1.0 INTRODUCTION

In Malaysia, deep basements have been extensively constructed, especially in the last 5 years, to effectively utilise the underground space for car parks and other usage in the expensive and congested urban area. The deepest excavation for basement that has been completed to date is 28.5m and is in Kuala Lumpur. Lately, there were a number of failures of support system or retaining wall used for deep excavation. The failures of the retaining wall or support system can be catastrophic affecting the serviceability of adjacent structures. This paper presents, in a selective way, the design and construction considerations for deep excavation. The commonly occurred mistakes and carelessness are also highlighted. Finally a case history describing the influence of excavation on the surrounding structures is presented and discussed.

2.0 DESIGN CONSIDERATIONS

In this paper, the major design considerations for deep excavation is divided into five sections as follows:
(a) Planning of subsurface investigation and laboratory testing.
(b) Evaluation of foundation of adjacent properties and their tolerances.
(c) Selection of type of retaining wall.
(d) Selection of type of support system.
(e) Design of retaining wall.

It is imperative that preliminary analyses be carried out for many options of the walls and support systems to assess on the cost and time of construction together with the technical requirements on the safety and its influence on the adjacent structures before the selection on the final option to produce safe and economical design.
2.1 PLANNING OF SUBSURFACE INVESTIGATION AND LABORATORY TESTING

As stated in BS8002:1994 “Code of practice for earth retaining structures”, sufficient information should be obtained on the ground and ground water conditions together with the strength and deformation properties of the soils which will be retained and the soils which will support the earth retaining structures. Geological maps memoirs and handbooks should be consulted together with any other source of local knowledge. Proper planning and supervision of subsurface investigation (S.I.) and laboratory testing is utmost important for the designer to produce safe and economical design for the deep basement construction. For the selection of geotechnical parameters for design of retaining walls, reference can be made to Gue (1997 & 1998) and CIRIA Report 104 (1984).

2.1.1 Field Tests

The code of practice for site investigation BS5930:1981 describes the general considerations to be taken into account and details the methods of site investigation available. Generally, a number of boreholes should be adequate to establish the ground conditions along the length of the wall and to ascertain the variability in those conditions. Piezocone tests would complement the boreholes particularly in sedimentary loose or soft deposits. For a large site, geophysical survey would be an advantage to optimise the subsurface investigation.

Prior to planning of S.I., it is important to acquire the geology formation of the site. The geological information of all states in Malaysia can be purchased from Geological Department of Malaysia. As recommended in BS8002:1994, the centres between boreholes will vary from site to site but should generally be at intervals of 10m to 50m along the length of the wall depending on the complexity of the geology, subsoil profile and adjacent structures. Some times, subsurface investigation outside the site should be carried out especially around the sensitive adjacent structures. In addition, dilapidation survey of the adjacent structures should also be carried out as required by local authorities and assessment of the effect due to the deep excavation as described in details in Sections 3.1 and 2.2 respectively.

In Malaysia, the most common field test carried out for design of retaining wall comprises of rotary wash boring (borehole) and includes Standard Penetration Tests (SPT),
Pressuremeter tests, collection of disturbed and undisturbed soil sampling for laboratory testing. If the site is underlain by soft clayey material, field tests such as piezocone tests, in-situ penetrating vane or vane shear tests in the borehole are commonly required. The details on the selection of suitable field tests for various types of subsoil of different geological history can refer to Gue (1997a), Geoguide 2 published by Geotechnical Control Office of Hong Kong in 1993.

In deep excavation design, an adequate knowledge of the ground water levels, seepage pressures and information on the existence of any hydrostatic uplift pressures are essential. Preliminary ground water conditions may be predictable from a knowledge of the local geology. Standpipies or piezometers should be installed in the drilled boreholes to determine and confirm the ground water conditions at site. It should be noted that water levels encountered during boring operations where water is used as a flushing medium are unreliable and seldom represent equilibrium conditions.

Sealing of the boreholes after completion is also important to prevent collapse of the soil causing loosening of the subsoil. Holes left open also pose a safety treat to human and animals. Boreholes in area with potential “blow-out” of ground water when carrying out excavation also need to be carefully sealed. Usually grout is used to seal the hole.

2.1.2 Laboratory Tests

Laboratory tests that are usually carried out in Malaysia on disturbed and undisturbed soil samples collected from the boreholes and are summarised as follows:

(a) Particles size distribution like sieve analysis including clay and silt separation using hydrometer

(b) Atterberg limits tests to determine liquid limit, plastic limit and plasticity index.

(c) Test to determine moisture content, porosity, unit weight, specific gravity.

(d) Chemical tests on pH, chloride, sulphate and organic content.

(e) Shear strength tests like Unconsolidated Undrained Triaxial Test (UU), Isotropically Consolidated Undrained Triaxial Test with pore water pressure measurements, Unconfined Compression Test (UCT), etc.

(f) Consolidation test.

(g) Test on rock cores using UCT for strength and strain gauges can be attached to the rock sample to measure modulus of the rock from the test.
2.2 **EVALUATION OF FOUNDATIONS OF ADJACENT PROPERTIES AND THEIR TOLERANCES**

Major concern during the planning and execution of deep excavation is the impact of construction related to ground movements on the adjacent properties and utilities. During excavation, the state of stresses in the ground mass around the excavation changes. The most common changes in stresses in the retained side are the stress relieve on the excavation face resulting in horizontal ground movement and follows by vertical movement for equilibrium and increase in vertical stress due to lowering of water table resulting in both immediate and consolidation settlement of the ground. These ground movements that vary away from the excavation can cause buildings, especially those on shallow foundation, to translate, rotate, deform, distort and finally sustain damage if the magnitude exceeded the tolerable limits as shown in Figure 1.

It is important to carry out analyses to estimate the magnitude and distribution of the ground movements due to the proposed excavation. Section 2.5 of this paper on “Serviceability Limit States” summarises some of the methods to predict deformation of the retained ground due to an excavation. The tolerance of the structures and utilities to the deformations and distortions sustained as a result of the ground movements should also be evaluated.

**Building Damage due to Ground Movements**

Structural damage affecting the stability structure is usually related to cracks or distortions in primary support elements such as beams, columns, and load bearing walls. Table 1 presents the classification system that provides a defined framework for the evaluation of the damages.

There are many approaches to address the subject of building damage due to ground movements. Usually, a simple and conservative method, which is the empirical approach, is needed in the preliminary assessment. Figure 2 shows the symbols and definitions of foundation movement normally used. This paper will only briefly elaborate simple empirical approach. For more rigorous and extensive methods, reference can be made to Boone (1996), Boscardin et al. (1989) and Poulos & Chen (1997).

**Empirical Method**

Skempton & MacDonald (1956) indentify a basis on which to determine allowable total and differential foundation settlement. Guidance for design has been largely based on their work and shown in Table 2. There are three important points to be noted in their studies:

(a) Confined to traditional mill-type steel-framed industrial buildings, reinforced-concrete framed buildings with traditional cladding, and some load bearing masonry wall buildings.
(b) The criterion for limiting deformation was the ‘angular distortion’ or relative rotation as shown in Figure 2.
(c) No classification of degree of architectural or visible damage was used.

Meyerhof (1956) and Polshin & Tokar (1957) recognise that unreinforced load bearing walls have a different mode of deformation from that of framed structures and recommended that the deflection ratio $\Delta/L$ to be used as stated in Table 2.

### 2.3 SELECTION OF TYPE OF RETAINING WALL

Selections of retaining wall type and support system are usually made on the basis of:

(a) Foundation of adjacent properties and services
(b) Designed limits on wall and retained ground movements
(c) Subsoil conditions and ground water 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 available construction plant
(h) Maintenance of the wall and support system in permanent condition

The commonly used retaining wall types in Malaysia to support excavations as shown in Figure 3 are:

(a) sheet pile wall
(b) soldier pile wall (soldier piles and horizontal lagging)
(c) contiguous bored piles wall
(d) secant piles wall
(e) diaphragm wall

### 2.3.1 Sheet Pile Wall

Sheet pile wall is commonly used as temporary retaining wall system in Malaysia. The suitability of sheet pile to be used in basement construction is generally influenced by the following factors:
(a) **Soil conditions and the ease of pile installation** :

The subsoil must allow the sheetpile to be easily driven in with Standard Penetration Test (SPT) ‘N’ values lower than 50 or else it would be difficult to achieve the required penetration. The selection of sheet pile to be used would depend on the requirement of the flexural strength and strength to resist driving. Driving of sheet piles in loose sandy soils can also result in settlements in adjacent ground.

(b) **Depth of excavation** :

Sheet pile is usually suitable for shallow excavation and as temporary works due to its lower stiffness compared to other types of retaining wall such as diaphragm wall, contiguous bored piles or secant piles.

