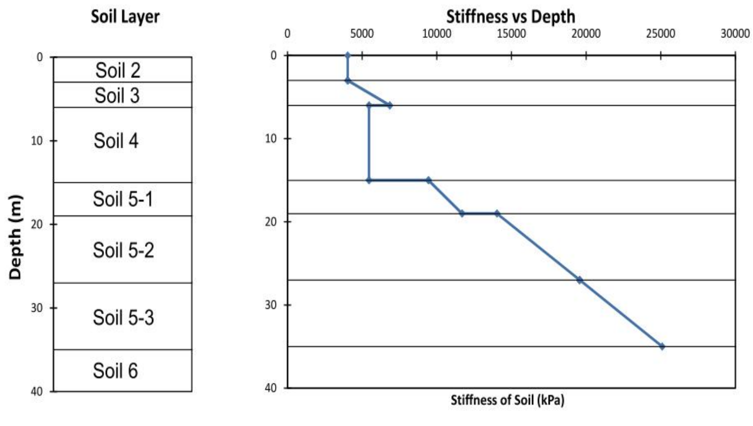


Figure 13: Soil stiffness adopted for back analysis

6.2 Comparison of Calculated Results with Observation

Section B-B represents typical support condition of the southern excavation. The calculated wall deflection of Section B-B agrees with the measured value. The calculated ground settlement is greater than measured value by about 25% and is about 50% of the calculated wall deflection. The measured and calculated settlement behavior is consistent with excavation in soft to medium clay described by Clough & O'Rourke (1990) and Peck (1969). The calculated upward movement of king posts is 18.6mm and is comparable to the measured values shown in Figure 6 and Table 5.

Section A-A is simulating the complicated ground condition between the intermediate and the northern diaphragm walls comprises of soil treated by DCM and JC, as well as a row separation piles. The calculated wall deflection and strut forces do not match very well with the observed behavior. It is because the soil model adopted cannot simulate the inhomogenity and behavior of treated soil. Also, effect of strut arrangement, corner effect and irregular geometry of the northern pit cannot be simulated by the 2-D model.

For the intermediate diaphragm wall, the calculated and observed mode of wall deflection are similar but the curvature at the point of maximum deflection is underestimated by the numerical model. For the northern diaphragm wall, the calculated curvature matches with observation, but the lateral displacement is over estimated.

The calculated settlement of the up track tunnel is 8.6mm and is greater than the measured settlement of 6.9mm. The calculated settlement of down track tunnel is 9mm and substantially greater than the measured settlement of 1.6mm.

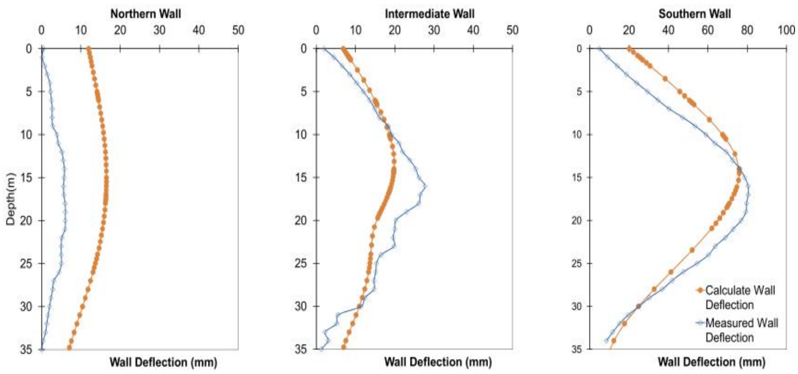


Figure 14a : Comparison of measured and calculated wall deflection at sections A-A and B-B

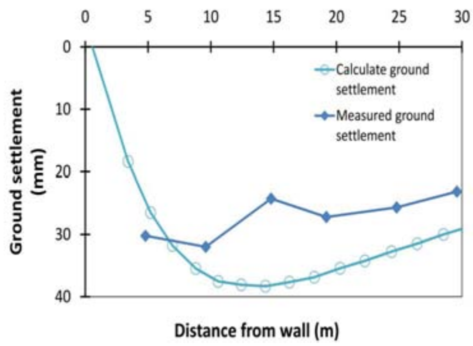


Figure 14b: Comparison of measure and calculated ground settlement at section B-B

Table 4: Summary of calculated and measured strut forces

Strut	Northern Section (A-A)		Southern Section (B-B)	
	Measured	Calculated	Measured	Calculated
1st level	261	382	267	144
2nd level	434	350	609	513
3rd level	303	356	542	606
4th level	194	323	165	162
Total force	1192	1411	1583	1425

