

Design of Retaining Wall and Support Systems for Deep Basement Construction – A Malaysian Experience

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Abstract: The design of retaining walls and support systems for deep basement construction requires careful analysis, design and monitoring of performance. This is especially critical for deep basement construction in urban areas where the need for space and high land prices justify the deep basement construction. Due to the close proximity of existing buildings in urban areas, careful selection of suitable retaining walls and support systems is important, taking into consideration criteria such as control of ground movement, lowering of the groundwater table, encroachment into neighbouring land, etc. The design of retaining walls and support systems requires careful evaluation of various possible failure modes, such as overall stability, basal heave failure, hydraulic failure, structural failure, etc. In addition to conventional “working state design”, the assessment of associated ground movements due to deep basement construction is also important to ensure neighbouring structures are not affected. The risk associated with deep basement construction works is high as failures of retaining wall or support systems will be catastrophic and will affect surrounding areas. As such, the design of retaining walls and support systems for deep basement construction works requires careful consideration of soil-structure interaction and this is usually accomplished using the finite element method (FEM). However, the use of the finite element method (FEM) requires proper understanding of the limitations associated with the method and also proper modelling of the structures in order to make a representative analysis. This paper presents design approaches commonly used to assess various potential failure modes, serviceability limits and recommended guidance on the use of finite element method for analysis and design of retaining walls and support systems for deep basement construction. In this paper two case histories on deep basement construction works are also presented.

1 INTRODUCTION

Due to scarcity of land, especially in urban areas, the need for basements to optimize the use of land has resulted in increasing depth of basements being constructed. In this paper, the approximate division between shallow and deep excavation is based on 6m which is guided by the definition used by CIRIA (Irvine & Smith (1992)) on trenching practice and Puller (1996). The design of retaining walls and support systems for deep basement construction requires careful analysis, design and monitoring of performance. This is because the risk associated with the works is high and recent high profile failures involving deep excavation (e.g. Nicoll Highway, Singapore ([Figure 1](#)) and Shanghai Metro, China) have highlighted the need for proper design and construction control. A recent study by Moh & Hwang (2007) has listed 43 failures since 2001 related to MRT works of which 8 failures were related to retaining walls and strutting works and some of the failures have resulted in death, collapsed buildings and economic losses in millions. Some of the recommendations by Moh & Hwang (2007) include having a proper risk management program associated with underground works and a sound understanding of geotechnical fundamentals to complement the use of computer codes. Proper implementation of risk management programmes and the use of computer codes require sound understanding of the design and construction considerations of underground works in order for the risk management to be effective and computer codes used properly. As such, this paper intends to highlight some of the important aspects of Malaysian experience on design of retaining walls and support systems for deep basement construction to ensure a safe and economical design.

2 DESIGN CONSIDERATIONS

In this paper, a brief discussion on the planning of subsurface investigation and testing and selection of retaining walls and support systems will be presented followed by a more detailed discussion of the design of retaining walls and support systems for deep basement excavation. The design of retaining walls and support systems for deep basement excavation will cover the following aspects:

- a) Overall stability
- b) Basal heave failure
- c) Hydraulic failure
- d) Axial stability
- e) Finite element analysis
- f) Ground movement associated with excavation

A short discussion of steel design for struts and some design aspects of reinforced concrete retaining walls is also presented. At the end of the paper, two case histories are presented to illustrate typical deep basement construction works in Malaysia.

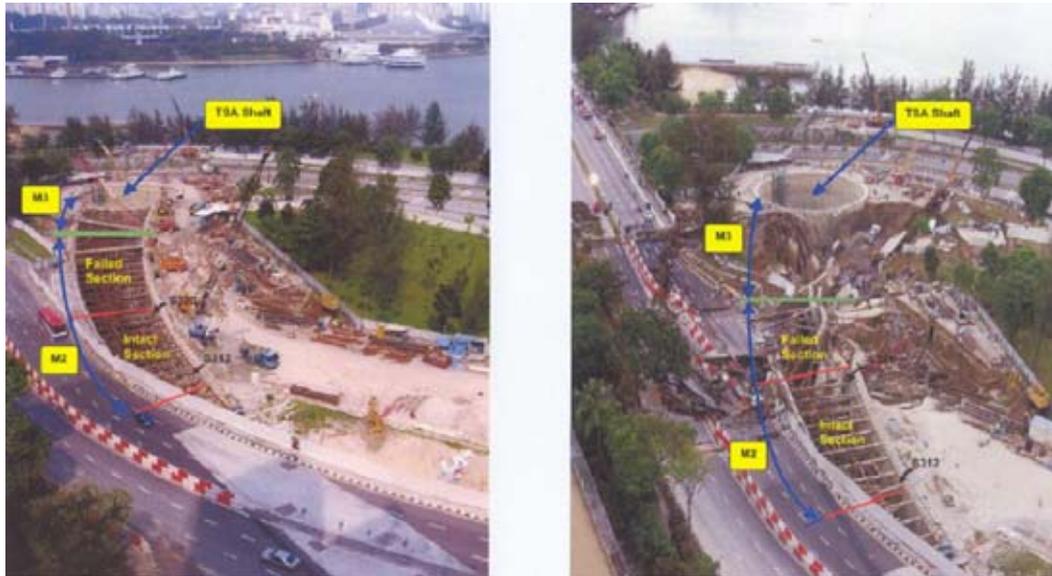


Fig. 1. View of Nicoll Highway site before (picture on the left) and after collapse (picture on the right) (COI (2005)).

3 PLANNING OF SUBSURFACE INVESTIGATION AND TESTING

Proper planning and supervision of subsurface investigation (SI) are of utmost importance to the designer in order to produce a safe and economical design for deep basement excavation. In this paper, a thorough discussion of the planning of SI, field and laboratory testing will not be included and interested readers may refer to publications by Geotechnical Control Office of Hong Kong (GEOGUIDE 2: Guide to Site Investigation) and Clayton et al. (1995). Generally the following soil parameters should be obtained from the SI:

- a) Shear strength parameters of soil (ϕ' and c')
- b) Stiffness of soil (E')
- c) Permeability of soil (k)
- d) Groundwater level

The above information is usually obtained from routine SI programmes except for soil stiffness which requires special testing techniques and interpretation of results. The use of pressuremeter tests is recommended to obtain representative soil stiffness values for design. Further discussion of the use of appropriate soil stiffness values will be presented in the next section. Recent advances in the use of seismic piezocone (e.g. Mayne (2000)) and seismic test (e.g. Massarsch (2004)) appears promising where small-strain stiffness can be obtained for design.

4 SOIL PARAMETERS FOR DESIGN OF RETAINING WALLS AND SUPPORT SYSTEMS

The design of retaining walls and support systems requires careful selection and interpretation of the appropriate soil parameters to be adopted. Some of the important soil parameters are discussed in the following sections.

4.1 Shear strength parameters

In Malaysia, the effective shear strength parameters of the soil (ϕ' and c') are commonly obtained from Isotropically Consolidated Undrained Triaxial (CIU) Test with pore pressure measurements. If a finite element is used, understanding the constitutive models and numerical algorithms adopted in the finite element software is important in order to model the problem appropriately. For example, for PLAXIS analysis, the following are recommended:

- a) Hardening soil model should be used to model excavation problems, as the conventional Mohr-Coulomb model is unable to model unload-reload problems properly. Mohr-Coulomb model is based on elastic behaviour and is unable to model density and shear hardening which renders it inaccurate for deformation problems.
- b) For undrained behaviour analysis, assumption of dilatancy angle has serious effects on results. Careful selection of appropriate dilatancy angles is important.
- c) Modelling of undrained behaviour is recommended to be performed in effective stresses and with effective stiffness and strength parameters, if possible.
- d) If information on effective strength parameters is not available, undrained strength parameters ($c = c_u$, $\phi = 0$, $\psi = 0$) with effective stiffness parameters can be used. Proper understanding of the constitutive soil models is important and further discussion of the undrained modelling of excavation problems will be presented in Section 7: Finite Element Analysis of Retaining Walls for Deep Basement.

The conversion of shear strength parameters between undrained strength (c_u) and effective strength parameters (ϕ' and c') should always be checked as per Figure 2 to ensure the parameters adopted are reasonable.

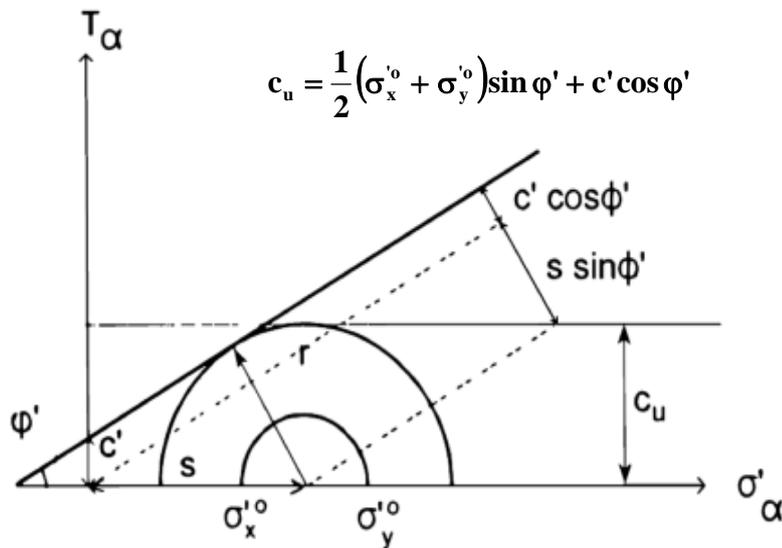


Fig. 2. Mohr circle for evaluating undrained shear strength (plane strain) (Tan,2007).

4.2 Soil permeability

For an economical design where coupled consolidation analysis is carried out in a finite element analysis, the soil permeability (k) is important to ensure the drained or undrained behaviour of the soil is modelled correctly. In-situ tests are recommended in order to take into account the complex soil stratigraphy at site which is not capable of being reproduced in the lab. Either rising, falling or constant head tests can be carried out. The values obtained should be compared to published values as a check to ensure the values obtained are reasonable for the given soil conditions. Figure 6 of BS8004: 1986 is useful as a simple check and it is reproduced here as Figure 3.

		Permeability k (m/s)											
		1	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}	10^{-10}	
Drainage		Good						Poor		Practically impervious			
Type of soil		Clean gravels		Clean sands and sand-gravel mixtures			Very fine sands, silts and clay-silt laminate		Unfissured clays and well mixed clay silts containing more than 20 % clay				
		Desiccated and fissured clays											
Recommended method of determining k		Pumping tests in situ						Flow from piezometer tips					
		Constant head permeameter tests						Equilibrium		Non-equilibrium			
		Estimation from grading curves											
		Falling head permeameter						Computed from oedometer or triaxial consolidation tests					
		Very reliable				Reliable							

Fig. 3. Permeability and drainage characteristics of soils (BS8004: 1986).

4.3 Soil stiffness

Current practice for estimation of soil stiffness is usually based on empirical correlations. This is because routine laboratory tests give soil stiffness parameters which are significantly less than the stiffness values derived from back analysis of field measurements. This is primarily due to disturbance to the soil samples and also testing at strain levels which are larger than the range which is appropriate for retaining walls. This is illustrated in Figure 4 which shows the strain dependent characteristics of soil stiffness.

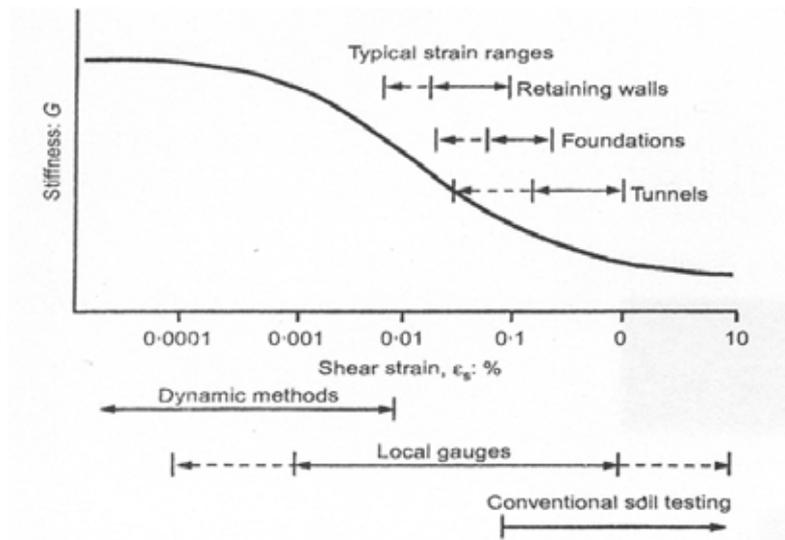


Fig. 4. Characteristic stiffness-strain behaviour of soil with typical strain ranges for laboratory tests and structures (Atkinson, 2000).

From Figure 4, it can be seen that the strain levels for retaining walls is relatively small compared to foundation and tunnel problems. As such, the use of dynamic testing methods or local gauges is recommended to obtain representative small-strain stiffness for design. As the use of local gauges is limited in Malaysia and generally requires a sophisticated testing laboratory, the use of in-situ testing such as seismic piezocone or seismic tests is recommended. However, the use of such in-situ tests is still limited in Malaysia and currently, the use of empirical correlations is the norm. Various empirical correlations are available to determine small-strain stiffness for design (e.g. Hardin (1978), Burland & Kalra (1986) and Tan (2001)). However, the designer should be aware of the basis of the empirical correlations as it is highly dependent on factors such as local soil conditions, constitutive models adopted and finite element programs used.

