Design and Construction of Deep Excavations in Taiwan

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Abstract: *Numerous deep excavations have been carried out in major cities in Taiwan for constructing infrastructures and basements for high-rise buildings. Discussed herein are the current status of the design and construction practice adopted. The concept of reference envelop is introduced and reference envelops have been established for the T2 Zone of the Taipei Basin for bottom-up excavations. Performance of individual diaphragm walls can be evaluated by comparing their deflection paths with reference envelops. Moreover, reference envelops can be used as a guide in performance-based design of diaphragm walls.*

INTRODUCTION

With the rapid economic growth in the past decades, Taiwan has undergone drastic social reform with construction industry playing an important role. As more and more high-rise buildings are constructed, basements tend to go deeper and deeper. Furthermore, the majority of stations in the Taipei Rapid Transit Systems (TRTS) and the Kaohsiung Mass Rapid Transit System (KMRT) are underground. As a result, there is significant advancement in both design concept and construction practice of underground works. Presented herein are the current status of deep excavations on the island. Also presented is the concept of wall deflection path and reference envelop for evaluating the performance of walls and the effects of various factors affecting wall deflections. Based on the observations obtained, reference envelops have been established for the T2 Zone of the Taipei Basin for bottom-up excavations and these envelops can be used to guide the design of diaphragm walls.

GROUND CHARACTERIZATION

Since the majority of the cases presented hereinafter are located in the Taipei Basin, a geological zoning map is given in Figure 1 (Lee, 1996) for the convenience of readers and an east-west soil profile across the Basin is presented in Figure 2. As can be noted that at the surface is a thick layer of Sungshan Formation underlain by the Chingmei Gravels. The Basin is divided into 21 zones and in the central city area (T2 Zone), where the Taipei Main Station (BL7/R13 Station) is located, the six-sublayer sequence is evident. Toward the east (K1, K2 and K3 Zones), silty clay dominates, and toward the west (B2 and B3 Zones) the stratigraphy becomes rather complex with silty sand and silty clay seams interbedded in these sublayers. The Chingmei Gravels contains gravels of various sizes and is extremely permeable. This gravelly layer is practically an underground reservoir and has been responsible for several major failures during the first stage construction of TRTS. As can be

noted from Figure 3 that the piezometric levels in the Chingmei Gravels were lowered by as much as 40m in the 70's as a result of excessive pumping of groundwater for industrial and domestic usages. The accompanying ground subsidence exceeded 2m. As pumping was banned in the late 70's, the piezometric levels in the Chingmei Gravels recovered rapidly. This imposes threats to basements constructed in the 60's and the 70's, if they were designed for the then low piezometric levels, as buoyancy increases and there is a tendency for these basements to float. The recovery of the piezometric level in the Chingmei Gravels, however, has been slowed down as a result of dewatering for constructing the rapid transit systems starting from the early 90's.
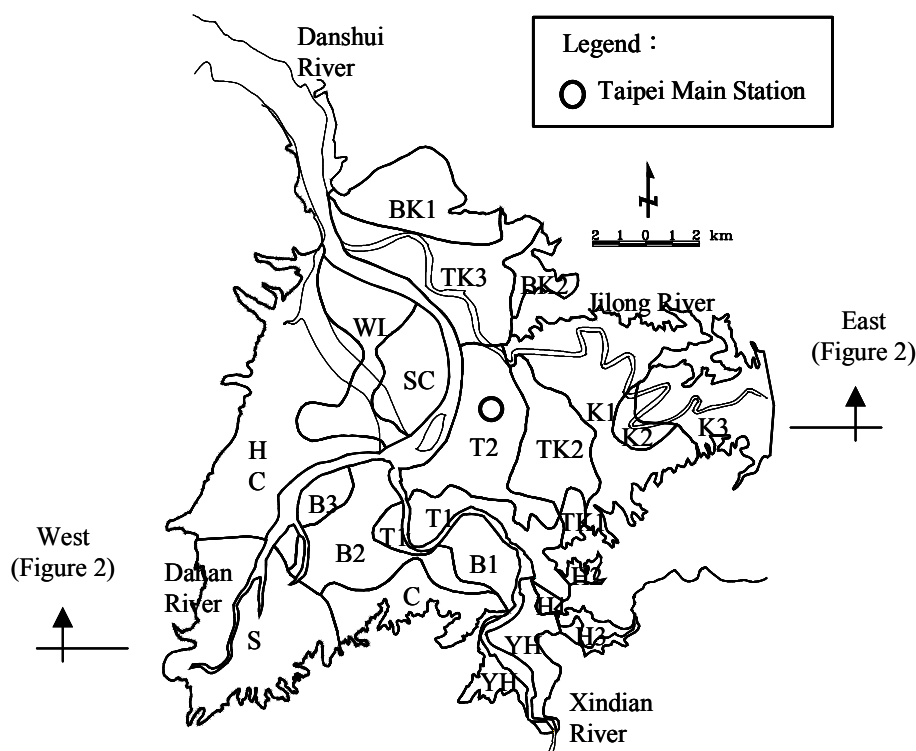
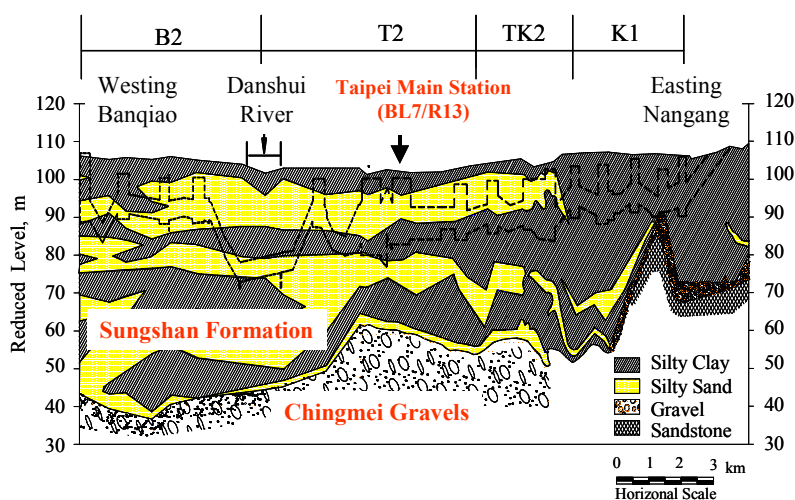Figure 1   Geological Zoning Map of the Taipei Basin