(c) **Watertightness** :

Some seepage is expected to pass through the interlocking steel sheet piling if there is a difference in hydraulic head.

(d) **Ability to withdraw temporary sheet pile after used** :

It would be more economical if extraction of any of the temporary sheet piles is allowed. However, extraction causes vibration unless silent piler is used and also lateral soil movement when the void created during extraction collapses.

### 2.3.2 Soldier Pile Walls (Soldier Piles and Horizontal Lagging)

Soldier pile wall has two major components; soldier piles (vertical component) and lagging (horizontal component). Soldier piles usually consist of steel H sections and are driven to maintained in full contact with the soil. Its installation resistance is quite similar to the sheet pile. Soldier piles provide the primary support to the retained soil, and lagging serves as a secondary support to the soil face. Lagging prevents progressive deterioration of the soil arching between piles.

In Malaysia, soldier pile wall is normally used for small shallow excavation in stiff soils and in soils above ground water table as temporary support. Soldier pile wall is not suitable for soft clays and loose sands.
2.3.3 **Contiguous Bored Piles Wall**

Contiguous bored piles wall can be both temporary or permanent wall for excavation. Usually contiguous bored piles wall is used in stiff soil and lower water table. The advantages of contiguous bored piles wall are lower cost and speed in construction for temporary or permanent wall where drilling conditions are conducive. The system has higher capacity to overcome obstructions like rock compared to other system. However additional works are needed to form an acceptable surface to the wall.

2.3.4 **Secant Piles Wall**

The major disadvantage of contiguous bore piles walls which is the lack of watertightness has been effectively overcome by interlocking as in secant piles wall. Other than watertightness, secant piles wall has similar advantages and disadvantages as contiguous bored piles wall. This method consists of boring and concreting primary piles, at centre to centre spacing of slightly less than twice the nominal pile diameter. Secondary piles are then bored at mid-distance between the primary piles before the concrete has achieved its full strength. Reinforcement is usually concentrated in secondary piles. The main advantage of the secant pile is the possible full temporary protection in sensitive and collapsible soils and ease of coring into rock.

2.3.5 **Diaphragm Wall**

Diaphragm wall is commonly used in Malaysia as a permanent wall system. Diaphragm wall offers most efficient watertightness compared to other wall types. Similar to contiguous bored piles and secant piles wall, diaphragm wall construction also causes minimum noise and vibration disturbance. However, it is not suitable for highly collapsible soil during trenching.
2.4 **SELECTION OF SUPPORT SYSTEM**

The above retaining wall types can be further divided into following three major categories according to the form of support provided as shown in Figure 4:

(a) cantilevered or unbraced wall (usually for shallow excavation)
(b) strutted or braced wall
(c) tied-back or anchored wall

Table 3, from Institution of Structural Engineers (1975), lists the advantages and disadvantages of each support system and shall be read in conjunction with Figure 4(a) to 4(m). The factors involved in the selection of a support system for a deep excavation as suggested in Navfac Design Manual 7.2 by US Navy (1982) are summarised in Table 4.

Other than the factors listed above, it is very important to note that although most of the time ground anchor support system looks very attractive with unobstructed excavation in centre of the site, there are some major factors that should be considered by the Engineer prior to adopting the system:

(a) Permanent ground anchors always pose great problem in maintenance in long term. Refer BS8081:1989 for details.
(b) If the local authorities require temporary ground anchors to be removed after use, then removal of temporary ground anchors may pose some problems if the system has not been proven at site to be fully removable.
(c) Approval from the adjacent owners should be acquired if there is encroachment of ground anchors into adjacent properties.
(d) Leakages and loss of fine through drill holes need additional precautionary measures in the construction.

2.5 **DESIGN OF RETAINING WALL**

In the design of earth retaining system for deep excavation, it requires both ultimate limit states and serviceability limit states to be considered. An ultimate limit state of a structure is deemed to have been reached when sufficient parts of the structure, the soil around it, or both have yield to result in the formation of a failure mechanism in the ground or severe damage in the principal structural components. A serviceability limit state of a structure is deemed to have been reached with the onset of excessive deformation or deterioration. Figure 5 shows some of the possible failure mechanisms of retaining wall. Other than the limit states need to be addressed, the short term...
(usually undrained for cohesive materials) and long term (drained) behaviour of the subsoil need to be considered in the design.

The check on the ultimate limit states of the wall includes check on the following:

(i) Overall Stability : the provision of sufficient embedment depth to prevent overturning of the wall and overall slope stability.

(ii) Basal Failure : the wall penetration depth must be sufficient to prevent basal failure in front of the wall after excavation to formation level.

(iii) Hydraulic Failure : the penetration of the wall must be sufficient to avoid piping or ‘blow out’ in front of the wall after excavation to formation level.

Other than the modes of failure stated above, the external forces acting on the walls and supports also need to be evaluated in the design and some of them are illustrated in Figure 5.

The serviceability limit state is to be considered in terms of wall and soil deformation at the rear of the wall. Usually the check on the serviceability limit state is carried out using computer program. There are also some empirical or semi-empirical methods to predict the deformation of the wall and retained soil. Special care should be given to control of ground water level in the retained ground in the design because lowering of ground water level would increase settlement and induce damages to the adjacent structures and services.

2.5.1 Overall Stability

The overall stability of both retaining walls is often evaluated using limit equilibrium methods of analysis in which the conditions of failure are postulated, and a factor of safety is applied to prevent its occurrence. There are a number of ways of applying the factor of safety for the overall stability. They are:

(a) Factor on embedment :
A factor of safety is applied to calculate embedment depth at limiting equilibrium. The method is described in the US Steel Sheet Piling Design Manual (United States Steel Corporation, 1975), the British Steel Corporation Pilling Handbook (1988) and by Symons (1983).
(b) **Factor on moments of gross pressure** :
This method applies a factor of safety to moments of gross pressure on the passive side only. Water pressure is not factored. The method is described in NAVFAC Design Manual 7.2 by US Navy (1982).

(c) **Factors on moments of net total pressure** :
The net horizontal pressure distribution acting on the wall is calculated and the factor of safety is defined as the ratio of moments of the net passive and active forces.

(d) **Factors on net passive resistance or Potts and Burland Method** :
Developed by Potts and Burland (1983) and is analogous to the calculation of the bearing capacity for a strip load. This method defines the factor of safety as the ratio of the moment of the net available passive resistance to the moment activated by the retained material including water and surcharge.

(e) **Factors on shear strength on both active and passive sides** :
Soil shear strengths are reduced by dividing $c'$ and $\tan \phi'$ by factor of safety, and the active and passive pressure diagrams are calculated using these reduced values. The reduced values approximate to mobilised values. Bending moments and prop loads derived from the calculation can be used for wall design if they are treated as ultimate limit state values. This method is recommended in BS8002, 1994.

(f) **Factor on shear strength of passive side only** :
The passive resistance is factored but no factor is applied to the active side.

The GCO Publication No. 1/90 titled “Review of Design Methods for Excavation” published by Geotechnical Control Office of Hong Kong in 1990 has presented some useful observations on the different methods and they are as follows:

(a) The factor on embedment is empirical and should be checked by applying a second method.

(b) The method of factoring moments on gross pressure may give excessive penetration at low angles of shearing resistance, so use varying factors for different ranges of $\phi'$.

(c) The factoring of net passive pressure moments tends to give high penetration values.

(d) The factors on modified net passive resistance as recommended by Burland and Potts (1983) appear to give consistent results in a reasonable range of soils and wall dimensions.

CIRIA Report 104 “Design of Retaining Walls Embedded in Stiff Clay” by Padfield and Mair (1984) has recommended some factors of safety for use in stiff clays with the above
methods and are listed in Table 5. Although applying to stiff clays, the factors of safety can also be used as indicative values in granular soils and residual soils.

For cantilever and single prop walls, particularly on sloping sites in soft clays and loose granular soils, deep-seated slip failure (slope stability failure) also needs to be evaluated.

2.5.2 Basal Heave Failure

Usually base failure to an excavation by upward heave applies particularly in very soft and soft clays and silty clays. Stiff soils less prone to encounter this problem. The basal heave failure is analogous to a bearing capacity failure, only in reverse being that stresses in the ground are relieved instead of increased.

There are many methods to examine the basal heave failure and may be broadly divided according to basic concepts such as those based on bearing capacity formulae and those based on examination of moment equilibrium. It is recommended that in the design, both methods are to be used for basal heave check.