5 THE SELECTION OF TYPES OF RETAINING WALLS AND SUPPORT SYSTEMS

Various types of retaining walls and support systems can be adopted for deep basement construction. The selection is usually made on the basis of:

- a) Foundation of adjacent properties and services
- b) Designed limits on walls and retained ground movements
- c) Subsoil conditions and groundwater level
- d) Working space requirements and site constraints
- e) Cost and time of construction
- f) Flexibility of the layout of the permanent works
- g) Local experience and availability of construction plant
- h) Maintenance of the walls and support systems in a permanent condition

Some of the retaining wall systems which are commonly used in Malaysia for deep basement construction are as follows:

- a) Steel sheet pile walls
- b) Soldier pile walls
- c) Contiguous bored pile walls (CBP walls)
- d) Diaphragm walls
- e) Secant pile walls
- f) Soil nail walls

For discussions of the advantages and disadvantages of the various wall systems, please see Gue & Tan (1998b) and Puller (1996).

For support systems, the following are commonly used in Malaysia:

- a) Internal horizontal steel struts
- b) Inclined steel struts
- c) Ground anchors
- d) Soil nails
- e) Top-down construction using floor slabs and structural frames as support

Support system selection is commonly affected by the following factors:

- a) Width of excavation
- b) The number of times it is necessary to move the props during construction
- c) The ease with which materials can be excavated
- d) Limits placed on walls and ground movements
- e) The distance that the props will span across the excavation
- f) The ease of fabrication of props
- g) Availability of props
- h) Build-ability

Further details on the selection and design of temporary propping systems can also be found in Institution of Structural Engineers (1975), CIRIA C517: Temporary propping of deep excavations – guidance on design (Twine & Roscoe (1999)) and Gue & Tan (1998b).

Other than the factors listed above, it is very important to note that even though for most of the time a ground anchor support system appears attractive where it offers unobstructed excavation for basement construction, the designer should be aware of the following factors prior to adopting the system:

- a) Permanent ground anchors always pose great difficulties in long-term maintenance. Reference can be made to BS8081 for further details. Therefore, it is not recommended to use permanent ground anchors unless no other options are available. In addition, a strict long term maintenance scheme should be in place and be strictly followed to prevent failure.
- b) If local authorities require temporary ground anchors to be removed after use, the removal of temporary ground anchors may pose problems if the system has not been proven at site to be fully removable. The use of U-turn temporary removable ground anchors which is quite common in Malaysia appears promising and reference can be made to Chua & Prasanthee (1997).
- c) Approval from adjacent land owners should be acquired if there is encroachment of ground anchors into adjacent properties. Checks should also be carried out to ensure the ground anchors do not affect any services or utilities, foundations, etc. at the adjacent land.
- d) Leakages and loss of fine (especially in loose sandy ground) through drill holes need additional precautionary measures during construction (e.g. full casing, etc).
- e) Proper sealing of temporary ground anchor holes upon completion of permanent basement construction works is important to ensure water-tightness.

6 DESIGN OF RETAINING WALLS FOR DEEP BASEMENTS

6.1 Overall stability

The overall stability of retaining walls is often evaluated using the limit equilibrium method of analysis where the conditions of failure are postulated, and a factor of safety is applied to prevent its occurrence. This is to ensure the provision of sufficient embedment depth to prevent overturning of the wall and to ensure overall slope stability. For excavation in soft ground, the Strength Factor Method as recommended by Padfield and Mair (1984) can be used to determine the penetration depth of the wall only. Limit equilibrium slope stability analysis is also carried out to check for both potential circular and non-circular slip failure using Bishop's simplified method and Spencer's method respectively. The Factor of Safety (FOS) adopted is 1.2 for short term and less critical structures while an FOS of 1.4 is adopted for long term or high risk to life structures. If a finite element computer program is available, it can also be used to carry out the check on overall stability. Figure 5 shows the examples of overall stability that need to be checked in design.

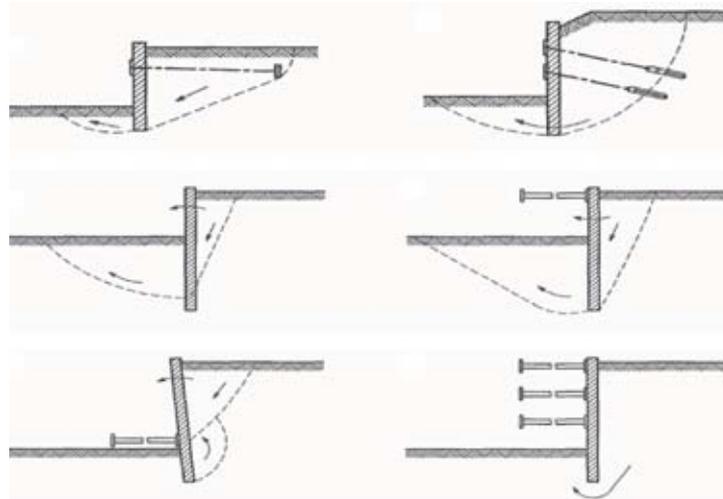


Fig 5. Examples of overall stability failures (extracted from EN1997-1:2004)

6.2 Basal heave failure

Basal heave failure is particularly critical for deep excavation in soft ground and less prone in stiff soils. The basal heave failure is analogous to a bearing capacity failure, only in reverse where the stresses in the ground are being relieved rather than increased. There are many methods to examine basal heave failure and they can be broadly divided into methods based on bearing capacity formulae and methods based on examination of moment equilibrium. The method based on bearing capacity formulae by Terzaghi (1943) is suitable for shallow and wide excavations while the method of Bjerrum & Eide (1956) is suitable for deep and narrow excavations. Based on the Authors' experiences, the moment equilibrium method is generally sufficient to check against basal heave failure. Figure 6 shows the moment equilibrium method. The required factor of safety (FOS) is 1.2 where the vertical shear resistance along retained ground shallower than the excavation is ignored (Kohsaka & Ishizuka (1995)).

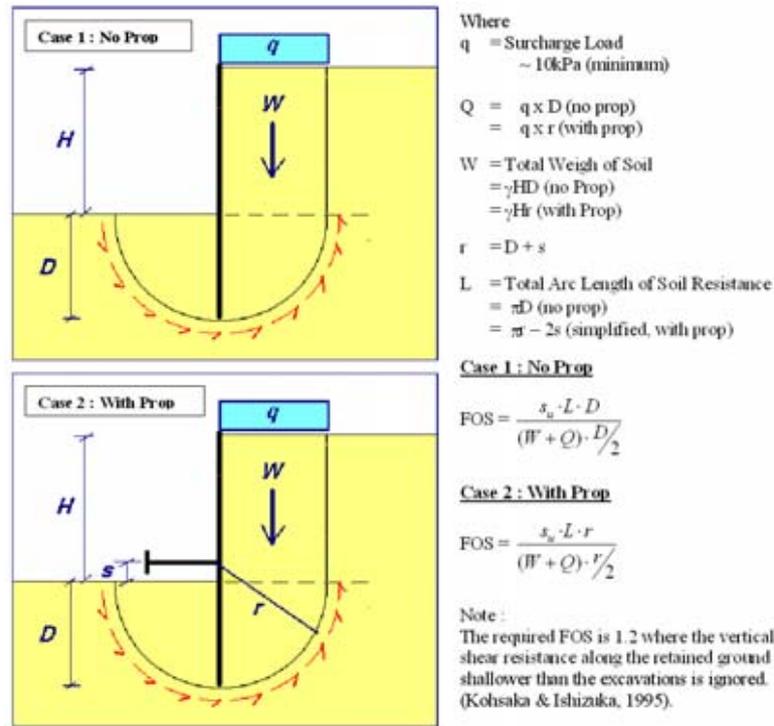


Fig. 6. Basal heave check based on the moment equilibrium method.

6.3 Hydraulic failure

For excavation at sites with groundwater on the retained side above the base of excavation or where artesian pressure is present, a hydraulic failure check needs to be carried out. If the toe of the wall does not penetrate into an impermeable layer or to a sufficient depth, base instability caused by piping will occur if the vertical seepage exit gradient at the base of the excavation is equal to or less than unity.

The potential for hydraulic failure of an excavation can be checked using Terzaghi's method, or the critical hydraulic gradient method which mainly considers vertical flow in the vicinity of the excavation bottom. Figure 7 illustrates the above two methods. A survey of common methods used in Japanese practice (Kohsaka & Ishizuka, 1995) indicates that the factor of safety (FOS) adopted for Terzaghi's method ranges from 1.2 to 1.5. For the critical hydraulic gradient method, the suggested FOS is 2.0.

Recent research by Tanaka & Verruijt (1999) has shown that the critical hydraulic gradient method is unconservative for situations where $D/T > 0.5$ (D – wall penetration depth, T – thickness of permeable layer). As such, Terzaghi's method is recommended for hydraulic failure checks for routine cases of one-layered soil. For other cases such as one-layered anisotropic soil, two-layered soil, or a one-layered soil with a loaded filter, the prismatic failure concept can be adopted (Tanaka & Verruijt, 1999).

To prevent heaving due to artesian pressure, equilibrium between overburden pressure and pore water pressure at the top surface of a confined aquifer (bottom surface of clayey soil) needs to be evaluated as shown in Figure 8. As this method only examines the balance of weight and does not consider mechanical properties of soil such as shear strength or adhesion strength of the ground and retaining wall, a smaller FOS of 1.0 to 1.2 is sufficient.

	Terzaghi's method	Critical hydraulic gradient method
Diagram	<p>$w = \gamma' L_d$</p> <p>$u = \frac{1}{2} \gamma_w h_w$</p> <p>$L_d$: penetration length</p> <p>h_w: difference of water head</p> <p>γ': submerged unit weight of soil γ_w: unit weight of water</p>	<p>l: length of stream line</p> <p>h_w: difference of water head</p> <p>G_s: specific gravity of soil particle e: void ratio</p>
Equation	$FOS = w/u = \frac{2\gamma' L_d}{\gamma_w h_w}$ <p>FOS \geq 1.2 (temporary works) FOS \geq 1.5 (permanent works)</p>	$FOS = i_c/i = \frac{G_s - 1}{1 + e} \cdot \frac{l}{h_w} = \frac{\gamma' l}{\gamma_w h_w}$ <p>FOS \geq 2.0</p>

Fig. 7. Hydraulic failure checks.

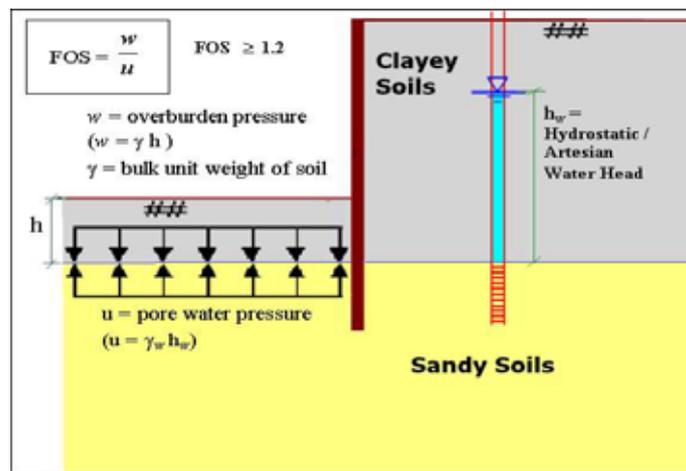


Fig. 8. Heaving due to artesian pressure check.

6.4 Axial stability

Axial stability is a simple check which is often overlooked but is important to ensure stability of the retaining wall and support system. The check ensures that the self-weight of the wall and any axial component of force exerted onto the wall (e.g. axial components of prestressed ground anchors) can be supported by the skin friction of the embedded portion of the wall. The skin friction on the wall above the base of the excavation and end-bearing of the wall should be ignored (unless proper cleaning of the wall toe can be carried out during construction). The factor of safety for axial stability checks is recommended to be 2.0. Figure 9 shows typical a vertical failure of an embedded wall that needs to be prevented in design.

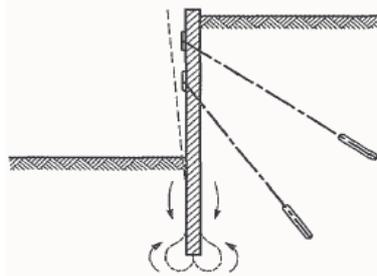


Fig.9. Example of vertical failure of embedded wall (EN1997-1:2004)

7.1 General

The design of retaining walls and support systems for deep basement construction requires consideration of soil-structure interaction and for this purpose, finite element (FEM) analysis is required. The use of finite element analysis is common and various commercial software packages are available to the designer. Some of the commonly used finite element software includes PLAXIS, CRISP, etc. Due to increasing user-friendliness of the commercial software packages, the use of finite element has become increasingly common and FEM is being routinely used by engineers of differing levels of experience and expertise. As such, various authors such as Potts (2003) and Wood (2004) had highlighted the importance of proper understanding of finite element analysis and also the coding and constitutive soil models used in the software. In an example quoted by Wood (2004) based on Schweiger (2003), the results of a benchmark problem analysed by different people using the same numerical analysis program (PLAXIS) and the same constitutive soil model with the same soil parameters are shown in Figure 10.