Table 5: Summary of calculated and observed upward movement of king post

	Observed (mm)			Back analysis
Depth of Excavation (m)	Minimum	Maximum	Average	
6.5	1.1	5.2	3.1	1.4
10.5	1.1	9.7	6.1	6.8
14.5	1.6	15.2	11	10.8
17.0	4.2	17.8	13.7	18.7
20.0	4.7	18.4	13.9	18.6

Table 6: Summary of calculated and measured tunnels settlement

Depth of excavation (m)	Up Track		Down Track	
	Measured	Calculated	Measured	Calculated
6.5	0.4	3.0	0.0	3.8
10.5	2.0	4.9	0.6	5.5
14.5	4.6	6.4	1.2	6.9
17.0	5.9	7.3	1.2	7.8
19.8	6.1	8.3	1.3	8.8
25	6.9	8.6	1.6	9

6.3 Effectiveness of Ground Treatment Work on Ground Movement Control

Based on the measured wall deflection and strut forces, the back analyzed in-situ stiffness of jet grouted soil at the passive side of the diaphragm wall is about 185MPa and is about 10 times stiffer than the original soil. The back analysed stiffness of jet grouted clay is comparable the laboratory testing results by Zhu (2004). According to numerical analysis result, deflection of the southern diaphragm wall would increase from 85mm to 108mm if jet grouting is omitted (see Figure 15).

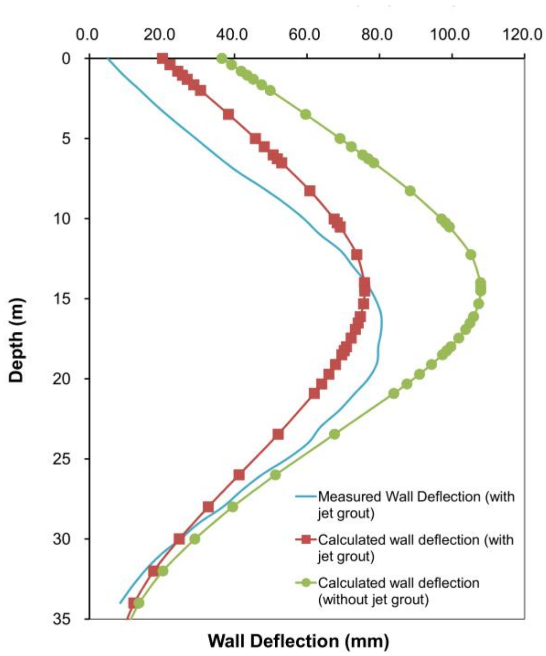


Figure 15:Evaluation of the effectiveness of jet grout at the passive side of the southern diaphragm

6.4 *Stiffness of In-situ Soil*

The shear strains at the active side of the analysed sections A-A and B-B are in the order of 0.1% (see Figures 16a and 16b). It falls into the typical range of stiffness suggested by Mair (1993) that giving reasonable designs for structures in London Clay. The back analysed ratios of Eu/Su adopted for matching the measured wall deflection and strut forces at the end of excavation range from 330 to 450 (see Table 3). These values are smaller than the minimum value of 600 for plasticity index of less than 30, determined from the correlation suggested by Duncan & Buchignani (1976).

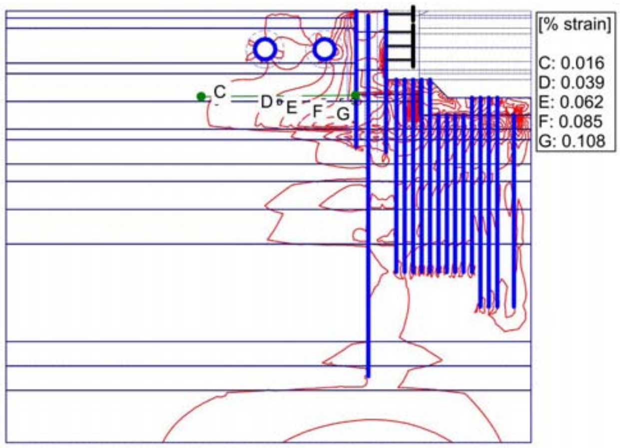


Figure 16a: Shear strain (section A-A)