**Method based on Bearing Capacity Formulae :**

The methods based on bearing capacity formulae are presented by Terzaghi (1943) for shallow and wide excavation, whereas that by Bjerrum & Eide (1956) is suitable for deep and narrow excavations. Both methods neglect the effect of wall penetration below foundation level and results may be conservative especially where stiffer clays exist with depth. Figure 6 shows the details of above two methods. For completeness, the methods for calculating factors of safety against basal heave in cohesionless soils, cuts in clay of considerable depth and cuts in clay limited by hard stratum as described in NAVFAC Design Manual 7.2 (1982) are presented in Figure 7. Generally the factor of safety against basal heave failure should not be less than 1.5.

**Method based on Moment Equilibrium :**

The methods to evaluate basal heave failure based on examination of moment equilibrium are described in Japanese Codes such as Architectural Institute of Japan (1988 Revision 1) and Japan Society of Civil Engineers (1986 Revision 6). Figure 8 presents the summary of the two moment equilibrium methods. In these methods, excavation width and excavation length can not be taking into consideration but it is possible to include variations of shear strength in the direction of depth. With the moment equilibrium methods, the factor of safety is generally required to be not less than 1.2.
### 2.5.3 Hydraulic Failure

For excavation at a site where groundwater on the retained side exists above the base of the excavation or under artesian pressure, analyses need to be carried out to prevent hydraulic failure. If the toe of the wall does not penetrate into an impermeable layer or to a sufficient depth, instability of the base caused by piping occurs if the vertical seepage exit gradient at the base of the excavation is equal to unity. Figures 9 and 10 present the design charts for wall penetration required for various safety factors against heave or piping in isotropic sands and in layered subsoil respectively as recommended in NAVFAC Design Manual 7.2 (1982). Usually a safety factor of 1.5 to 2.0 is provided to prevent piping.

The boiling (piping) of the excavation base can also be checked using Terzaghi’s method and the critical hydraulic gradient method that mainly consider vertical flow in the vicinity of the excavation bottom. Figure 11(a) shows the summary of the above two methods. The Japanese codes mostly suggest the use of Terzaghi’s method with factor of safety ranges from 1.2 to 1.5 for temporary and permanent works respectively. For the critical hydraulic gradient method, the suggested factor of safety is 2.0.

To prevent heaving due to artesian pressure, the equilibrium between overburden pressure and pore water pressure at the top surface of confined aquifer (bottom surface of clayey soil) need to be evaluated as shown in Figure 11(b) and usually factor of safety of 1.2 is sufficient.

### 2.5.4 Earth Pressures for Structural Design

For deep excavation, usually the multi-level supported walls are used instead of cantilever or singly supported walls. The design requirements and analyses for multi-level supported walls are different from cantilever or singly supported wall. The earth pressures that act on multi-level strutted walls or multi-level tied-back walls depend on the wall stiffness relative to the soil, the support spacing and the prestress load. The method of construction of these walls is usually sequential, installing the wall and excavation in stages followed by installation of support like anchor or prop at each installation stage. The available methods for analysis and design of multi-level supported walls can be categorised as follows:

(a) **Empirical Methods**:

Usually based on strut load envelopes recommended by Peck (1969) or Terzaghi and Peck (1967) for three categories of soil: sands, soft to medium clays and stiff clays.
(b) **Computer Methods based on Winkler Spring Theory**:

This method is sometimes called Beam-Spring Approaches.

(c) **Full Soil-Structural Interaction Analysis**:

Employ either Finite Element Method (FEM), Boundary Element Method or Finite Difference Method.

(a) **Empirical Methods**

The strut load envelopes developed by Terzaghi & Peck (1967) are presented in Figure 12. These diagrams do not represent the actual earth pressure or its distribution with depth, but load envelopes from which strut loads can be evaluated. Clay is assumed “undrained” and only considers total stresses. Sands are assumed “drained”. If non-permeable wall is used, hydrostatically distributed water pressure should be added to strut loads. Wall can be designed using the Coulomb earth pressure distribution with hydrostatic water pressure except full drainage occurs through the wall.

Gue and Tan (1998) observe that for anchored diaphragm walls in Kenny Hill residual soils in Kuala Lumpur, the apparent lateral earth pressures that were obtained from the load cells indicated that for anchors at depths greater than 60% of the maximum excavation depth of more than 20m, the apparent earth pressure obtained is larger than the values suggested by Terzaghi & Peck (1967).

(b) **Beam Spring Approach**

This method needs the help of computer program. The soil is either modelled by a set of unconnected vertical and horizontal springs (Borin, 1989) or a set of linear elastic interaction factors (Papin et al., 1985). This approach allows the deformation of the wall to be predicted but can not calculate the deformation of the soil in front or behind the wall. Props or anchors are modelled as simple springs.

An example of this type computer program widely used in Malaysia, is WALLAP (Borin, 1989) and a more sophisticated implementation is provided by FREW (Pappin et al., 1986). The user is allowed to impose active or passive limits on the effective pressures applied to the wall. Gue and Tan (1998) show that the ground anchor loads obtained from analysis using computer program FREW are within 10% of the loads measured from the
load cells installed at the anchor heads of diaphragm walls installed in Kenny Hill residual soil, Kuala Lumpur.

(c) Soil-Structure Interaction Methods

This method will be able to model wall and soil deformation and stress in a realistic stages of operations that follow actual construction sequence. Pre-judged failure modes are not required in the analysis. This method can be carried out in two or three-dimensional depending on the computer codes used. Usually two-dimensional is sufficient.

This method is particularly useful in predicting deformations of wall and soil for serviceability checks especially there are deformation sensitive structures around the excavation. Use of soil stiffness at low strain value is essential in this approach. Tan (1997) has presented correlation to acquire the stiffness of Kenny Hill residual soils at low strain.

Some of the computer packages in this category include Finite Element program CRISP-90 (Britto and Gunn, 1987) and Plaxis, and the finite difference package FLAC (ITASCA, 1991).

2.5.5 Serviceability Limit States

Serviceability limit state check for retaining walls involves solution of soil-structures interaction problems that require the use of deformation parameters and generally divided into two major items:

(a) Deformation of Wall

The acceptable limits of the wall deformation will depend on the purpose of the excavation and whether the works are temporary, permanent and the permissible deformation of soil behind the wall.

(b) Deformation of Soil behind the Wall

Settlement and lateral movements of the soil behind wall must not exceed the permitted deformation of surrounding buildings and services. The guide on the limiting deformation of framed buildings, reinforced and unreinforced load bearing walls are indicated in Table 2.
Primary factors influencing the deformation of the wall and the retained ground are:
(a) Type of ground
(b) Depth and width of excavation
(c) Stability of the bottom of excavation
(d) Stiffness the support system and preload forces
(e) Rigidity of the wall.
(f) Construction technique

Finite element method (FEM) is most well suited to this type of deformation analysis, however, FEM is too cumbersome and costly for general application and furthermore accurate determination of the required soil parameters is also difficult. In view of this, the magnitude and distribution of ground movements is generally considered based on empirical method such as recorded case histories and often using collations such as Clough and O'Rourke (1990) or methods suggested in by Japanese in Figure 13.

The first empirical method for estimating movements for in-situ wall system was proposed by Peck (1969) as in Figure 14 from data compiled on settlement of the ground adjacent to temporary braced sheet pile and soldier pile walls. Since then, there were improvement and progress in control of movements with the use of updated design and construction technologies.

Clough and O'Rourke (1990) present the settlement profile of retained soil for properly designed and constructed excavation works using different retaining wall types and support system, in different soil types as follows:

(a) Excavation in Sand:
Figure 15(a) summarises settlements for excavation in predominantly sand and granular soils. The maximum settlements are typically less than 0.3% and extended with decreasing values up to a distance of 2 times the maximum excavation depth.

(b) Excavation in Stiff to Very Hard Clays:
Settlements and horizontal movements for excavation sites in stiff to very hard clays are summarised in Figure 15(b). The settlements are only a small percentage of excavation depth, with maximum settlement usually less than 0.3%, but are distributed over 3 times the excavation depth from the edge of the excavation. Generally the average horizontal and vertical movements is about 0.2% and 0.15% of the depth of excavation respectively.
(c) Excavation in Soft to Medium Clays:

Figure 15(c) shows the settlements for excavation in soft to medium clay. Zones pertaining to various levels of workmanship and soil conditions as described by Peck (1969) are also shown in this figure. Different from stiffer soils, basal stability dominates deflections of the excavation in soft to medium clays. Mana and Clough (1981) and Clough et al. (1989) defines movements in terms of factor of safety against basal heave failure and take into consideration the influence of wall stiffness and support spacing as shown in Figure 16. This chart (Figure 16) can be used to assist in predicting maximum lateral wall movements in clayey soils. Generally for soft to medium clays, the maximum settlement is equal to maximum horizontal wall movement. However the chart should be used with caution when FOS is lower than 1.5.