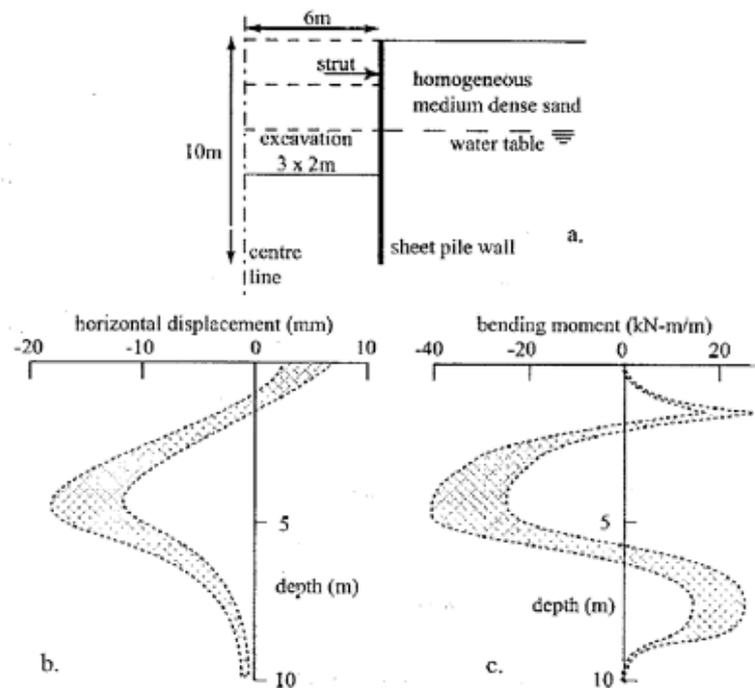


Fig. 10. Benchmark comparisons of results of numerical analysis of strutted sheet pile wall retaining dense sand using PLAXIS: (a) outline of problem analysed; (b) approximate range of predictions of horizontal displacement of walls; (c) approximate range of predictions of bending moment in walls (Wood, 2004).

As can be seen from Figure 10, there is scatter in the results, even for such a 'simple' excavation problem. As such, the designer should be aware of the following factors which may affect the results of FEM analysis:

- a) Locations of the boundaries of the problem. The problem boundary should be located far enough away such that there is no stress rotation near the boundary. For undrained analysis, the extent of the model required will be greater.
- b) Details of mesh. Higher order elements are to be preferred to simple elements, especially if high strain gradients are anticipated, or for failure analysis. More, smaller, elements need to be placed where gradients are expected to be highest, and at regions of stress concentration (Wood (2004)).
- c) Long, thin elements will lead to calculation instability. As such, the layout of the model and mesh should avoid long, thin elements.
- d) Stages of construction. As soils are non-linear, history dependent materials, proper modelling of the soil at various stages from the past to its construction stages needs to be carried out.
- e) Modelling of interfaces. Improper modelling or use of unconservative interface reduction factors may lead to dangerously unsafe design.
- f) Use of suitable constitutive soil models to model different geotechnical problems.
- g) Sensitivity of various soil parameters. For different constitutive soil models adopted in different FEM software packages, different soil parameters may have different effects on the analysis results. Some of the important parameters include:
 - i. Shear strength parameters (c' and ϕ')
 - ii. Stiffness parameter (E)
 - iii. Coefficient of earth pressure at rest (K_0)
 - iv. Wall-Ground Interface factor (δ)
 - v. Permeability of soil (k)

The amount of shear stress which can be mobilised at the wall-ground interface is governed by the wall-ground interface factor (δ) which can be significant for deep excavation.

$$\delta = k \cdot \phi_{cv}'$$

where, ϕ_{cv}' = critical state angle of shearing resistance in terms of effective stress

EC7 (EN1997-1:2004) recommendations are as follows:

- for precast concrete or steel sheet pile, k should not exceed 2/3
- for concrete cast against soil, k can be assumed as 1.0.

The designer should be aware of the modelling techniques required for different FEM software and also its limitations. As such, a simple hand calculation should always be carried out together with FEM analysis as a check against the validity of the FEM analysis.

7.2 Geometrical data

In FEM analysis, the geometry of the model should reflect the actual field conditions closely. In Malaysia, the following criteria are normally observed in both FEM and manual calculation:

b) Provision for Over-Excavation (Δa):

The stability of the retaining wall depends on the passive ground resistance in front of the structure and therefore it is prudent to allow for over-excavation (Δa) in the design (EN1997-1:2004) depending on the site control.

- for cantilever walls, Δa should equal to 10% of the wall height above excavation level, limited to a maximum of 0.5m;
- for a supported wall, Δa should equal 10% of the distance between the lowest support and the excavation level, limited to a maximum of 0.5m.

A smaller value of Δa can be used when the excavation surface can be controlled reliably throughout the excavation works. The over-excavation (Δa) provided in design is not meant for lack of control at site which is very important for all excavation works.

c) Water levels:

The selection of design groundwater level (free water and phreatic surfaces) should be based on information collected during subsurface investigation through monitoring of standpipes or other means. If the site is prone to flooding, as in many areas of Malaysia, the flood level should be taken into consideration depending on the permeability of the subsoil.

d) Surcharge:

The surcharge value should take into account the site conditions and control at site. Site conditions such as loadings from adjacent buildings, vehicles, services, etc. should be taken into consideration in the design. It is prudent to incorporate a minimum surcharge of 10kPa to cater for construction loads and unforeseen circumstances. During tender and construction, it is very important for the Contractor to be aware and follow the assumptions adopted by the designer to prevent causing problems to the retaining system due to uncontrolled stacking of materials (loading) on the retained side.

7.3 Constitutive soil models

In FEM analysis, proper understanding of the constitutive soil models is important in order to produce safe design. Various soil models have been incorporated in commercial software packages ranging from elastic-perfectly plastic Mohr-Coulomb model to the Cam clay model. For example, in the FEM computer program PLAXIS, there are various soil models for different application, i.e. the Mohr-Coulomb, Soft Soil Model and Hardening Soil Model. An example of incorrect use of soil models is best illustrated in the recent Nicoll Highway collapse (Yong & Lee, 2007). In the design of the diaphragm wall with internal strutting, the Contractor had used effective stress parameters (c' and ϕ') with the Mohr-Coulomb model to simulate the undrained behaviour of soft clay (known as Method A among PLAXIS users). This method overestimated the undrained shear strength of the marine clay as illustrated in the stress path diagrams (Figure 11). The undrained shear strength, c_u in the 'real' soil from the test is much lower than that predicted using Method A. The consequence of using Method A with the Mohr-Coulomb model in the Nicoll Highway project led to an under-estimation of wall deflection, bending moment and strut forces in the design. The original design estimated a maximum deflection of 145mm whereas 450mm would have been computed if the lower c_u value in the 'real' soil was used (Yong & Lee, 2007).

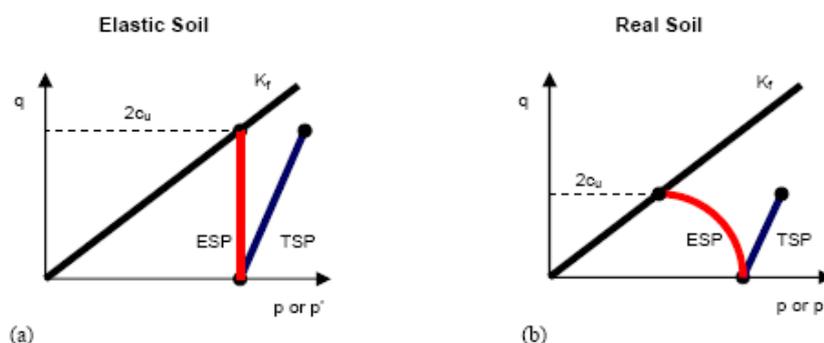


Fig. 11. Comparison of undrained strength (c_u) of a soft clay in consolidated undrained triaxial compression test: (a) stress path determined from finite element analysis using Mohr-Coulomb model with effective stress parameters (Method A) and (b) stress path of a real soil.

7.4 FEM analysis of limit state

By 2010, the Eurocodes will have replaced current British Standards and it is apparent that Malaysia will follow a similar route in adopting the Eurocodes for design with its own national annexes. The Authors have started to incorporate the Eurocodes concept into their geotechnical analysis and design. Following are the procedures suggested by the Authors to merge with current practice in Malaysia.

In Malaysia, the analysis and design of retaining walls for basement excavation is commonly carried out using “working state design” or “conventional method”. In the “working stage design”, the focus and concept of design is to model what is expected to happen at site with the construction performing in a successful manner. Once the actions and stresses (e.g. struts/anchors forces, bending moments & shear forces) which will be mobilised in this working state are calculated, factors of safety (FOS) are imposed to cater for unforeseen circumstances and to ensure safety.

For those who are not familiar with EC7 (EN1997-1:2004), following are some common terms used as extracted from Simpson & Yazdchi (2003):

Table 1. Common terms used in EC7.

<i>Design value</i>	A value used in a calculation, possibly already factored or adjusted from another value, such as a characteristic value.
<i>Characteristic value of a structural parameter</i>	Usually a value derived as a fractile of results from a prescribed test.
<i>Characteristic value of a ground parameter</i>	“A cautious estimate of the value affecting the occurrence of the limit state” as defined in EC7. It is intended to indicate that the value, though cautious, is the engineer’s assessment of the ground behaviour actually governing the occurrence of the limit state under consideration. In conventional design, they are called “moderately conservative values” usually selected by a Geotechnical Engineer for analysis and design.

EC7 (EN1997-1:2004) and Simpson & Yazdchi (2003) defined and explained clearly the limit state design. FEM is usually used to analyse expected behaviour (conventional design) but it can also be used to check for unexpected states in order to demonstrate that limit states are sufficiently unlikely to occur. Partial factors are applied to the strength of the ground and to actions as in Design Approach 1, Combinations 1 & 2 of EC7 (EN1997-1:2004), as listed below:

Table 2. Partial factors for Design Approach 1.

Partial factors for DA1 – Extracted from UK National Annex			Design Approach 1	
			Combination 1 Partial Factors	Combination 2 Partial Factors
Actions	Permanent	Unfavourable	1.35	1.0
		Favourable	1.0	1.0
	Variable	Unfavourable	1.5	1.3
		Favourable	0	0
Soil Parameters	$\tan \phi'$		1.0	1.25
	c' (effective cohesion)		1.0	1.25
	s_u (undrained shear strength)		1.0	1.4
	Unconfined strength		1.0	1.4
	Weight / Density		1.0	1.0

One of the important features in FEM is that the complete “stress history” of the soil and the structure can be simulated and accounted for. Simpson & Yazdchi (2003) proposed three possible schemes to perform limit states design using FEM:

- (1) Perform all the calculations with design (factor) values of ground and action parameters: the complete history is then simulated for soil parameters at design level.
- (2) Simulate the whole stress history using ground parameters at characteristic levels and check the safety at the relevant stages by performing a stepwise reduction of the soil strength parameters, the initial stress field at each of these reductions being the current characteristic stress field.
- (3) Simulate the whole stress history using ground parameters at characteristic levels and multiplying these values by the load factor (which then in fact acts as a model factor on the effects of actions).

Simpson & Yazdchi (2003) also highlighted that the two key points stated above are that:

- a) part of the reason for factors of safety is to cover human error.
- b) limit state analyses investigate unrealistic states, especially in the Ultimate Limit State (ULS) analysis.

The purpose of the analysis is to establish that the limit state is sufficiently unlikely to occur. Simpson & Yazdchi (2003) recommended that both Schemes 2 and 3 should be carried out. Scheme 2 represents EC7 Design Approach 1 Combination 2 (DA1-C2) and Scheme 3 represents Design Approach 1 Combination 1 (DA1-C1).

In this paper, the above two schemes are summarised into design procedures using FEM as follows:

Step 1: Carry out each stage of excavation and construction of support in FEM using Characteristic Values strength parameters (similar to EC7 Design Approach 1 Combination 1)

- Adopt characteristic values for soil strength parameters (partial factors = 1.0)
- Adopt a wall-ground interface factor (δ) depending on the retaining structures adopted (see Section 7.1 above)
- Adopt a representative conservative ground water level
- Apply surcharge at the retained soil side (if proper control can be established at site, surcharge loadings can be omitted)
- Generally use over-excavation of 0.5m (if proper control can be established at site, provisions for over-excavation can be omitted)
- The envelopes of bending moment (BM), shear force (SF) and reactions (struts/anchors forces) obtained using *characteristic values* are *working state values*.
- The need to apply Factor of Safety (FOS) = 1.35 (as in EC7 DA1-C1) or 1.4 (as in conventional design) to convert *working state values* to *Ultimate BM* or *Ultimate Shear Force* for structural design.
- For struts or anchors, to impose FOS = 1.5 to 2.0 depending on the criticality of the support on the allowable movements, overall stability of the retaining structures and adjacent properties or services.

Step 2: After completing the entire Step 1 of FEM analysis, “insert” between each stage a “critical stage” as an ultimate limit state (ULS) check following EC7 Design Approach 1 Combination 2 (EC7 DA1-C2)

- Adopt *design strength values* for soil strength parameters.
- *Design Soil Strength* = Characteristic Soil Strength / Partial Factors. The partial factors should refer to Design Approach 1 Combination 2 values as listed in Table 2.
- Adopt a wall-ground interface factor (δ) depending on the retaining structures adopted (see Section 7.1 above)
- Adopt *worst* possible ground water level (also check for artesian or flood level, if any).
- Adopt *worst* possible surcharge at the retained soil (if proper control can be established at site, surcharge loadings can be omitted). If not sure, use at least 10kPa.
- Adopt over-excavation of 0.5m.
- The envelopes of bending moment (BM), shear force (SF) and reactions (struts/anchors forces) obtained using *design values* are *ultimate state values*.

Step 3: Compile the results of both Step 1 and Step 2 to carry out design

- Adopt the larger value of Ultimate State BM envelopes, Ultimate State SF envelopes and Ultimate State Actions (struts / anchors forces) obtained from both Step 1 and Step 2.
- Carry out design of the retaining structures and support systems.
- Deformation of the wall and adjacent ground should be assessed based on results from Step 1.