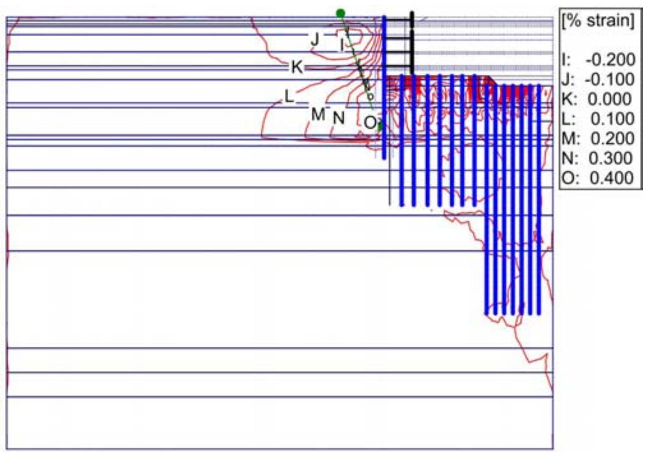


Figure 16b: Shear strain (section B-B)

7 CONCLUSIONS

- The excavation strategy adopted was to minimize the time dependent deflection of diaphragm wall in order to mitigate the potential tunnel movement due to the excavation works in its proximity.
- Soil improvement by means of jet grouting and deep cement mixing was found to be effective in control of ground movement in the soft deposits in Shanghai.
- Comprehensive geotechnical instrumentation and monitoring plan, and qualified supervision were essential to safe

execution of complicated geotechnical works adjacent to sensitive structures.
- The back analysed ratios of Eu/Su range from 330 to 450 with estimated shear strain in the order of 0.1% in the soil around the excavation.
- Numerical modeling by adopting Mohr-Coulomb model for linear elastic-perfectly plastic analysis can reasonably evaluate the behavior of excavation to provide essential information on ground movement, structural deformation and stability of the excavation.
- Understanding of numerical modeling and soil behavior are of the essence in geotechnical design by using numerical modeling.

ACKNOWLEDGEMENTS

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Geotechnical Design and Construction Considerations for the Adelaide Desalination Plant Shafts, Australia

A.D. Mackay
Snowy Mountains Engineering Corporation, Hong Kong

ABSTRACT

Due to the shortage of water replenishing the existing reservoirs from the Murray River, Adelaide, Victoria, Australia, it was decided that a desalination plant was required to supply additional water. The construction is presently underway and is due for completion late 2011. The plant is located on the coastline at Port Stanvac, Adelaide and, due to the inaccessible terrain in this area a large portion of the desalination processing plant has been accommodated below the ground surface. The sub-surface structures include two 4m outside diameter water intake and discharge tunnels, extending for about 1.3 and 1.05 kilometres beyond the coastline respectively. The tunnels join at a 10m diameter, 50m deep working shaft which in turn connects to two 45m deep outfall and intake shafts. The intake shaft extends into a cavern of dimensions 60m length, 15m width and 25m depth, used for initial salt water processing prior to discharge via riser pipes to the surface. This paper discusses the geotechnical considerations for the working, outfall, intake and riser shaft design and construction. All shafts were excavated within weak rock prone to variable weathering, referred to as the Brachima Formation. The rock has variable physical properties dominated by the north east to south west trending sub-vertical bedding, and localized weakening affected by the presence of persistent shear zones. An emphasis on the variable geotechnical properties and their impact on the shaft design and construction is provided.

1 INTRODUCTION

The Adelaide Desalination Plant will eventually supply about 50 billion litres of water, or about 25% of Adelaide's annual water demand, and will commence operation towards the end of 2011. The surface desalination plant will occupy an approximate 40 hectares site and water processing will be carried out by drawing water in through the intake tunnel extending offshore. As the salt water enters the cavern wet well it is processed using reverse osmosis techniques, which strain the majority of the salt and impurities by passing the water through membranes at high pressure, after which it is discharged to the surface via riser shafts for further processing. Following desalination the water is discharged offshore through the outfall shaft and discharge tunnel. Refer to Figures 1 to 6 for the site location and the desalination plant configuration. All shafts were constructed within the Brachima geological formation, the temporary support design was based on the Norwegian Geotechnical Institution (NGI) Q rock mass classification (Barton 2002) and final lining installed using either fibre reinforced shotcrete or cast in-situ concrete. This paper describes the design and construction considerations for the shafts formed for the desalination plant.