Figure 17 presents dimensionless settlement profiles recommended by Clough and O’Rourke (1990) as a basis for estimating vertical movement patterns adjacent to excavations in different soil types. These diagrams only pertain to settlements caused during excavation and bracing stages of construction. Settlement due to other activities, such as dewatering, deep foundation removal or construction, and wall installation, should be estimated separately.

Bearing in mind that all the empirical methods above are based from the experiences in other countries. Therefore, they may not be relevant and should be used as a preliminary estimate only. Local experiences where available should be referred. In Malaysia, for example there has been cases of lowering of ground water table in the retained soil has caused settlement and extended to more than those quoted above, and could be more than 40 times the depth of excavation.

3.0 CONSTRUCTION CONSIDERATIONS

The complexity of the interaction between the ground and the retaining structures for deep excavation sometimes make it difficult to predict the behaviour of a retaining structure in detail and accurately before the actual execution of the works. Therefore, the involvement of design engineer does not stop after designing the retaining structures. Instead, the design engineer should closely supervise the construction works at site and review the performance of the retaining structure and compare to the design requirements and predictions, and take necessary actions to prevent the occurrence of the critical limit state like large displacement of the wall causing damage.
to nearby structures or services. “Observational Method” proposed by Peck, (1969) is often employed.

Major considerations to be taken during construction can be broadly divided into three major sections as follows:
(a) Dilapidation survey of adjacent structures
(b) Instrumentation and monitoring program
(c) Supervision and construction control

3.1 DILAPIDATION SURVEY OF ADJACENT STRUCTURES

For deep excavation, dilapidation survey of adjacent structures is necessary to prevent unnecessary contractual conflict or even lawsuit. Dilapidation survey also forms part of the requirements by the local authorities. Dilapidation survey should be carried out prior to any construction activities at the site.

Normally some of the adjacent structures, especially old buildings, may have already suffered some cracks prior to construction activities at the site. Therefore, by carrying out dilapidation survey, the developer, consultants, contractor and even the owners of adjacent structures will have a clear picture of the conditions of the structures adjacent to the site. The dilapidation survey report also serves as a reference for any claims by the owner of the adjacent properties on the damages caused by the excavation (if any).

In most of the deep excavation, some movements of the retained ground is commonly expected. The dilapidation survey should cover the area within and beyond the influence zone of the excavation. The dilapidation survey should be thorough and with approval of the adjacent owners. The dilapidation survey should also include the interior of the adjacent structures and settlement of ground especially between suspended and non-suspended structures. If there are cracks on the adjacent structures, the direction of the cracks, the dimension of the cracks like length, width of the cracks should be measured and reported. Photographs should also be taken together with the measuring equipment (measurement tape or ruler) and included in the report.
3.2 INSTRUMENTATION AND MONITORING PROGRAM

In Malaysia, most of the sites requiring deep excavation are located in the urban area with all sides either surrounded by roads, buildings or services. It is very important to have an effective instrumentation and monitoring scheme to make sure that during excavation and construction of the basement, the safety of the surrounding properties can be secured. The instrumentation and monitoring scheme also allows the design engineer to validate the design and to identify the need for remedial measures or alterations to the construction sequence before the serviceability of the retaining structures or the surrounding buildings and services are affected. The instruments particularly the ground settlement markers and piezometers should be extended beyond the normally expected distance perpendicular away from the retaining wall.

For deep excavation, the commonly used instruments and their functions are as follows:

1. **Inclinometers**
   The function of Inclinometer is to allow the deformation of the wall with depth to be measured. Inclinometer access tubes are usually installed in the wall except in sheet pile and soldier pile walls where they are usually installed in the subsoil immediately behind the wall. If the inclinometers are installed in the retained soil behind the wall, the lateral deformation of the subsoil with depth can be measured instead of direct measurement of wall deformation. Usually the tubes are extended about 3m below the toe of the wall so that the wall toe movements can be measured. From the deformation profile obtained from inclinometers for each stage of excavation, the design engineer can validate the design and to take early precautionary or remedial measures if the deformation of the wall exceeds the design limit.

2. **Piezometers or Standpipe Piezometers**
   Piezometer or standpipe piezometer allows the changes of the ground water level in the subsoil to be monitored. It should be located in lines perpendicular to the wall to establish the profile. Piezometer either pneumatic or vibrating wire type also allows sudden change of pore water pressure in the subsoil to be measured. Changes of ground water level are very important because lowering of ground water level would cause settlement of the retained soil. On the other hand, rising of ground water unexpectedly to a level higher than design value can cause additional lateral pressure leading to failure of retaining wall.

3. **Settlement Markers / Displacement Markers**
   Settlement markers are installed on the ground in lines perpendicular to the wall to establish the settlement of the ground surface profile using levelling. If lateral movements of the ground surface need to be measured, displacement markers will be used and surveyed using precision theodolite and electronic distance measurement (EDM).
Sometimes, settlement markers or displacement markers are placed at the columns and floors of surrounding buildings (usually for buildings on shallow foundation) or services to measure the movements. Excessive settlement/lateral movement of the retained soil can cause damage to the adjacent properties as described in Section 2.2.

4. Deep Extensometers
Deep extensometers allow the settlement or heave of the subsoil at different depths to be measured. Deep extensometers sometimes are installed in the site to monitor the heaving of the subsoil due to excavation and as an early indicator of base heave and to establish differential base heave between podium and tower building of a project.

5. Load Cells
Load cells can be installed at the struts or anchors to measure the changes of load with each stage of excavation. The measurement of the load will allow the design engineer to validate the design of the support system and to ensure the support system functions properly.

6. Tiltmeters
Tiltmeters sometimes are used to monitor the change in inclination (rotation) of structural element. They are used in surrounding buildings that are sensitive to tilt as an early warning system.

Practical monitoring program is essential to the success of the instrumentation and monitoring scheme. For deep excavation, usually the monitoring of all instruments are carried out weekly. In the area where activities like construction of the wall, bulk excavation in front of the wall, drilling of ground anchor etc, the frequency of monitoring for the instruments at the affected area should be increased to daily. If there is any sign of increasing wall movement, strut or ground anchor load above the values designed, the frequency of monitor should be intensified to daily until the causes identified and remedial measures carried out.

3.3 SUPERVISION AND CONSTRUCTION CONTROL

In order to ensure the quality of works and safety of the retaining wall for deep excavation, the construction and workmanship shall be closely supervised. It is also a requirement of Uniform Building By-Laws (1984) that the submitting person or his representatives shall supervise the construction ensuring it complies to drawings, specification and use the approved method
statement. Followings are some of the commonly encountered problems at site that requires close supervision and construction control:

1. During the excavation or drilling of the wall, the construction records should clearly highlight types of soils encountered, rock level (if any), any abnormalities like sudden drop of drilling fluid or water gushing out (artesian). Design engineer has to review the records and confirm the validity of the design assumptions like subsoil types, rock levels and water conditions.

2. Design engineer should review the monitoring records of the instruments and carry out back-analysis to validate the design and also to check the performance of the wall as described in Section 3.2.

3. During each stage of bulk excavation in front of the wall, the supervising engineer should make sure that the Contractor follows the predetermined design level to prevent over excavation deeper than the design level. Over-excavation increases additional stresses resulting in the increase in wall movement and deformation of the retained soil. Excessive over-excavation might even cause catastrophic failure of the wall.

4. Surcharging at the retained side of the wall should also be closely monitored. Extra surcharge above the design value increases the soil pressure on the retained soils and may cause increase in wall movement and even failures.

5. The drainage system of the excavated platform in front of the wall is very important because bad surface drainage would cause soaking and softening of the soil in front of the wall and reduces the passive resistance supporting the wall.

6. If prestressed ground anchors are being used as support for the retaining wall, prestressing and locking off of anchors should be carefully carried out at the site. Overstressing of the anchors or locking off the anchors at loads higher than predetermined design values can sometimes cause increase in the anchor load and even to failure. On the other hand, the wall would experience larger movements if the anchors are understressed.

7. Special care in sealing the anchor holes for temporary ground anchors during construction and after construction is very important to prevent further lowering of the ground water level on the retained ground.

8. The drilling technique of ground anchors proposed by the Contractor in the method statement should also be reviewed by the Consultant. For loose soil that is sensitive to loss
of materials through the anchor holes, double acting drilling method with temporary casing should be used.

9. If internal struts are used, the connections between the waler beam and struts should be in full contact and ensure that the required prestress are applied when specified to reduce wall movement.

4.0 CASE HISTORY

Complaints of cracks on walls and some beams on some old shophouses were reported to a local authority. The number of complaints reported increased with time and within a few months, some 344 shophouses were reported to have suffered damages of various severity.

Some of these shophouses had to be propped up to prevent mishap. Figure 18 shows some of the damages. In addition to the cracks, ground settlements were observed, particularly on the non-suspended slab around the new high-rise building as shown in Figure 19. The main activities near the vicinity of the shophouses were the basement excavation and piling work for two proposed commercial complexes. The affected shophouses were within 300m from the edge of the basement excavation.