7.5 FEM analysis via Plaxis

FEM computer program Plaxis is one of the most widely used FEM software programs in Malaysia to analyse retaining walls for deep basement construction. The following section describes some of the key processes to observe when using Plaxis to simulate basement excavation.

It is recommended to carry out effective stress analysis in Plaxis compared to total stress analysis. The hardening soil model is recommended, rather than the Mohr-Coulomb model as discussed in Section 4.1. Two methods commonly used are:

- 1) Method A : Effective Stress Analysis
 - Type of material modelled is set to “undrained”.
 - Soil Strength = Effective Stress Strength (c' , ϕ' , ψ')
 - Soil Stiffness = E' , ν' , λ^* , κ^*
 - Generally used for granular residual soils or sandy materials as these soils are generally governed by effective stresses.
 - For soft clay, it is better to use Method B unless the undrained strength parameters can be correlated properly with effective strength parameters (care should be exercised) as highlighted in Sections 4.1 & 7.2. Method A using Mohr-Coulomb’s model will produce unsafe design as discussed in Section 7.3.
 - Allows change of strength with change in effective stress (due to loading, unloading or water pressures).
 - Essential for exploiting features of advanced soil models such as the Hardening Soil Model (Vermeer & Brinkgreve, 2002), the Soft Soil Model and the Soft Soil Creep Model in Plaxis.

- 2) Method B : Effective Stress Analysis
 - Type of material modelled is set to “undrained”.
 - Soil Strength = Undrained Shear Strength (c_u)
 - Soil Stiffness = E' , ν'
 - Suitable for low permeability fine grained soil (e.g. soft clay)
 - Use this method when there is no information on effective strength parameters.
 - Can only use the Mohr-Coulomb and Hardening Soil Models.
 - Limitation is that change in effective stress will not change the strength of the subsoil.

When setting initial conditions in FEM analysis, it is important to use representative OCR or K_0 in the generation of initial stress conditions. For each stage of excavation, seepage analysis should be carried out to obtain the representative water pressures for the whole system. It is more representative of the actual site conditions if transient flow calculation is carried out, provided that the permeability of the subsoil and construction timing/duration are well defined. If these two key pieces of information are lacking, then steady state flow calculation will generally err on the conservative side.

8 GROUND MOVEMENT ASSOCIATED WITH EXCAVATION

In the design of deep excavation works, the designer should be aware that ground movements related to open excavation can include a substantial component of horizontal strain. This is in addition to settlements of buildings caused by their own weight. The traditional criteria based on differential settlement or angular distortion alone is therefore inadequate for assessment of building response due to deep excavation works. Typical limiting values of angular distortion for buildings where only building settlement is assessed are shown in Figure 12.

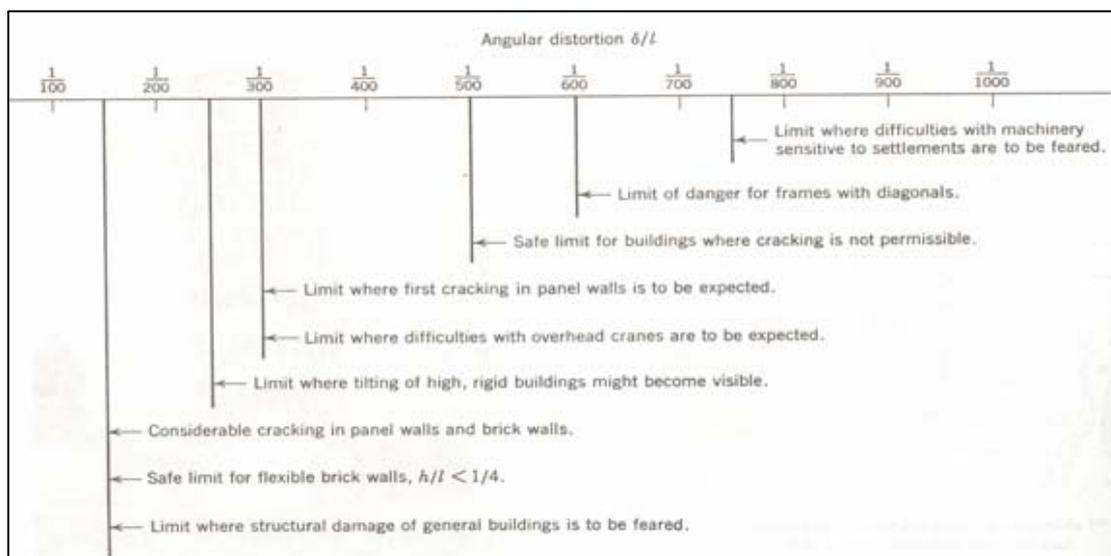


Fig. 12. Limiting angular distortion (Bjerrum, 1963).

Boscardin & Cording (1989) summarise the following important considerations for assessment of building response to excavation:

- Buildings sited adjacent to excavations are generally less tolerant to excavation-induced differential settlements than similar structures settling under their own weight. This is due to the lateral strains that develop in response to most excavations.
- As a structure is subjected to increasing lateral strains, its tolerance to differential settlement decreases. As a consequence, measures to mitigate excavation-related building damage should include provisions to reduce the lateral strains sustained by the ground.
- Horizontal ties in the form of reinforced concrete grade beams or similar items are effective means of controlling the strains and distortions in both bearing wall and frame structures adjacent to an excavation.

Various damage category criteria have been proposed such as the methods of Rankin (1988), Boscardin & Cording (1989) and Boone (1996). The method of Boscardin & Cording (1989) appears to produce reasonably accurate predictions as shown in Figure 13. As such, the damage criteria as proposed by Boscardin & Cording (1989) are recommended and the limiting values are summarised in Table 3.

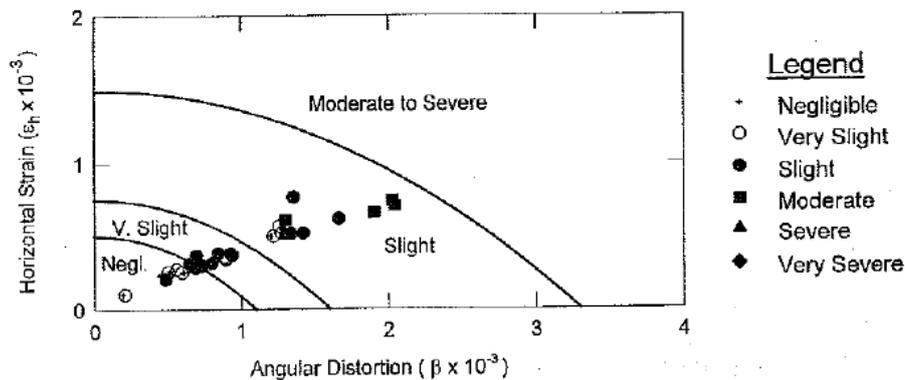


Fig. 13. Summary of plots of damage estimation using the method of Boscardin & Cording (1989) and assessed ground movement behaviour (Boone, 1999).

Table 3. Damage category criteria (Boscardin & Cording, 1989).

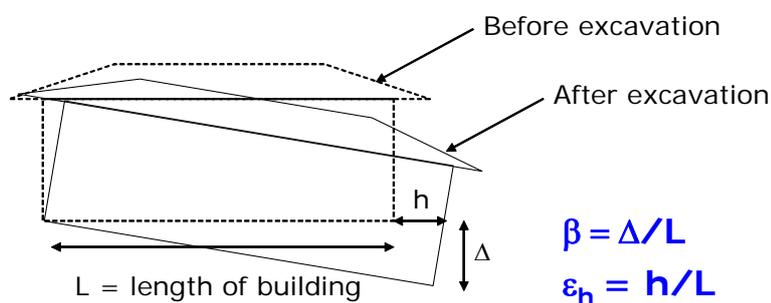
CATEGORY	ANGULAR DISTORTION β ($\times 10^{-3}$)	HORIZONTAL STRAIN ϵ_h ($\times 10^{-3}$)
Negligible	$< \sim 1.1$	< 0.5
Very slight	$\sim 1.1 < \beta < \sim 1.6$	$0.5 < \epsilon_h < 0.75$
Slight	$\sim 1.6 < \beta < \sim 3.3$	$0.75 < \epsilon_h < 1.5$
Moderate	$\sim 3.3 < \beta < \sim 6.7$	$1.5 < \epsilon_h < 3.0$
Severe	$> \sim 6.7$	> 3.0

For design purposes, the designer should normally limit the angular distortion and horizontal strain such that the damage category does not exceed “slight” in Table 3. The damage category is based on the work of Burland et al. (1977) and is shown in Table 4. Therefore, the limiting value of angular distortion (β) and horizontal strain (ϵ_h) for the slight category is 3.3×10^{-3} and 1.5×10^{-3} respectively. Figure 14 illustrates a simple example of the limiting values of angular distortion and horizontal strain.

Table 4. Severity of cracking damage (modified after Burland et al., 1977).

DAMAGE CATEGORY	DESCRIPTION OF TYPICAL DAMAGE	APPROXIMATE WIDTH OF CRACKS, mm
Negligible	Hairline cracks	< 0.1
Very slight	Very slight damage includes fine cracks which can be easily treated during normal decoration, perhaps an isolated slight fracture in building, and cracks in external brickwork visible on close inspection	< 1
Slight	Slight damage includes cracks which can be easily filled and redecoration would probably be required, several slight fractures may appear showing the inside of the building, cracks which are visible externally and some repointing may be required, and doors and windows may stick	< 5

Moderate	Moderate damage includes cracks that require some opening up and can be patched by a mason, recurrent cracks that can be masked by suitable linings, repointing of external brickwork and possibly a small amount of brickwork replacement may be required, doors and windows stick, service pipes may fracture, and watertightness is often impaired	5 to 15 or several cracks > 3mm
Severe	Severe damage includes large cracks requiring extensive repair work involving breaking-out and replacing sections of walls (especially over doors and windows), distorted windows and door frames, noticeably sloping floors, leaning or bulging walls, some loss of bearing in beams, and disrupted service pipes	15 to 25 also depends on number of cracks
Very severe	Very severe damage often requires a major repair job involving partial or complete rebuilding, beams lose bearing, walls lean and require shoring, windows are broken with distortion, and there is danger of structural instability	Usually > 25 depends on number of cracks



E.g.

Building length = 10m

Allowable lateral movement = 15mm

$$(\epsilon_h = 1.5 \times 10^{-3})$$

Fig. 14. Example of limiting values of angular distortion and horizontal strain.

The estimation of ground movement due to deep excavation requires the use of finite element analysis and the results from finite element analysis are then compared with the criteria given above. However, for simple preliminary checking, empirical methods by Clough & O'Rourke (1990) and Peck (1969) can be used.

The empirical method for estimating movements for in-situ wall system proposed by Peck (1969) is shown in Figure 15 from data compiled on the settlement of the ground adjacent to temporary braced sheet pile and soldier pile walls. Clough & O'Rourke (1990) present settlement profile for retaining walls in different soil types and is generally more detailed compared to the earlier work by Peck (1969).

In Malaysia, the use of the above empirical methods is for preliminary assessment only and detailed analyses are usually carried out using FEM. This is because the above empirical data is based on experience from other countries that may not reflect local experiences. For example, there have been cases of lowering of the groundwater table in the retained soil which causes settlement extending beyond the influence zone recorded in the above methods and distance exceeding 50 times the depth of excavation has been recorded. As such, the designer should always compare the preliminary assessment based on empirical methods with results from FEM analysis. The designer should pay particular attention to structures within the influence zone obtained from FEM analysis and therefore, dilapidation survey should also cover buildings within this zone.

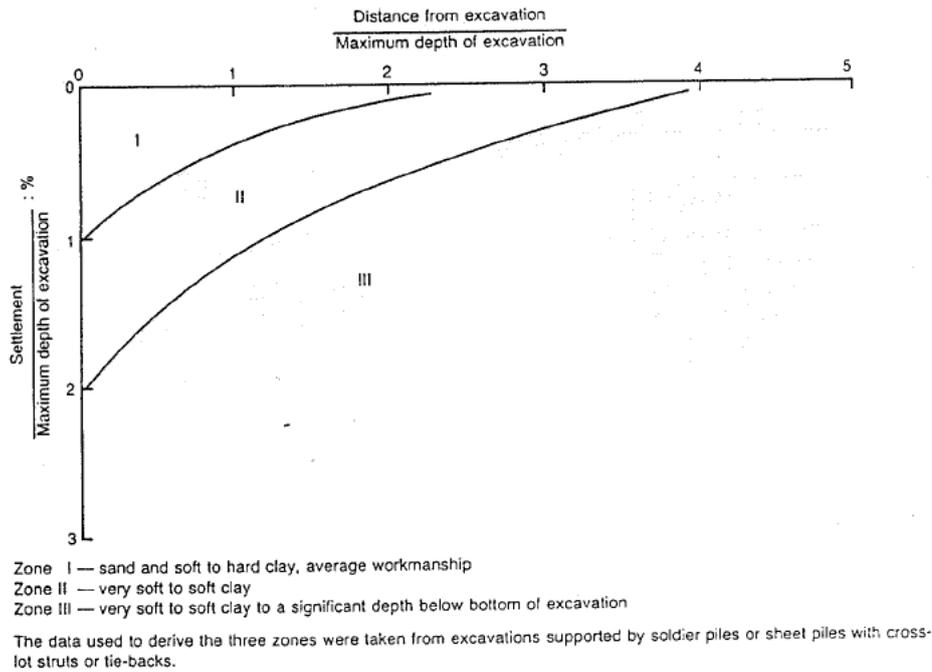


Fig. 15. Observed settlements behind excavations (Peck, 1969).