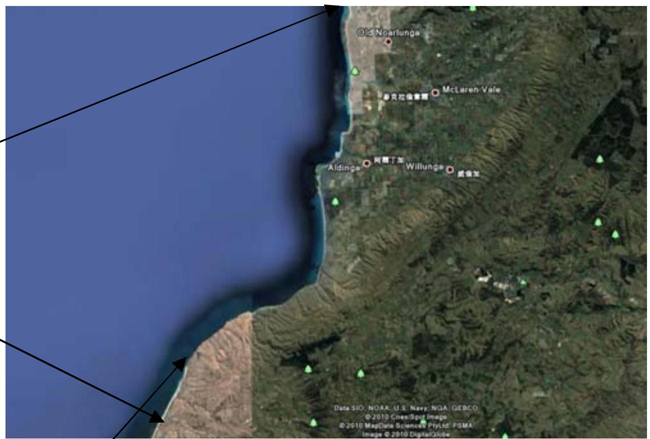

Figure 1: location of the Adelaide Desalination Plant

Figure 2: Location, Port Stanvac

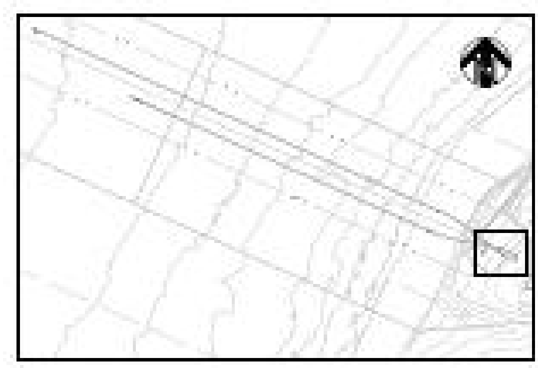

Figure 3: General plan, Adelaide Desalination Plant.

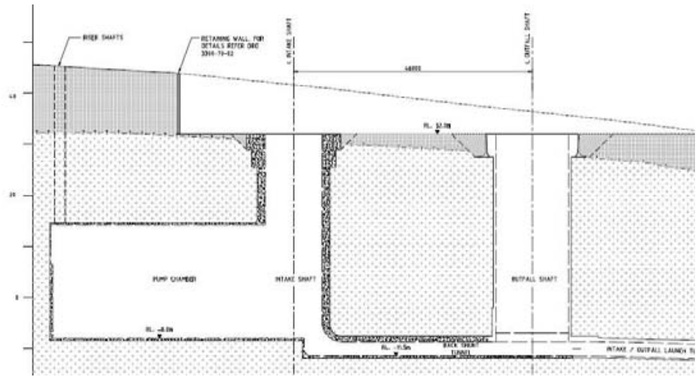

Figure 4: Section through intake and working shafts

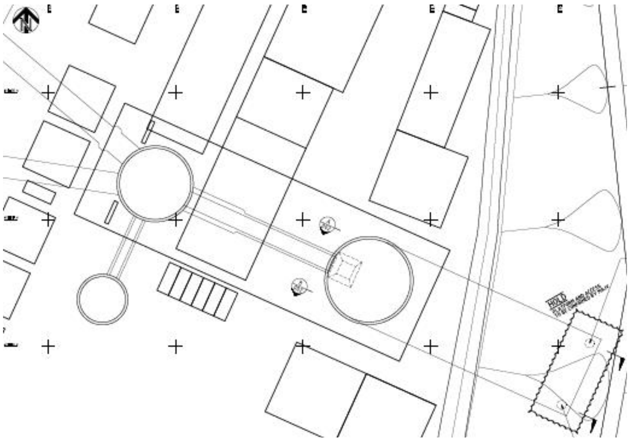

Figure 5: Shaft locations, lower levels

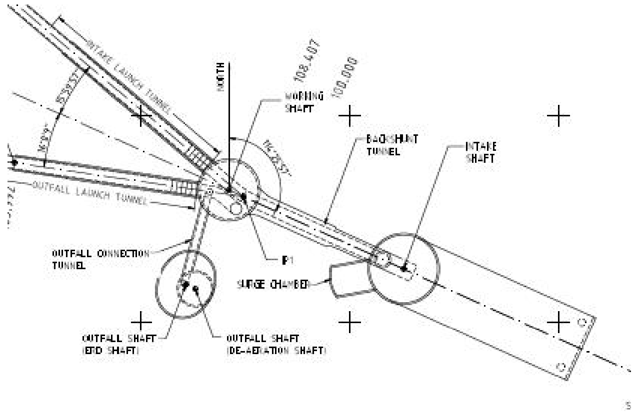

Figure 6: Shaft locations, upper levels

2 SITE SETTING

The Port Stanvac site is located immediately inland from the Gulf of St Vincent and was chosen due to its relatively deep sea water location, marine dispersion characteristics, access to the Adelaide water supply network, land availability and construction costs. The coastline has a north north east to south south west (NNE – SSW) trend and is bounded by a 20m vertical rock precipice. The ground surface rises at a gentle gradient of between 5 to 10 degrees from the coastline. To allow construction of the shafts and cavern the site was leveled in the vicinity of the shafts and above the cavern footprint. Refer to Plate 1 for the site formation and Plate 2 for the Intake Shaft excavation.