Project A is a proposed shopping mall with two levels of basement parking and to be supported on 450mm diameter spun piles. The piles along the perimeter of the site had been driven and temporary cofferdam using 15m sheetpile wall supported by internal struts had been completed. Figure 20 shows a typical section of the temporary sheetpile wall during the complaints. The plan area involved was approximately 20,000m². The general depth of excavation was 7m although at locations of lift pits the excavation has gone down to about 10m. Some 40% of the plan area have reached 7m below ground level. After the stop work order was issued by the local authority, water level in the excavation pit increased to about 4m below ground level.

Project B is a proposed 5-storey shop building with two levels of basement and to be founded on spun piles. The spun piles and 15m to 21m of temporary sheetpiles with internal struts had been completed. However, only about 800m² of the total of about, 3,500m² have been excavated and reached 9m below ground level. At the time of the stop work order, the basement was ready for casting of pilecaps and basement slab.

Majority of the complaints was near the Project A and this paper will mainly present the investigation around Project A.
4.1 INVESTIGATION AND FINDINGS

The proposed projects lie within the Quaternary deposits of marine clays. The site is within the old river which has been reclaimed. The boreholes carried out at Project A show that the interbedded layers silty sand in the marine clay as depicted in Figure 21.

During the investigation, 26 number of boreholes were drilled outside of the two proposed projects with the objectives of mapping the subsoil profile, water profile and soil properties within the proposed project sites. Piezometers were installed in most of the boreholes to monitor the water profile almost perpendicular to the edge of the excavation. Settlement profiles along the lines were also measured. In addition, some 306 houses were monitored for the activities of the cracks using tell-tale glass and crack gauge.

The results of the original ground investigation indicate the present of sand layers within the marine clay especially near the toe of the sheetpiles, where a layer of sand of about 7m thick present. The surface water profile during the investigation as indicated by Figure 22 indicates the groundwater on the retending side has dropped significantly especially near the excavation indicating seepage through the sheetpile wall. The seapage continued and resulted in the profile shown in Figure 20. Every drop of a metre of groundwater would increase the effective overburden pressure by 10kPa which is equivalent to about half a metre of compacted earthfill. As the drop of water level reduces with distance away from the excavation, the increase in effective overburden also reduces respectively. The increase in effective overburden pressure induces immediate settlement to cohesionless soil and consolidation settlement to cohesive soil. It was the differential settlement or distortion that caused the formation of new cracks and widened with time.

The remedial proposal for Project A was carried by installing an additional row of sheetpiles of about 30m penetrating into the relatively impermeable clayey soil underlying the sandy soils together with recharging well and these had effectively restored the water table to its original level. Resumption of further basement excavation was only granted after the monitoring confirmed the effectiveness of the remedial measures. Similar trend of the drop in water level was observed at Project B where the water level near the excavation has dropped by 5.3 m.

The remedial proposal included the casting of its basement for the excavated portion to prevent further seepage and recharging wells were also introduced to expedite the recovery of the water level. The investigation also found that more reports of cracks were received from shophouses on or near the old Prangin River or its tributaries and swamps which had been reclaimed more than 100 years ago as shown in Figure 23.
5.0 CONCLUSION

The success of the design and construction of a deep excavation begins from well planned and closely supervised subsurface investigation works including field and laboratory testing. The design of retaining wall and support system should follow the appropriate standards, guidelines, and good practices. Intimate input from design engineer at every stage of the construction starting from setting out and establishing monitoring instruments before any excavation is essential. The construction should also follow the approved method statement and have a checklist on supervision to prevent mistakes or carelessness in the execution of works especially those highlighted in this paper.

Quality assurance system should be implemented to ensure design and construction are carried out systematically with the necessary checklists. Due care and diligence with full conscience from the design and supervision teams are imperative to ensure success of engineering works in particular for deep excavation which is risky not only to the work but also to adjacent structures.

ACKNOWLEDGEMENT

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CANTILEVERED WALL

BRACED WALL

FIG. 1  EFFECT OF DEFORMATION OF RETAINED GROUND ON ADJACENT PROPERTIES.

NOTE:
WALL DEFORMATION EXAGGERATED FOR ILLUSTRATION PURPOSE.
FIG. 2  SYMBOLS AND DEFINITIONS OF FOUNDATION MOVEMENT (After Institution of Structural Engineers, 1989)
FIG. 3 TYPES OF RETAINING WALL

(A) SHEET PILE WALL

(B) SOLDIER PILE WALL

(C) CONTINUOUS BORED PILE WALL

(D) SECANT PILE WALL

(E) DIAPHRAGM WALL
(a) Temporary Support Strutted against Central Dumpling

(b) Temporary Support by Fully Braced Trench

Note: Advantages and disadvantages of these systems are given in Table 1.

FIG 4(a) TYPES OF TEMPORARY AND PERMANENT SUPPORT SYSTEMS (After Institution of Structural Engineers, 1975) Sheet 1 of 7
(c) Fully Braced Temporary Support

(d) Permanent Wall Constructed Prior to Excavation and Braced from Central Dumpling

Note: Advantages and disadvantages of these systems are given in Table 1.

FIG 4(b) TYPES OF TEMPORARY AND PERMANENT SUPPORT SYSTEMS (After Institution of Structural Engineers, 1975) Sheet 2 of 7
(e) Permanent Wall Constructed Prior to Excavation (Diaphragm Wall or Contiguous Bored Pile Wall) and Braced from Central Permanent Construction

(f) Ground Anchors

Note: Advantages and disadvantages of these systems are given in Table 1.

FIG 4(c) TYPES OF TEMPORARY AND PERMANENT SUPPORT SYSTEMS (After Institution of Structural Engineers, 1975) Sheet 3 of 7
Sheet or bored piling, diaphragm wall or H-piles with horizontal sheeting

**g) Cantilevered Wall**

Concreting at ground level

Bentonite skin to reduce friction

Cutting edge

Simultaneous basement excavation

**h) Caissons**

Light piling or sleep batter

- Backing material stabilized by either cement/chemical/clay grouting or by freezing

**j) Stabilization Systems**

Note: Advantages and disadvantages of these systems are given in Table 1.
(k) Concurrent Upward and Downward Construction  
(Involving Progressive Underpinning)

Note: Advantages and disadvantages of these systems are given in Table 1.

FIG 4(e) TYPES OF TEMPORARY AND PERMANENT SUPPORT SYSTEMS (After Institution of Structural Engineers, 1975) Sheet 5 of 7
(1) Long Flying Shores Across Excavation

Note: Advantages and disadvantages of these systems are given in Table 1.

FIG 4(f) TYPES OF TEMPORARY AND PERMANENT SUPPORT SYSTEMS (After Institution of Structural Engineers, 1975) Sheet 6 of 7
FIG 4(g) TYPES OF TEMPORARY AND PERMANENT SUPPORT SYSTEMS (After Institution of Structural Engineers, 1975) Sheet 7 of 7
FIG. 5

POSSIBLE FAILURE MECHANISMS OF RETAINING WALL

<table>
<thead>
<tr>
<th>Failure form</th>
<th>Ground condition</th>
<th>Heaving</th>
<th>Boiling (piping)</th>
<th>Heaving due to artesian pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
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<tr>
<td>(2)</td>
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<tr>
<td>(3)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Factor of safety = $\frac{C_u N_c}{H + q}$

(a) For Deep Excavations with $\frac{H}{B} > 1$ (Bjerrum & Eide, 1956)

Factor of safety = $\frac{C_u N_c}{H (\gamma - \frac{C_u}{0.7B})}$

(b) For Shallow or Wide Excavations with $\frac{H}{B} < 1$ (Terzaghi, 1943)

Legend:

$C_u$ Undrained shear strength of soil
$\gamma$ Unit weight of soil

FIG. 6 METHODS OF BASAL HEAVE ANALYSIS BASED ON BEARING CAPACITY FORMULA
Stability is independent of \( H \) and \( D \), but varies with \( \gamma \), \( \phi \), and sample condition.

Safety factor \( F_s = \frac{N_c}{\gamma H + q} \)
where \( N_c \) is the bearing capacity factor.

If groundwater is at a depth of \( D \) or more below base of cut \( y_g \) and \( y_s \) are taken as moist unit weights.
If groundwater is static at base of cut \( y_s \) is the moist weight and \( y_g \) the submerged weight.
If slope is moving upward to base of cut \( y_s = \) (saturated unit weight) \( - \) (split pressure).

If sheeting terminates at base of cut the

safety factor, \( F_s = \frac{N_c C}{\gamma H + q} \)
where \( N_c \) = bearing capacity factor (Fig. 5.28) which depends on dimensions of the excavation: \( B \), \( L \), and \( H \) (base \( H = Z \)).