9 STRUCTURAL DESIGN OF RETAINING WALLS AND SUPPORT SYSTEMS

The structural design of retaining walls and support systems for deep excavation is also very important to ensure satisfactory performance and to prevent catastrophic failure. In this paper, some aspects of reinforced concrete and steel design related to deep basement excavation are presented. This is in light of recent high profile failures of deep excavation retaining walls and support systems (e.g. Nicoll Highway, Singapore) where deficiency in structural design is believed to have contributed to the failures.

Current approach to design of retaining walls and support systems will invariably involve some form of finite element analysis (e.g. PLAXIS, SAGE-CRISP, etc.). As such, the forces and stresses extracted from finite element analyses should be factored appropriately for structural design. Where unfactored soil parameters (characteristic soil parameters) are used in the analysis, the following factors of safety are generally applied in structural design:

- a) A minimum factor of safety (FOS) of 1.4 should be applied to walls and sheeting moments.
- b) A minimum FOS of 1.5 to 2.0 should be applied to strut or anchor loads.

It can be observed that a higher FOS is applied to strut or anchor loads, as conventional plane-strain analysis tends to underestimate the strut or anchor loads due to the 3-dimensional arching effect.

9.1 Some common mistakes in steel design (Chiew & Leow, 2006)

This section highlights some of the common mistakes of steel design for deep basement support systems using the steel strut-waler system. The designers should be aware of the serious consequences of inadequate design of the steel strut-waler system which will lead to catastrophic failure of the entire retaining walls and support systems. The materials presented in this section are primarily drawn from Chiew & Leow (2006).

One of the commonly used support systems used in conjunction with retaining walls for deep basement excavation is the steel strut-waler system. The struts usually consist of an H-section with walers laid across the walls to ensure continuity. The struts and walers used as a support system are different compared to beams and columns in terms of connection behaviour. For beam-column connection typically found in building frames, the webs of the beam and column are usually in the same plane. On the other hand, for strut-waler connection, the webs of the strut and waler are in two different planes perpendicular to each other. This key difference in the orientation of the H-section strut is significant in terms of load transfer; the load from the waler is not transferred to the strut in a straightforward manner as compared to the beam-column situation. In addition, the two strut flanges are abutting at the edges of the waler flanges; consequently, the load transfer is more critical and the strut-waler connection is more prone to sway instability compared to the beam-column connection. Due to these reasons, strut-waler connections are usually stiffened because it is the weakest link in the entire strut-waler system. Not many designers recognize this difference and therefore, inadequate attention is paid to the design of this connection.

The formulas for web bearing and web buckling capacities extracted from the 1990 and 2000 versions of BS5950 are shown in Table 5.

Table 5. Web bearing and buckling equations.

Design criteria	BS5950-1: 1990	BS5950-1: 2000
Web bearing	$P = (b_1 + n_2) t p_{yw}$	$P_{bw} = (b_1 + nk) t p_{yw}$
Web buckling	$P_w = (b_1 + n_1) t p_c$	$P_x = \frac{25\epsilon t}{\sqrt{(b_1 + nk)d}} P_{bw}$

Note:

b_1 – stiff bearing length

n_1 – length obtained by dispersion at 45° through half the depth of the section

n_2 – length obtained by dispersion through the flange to the flange to web connection at a slope of 1:2.5 to the plane of the flange

t – thickness of the web

p_c – compressive strength of the web

p_{yw} – design strength of the web

n, k – design coefficients as given by Clause 4.5.2.1 of BS5950-1:2000

d – depth of the web

$\epsilon = (275/p_y)^{0.5}$

From the table above, it can be seen that the stiff bearing length, b_1 plays an important role in the resistance of the unstiffened web to bearing and buckling. Stiff bearing length is defined as the length of support that cannot deform appreciably in bending. This stiff bearing length is usually smaller than the width of the cover plate and using the cover plate width as the stiff bearing length would lead to an overestimation of the web bearing capacity by approximately 2 times. The formula for determining the stiff bearing length is now given in Figure 13 of BS5950-1:2000. Figure 16 shows the correct diagram and formula to be used in determining the stiff bearing length for a strut-waler connection.

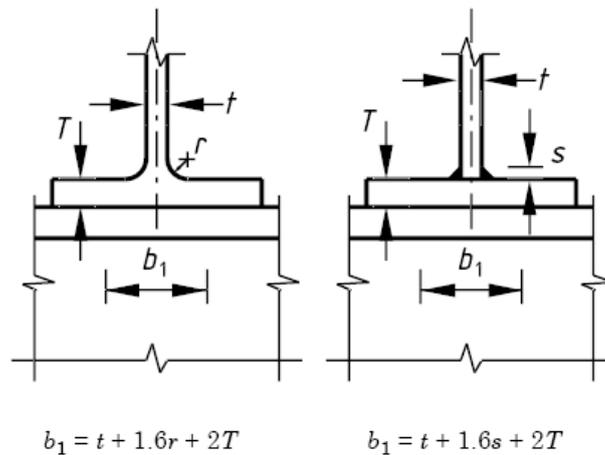


Fig. 16. Stiff bearing length for strut-waler connection (BS5950-1:2000).

For web buckling capacity, as with all buckling phenomena, the effective length and slenderness of the unstiffened web under axial compression affects the buckling capacity. In the 1990 version, the slenderness λ of the web can be taken as $2.5d/t$ (derived based on an effective length factor of 0.7) provided that the flange of the waler is effectively restrained against both rotation relative to the web and lateral movement relative to the other flange. In the 2000 code, it can be seen that there is no effective length or slenderness parameter in the web buckling formula. This is because similar to the 1990 version, the 2000 version also defined that this formula is valid only when the flange of the waler is effectively restrained against rotation relative to the web and lateral movement relative to the other flange. When these two conditions are satisfied, this means that the flange of the waler will not sway sideways under the strut loading and the effective length of the web is 0.7 times the depth of the web, similar in concept to the 1990 version. However, more often than not, there is no lateral restraint to the flange of the waler as they are placed horizontally across the walls. This means that the waler flange can sway sideways and thus causes the effective length of the web to increase. The 1990 version requires the user to check the web for the effects of strut action. The 2000 version takes this into account by introducing a reduction factor, based on the ratio of the effective length under sway and non-sway mode, to the web buckling capacity under non-sway condition as shown in Eq. 1.

$$P_{xr} = \frac{0.7d}{L_E} P_x \quad (1)$$

If the unstiffened web is required to be strengthened, the type of stiffener detail adopted for the connection will also have a huge impact on the connection performance. The conventional method of welding steel plates between the flanges of the waler increases not only the bearing and buckling capacity of the web but also the robustness and ductility of the connection. The plates, together with the waler web, form a cruciform section and this increases the stiffness of the waler about its minor axis. Connections using this plate stiffener detail tend to be ductile enough to undergo a substantial amount of plastic deformation before failure occurs. Failure is mainly due to the in-plane buckling of the plate and/or web with little or no side sway of the waler flange.

Another common way of strengthening the web involves casting a concrete block in place of welding the steel plates. Connections of this form may have a higher bearing and buckling capacity than steel plate stiffeners due to the high compressive resistance of the concrete. However, failure occurs under a much lesser post-peak plastic deformation and this indicates that the failure mode is more sudden and brittle.

Another form of strengthening detail is to weld steel C-channels instead of steel plates. Recent research carried out by Chiew & Yu (2006) has shown that this form of connection has the worst performance amongst the three details. Failure is caused by the side-sway of the C-channels and the web in a sudden manner with a drastic reduction in web capacity. The web and C-channels do not interact as one section unlike the cruciform section for a plate stiffener, as each member tends to bend about its own axis. Connections adopting C-channels as stiffeners are most sensitive to the effective length used in calculating the web capacity as compared to the other two types of connection details.

In order to illustrate the sensitivity of the web buckling capacity to changes in stiff bearing length and effective lengths, a total of three different strut-waler connection details as illustrated in Figure 17 were studied by Chiew & Leow (2006). The first case is an unstiffened waler. The second and third cases are waler stiffened with steel plates and C-channels on both sides respectively. The effective length factor used is 1.0, 1.2, 1.5 and 2.0. The design is based on BS5950-1:2000, Clause 4.5.3.1. The design parameters are given in Figure 17 and results tabulated in Tables 6 and 7.

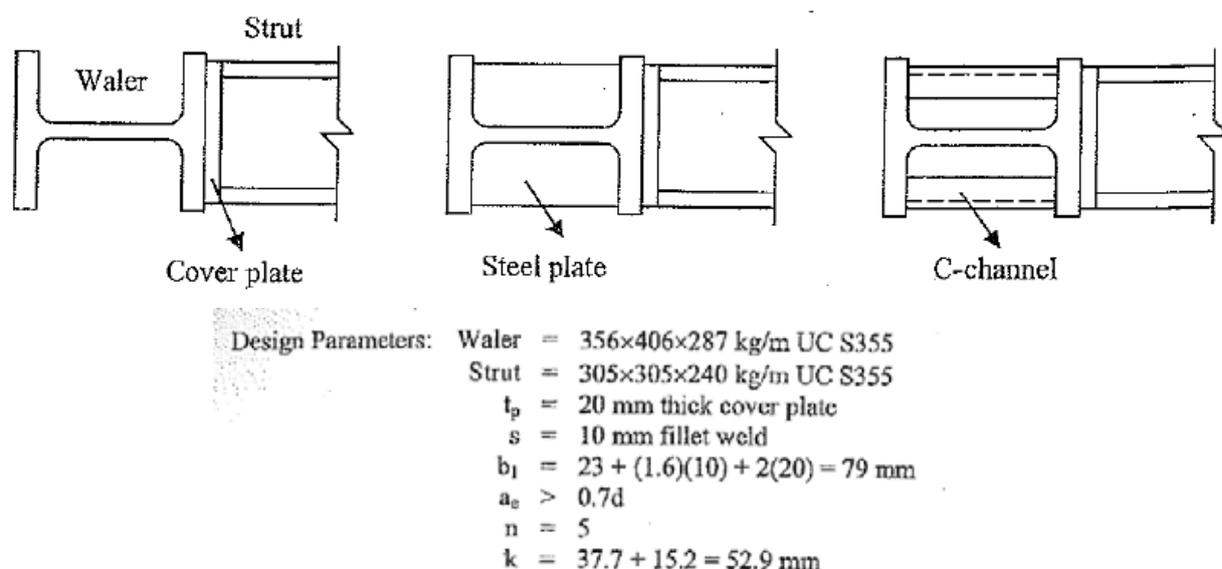


Fig. 17. Strut-waler connections with different stiffening details (Chiew & Leow, 2006).

Table 6. Influence of stiff bearing length (Chiew & Leow, 2006).

Stiff Bearing Length (b_1) (mm)	Unstiffened waler P_x (kN)	Steel plate stiffener 100x12mm S355 P_x (kN)	C-channel stiffener 127x64x15 kg/m S355 P_x (kN)
317.9 (full width)	4494	5198	5805
79 (Fig. 13 BS5950)	2631	3335	3942

Table 7. Influence of effective length (Chiew & Leow, 2006).

Effective Length	Unstiffened waler P_x (kN)	Steel plate stiffener 100x12mm S355 P_x (kN)	C-channel stiffener 127x64x15 kg/m S355 P_x (kN)
$L_E = 1.0d$	4242	6114	5539
$L_E = 1.2d$	2474	6114	3747
$L_E = 1.5d$	1979	5994	3213
$L_E = 2.0d$	1485	5773	2649

It can easily be seen from Tables 6 and 7 that the values of stiff bearing and effective length assumed in design have a major influence for both unstiffened walers and walers stiffened with C-channels. Walers stiffened with steel plates are less sensitive to changes in effective length due to the high stiffness of their cruciform sections. The correct stiff bearing length should be used, and it is recommended that an effective length factor of at least 1.2 be used for strut-waler connections.

9.2 Some aspects of reinforced concrete design of retaining walls for deep basements

The design of reinforced concrete walls for deep basements requires consideration of the water-tightness requirements of the basement. As such, the structural design of the basement wall using systems such as diaphragm walls would require the diaphragm wall to be designed as a water retaining structure. In Malaysia, the diaphragm wall is designed in accordance with BS8007:1987 where the maximum design surface crack widths are:

- a. Severe or very severe exposure: 0.2mm
- b. Critical aesthetic appearance: 0.1mm

For normal car parks in Malaysia, a design surface crack width of 0.2mm is adequate. For routine designs, Cheng (1996) is useful for the determination of reinforcement requirements. The minimum concrete grade for diaphragm walls should be Grade 35.

Good detailing practice is also important to ensure satisfactory performance of reinforced concrete retaining wall systems (e.g. diaphragm walls). Some common detailing practices are as follows (BS8110: Part 1):

- a. Every corner bar is to be supported by a link passing round the bar.
- b. Links should be arranged such that there is no longitudinal tension bar > 150mm from a vertical leg.
- c. Size of links = $\frac{1}{4} \times d$ of largest compression bar or 6mm whichever is greater.
- d. Maximum link spacing = $12 \times d$ of smallest compression bar.

The above requirements are to prevent buckling of compression bars and peeling of concrete covers. They also confine the concrete core which improves its axial capacity. This is important, as usually diaphragm walls are also designed to support some of the superstructure loads.

In addition, the construction of diaphragm walls should also ensure that the concrete can flow around the reinforcement and inserts and fill the entire space within the slurry trench. In order to achieve this, detailing recommendations of DIN Standard 4126 are recommended:

- a. The clear distance between the reinforcement and the bottom of the trench should be not less than 200mm.
- b. Minimum clear spacing between vertical and horizontal reinforcement should be provided as per Figure 18.