Plate 1: Shaft locations

Plate 2: Working shaft excavation

3 GROUND AND GROUNDWATER CONDITIONS

The geology in the vicinity of the shaft and cavern footprint comprises superficial deposits of sand and clay outwash and lateritic sands and gravel, over the Brachima geological formation, comprising siltstone, meta-siltstone with occasional sandstone and quartz veining. The superficial deposits were removed during preparation of the leveled site formation. The main geological structures in the area include an asymmetrical anticline, with a fold hinge located about 1.5 kilometres inland with a NNE to SSW trend, similar to the coastline orientation, and the Eden – Burnside Fault, which is located about 2km offshore, has a NE to SW trend, dips sub-vertically and has a significant displacement of about 70m (Thomson et al. 1962). The geological discontinuities are strongly influenced by both structural features; as the site is located on the western limb of the anticline, the bedding dips 60 to 80 degrees towards the NW.

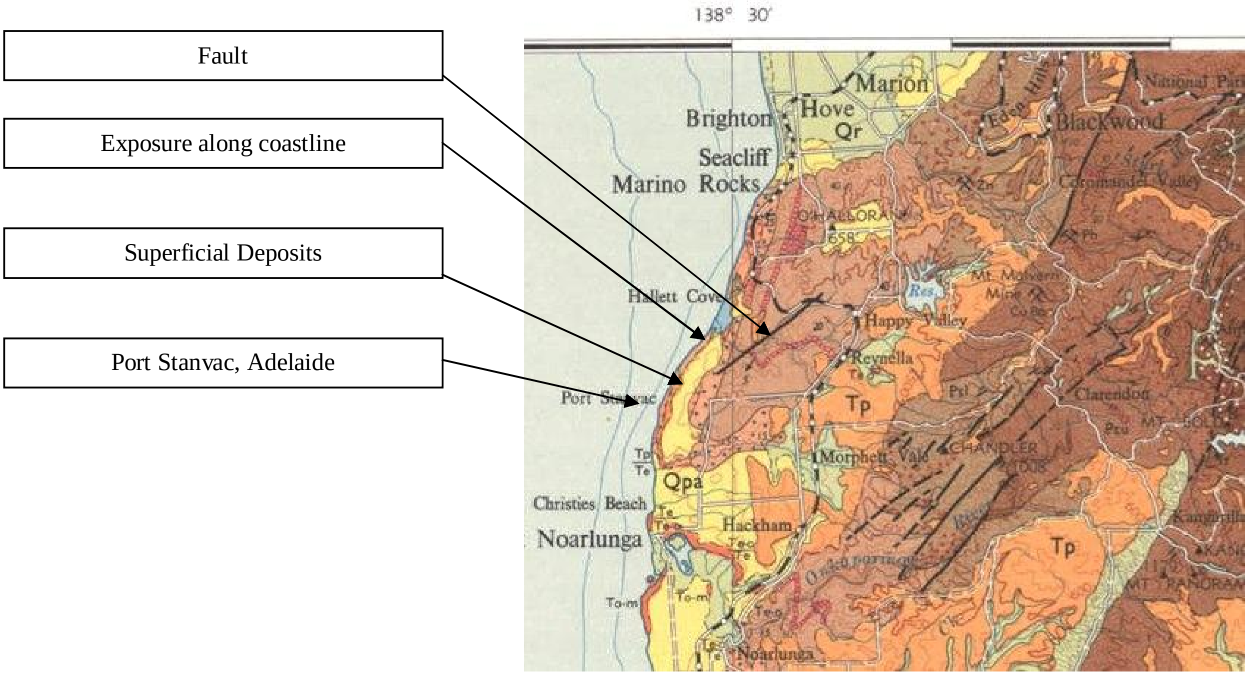

Figure 7: General geological conditions (Port Stanvac)

The ground investigation (GI) included vertical and inclined drilling with hydraulic conductivity and discontinuity survey in-situ testing and Point Load, Unconfined Compressive Strength (UCS) and Poisson's Ratio rock laboratory testing. The findings of the GI are summarized in Table 1.