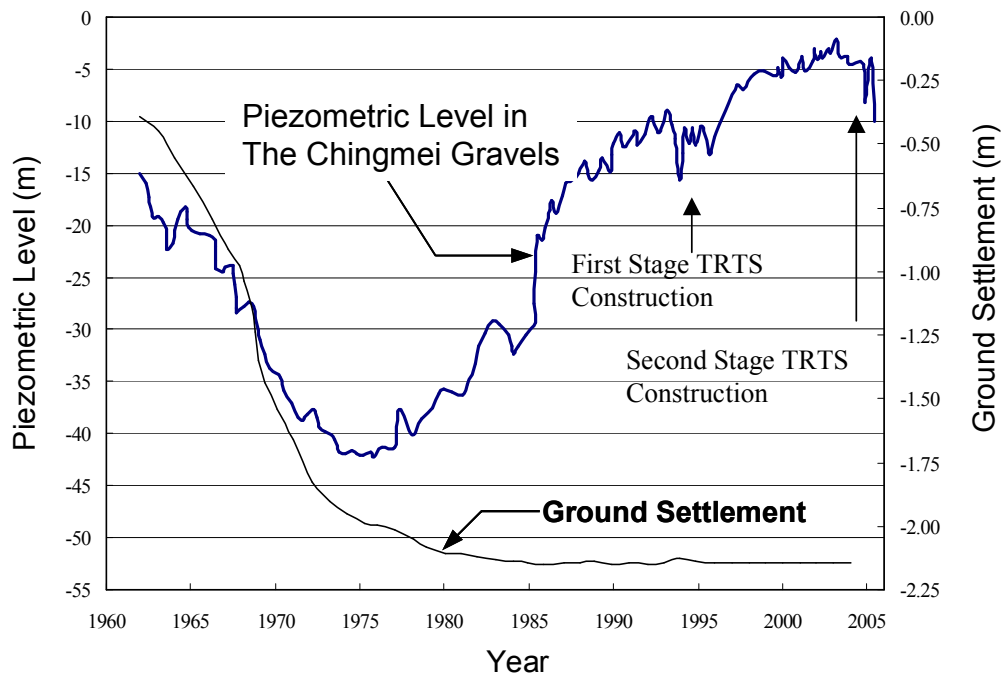Figure 2   East-West Geological Profile of the Taipei Basin