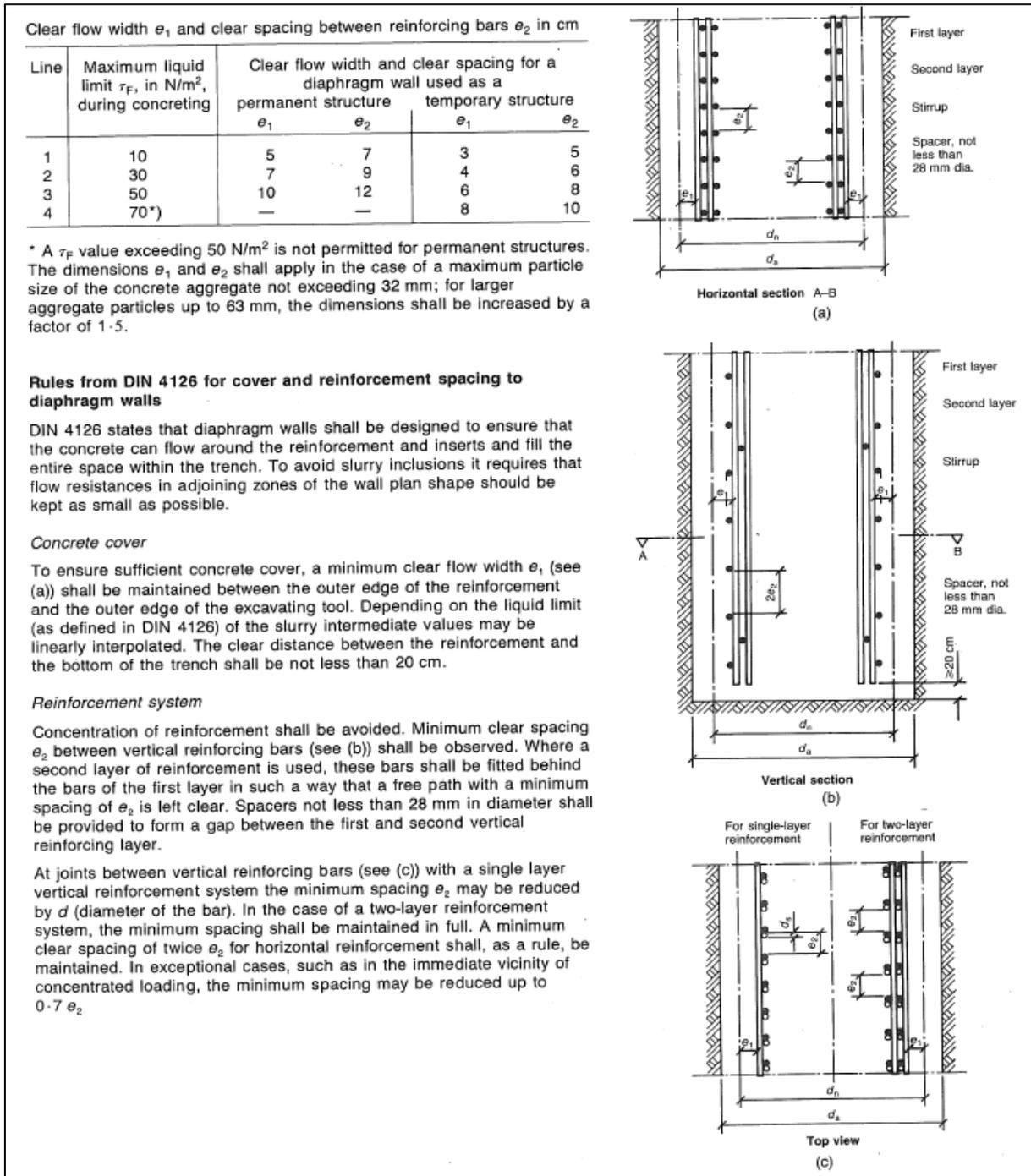


Fig. 18. Minimum spacing and concrete cover to reinforcement in diaphragm walls as recommended by DIN Standard 4126 (Puller (1996)).

10 CASE HISTORIES

In this section, some case histories on retaining walls and support systems for deep basement are presented to illustrate the successful application of the principles highlighted in this paper.

10.1 Berjaya Times Square

The performance of 1.2 m thick anchored diaphragm walls for a 6-level deep basement for a mixed development of Berjaya Times Square in Kuala Lumpur, Malaysia, is presented here (Tan et. al., 2001). Figure 19 shows the layout of the site. The depth of the basement excavation ranges from 24.5 m to a maximum of 28.5 m and the walls were constructed in residual soils derived from Kenny Hill Formation.

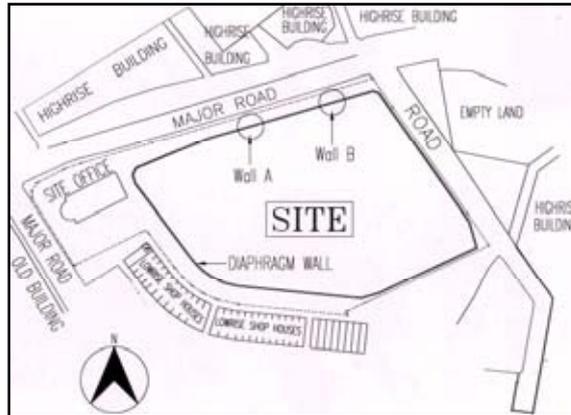


Fig. 19. Location plan of the two walls.

The subsurface investigations carried out comprise rotary wash boring boreholes along the perimeter of the diaphragm wall and inside the site. Field testing such as Standard Penetration Tests (SPT) and pressuremeter tests were also carried out in the boreholes. Standpipe piezometers were installed in the boreholes to measure changes of groundwater levels with time for design. The groundwater level measured was generally located at about R.L.+36.0m before excavation. In addition, disturbed and undisturbed soil samples collected from the boreholes were also tested in the laboratory to acquire the necessary soil parameters. The typical subsoil profile at the site, as shown in Figure 20, indicates that the Kenny Hill residual soils mainly consist of stiff to hard clayey SILT, and silty CLAY with sand or gravel. In between these materials are layers of medium dense to very dense clayey and silty SAND and GRAVEL.

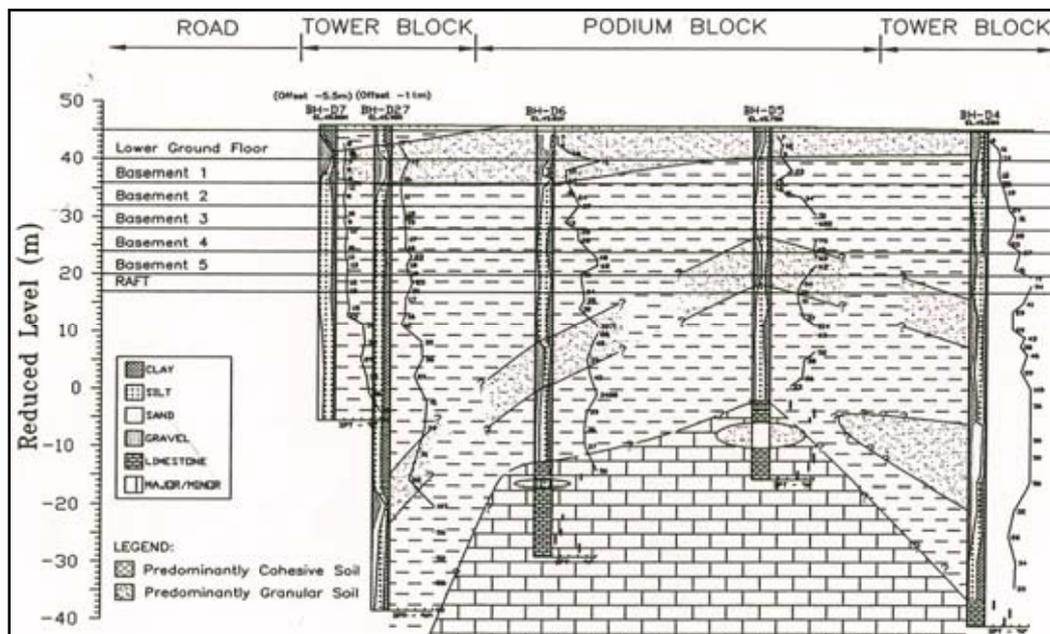


Fig. 20. Subsoil profile

Although the subsoil profile presented in Figure 20 shows variable successions of strata and for analysis purposes, the subsoil were divided into two major components, namely predominantly Granular materials and predominantly Cohesive materials for the design and back-analysis of the two selected typical panels of the diaphragm walls with different ground anchor configurations (Wall A

and Wall B). These two panels were located on one side of the site as shown in Figure 19. Based on the type of subsoil and variation of SPT 'N' values, the subsoils were further divided into four different layers in the analyses

The finite element method (FEM) computer program PLAXIS, was used to back-analyse the deep basement excavation supported by anchored diaphragm walls. The residual soil was modelled using the "Hardening Soil Model" (Vermeer & Brinkgreve, 2002) and the 1.2 m thick diaphragm wall was modelled as beam elements. The free length of pre-stressed temporary ground anchors was simulated as node-to-node spring while the fixed length grout body was simulated as slender objects with an axial stiffness but with no bending stiffness and can only sustain tensile forces only. In addition, interface elements were introduced to simulate soil-structure interactive behaviour. Figure 21 shows the plane strain condition FEM model of Wall A in initial stage utilising 6-node elements under a 2-D plane strain condition.

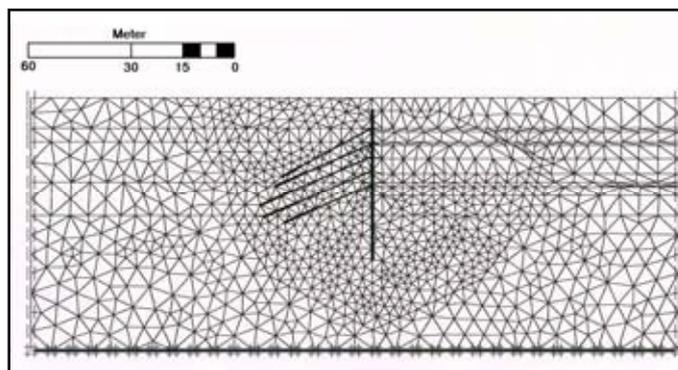


Fig. 21. Finite element modelling of excavation.

Table 8. Soil parameters for FEM back-analysis.

Layer	SPT 'N' (blow/300mm)	γ_b (kN/m ³)	E' (kN/m ²)	E' _{ur} (kN/m ²)	c' (kN/m ²)	ϕ' (degree)	K _o
> R.L. 32m	10	18.5	20,000 (35,000)	60,000	0	31	1.0
R.L.32m to R.L. 20m	28	18.5	56,000 (98,000)	168,000	5	32	0.8
R.L.20m to R.L. 10m	34	19.0	68,000 (119,000)	204,000	5	32	0.8
< R.L. 20m	42	19.0	84,000 (147,000)	252,999	5	34	0.8

Note: The E' values in brackets are the stiffness used in the original design of the diaphragm wall (Gue & Tan, 1998a).

Table 9. Details of ground anchors.

Prestressed Ground Anchors at	Wall A				
	Level – A	Level – B	Level – C	Level – D	Level – E
Reduced Level (m)	36.2	31.5	27.8	23.5	19.3
Angle of Inclination (degree)	30	25	25	25	25
Working Load (kN)	740	730	1008	1129	891
Horizontal Spacing of Anchor (m)	1.57	1.05	1.05	1.05	1.05
Prestressed Lock-Off Load (kN)	689	626	847	873	715

Prestressed Ground Anchors at	Wall B			
	Level – A	Level – B	Level – C	Level – D
Reduced Level (m)	36.3	28.7	24.5	20.7
Angle of Inclination (degree)	30	25	25	25
Working Load (kN)	879	811	1148	1157
Horizontal Spacing of Anchor (m)	1.57	1.05	1.05	1.05
Prestressed Lock-Off Load (kN)	671	624	901	924

The key input soil parameters (all in effective stress) are listed in Table 8 and the details of the prestressed ground anchors are listed in Table 9. The effective Young's Modulus (E') of the soils was interpreted from pressuremeter tests performed at different depths in the boreholes. The effective unloading/reloading stiffness (E'_{ur}) is taken as three times the effective Young's Modulus which

matches the values from the unload-reload cycle of pressuremeter tests. The initial coefficients of earth pressure at rest, K_0 , were also obtained from pressuremeter tests.

In the back-analysis, the soil stiffness which has considerable influence on the wall movement has been modified from the values used in the original design (Gue & Tan, 1998a). The original design of the diaphragm wall was carried out using the computer program FREW, a quasi-finite element program which allows the interaction between the wall and the soil to be modelled. Full details of the computer program FREW are described by Pappin et al. (1986).

In the numerical modelling, the diaphragm wall was assumed as “wished-in-place” after the bulk earthworks excavation in the middle of the site but before the start of excavation immediately in front of the wall. For each stage of excavation in the undrained analysis, steady state groundwater calculation was performed. The excavation sequences simulated in the FEM analysis are as follows :

0. Bulk earthworks excavation at the centre of the site (Figure 22).
1. Installation of diaphragm wall and excavate to 1st level (R.L.35.5m for Wall A and R.L. 36.0m for Wall B).
2. Install and Prestress Anchors (Level-A) and excavate 2nd level (R.L.31.0m for Wall A and R.L. 28.0m for Wall B).
3. Install and Prestress Anchors (Level-B) and excavate 3rd level (R.L.27.0m for Wall A and R.L. 23.6m for Wall B).
4. Install and Prestress Anchors (Level-C) and excavate 4th level (R.L.22.5m for Wall A and R.L. 20.0m for Wall B).
5. Install and Prestress Anchors (Level-D) and excavate 5th level (R.L.18.5m for Wall A and final level R.L. 16.7m for Wall B).
6. Install and Prestress Anchors (Level-E) and excavate 6th level (final level R.L.16.7 m for Wall A).