Table 1: Findings from the site investigation

Feature	Summary
Zones of weakness	Five potential shear zones, with thickness ranging between 30 to 200mm, were identified potentially intercepting the cavern and shafts. The majority of these shear zones were orientated parallel to the bedding; one shear zone was orientated sub-parallel to the sub-horizontal joint set.
UCS values	The results were influenced by a high degree of anisotropy, strongly influenced by the bedding and ranged from 25 to 200 MPa.
Hydraulic Conductivity	The permeability values were estimated from about 30 test results taken between 12 to 45m below the ground surface and ranged between 1 to 5 Lugeons. The results were strongly influenced by the discontinuity intensity over the test range.
In-situ stress field	The results were estimated from empirical data taken from results in the vicinity and had a strong influence from the tectonic setting. The maximum horizontal to vertical stress ratio was 2, ranging from 1.5 to 3 and orientated east to west; the minimum stress ratio was 1.5, orientated north to south.
Discontinuity sets.	Estimated from available exposures and borehole televiewer surveys carried out in vertical drillholes and therefore had bias from the discontinuity intercept. The results revealed a strong correlation between the main geological structures present in the area and the discontinuity orientation. The dominant discontinuity set included the bedding planes, referenced J1, dipping sub-vertically and trending NE to SW. Two minor joint sets, referred to as J2 and J3, were identified trending trending NW to SE and dipping a sub vertically and sub horizontally respectively.
Groundwater level	The groundwater level had a consistent rise of about 6m vertical for every 100m from the coastline and ranged in level between +1 to +6m above datum in the vicinity of the shafts. Due to low rainfall levels in the area, groundwater fluctuations were minimal.
Rockhead level.	Rock-head referenced as being moderately weathered rock, which was encountered at a consistent depth between 8 to 12 m below ground surface, defined as rock mass classification, Brachima Formation (BF), 3a or better. Notwithstanding the weathering front was highly variable resulting from the presence of differential weathering along the sub-vertical discontinuities and zones of increased fracture and decreased discontinuity spacing.

As the rock mass condition was revealed to be consistent, based on the findings of the site investigation and subsequent geological mapping, to allow design parameter interpretation, the rock was classified into Brachima Formation (BF) rock mass units, as summarized in Table 2.

Table 2: Summary of interpreted rock mass units and parameters for use in the design

Rock Mass	Description, based on British Standard (BS 5930 1999)	Density (kN/m^3)	UCS (MPa)	Cohesion (kN/m^2)	Defn Mod. (MPa)
BF1	High to very high strength, slight to fresh weathering, slight fracturing and 300 to 600mm discontinuity spacing	27 - 28	50 - >200	3000	28000
BF2	High to very high strength, moderate weathering, slight fracturing and 100 to 300mm discontinuity spacing	26 – 27	25 – 50	1850	16500
BF2a	High to very high strength, slight weathering, slight fracturing and 100 to 300mm discontinuity spacing	26 - 27	50 - 100	-	-
BF3a	High to very high strength, moderate weathering, 30 to 100mm discontinuity spacing, locally more intensely fractured.	26 – 27	25 – 50	500	3800
BF3b	Very low to very high strength slightly weathered and highly fractured (SHEAR ZONE).	27 - 28	>100	500	1500
BF4	Low to high strength moderately weathered and highly fractured.	23 - 25	25 - 50	250	1200

To verify the findings of the site investigation, detailed geological face mapping was carried out to ascertain the ground model and the NGI Q value for temporary support. Refer to Plates 2 and 3 for the ground conditions encountered.

Plate 3: Working shaft exposure, BF3a

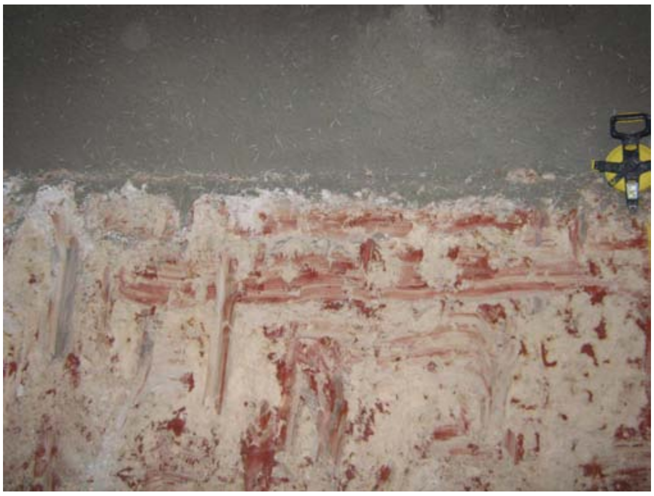

Plate 4: Working shaft exposure, silty clay, uppermost levels

4 THE SHAFTS AND CONSTRUCTION METHODOLOGY

A summary of the purpose, configuration and ground conditions for each shaft is summarized in Table 3:

Table 3: Shaft summary

Shaft	Purpose	Configuration	Interpreted Ground Conditions
Working	Temporary shaft to allow launch and retrieval of the Tunnel Boring Machines used to form the water intake and discharge tunnels.	15m inside diameter (ID) to 10m depth; 14m ID below this level.	0-3m silty clay, 3-10m BF4, 10-25m BF2a with shear zones and 25 – 52m BF1/2 with potential shear zones
Outfall	Allow discharge from the surface to the discharge tunnel. Due to the space and hydraulic requirements the uppermost shaft had an elliptical configuration and the lower shaft circular.	Uppermost shaft 16m max. and 13m min ID to 19m depth; 8m ID below this level.	0-3m silty clay, 3-8m BF4, 8-25m BF2a and 25 – 45m BF1/2 with potential shear zones
Intake	Allow access to the desalination plant within the cavern. As compatibility between the gantry crane, to be installed within the uppermost levels of the cavern, and lifting to the surface was required the shaft and cavern diameter had the same dimensions.	16.36m ID, compatible with the cavern ID.	0-6m silty clay, 6-11m BF4, 11-17m BF3a, 17 - 25m BF2a and 24 – 40m BF1/2
Riser	To discharge water from the cavern to the surface. There riser shafts located at the end of the cavern and are spaced approximately 15m c/c.	Both shafts have 1.8m ID	0-6m silty clay, 6-11m BF4, 11-17m BF3a, 17 - 25m BF2a and 24 – 40m BF1/2

The construction methodology for the working, outfall and intake shafts involved initial installation of a 0.5m thick, reinforced concrete (RC) collar to 2m depth achieving a 20MPa final strength. The subsequent foreshaft construction, in soil comprising silty clay and rock mass unit BF4, was then carried out in 1.5m deep advances with installation of Fibre Reinforced Shotcrete (SFR) with timber block off at the base. Upon reaching competent rock, i.e. BF3a or better, the advancement depth would be increased depending on the Q value encountered and the method of advancement, using a greater advancement rate for mechanical and reduced rate for drill and blast excavation. Sub- vertical spiling bars were installed at 0.5m c/c within the circumference at the base to act as tie backs for the shotcrete. Upon completion of excavation cycle the Q value would be assessed and suitable rock bolt installation allocated accordingly.

The riser shafts were excavated using Reverse Circulation Drilling (RCD) piles with a steel casing temporary support installation. A Glass Fibre Pipe (GRP) final lining was then installed and backfilled with 0.5m thick concrete annulus with grout tubes for later grout injection. The shaft was then backfilled to allow cavern excavation beneath the shaft invert to be carried out. Upon exposure of the shaft invert an RC collar with rock bolt tie backs was installed to provide lining support from the base.

5 TEMPORARY SUPPORT DESIGN

The Q values estimated from the site investigation revealed a high degree of variability, mainly due difficulties in interpretation from core loss, bias of the televiewer survey results and relatively limited data retrieved from drillholes; local decreases to less than 0.1, influenced by intense fracture and discontinuity spacing, were revealed. The Q value estimation following excavation however showed a greater consistency with fewer abrupt decreases in the estimation, often associated with the presence of intensely fractured shear zones. The shear zones and potential block failures indentified from the mapping within the shaft were supported by suitable rock bolt installation and shotcrete passing through the rock block or spanning across the shear zone. Refer to Figures 7 to 10 for the rock bolt installation used to support loose blocks and shear zones.