Figure 3   Piezometric Levels of Groundwater in the Chingmei Gravels

A typical CPT profile obtained in the central city area of Taipei (T2 Zone) is shown in Figure 4.   As can be noted that the six-layer sequence in the Sungshan Formation is clearly identifiable. The various soil strata can better be identified in the porewater pressure profile than tip resistance or local friction.   However, it should be noted that porewater pressures induced are dependent on the type of the cone used and, more importantly, are affected by the workmanship.   Figure 5 shows a comparison of the results obtained at two neighboring locations.   At CPT-39, the piezometer tip was simply submerged in water for 24 hours before the test.   The pore pressure response was poor and the boundaries between consecutive layers could not be identified.   The test was repeated at CPT-39A, which was only 2m away from the location of CPT-39, with the piezometer tip submerged in a water-glycerin mix and boiled for 10 minutes to drive air bulbs out.   The results obtained show drastically improvement as compared to those obtained previously.

METHODS OF EXCAVATION

In the lack of precedents, the authors propose the following definitions and hope for a common understanding among professionals:

shallow excavations: up to 5m in depth, or 1-level basement
mid-depth excavations: 5m to 10m in depth, or 2-level to 3-level basements
deep excavations: 10m to 20m in depth, or 4-level to 5-level basements
very deep excavations: 20m to 30m in depth, or 6-level or more basements
extremely deep excavations: 30m or greater in depth

In the following paragraphs, a few cases which are benchmarks of underground works on the island are presented. Their successful completion is an index of the technology level and the quality of local construction industry.
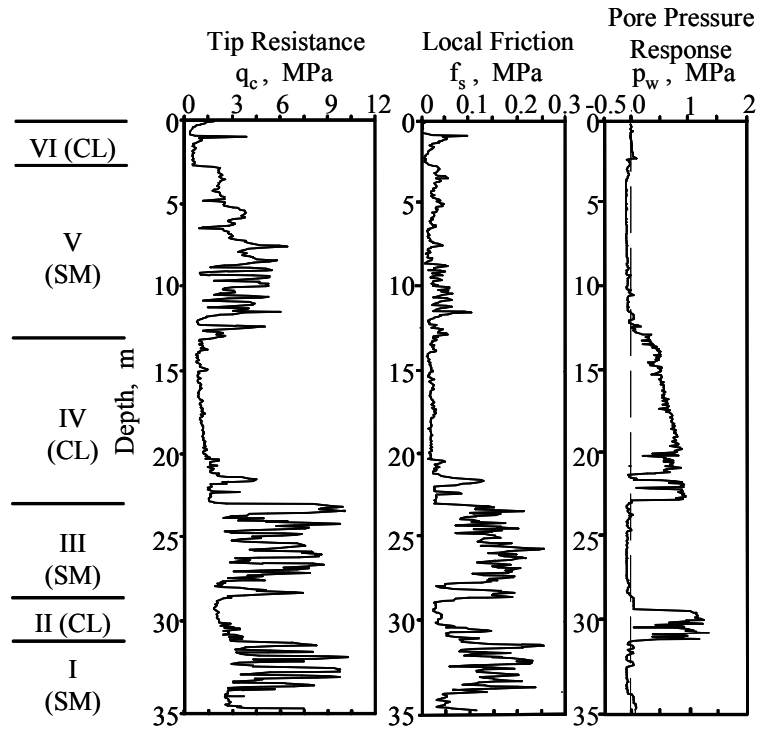
Tip Resistance
q_c, MPa