\( C \) = undrained shear strength of clay in failure zone beneath and surrounding base of cut.
\( q \) = surface surcharge.

If safety factor is less than 1.5, sheeting must be carried below base of cut to ensure stability.

Force on bentil length:

if \( H_H = \frac{2 \sqrt{2} \gamma H}{3 \sqrt{2} B} 
\)

if \( H_H = \frac{2 \sqrt{2} \gamma H}{3 \sqrt{2} B} = 1.5 \gamma H \)

Continuous excavation, \( F_s = \frac{N_c C}{\gamma H + q} \)

Rectangular excavation, \( F_s = \frac{N_c C}{\gamma H + q} \)

\( N_c1 \) and \( N_c2 \) = bearing capacity factors (Fig. 5.29) which depend on the dimensions of the excavation: \( B \), \( L \), and \( H \) (base \( H = Z \))

---

**FIG. 7 METHODS OF BASAL HEAVE ANALYSIS IN**

**COHESIONLESS SOIL; (b) CUTS IN CLAY; (c) CUTS IN CLAY LIMITED BY HARD STRATUM after NAVEC 1981**

---

**Assumed conditions**

1. \( D = B \)
2. Soil is uniform to depth \( d_u = D \)
3. Water level lower than \( d_u \) below base of footing
4. Vertical load concentric
5. Friction and adhesion on vertical sides of footing are neglected
6. Foundation soil with properties \( C, \phi, \gamma \)
Based on equilibrium of moments

\[ F_s = \frac{M_r}{M_d} \]

<table>
<thead>
<tr>
<th>Architectural Institute of Japan (1988)</th>
<th>Japan Society of Civil Engineers (1986)</th>
</tr>
</thead>
</table>
| *Assuming a circular sliding surface about the lowest strut.  
*Not considering vertical shear resistance along the retained ground shallower than the excavation bottom level.  
*Possible to include variation of shear strength with depth. | *Assuming a circular sliding surface about the excavation bottom.  
*Not considering vertical shear resistance along the retained ground shallower than the excavation bottom level.  
*Possible to include variation of shear strength with depth. |
| 1.2 | 1.2 |

**FIG. 8** METHODS OF BASAL HEAVE ANALYSIS BASED ON MOMENT EQUILIBRIUM
FIG. 9 PENETRATION OF CUT-OFF WALL TO PREVENT HYDRAULIC FAILURE IN HOMOGENEOUS SAND (after NAVFAC, 1982a)
(a) Coarse Sand Underlying Fine Sand
Presence of coarse layer makes flow in the fine material more nearly vertical and generally increases seepage gradients in the fine material compared to the homogeneous cross-sections of Figure 9.
If top of coarse layer is below toe of cut-off wall at a depth greater than width of excavation, safety factors of Figure 9 (a) for infinite depth apply.
If top of coarse layer is below toe of cut-off wall at a depth less than width of excavation, then uplift pressures are greater than for the homogeneous cross-sections. If permeability of coarse layer is more than ten times that of fine layer, failure head ($H_f$) = thickness of fine layer ($H_s$)

(b) Fine Sand Underlying Coarse Sand
Presence of fine layer constricts flow beneath cut-off wall and generally decreases seepage gradients in the coarse layer. If top of fine layer lies below toe of cut-off wall, safety factors are intermediate between those derived from Figure 9 for the case of an impermeable boundary at (i) the top of fine layer, and (ii) the bottom of the fine layer assuming coarse sand above the impermeable boundary throughout. If top of fine layer lies above toe of cut-off wall, safety factors of Figure 9 are somewhat conservative for penetration required.

(c) Very Fine Layer in Homogeneous Sand
If top of very fine layer is below toe of cut-off wall at depth greater than width of excavation, safety factors of Figure 9 assuming impermeable boundary at top of fine layer apply. If top of very fine layer is below toe of cut-off wall at a depth less than width of excavation, pressure relief is required so that unbalanced head below fine layer does not exceed height of soil above base of layer.

To avoid bottom heave when toe of cut-off wall is in or through the very fine layer, $(\gamma_s H_s + \gamma_c H_c)$ should be greater than $\gamma_w H_w$,

\[ \gamma_s = \text{saturated unit weight of sand} \]
\[ \gamma_c = \text{saturated unit weight of clay} \]
\[ \gamma_w = \text{unit weight of water} \]

If fine layer lies above subgrade of excavation, final condition is safer than homogeneous case, but dangerous condition may arise during excavation above fine layer and pressure relief is required as in the preceding case.

FIG. 10 PENETRATION OF CUT-OFF WALL TO PREVENT HYDRAULIC FAILURE IN STRATIFIED SOIL (after NAVFAC, 1982a)
<table>
<thead>
<tr>
<th>(1) Terzaghi's Method</th>
<th>(2) Critical hydraulic gradient method</th>
</tr>
</thead>
<tbody>
<tr>
<td>[ h_w = \gamma_s L_d ]</td>
<td>[ h_w = \text{difference of water head} ]</td>
</tr>
<tr>
<td>[ w = \gamma \cdot u = \frac{2 \gamma_s L_d}{\gamma_w h_w} ]</td>
<td>[ l = \text{length of stream line} ]</td>
</tr>
<tr>
<td>[ F_s = \frac{w}{u} = \frac{2 \gamma \cdot L_d}{\gamma_w \cdot h_w} ]</td>
<td>[ G_s = \text{specific gravity of soil particle} ]</td>
</tr>
<tr>
<td>[ e = \text{void ratio} ]</td>
<td>[ i = \text{critical hydraulic gradient} ]</td>
</tr>
<tr>
<td>[ F_i = \frac{i}{i} ]</td>
<td>[ F_i = \frac{G_i - 1}{1 + e} ]</td>
</tr>
</tbody>
</table>

**Diagram**

- Failure width by boiling is equivalent to half of penetration length of wall.
- Neglecting head loss in the retained ground above excavation bottom.
- Groundwater level in the retained ground is unchanged.

**Remarks**

- The same equation as of Terzaghi's method is obtained if the length of stream line \( l \) is being double to the penetration length of wall.

**Formula**

- \[ F_s = \frac{w}{u} = \frac{2 \gamma \cdot L_d}{\gamma_w \cdot h_w} \]
- \[ F_i = \frac{i}{i} \]
- \[ F_i = \frac{G_i - 1}{1 + e} \]

**Figure 11(a): Examination Methods of Boiling and Heaving Due to Water Pressure**
FIG. 11(b)  HEAVING DUE TO ARTESIAN PRESSURE

\[ F_s = \frac{w}{u} \]

- \( w \): overburden pressure
- \( h \): wet unit weight of soil
- \( u \): pore water pressure
- \( h_w \): artesian water head

\( w = \gamma' h \) and \( u = \gamma' h_w \)
APPARENT PRESSURE DIAGRAMS FOR STRUT LOADS (after Terzaghi and Peck, 1967)

- Struts
- Sands
- Soft or Medium Clays
- Stiff or Fissured Clays

\[ K_A = 1 - m \frac{\gamma H}{4 s u} \]

\[ m = 1.0 \text{ except as noted} \]

\[ K_A = 1 - m \frac{\gamma H}{4 s u} \]

\[ K_A = 1 - m \frac{\gamma H}{4 s u} \]

\[ 0.55 K_A \frac{\gamma H}{4 s u} \]

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\[ 0.55 K_A \frac{\gamma H}{4 s u} \]

\[ 0.55 K_A \frac{\gamma H}{4 s u} \]