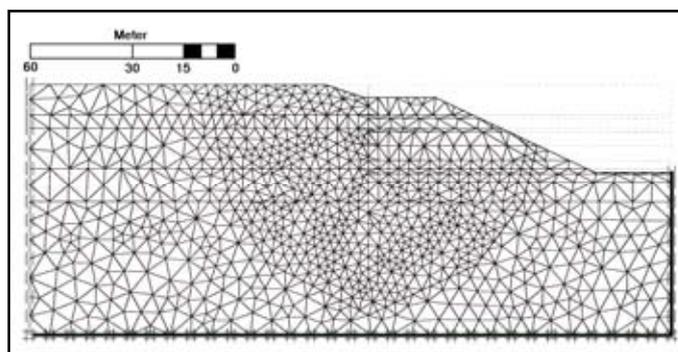


Fig. 22. Stage 0: Bulk earthworks excavation carried out at the centre of the site before installation of the wall.

The back-analysed results presented in this paper include relative lateral displacement of the walls and settlement of the retained ground behind the walls due to excavation. Figures 23 and 24 show the relative lateral displacement measured from inclinometers installed at Wall A and Wall B respectively. In these figures, the back-analysed wall displacements were also presented. Generally the FEM results matched the measured wall profile well for all stages of excavation for both Wall A and Wall B except for slight under-estimation of the wall top displacements. In view of the good match between the measured and back-analysed wall relative lateral displacement profile, it is evident that residual soils can be properly modelled using the “hardening soil” model for the deep excavation problem.

The ground surface settlement measured from settlement markers installed perpendicular to the wall in the retained ground and the FEM back-analysed results for Wall B are presented in Figure 25. In the FEM analysis, the maximum ground settlements are located at a distance of 15m to 20m from the wall and indicate close agreement with field measurements. The analysed settlement profile overestimated the ground surface settlement for Stage 0 after bulk earthwork excavation at the centre of the site. The results improved for Stages 1 to 3 where the analysed and measured ground surface settlement profiles agree well for a distance of 50m before the analysed results overestimated the settlement. The reason for smaller measured settlement after a distance of 50m is due to the presence of high-rise buildings supported by deep foundations obstructing the propagation of the ground settlement. In the last two stages (Stage 4 and Stage 5), the analysed results underestimated the ground surface settlement which might be due to consolidation of the subsoil due to lowering of ground water level caused by seepage flow during the process of excavation. In order to model this effect, coupled consolidation analysis needs to be carried out

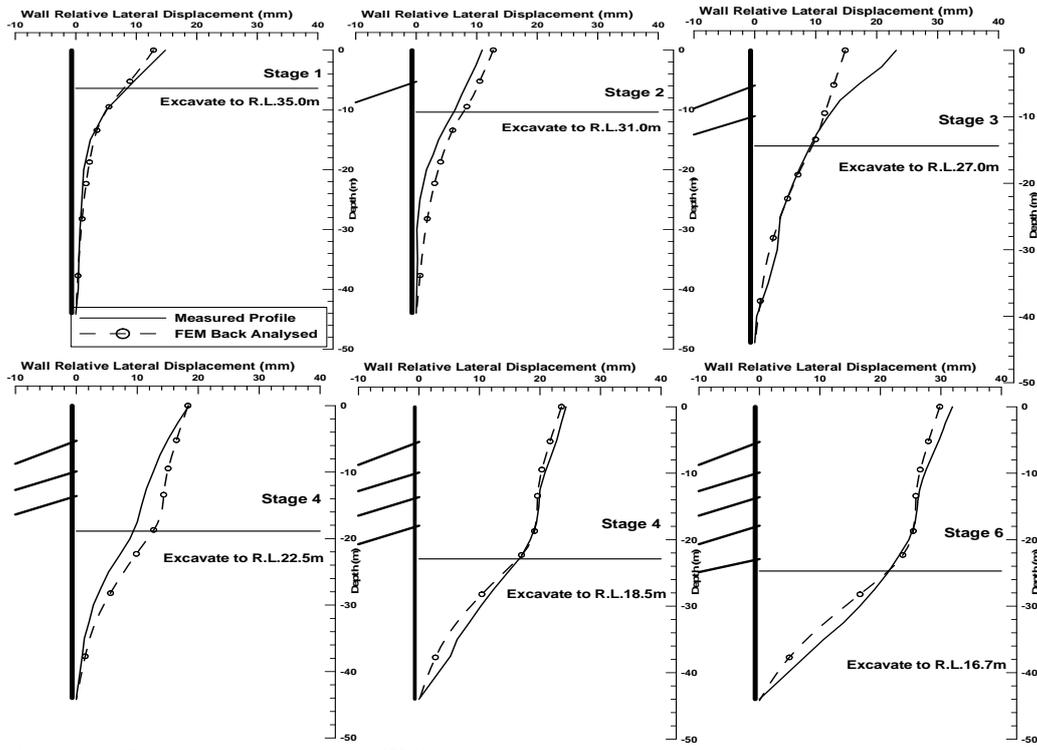


Fig. 23. Relative lateral displacement profile for Wall A

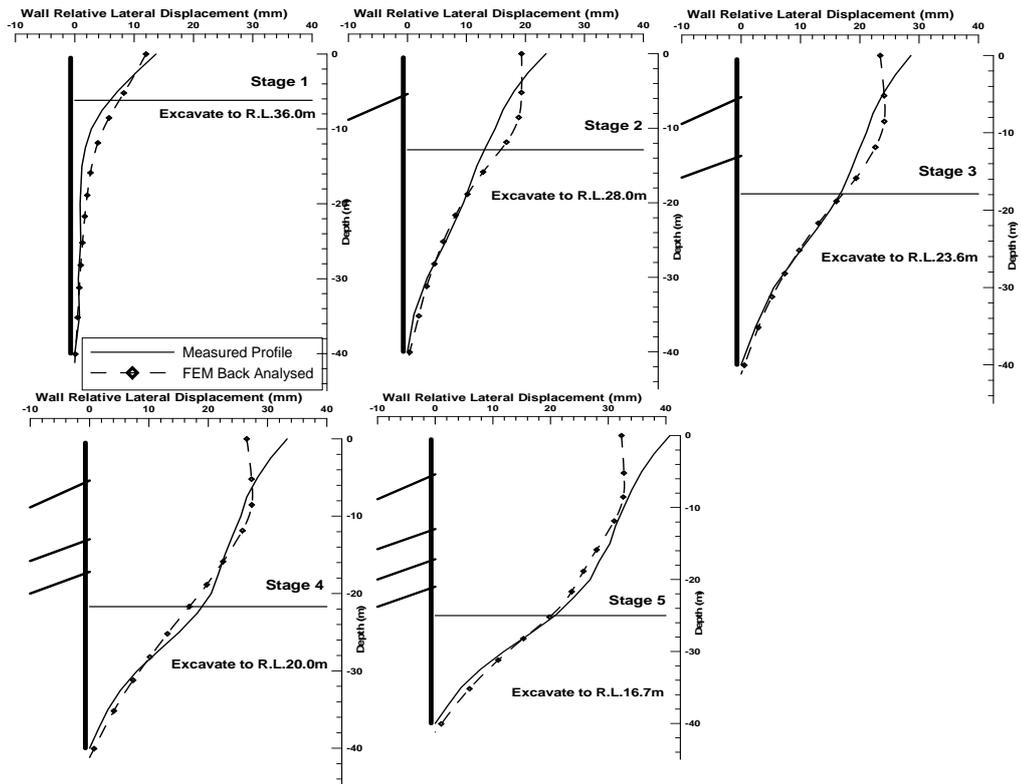


Fig. 24. Relative lateral displacement profile for Wall B

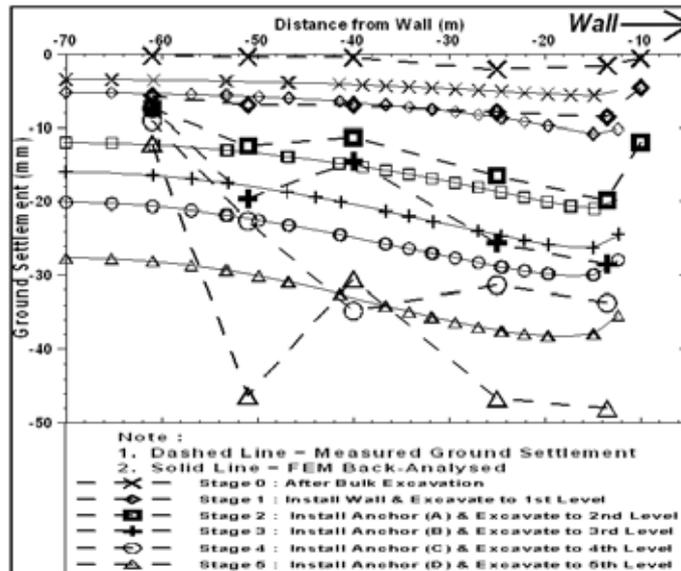


Fig. 25. Ground surface settlement profile

Figure 26 shows the analysed and measured ground surface settlement profiles plotted in dimensionless settlement profiles recommended by Clough & O'Rourke (1990). It is observed that the settlement profile generally extended up to a distance of 5 to 12 times the depth of excavation instead of 3 times presented by Clough & O'Rourke (1990) for stiff to very hard clay. This is because residual soil is usually of higher permeability than clay therefore seepage flow might have caused some lowering of groundwater level in the retained ground which can extend to distances well beyond 3 times the depth of excavation. For excavation in soft clay, the extent of the ground settlement can extend to as far as 50 times the depth of excavation if control of seepage was not properly carried out (Gue & Tan, 1998b). On the other hand, the percentage of magnitude of maximum settlement over the depth of excavation for both measured and back-analysed is about 0.2% which is smaller than the 0.3% presented by Clough & O'Rourke (1990).

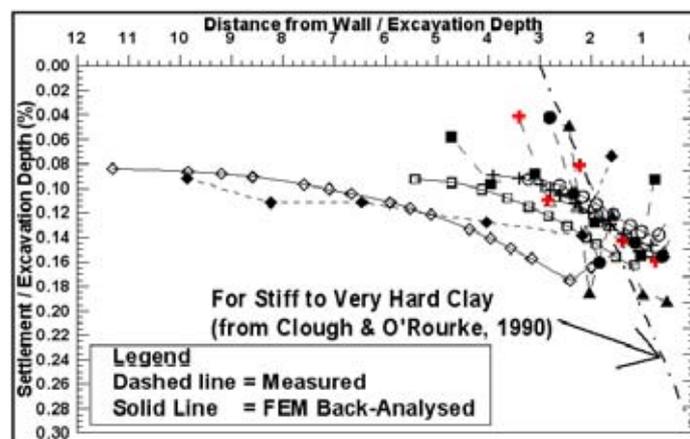


Fig. 26. Dimensionless ground surface settlement profile.

In Malaysia, Standard Penetration Tests (SPT) which are commonly carried out at site to obtain SPT 'N' values were correlated to obtain strength and stiffness parameters of the residual soils for design because this is economical and easily available. This is based on the contention that the SPT 'N' values (blows/300mm), which were obtained extensively at site, adequately represent the mass of the residual soil which is very heterogeneous. Sampling and testing for representative parameters in the very heterogeneous residual soil are difficult to carry out. Pressuremeter tests are also carried out at times to refine the selected soil parameters.

From the FEM back-analysis, the suggested correlations between SPT 'N' values and the effective Young's Modulus (E') and effective unloading/reloading stiffness (E'_{ur}) used in the "Hardening Soil" model are as follows:

$$E' = 2000 \times \text{SPT 'N'} \text{ (kN/m}^2\text{)} \quad (1)$$

$$E'_{ur} = 3 \times E' = 6000 \times \text{SPT 'N'} \text{ (kN/m}^2\text{)} \quad (2)$$

10.2 Solaris 2 Dutamas

This case history is somewhat unique as the retaining wall system used is the soil nail wall instead of the more traditional contiguous bored pile (CBP) wall, diaphragm wall or secant pile wall. The total depth of the excavation is approximately 30m and the close proximity of adjacent structures to the deep excavation pose unique challenges to the design of the soil nailing. The cross-section of the soil nail wall is shown in Figure 27. Figure 28 shows the soil nail wall during construction taken in 2006. As can be seen from Figure 28, existing double-storey residential houses are in close proximity to the soil nail wall and therefore, the movement criteria of the soil nail are very important.