Local Friction
f_s, MPa

Pore Pressure
Response
p_w, MPa

Figure 4   CPT Results in Central Taipei City (T2 Zone)
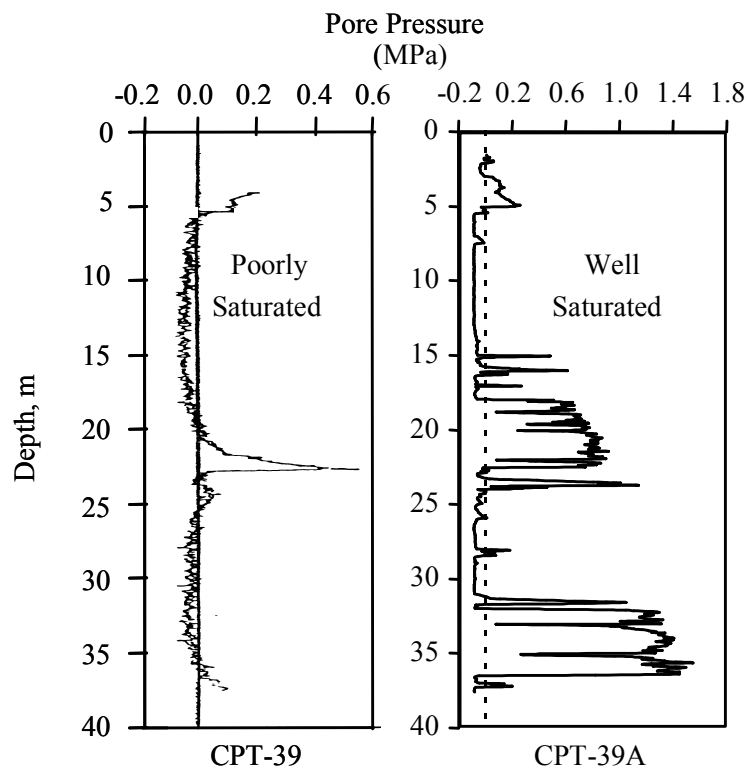
Pore Pressure
(MPa)

CPT-39

CPT-39A

Figure 5   Results of Piezocone Tests as Affected by Saturation of Piezometer

Taipei Rapid Transit System
Of the 34 underground stations in the first stage construction of TRTS, 23 were constructed by using the bottom-up method, 10 by using the semi-top down method but only 1 by using the top-down method.   It is clear that the top-down method was not favoured in constructions of rapid systems

Because of the absence of a competent stratum within a reasonable depth for bonding ground anchors, all the excavations were strutted.   The ventilation shaft in the Zhonghe Line was the only circular excavation in TRTS constructions and the rest of excavations were all rectangular.   As ground conditions are poor and excavations were deep, diaphragm walls were exclusively used for station excavations. For shallower excavations, for example, at entrances, contiguous bored piles and sheet piles were sometimes used.   There is a growing concern on the use of diaphragm walls as permanent walls because of their poor water-tightness, and as a result, single-wall system is gradually phased out and the double-wall system has become more popular nowadays.   At a few stations, composite-wall system, in which the permanent wall is structurally connected to the diaphragm wall by dowels, was adopted.   Because the diaphragm wall and the permanent wall form a single structural element, the thickness of the permanent wall can be reduced saving some space. However, in hindsight, composite walls appear to be a nuisance.   First of all, the provision of dowels makes the tremieing of concrete difficult.   Secondly, these dowels have to be manually exposed and bent for the permanent wall to be cast.   Unless space is crucial, the use of composite walls should be discouraged.

Groundwater was a major concern during excavations and large scale pumping was carried out in the period of 1993 to 1995 at three sites where extremely deep excavations were carried out for lowering the groundwater table, by 10m or so, for maintaining sufficient factor of safety against blow-in.   Pumping rates ranged from 2,000 to 4,200 cubic meters per hour (50,000 to 100,000 tons per day) and pumping lasted for six months or longer in each case. Detailed discussions on the measures taken against blow-in are available in Moh, Chuay and Hwang (1996).

Largest Circular Excavation on the Island
With an inner diameter of 140m, the excavation for Formosa Boulevard Station (formerly, Da-Kang-Pu Station, O5/R10) of Kaohsiung MRT is the largest circular excavation on the island and is one of the largest circular excavations in the world.   The ring was made of 146 panels of diaphragm walls of 1.8m in thickness with their toes embedded in stiff clays at a depth of 60m to provide seepage cutoff.   Excavation was first carried out to a depth of 19.74m for housing the concourse at the U1 Level and the two tunnels and part of O5 Station of the Orange Line at the U2 Level, with only a ring beam at the very top of diaphragm walls but without any inner bracing otherwise.   A north-southward trench was then made to the final depth of 27.13m for constructing R10 Station of the Red Line at the U3 Level with sheetpiles as retaining walls on the two sides. As of this date (December, 2005), excavation has been completed and all the floors have been cast.   Despite the poor ground conditions, wall deflections were generally less than 35mm.

Deepest Basement on the Island
The 7-level basement of Core Pacific City is the deepest building basement on the island. Excavation was carried out to a maximum depth of 31.68m by using the top-down

construction method and was completed in mid-2000. Diaphragm walls of 1.5m in thickness were installed to a depth of 52m and braced only by basement floors. Wall deflections ranging from 90mm to 166mm were observed.