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\[ 0.55 K_A \frac{\gamma H}{4 s u} \]

\[ 0.55 K_A \frac{\gamma H}{4 s u} \]

\[ 0.55 K_A \frac{\gamma H}{4 s u} \]
<table>
<thead>
<tr>
<th>Proponents</th>
<th>Outline of Prediction Method</th>
<th>Ground</th>
<th>Conceptual Diagram of the Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Naiño, et al. (1958)</td>
<td>Solve for volume $A_0$, (amount of deformed soil) and $A_1$ (amount of settlement of ground surface), due to deformation of walls, which are the same. In recent years, amount of deformation of walls is computed by the “elasto-plastic method,” and amount of settlement is predicted from the relationship shown in the conceptual diagram.</td>
<td>Plastic cohesive soil</td>
<td>![Example of assumed slip surface]</td>
</tr>
<tr>
<td>Maruoica, Ikuta, Aoki, Sato, et al. (1979)</td>
<td>Determination of the relationship between the maximum amount of settlement $S_{max}$ (calculated as the following condition: a sliding surface is assumed according to the stress characteristic curve defined by irtricity (1973), and the wall displaces along with the sliding surface and appears on the ground surface), and the heaving stability number $N_h$, proposed by Peck. After that, sliding surfaces are assumed by various methods, and the sliding surfaces are patterned for computing settlement of surface ground. The settlements are compared with measured data. $D$ is the depth of excavation.</td>
<td>Soft cohesive soil</td>
<td>![Wall displacement, mode 3]</td>
</tr>
<tr>
<td>Japanese National Railways (1979)</td>
<td>Relationship between N-value measured near the bottom of excavation and the maximum horizontal displacement of wall, 10 divided by the final excavation depth $D$ are examined, and if N-value becomes smaller than 5, the maximum horizontal displacement of wall becomes larger. This relationship is very similar to the relationship between $S_{max}$ and $F_{min}$ that was defined by Matsumo and Kawamura (as shown below) and the relationship defined by Mana and Clough (1981).</td>
<td>Cohesive to sandy soils</td>
<td>![Cohesive to sandy soils]</td>
</tr>
<tr>
<td>Matsumo and Kawamura (1981)</td>
<td>Showing that when the minimum safety factor Fmin in a circular sliding surface becomes less than 1.15, the maximum amount of settlement, Smax suddenly increases (based on construction in soft cohesive soil). The relational expression for $S_{max}$ and Fmin is: $S_{max} = 1/(0.654 F_{min} - 0.719)$ (where $F_{min} \geq 1.10$)</td>
<td>Soft or medium cohesive soil</td>
<td>![Soft or medium cohesive soil]</td>
</tr>
<tr>
<td>Manoka and Ikuta (1986)</td>
<td>Data collected from soft cohesive alluvial soil or sandy alluvial soil with N-value less than 10 are summarized for relationship between displacement of wall and adjacent ground.</td>
<td>Soft cohesive and alluvial sandy soil with SPT N less than 10</td>
<td>![Soft cohesive and alluvial sandy soil with SPT N less than 10]</td>
</tr>
<tr>
<td>Sugimoto and Susuki (1987)</td>
<td>The maximum amount of settlement of surface ground, $S_{max}$, will be 0.5 to 1.0 times the maximum horizontal displacement of wall, $B_{max}$, and is confirmed based on measured data, and proved by the FEM analyzing method. These results are similar to results obtained by Mana and Clough (1981).</td>
<td>Cohesive to sandy soils</td>
<td>![Cohesive to sandy soils]</td>
</tr>
<tr>
<td>Sugimoto (1986)</td>
<td>Factors affecting the maximum amount of ground surface settlement, $S_{max}$ are extracted by using the many measured data quantification theory, and these factors are combined to find the excavation coefficient $ce$. Then the relationship between $ce$ and the maximum amount of ground settlement of $S_{max}$ is determined. This relationship is verified by the FEM analyzing method.</td>
<td>Cohesive to sandy soils</td>
<td>![Cohesive to sandy soils]</td>
</tr>
</tbody>
</table>

Note: The contents in parenthesis under the proponents column are the category of the prediction method, either theoretical or empirical.

**FIG. 13 METHODS FOR PREDICTING SETTLEMENT OF ADJACENT GROUND FOR BRACED EXCAVATIONS**
FIG. 14
OBSERVED SETTLEMENT BEHIND EXCAVATIONS
(Peck, 1969)

Legend:
- Sand and hard clay, average workmanship
- Very soft to soft clay
- Very soft to soft clay to a significant depth below bottom of excavation

The data used to derive the three zones were taken from excavations supported by soldier piles or sheet piles with cross-
lot struts or tie-backs.

Zone I — sand and hard clay, average workmanship
Zone II — very soft to soft clay
Zone III — very soft to soft clay to a significant depth below bottom of excavation

Maximum depth of excavation
Maximum depth of excavation
Settlement
%
FIG. 15(a) SUMMARY OF MEASURED SETTLEMENT ADJACENT TO EXCAVATIONS IN SAND (From Clough & O'Rourke, 1990)

Legend:
- Hatfield (50)
- Bergshamra (47)
- 7th & G Sts (40)
- G St Test Site (40)
- 8th & G St (40)
- OCC Bldg. (55)
- Charter Station (14)
FIG. 15(b) SUMMARY OF MEASURED SETTLEMENT TO EXCAVATIONS IN STIFF TO VERY HARD CLAY (From Clough & O’Rourke, 1990)
FIG. 15(c) SUMMARY OF MEASURED EXCAVATIONS IN SOFT TO MEDIUM CLAY (From Clough & O’Rourke, 1990)
FIG. 17  DIMENSIONLESS SETTLEMENT PROFILES RECOMMENDED FOR ESTIMATING THE DISTRIBUTION OF SETTLEMENT ADJACENT TO EXCAVATIONS IN DIFFERENT SOIL TYPES (From Clough & O’Rourke, 1990)
FIG. 18 (a) and (b) SOME DAMAGES OF ADJACENT BUILDINGS
**Fig. 19 (a)**

**Fig. 19 (b)**

**FIG. 19 (a) and (b) SETTLEMENTS OF A NON-SUSPENDED SLAB**
FIG. 21
SOIL PROFILE WITHIN PRANGIN MALL, KOMTAR
FIG. 23 OLD RIVER, TRIBUTARIES AND COASTLINE BEFORE RECLAMATION
<table>
<thead>
<tr>
<th>Class of damage (1)</th>
<th>Description of damage* (2)</th>
<th>Approximate width* of cracks, mm (3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Negligible</td>
<td>Hairline cracks</td>
<td>&lt;0.1</td>
</tr>
<tr>
<td>Very Slight</td>
<td>Fine cracks easily treated during normal redecoration. Perhaps isolated slight fracture in building. Cracks in exterior brickwork visible upon close inspection.</td>
<td>&lt;1</td>
</tr>
<tr>
<td>Slight</td>
<td>Cracks easily filled. Re-decoration probably required. Several slight fractures inside building. Exterior cracks visible, some re-pointing may be required for weathertightness. Doors and windows may stick slightly.</td>
<td>&lt;5</td>
</tr>
<tr>
<td>Moderate</td>
<td>Cracks may require cutting out and patching. Recurrent cracks can be masked by suitable linings. Tuck-pointing and possibly replacement of a small amount of exterior brickwork may be required. Doors and windows sticking. Utility service may be interrupted. Weathertightness often impaired.</td>
<td>5 to 15 or several cracks &gt; 3 mm</td>
</tr>
<tr>
<td>Severe</td>
<td>Extensive repair involving removal and replacement of sections of walls, especially over doors and windows required. Windows and door frames distorted, floor slopes noticeably. Walls lean or bulge noticeably, some loss of bearing in beams. Utility service disrupted.</td>
<td>15 to 25 also depends on number of cracks</td>
</tr>
<tr>
<td>Very Severe</td>
<td>Major repair required involving partial or complete re-construction. Beams lose bearing, walls lean badly and require shoring. Windows broken by distortion. Danger of instability.</td>
<td>usually &gt;25 depends on number of cracks</td>
</tr>
</tbody>
</table>

*Location of damage in the building or structure must be considered when classifying degree of damage.

*Crack width is only one aspect of damage and should not be used on alone as a direct measure of it.

Note: Modified from Burland et al. (1977)
### (a) Framed buildings and reinforced loadbearing walls

Limiting values of relative rotation (angular distortion) $\beta$

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>structural damage</td>
<td>1/150</td>
<td>1/250</td>
<td>1/200</td>
<td>1/150</td>
</tr>
<tr>
<td>cracking in walls</td>
<td>1/300</td>
<td>1/500</td>
<td>1/500</td>
<td>1/500</td>
</tr>
<tr>
<td>and partitions</td>
<td>(but 1/500 recommended)</td>
<td></td>
<td>(0.7/1000 to 1/1000 for end bays)</td>
<td></td>
</tr>
</tbody>
</table>

### (b) Unreinforced loadbearing walls

Limiting values of deflection ratio $\Delta/L$ for the onset of visible cracking

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>sagging</td>
<td>1/2500</td>
<td>$L/H &lt; 3$; 1/3500 to 1/2500 $L/H &lt; 5$; 1/2000 to 1/1500</td>
<td>1/2500 at $L/H = 1$ 1/1250 at $L/H = 5$</td>
</tr>
<tr>
<td>hogging (unreinforced)</td>
<td>-</td>
<td></td>
<td>1/5000 at $L/H = 1$ 1/2500 at $L/H = 5$</td>
</tr>
</tbody>
</table>