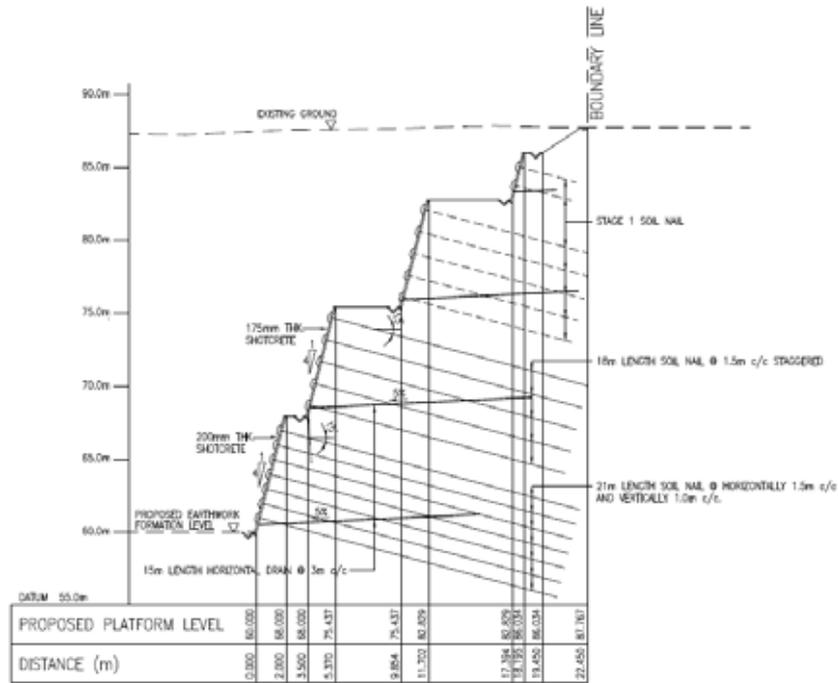


Fig. 27. Typical cross-section of soil nail wall.



Fig. 28. View of soil nail wall during construction (photo taken in 2006).

The design of the soil nail wall had been carried out in accordance with FHWA (1998) and the design method has been summarised by Chow & Tan (2006). The soil nail wall consists of nails ranging from 4m to 21m in length.

This case history serves to illustrate the benefits of geotechnical input during the early stages of the project, as modifications to the architectural layout to accommodate the soil nail wall have resulted in significant cost savings to the basement construction while maintaining the functionality of the building. Cooperation between the client, architect, structural engineer and geotechnical engineer during the initial planning of the building has resulted in a significantly cheaper retaining system for the basement construction as illustrated in Figure 29. The original location of the basement wall would have resulted in construction of retaining walls and support systems to support a vertical wall more than 25m deep with close proximity to structures on the adjacent land. The design of such high walls will be costly and it will also affect the construction programme significantly as top-down construction with the floor slabs acting as support for the wall would be required. The changes made to the architectural layout of the buildings to accommodate the soil nail wall have resulted in significant cost and time savings to the project.

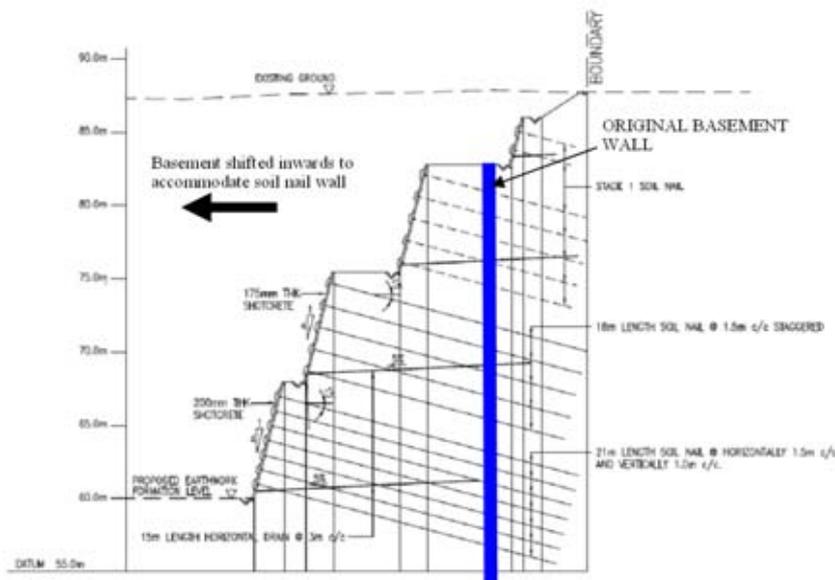


Fig. 29. Shifting of the basement to accommodate a soil nail wall resulted in significant cost and time savings to the project.

A monitoring programme consisting of settlement markers, standpipe piezometers and inclinometers was implemented to monitor the performance of the soil nail wall. Monitoring works were carried out for approximately 1.5 years starting from May 2006 to October 2007. Total movement of the wall is very small where the recorded movement of the inclinometers is less than 5mm. Figure 30 shows the readings of one of the inclinometers (a total of 3 inclinometers were installed). The very small movement recorded is probably due to the inclinometer being installed after the first two berms of soil nails had been installed, where some movement is believed to have taken place. Nevertheless, the monitoring performance has indicated that the performance of the soil nail wall is satisfactory and no cracks on the adjacent houses were reported.

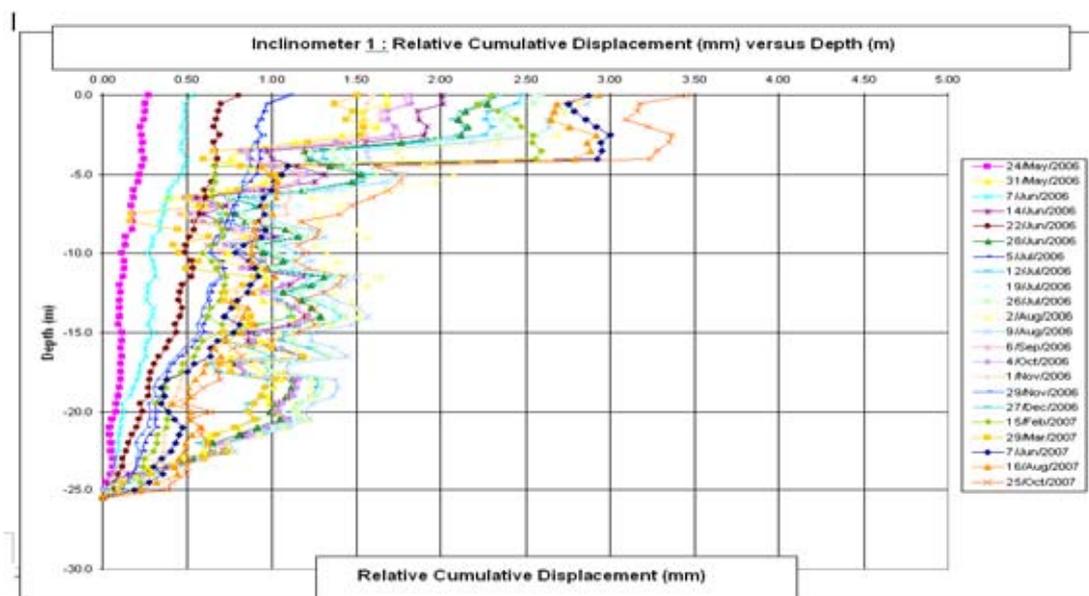


Fig. 30. Inclinometer readings for Inclinometer 1.

REFERENCES

- Atkinson, J.H. 2000. Non-linear soil stiffness in routine design (40th Rankine Lecture). *Geotechnique*. Vol. 50 (5): 487-508.
- Bjerrum, L. 1963. *Discussion Proc. Eur. Conf. SM&FE*. Wiesbaden, Vol. 2: 135.
- Boone, S.J., Westland, J. & Nusink, R. 1999. Comparative evaluation of building responses to an adjacent braced excavation. *Can. Geotech. J.* 36: 210-223.
- Boscardin, M.D. & Cording, E.J. 1989. Building response to excavation-induced settlement. *Journal of Geotechnical Engineering*. Vol. 115, No. 1: 1-21.
- Burland, J.B. & Kalra, J.C. 1986. Queen Elizabeth II Conference Centre: geotechnical aspects. *Proc. Instn Civ. Engrs.* Part 1, 80: 1479-1503.
- Cheng, R. 1996. *Design Tables to BS8007: Design of Concrete Structures for Retaining Aqueous Liquids*. Thomas Telford, London.
- Chiew, S.P. & Leow, B.H. 2006. Common Mistakes in Steel Design. *International Symposium on Worldwide Trend and Development in Codified Design of Steel Structures*. Kuala Lumpur, Malaysia.
- Chiew, S.P. & Yu, Y. 2006. Behaviour of Strut-Waler Connections with Different Stiffening Details. *The Structural Engineer, Journal of the Institution of Structural Engineers*. Vol. 84, No. 14: 28-31.
- Chow, C.M. & Tan, Y.C. 2006. Soil Nail Design: A Malaysian Perspective. *International Conference on Slopes*. Malaysia.
- Chua, T.S. & Prasanthee, R. 1997. Performance of temporary removable U-turn ground anchors in Singapore. *Proceedings of 3YGE, Singapore*.
- Clayton, C.R.I., Matthews, M.C. & Simons, N.E. 1995. *Soil Investigation (2nd Edition)*. Blackwell Science, Oxford.
- Clough, G.W. & O'Rourke, T.D. 1990. Constructed Induced Movements of In-situ walls. *Proc. ASCE Specialty Conference, Cornell*. ASCE Geo Special Publication 25.
- COI. 2005. *Report of the Committee of Inquiry into the Incident of the MRT Circle Line Worksite that led to the Collapse of Nicoll Highway on 20 April 2004*, presented to Ministry of Manpower on 10 May 2005.
- FHWA. 1998. *Manual for Design & Construction Monitoring of Soil Nail Walls*. Federal Highway Administration, US Department of Transportation, USA.
- Gue, S.S. & Tan, Y.C. 1998a. Performance of Anchored Diaphragm Walls for Deep Basement in Kuala Lumpur, Malaysia. *Proc. of the 13th Southeast Asian Geotechnical Conference*, Taiwan: 511-517.
- Gue, S.S. & Tan, Y.C. 1998b. Design and Construction Considerations for Deep Excavation. *Lecture Notes*, The Institution of Engineers, Northern Branch, Penang.
- Hardin, B.O. 1978. The Nature of Stress-strain Behavior for Soils. *Conference on Earthquake Engineering and Soil Dynamics*. ASCE: 3-90.
- Institution of Structural Engineers. 1975. *Design and Construction of Deep Basements*. Institution of Structural Engineers. London.
- Kohsaka, N. & Ishizuka, K. 1995. Stability of bottom during deep excavations – A survey of Japanese codes. In Fujita & Kusakabe (eds), *Underground Construction in Soft Ground*: 323-328. Balkema, Rotterdam.
- Massarsch, K.R. 2004. Deformation properties of fine-grained soils from seismic tests. *Keynote Lecture*, International Conf. Site Characterization. Porto.
- Mayne, P.W. 2000. Enhanced Geotechnical Site Characterization by Seismic Piezocone Penetration Tests. *Invited Lecture*, Fourth International Geotechnical Conference. Cairo University: 95-120.
- Moh, Z.C. & Hwang, R.N. 2007. Lessons Learned from Recent MRT Construction Failures in Asia Pacific (Opening Keynote Address). *Proc. 16th SEAGC*. Subang Jaya, Malaysia.
- Padfield, C.J. & Mair, R.J. 1984. Design of retaining walls embedded in stiff clay. *CIRIA Report No. 104*. Construction Industry Research and Information Association, London.
- Pappin, J.W., Simpson, B., Felton, P.J., & Raison, C. 1986. Numerical Analysis of Flexible Retaining Walls. *Symp. on Computer Applications in Geotechnical Engineering*, the Midland Geotechnical Society.
- Peck, R.B. 1969. Deep excavations and tunnelling in soft ground. *Proc. 7th ICSMFE*. Mexico, State-of-the-art vol.: 225-290.
- Potts, D.M. 2003. Numerical analysis: A virtual dream or practical reality? (42nd Rankine Lecture). *Geotechnique*. Vol. 53 (6): 535-573.
- Puller, M. 1996. *Deep Excavations: A practical manual (1st Edition)*. Thomas Telford, London.
- Schweiger, H.F. 2003. PLAXIS benchmark no. 2: Excavation 1: results. *Bulletin of the PLAXIS Users Association*. Delft (13): 5-8.
- Simpson, B. & Yazdchi, M. 2003. Use of finite element methods in geotechnical limit state design. *LSD2003: International Workshop on Limit State Design in Geotechnical Engineering Practice*. Cambridge, Massachusetts.
- Tan, S.A. 2007. 2-Day Workshop on Advanced Computational Geotechnics: Finite Element Analysis. *Course Notes*. Petaling Jaya, Selangor, Malaysia.
- Tan, Y.C. 1997. Deformations of Anchored Diaphragm Walls for Deep Basement at Berjaya Star City, Kuala Lumpur. *Proc. of the Third Asian Young Geotechnical Engineers Conference*. Singapore: 121-129.
- Tan, Y.C., Liew, S.S., Gue, S.S. & Talha, M.R. 2001. A Numerical Analysis of Anchored Diaphragm Walls for a Deep Basement in Kuala Lumpur, Malaysia. *Proc. of the 14th Southeast Asian Geotechnical Conference*. Hong Kong : Vol. 1 : 453-458
- Tanaka, T. & Verruijt, A. 1999. Seepage failure of sand behind sheet piles – The mechanism and practical approach to analyze. *Soils and Foundations*. Vol. 39, No. 3: 27-35.
- Terzaghi, K. 1943. *Theoretical soil mechanics*. Wiley, New York.
- Twine, D. & Roscoe, H. 1999. *Temporary propping of deep excavations – guidance on design (CIRIA C517)*, CIRIA, London.

- Vermeer, P.A. & Brinkgreve, R.B.J. 2002. PLAXIS: Finite Element Code for Soil and Rock analyses (Version 8). Rotterdam, Balkema.
- Wood, D.W. 2004. *Geotechnical Modelling*. Taylor & Francis, London.
- Yong, K.Y. & Lee, S.L. 2007. Collapse of Nicoll Highway – A Global Failure at the Curved Section of a Cut-and-Cover Tunnel Construction (Chin Fung Kee Lecture). *Proc. 16th SEAGC*. Subang Jaya, Malaysia.