Basement Excavation for the Tallest Building in the World
Being the tallest building in the world at this moment for its height of 508m, Taipei 101 certainly deserves a space herein. The excavation for the 5-level basement of the 6-story podium block was carried out to a maximum depth of 21.65m by using the top-down construction method while that for the 101-story tower block was carried out to a maximum depth of 27.5m by using the bottom-up construction method. The pit was enclosed by 1.2m thick diaphragm walls to a maximum depth of 52m. Buttresses with a length of 6m and thicknesses of 900mm and 1200mm were used to reinforce diaphragm walls for reducing wall deflections. The mats of both the podium block and the tower block are supported on bored piles of 1500mm and 2000mm in diameter to a maximum depth of 72m. Excavation for the basement of the tower block was completed in March, 2000 and that for the basement of the podium block was completed a few months later. Wall deflections observed were 50mm or less.

Deepest Excavation on the Island
The 50.27m excavations carried out for 3 LNG (liquefied natural gas) tanks in Yun An Receiving Station (Phase II Project) are so far the deepest on the island (Chung and Chen, 1994). They are circular in shape with outer diameters of 70m and inner diameters of 64m. Diaphragm walls of 1.2m in thickness and 90m in length were installed by using the MHL and BW methods to provide seepage cutoff. The site was reclaimed in the period of 1985 to 1988 and the three tanks were completed in the year of 1996. The excavations are unique in that there were no internal bracing and diaphragm walls were retained by ring beams at the very top till a depth of excavation of 40.3m was reached. A section of the inner wall, 2.8m in thickness, was then cast between depths of 29.5m and 37m in each pit to serve as a ring beam for the remaining excavation to proceed.

Deepest Excavation in the Taipei Basin
With a final depth of 40m, the excavation currently being carried out for constructing the crossover box at the west bank of Danshui River for the construction of the Xinzhuang Line of TRTS is so far the deepest in the Taipei Basin. Diaphragm walls of 2000mm in thickness are used and are braced by 13 levels of struts. This excavation is rated as one of the most risky works in TRTS not only because of its greatest depth but also because of the proximity of the site to the Danshui River and the fact that the Chingmei Gravels lies at a short depth of only 20m below the formation level. To ensure the safety of construction, a slab of 5m in thickness has been cast by using the double-packer grouting method to serve as a blanket to cut off seepage and pumping is being carried out to lower the piezometric levels in the Chingmei Gravels below this slab by 20m for maintaining an adequate factor of safety against upheave and piping. Excavation has reached a depth of 33m as of this date (December, 2005) and the performance is as predicted.

CONCEPT OF PERFORMANCE-BASED DESIGN
As mentioned above, for deep excavations in soft ground in major cities, diaphragm walls were exclusively used with other types of retaining structures used in very rare occasions.

Prior to TRTS constructions (say, 1990 and earlier), diaphragm walls were generally designed in consideration of their structural capacity and the stability of the ground below the formation levels without due consideration given to their lateral deflections. As deep excavations are normally carried out in densely populated areas and people have become more and more aware of their own rights, protection of adjacent buildings and properties is a serious concern nowadays for underground constructions. Experience indicates that rectification of buildings and structures which have been affected by ground movements is both costly and ineffective and it will be much wiser to minimize ground movements at source. The old saying that "An ounce of prevention is worth of a pound of cure" certainly holds true for underground constructions. As it is obvious that ground movements are primarily caused by wall deflections, the concept of performance-based design, instead of capacity-based design, has thus been adopted since the early 90's for the purpose of limiting wall deflections, hence, ground movements behind walls.

In the early stage of the TRTS constructions, ground settlements were limited to 25mm by specifications and wall deflections were limited to a similar magnitude. This was found to be impractical and subsequently, the specifications were revised so that designers have to evaluate the conditions of adjacent buildings and properties, determine allowable ground settlements and wall deflections, and design the walls accordingly. In most cases, wall deflections were limited to 30mm to 60mm. To achieve this, thicker walls, generally 200mm to 300mm thicker in comparison with those designed based on their structural capacities, were used, and struts were preloaded to 50% to 60% of their design loads. These precautionary measures indeed paid off as damages to adjacent buildings and properties due to wall movements were greatly minimized.