**TABLE 2  SUMMARY OF LIMITING DEFORMATION**
<table>
<thead>
<tr>
<th>Types of Temporary and Permanent Support Systems</th>
<th>Figure</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
</table>
| 1. Temporary support strutted against central dumping | 1 (a)  | (i) Suitable for large excavations in plan rather than in depth.  
(ii) Evades groundwater problems if sheet piling can effect seal in underlying stratum. | (i) Slow and radically constrains programme and access.  
(ii) Wall has to be self-supporting to withstand soil pressures when dumping removed. |
| 2. Temporary support by fully braced trench | 1 (b)  | (i) Suitable for excavations relatively large in extent rather than depth.  
(ii) Evades groundwater problems if sheet piling can effect seal in underlying stratum. | (i) Slow and restrains construction programme.  
(ii) Wall has to be self-supporting against soil pressures when basement area is excavated. |
| 3. Fully braced temporary support | 1 (c)  | (i) Suitable for very deep excavations.  
(ii) Traditional.  
(iii) With incorporation of jacks preloading can be used where movements must be restricted to minimum. | (i) Slow, sure and very costly, particularly as width of excavation increases.  
(ii) Constrains construction programme greatly because of access difficulties. |
| 4. Permanent wall constructed prior to excavation and braced from central dumping | 1 (d)  | Variant of 1(e), somewhat cheaper, some advantages apply. | Some disadvantages as 1(a) apply, except that wall is not so massive, support usually derived from part of permanent work when dumping removed. |
| 5. Permanent wall constructed prior to excavation and braced from central permanent construction | 1 (e)  | (i) Good for relatively deep excavations provided that distance of perimeter wall from central construction is not too great.  
(ii) Can exercise control of movements satisfactorily with prestressing.  
(iii) Can exploit use of permanent work in temporary condition. | (i) Central structure must be massive.  
(ii) Passive soil buttress not so effective in restraining lateral movement of wall as a strut. |
| 6. Ground anchors | 1 (f)  | (i) Provides clean unobstructed area for basement construction.  
(ii) Specially attractive when anchored into rock foundation. | Great care necessary to prevent movement of ground anchors and hence wall. |

**TABLE 3(a)** ADVANTAGES AND DISADVANTAGES OF VARIOUS TEMPORARY AND PERMANENT SUPPORT SYSTEMS (After Institution of Structural Engineers, 1975) (Sheet 1 of 2)
<table>
<thead>
<tr>
<th>Types of Temporary and Permanent Support Systems</th>
<th>Figure</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
</table>
| 7. Cantilevered walls                           | 1 (g)  | (i) Provides unobstructed area for construction.  
(ii) Quick to install. | Great care required during concreting of deep walls especially if water exclusion is also a requirement. |
| 8. Caissons                                     | 1 (h)  | (i) Rapid construction once started.  
(ii) Provides unobstructed working space inside.  
(iii) More suited to circular basements. | (i) Extremely careful construction required to achieve verticality and positioning.  
(ii) Boulders can retard construction.  
Hence, very careful ground investigation required. |
| 9. Stabilization systems                        | 1 (j)  | Only used as last resort when other methods do not work. | (i) Usually costly and time consuming.  
(ii) May not eliminate groundwater problems. |
| 10. Concurrent upward and downward construction  | 1 (k)  | (i) Good for deep excavations.  
(ii) Affords speedier construction on superstructure. | Excavation and removal of spoil from enclosed area relatively difficult. |
| 11. Long flying shores across excavations       | 1 (l)  | Variant of 1(c), but suitable for narrower excavations. | (i) Impedes construction.  
(ii) Incorporation of monitoring jacks more difficult than for method 1(c). |
| 12. Floors cast on ground with excavation continuing below | 1 (m)  | (i) Good method for deep excavation.  
(ii) Temporary strutting eliminated.  
(iii) Temporary passive soil buttresses eliminated. | Excavation under slabs and removal of spoil relatively difficult. |

**TABLE 3(b)** ADVANTAGES AND DISADVANTAGES OF VARIOUS TEMPORARY AND PERMANENT SUPPORT SYSTEMS (after Institution of Structural Engineers, 1975) (Sheet 2 of 2)
<table>
<thead>
<tr>
<th>Requirements</th>
<th>Lends Itself to Use of</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Open excavation area</td>
<td>Tie-backs, raking struts or cantilevered walls</td>
<td></td>
</tr>
<tr>
<td>2. Low initial cost</td>
<td>Soldier pile or sheet pile walls; combined soil batter with wall</td>
<td></td>
</tr>
<tr>
<td>3. Use as part of permanent structure</td>
<td>Diaphragm or cylinder pile walls</td>
<td>Diaphragm walls most common as permanent walls</td>
</tr>
<tr>
<td>4. Deep, soft clay subsurface conditions</td>
<td>Strutted or raker-supported diaphragm or cylinder pile walls</td>
<td>Tie-back capacity not adequate in soft clays</td>
</tr>
<tr>
<td>5. Dense, gravelly sand or clay subsoils</td>
<td>Soldier pile, diaphragm or cylinder pile walls</td>
<td>Sheet piles may lose interlock on hard driving</td>
</tr>
<tr>
<td>6. Deep, overconsolidated clays</td>
<td>Struts, long tie-backs or combination tie-backs and struts</td>
<td>High in situ lateral stresses are relieved in overconsolidated soils. Lateral movements may be large and extend deep into soil</td>
</tr>
<tr>
<td>7. Avoid dewatering</td>
<td>Diaphragm walls, possibly sheet pile walls in soft subsoils</td>
<td>Soldier pile walls are pervious</td>
</tr>
<tr>
<td>8. Minimize movements</td>
<td>High preloads on slift strutted or tied-back walls</td>
<td>Analyze for stability of bottom of excavation</td>
</tr>
<tr>
<td>9. Wide excavation (greater than 20 m wide)</td>
<td>Tie-backs or raking struts</td>
<td>Tie-backs preferred except in very soft clay subsoils</td>
</tr>
<tr>
<td>10. Narrow excavation (less than 20 m wide)</td>
<td>Cross-lot struts</td>
<td>Struts more economical but tie-backs still may be preferred to keep excavation open</td>
</tr>
</tbody>
</table>

**TABLE 4** FACTORS INVOLVED IN THE CHOICE OF A SUPPORT SYSTEM FOR A DEEP EXCAVATION (NAVFAC, 1982b)
<table>
<thead>
<tr>
<th>Method</th>
<th>Design approach A: recommended range for moderately conservative parameters ($c'$, $\phi'$, or $c$)</th>
<th>Design approach B: recommended minimum values for worst credible parameters ($c'$ = 0, $\phi'$)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><strong>Temporary works</strong></td>
<td><strong>Permanent works</strong></td>
<td></td>
</tr>
<tr>
<td>Factor on embedment, $F_d$</td>
<td>Effective stress 1-1 to 1-2 (usually 1-2)</td>
<td>1-2 to 1-6 (usually 1-5)</td>
<td>Not recommended</td>
</tr>
<tr>
<td></td>
<td>Total stress* 2-0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strength factor method, $F_s$</td>
<td>Effective stress 1-1 to 1-2 (usually 1-2)</td>
<td>1-2 to 1-5 (usually 1-5)</td>
<td>1-0</td>
</tr>
<tr>
<td></td>
<td>except for $\phi' &gt; 30^\circ$ when lower value may be used</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total stress* 1-5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Factor on moments: CP2 method, $F_p$</td>
<td>Effective stress 1-2 to 1-5</td>
<td>1-5 to 2-0</td>
<td>1-0</td>
</tr>
<tr>
<td></td>
<td>$\phi' \geq 30^\circ$</td>
<td>2-0</td>
<td>1-0</td>
</tr>
<tr>
<td></td>
<td>$\phi' = 20$ to $30^\circ$</td>
<td>1-2 to 1-5</td>
<td>1-5 to 2-0</td>
</tr>
<tr>
<td></td>
<td>$\phi' \leq 20^\circ$</td>
<td>1-2</td>
<td>1-5</td>
</tr>
<tr>
<td></td>
<td>Total stress* 2-0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Factor on moments: Berland-Potts method, $F_t$</td>
<td>Effective stress 1-3 to 1-5 (usually 1-5)</td>
<td>1-5 to 2-0 (usually 2-0)</td>
<td>1-0</td>
</tr>
<tr>
<td></td>
<td>Total stress* 2-0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Speculative, treat with caution

Note 1. In any situation where significant uncertainty exists, whether design approach A or B is adopted, a sensitivity study is always recommended, so that an appreciation of the importance of various parameters can be gained.

Note 2. Only a few of the factors of safety recommended are based on extensive practice experience, and even this experience is recent. At present, there is no well-documented evidence of the long-term performance of walls constructed in stiff clays, particularly in relation to serviceability and movements.

However, the factors recommended are based on the present framework of current knowledge and good practice.

Note 3. Of the four factors of safety recommended, only $F_s$ depends on the value of $\phi'$.

TABLE 5 FACTORS OF SAFETY FOR METHODS OF ANALYSING EMBEDMENT (Padfield and Mair)