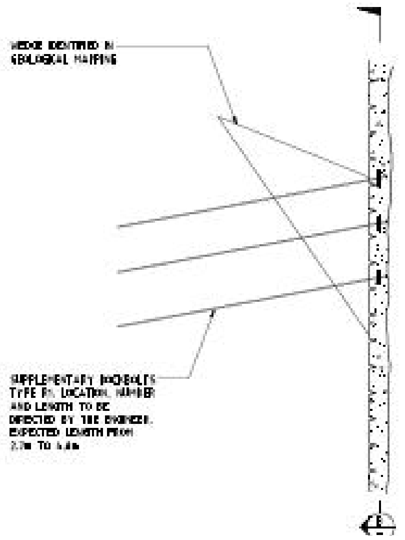

Figure 7: Section, wedge support

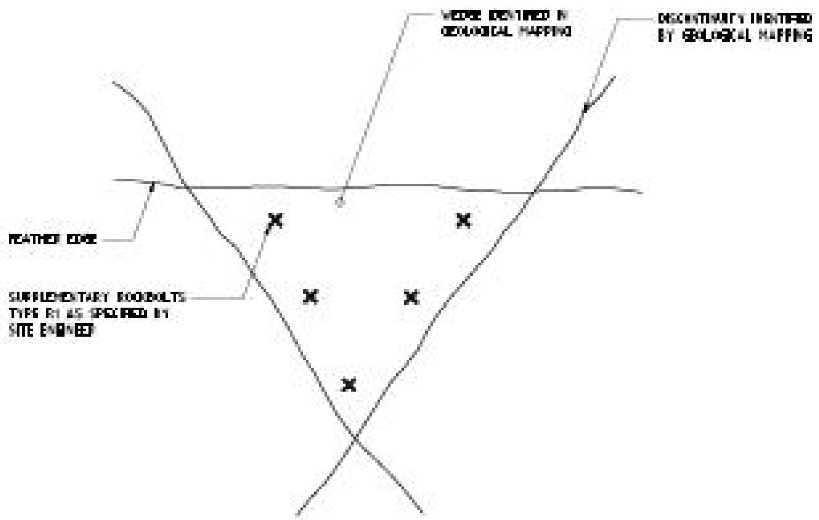

Figure 8: Plan, wedge support

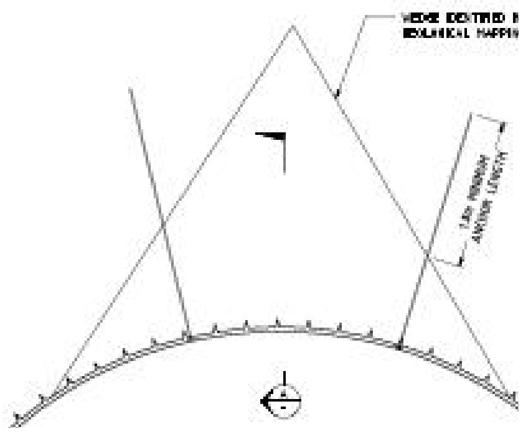

Figure 9: Section, wedge support

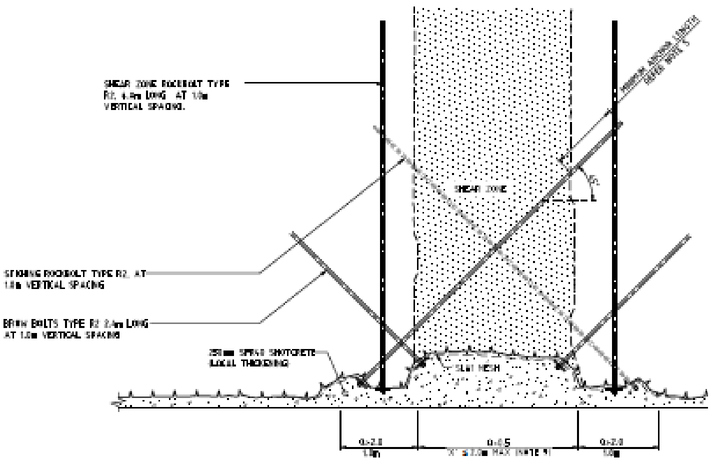

Figure 10: Section, shear zone support

6 PERMANENT LINING DESIGN

The lining design included cast in-situ concrete within the intake shaft above Reduced Level +16m and within the riser shafts. The remaining shafts were supported by SFR, which had the advantage of gaining strength and allowing deformation with time, thickness adjustment to suit the ground conditions encountered and having a quantifiable tensile strength depending upon the quantity of fibre added.

7 CONCLUSION

The accommodation of the Adelaide Desalination Plant below ground surface is an example of an innovative solution to increase the efficiency of the de-salination process. In addition the construction and design verification in potentially complicated ground conditions was carried out with an efficient interface between the Contractor, designer, Independent Checking Engineer and other interested parties as the works progressed.

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