FIELD OBSERVATIONS AND PERFORMANCE OF WALLS

Instrumentation and Data Interpretation

Instrumentation plays a vital role in safety management during construction. Unfortunately, not sufficient attention has been given to instrumentation and monitoring by contractors and, as a result, instruments were frequently improperly installed and data were frequently misread. This, sometimes, could lead to serious consequences as early warnings will not be able to obtain and the purpose of instrumentation is defeated (COI, 2005).

Lateral deflections of walls are routinely monitored by using inclinometers which are amazingly accurate and can be considered as one of the most reliable types of geotechnical instruments. However, this does not mean that inclinometers always faithfully report wall deflections. For deep excavations in soft ground, Figure 6 shows the results normally expected from monitoring of wall deflections. The wall behaves as a cantilever in the first stage of excavation and significant movement would normally occur in soft ground before the first level of struts are installed. During this stage of excavation, the rigidity of the wall contributes very little in reducing wall deflections. Once the first level of struts are installed and preloaded, the wall behaves as a beam supported at its upper end and the rigidity of the wall starts to show its significance. The wall would bulge in toward the pit in subsequent stages of excavation while the movements of the wall at each of the strut levels are mainly induced by the shortening of struts and are expected to be small once struts are preloaded.

The toes of inclinometers are normally assumed to be fixed and the movements at all other depths are computed in relation to the toes. Because diaphragm walls usually are not designed to have zero movements at their toes, so the toes of inclinometers are expected to

move if inclinometers stop at the same levels as the walls. In such cases, the readings obtained become misleading and have to be corrected. It is a good practice to check the movements of the top of casing by precision survey for calibrating readings at other depths. However, this sometimes may become difficult to carry out because of site constrains or difficult to enforce because of lack of supervision. Therefore, it will be a good idea to specify that inclinometers should be buried in stable strata as depicted in Figure 6 or extended to sufficient depths so the toe movements will be insignificant.
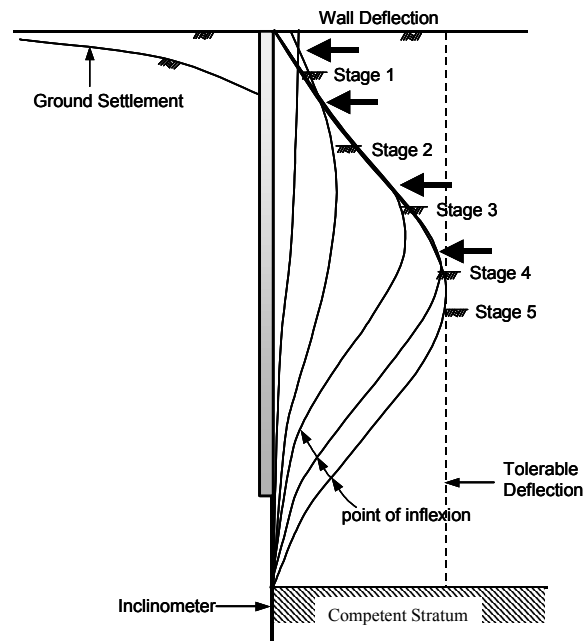

Figure 6   Ideal Wall Deflection Profile

To illustrate this argument, Figure 7(a) shows the inclinometer readings obtained for a 12m excavation in the T2 Zone (i.e., the central city area, refer to Figure 1 for location) in which soft deposits extending to a depth of 50m or more (refer to Figure 2 for soil profile).   The pit was retained by 600mm diaphragm walls installed to a depth of 24m. In the first stage of excavation (to a depth of 1.5m), the wall behaved as a cantilever and a maximum movement of 17mm was recorded.   The top of the wall was pushed back by 5mm as the first level of struts were preloaded. The lower portion of the wall bulged in toward the pit while the top of the wall was further pushed back by 6mm in the second stage of excavation. The same trend was observed in the subsequent stages of excavation and a maximum inward deflection of 18mm was obtained at the end of Stage 5 excavation. The top of the wall moved outward beyond its original position by 25mm at the end of Stage 5 excavation and the wall at the first level of struts moved by more than 33mm since these struts were preloaded.   This is highly unlikely to be realistic because, firstly, this would mean that the soils behind the wall would be in a passive state, but the preloads in all the struts would not be sufficient to make this to happen.   Secondly, should the wall indeed moved as shown, tensions would have developed in the upper levels of struts.   This is contrary to the observation that the loads in the upper levels of struts indeed increased as excavation proceeded.

Wall Deflection, mm

(a) Before Adjustment for Toe Movements

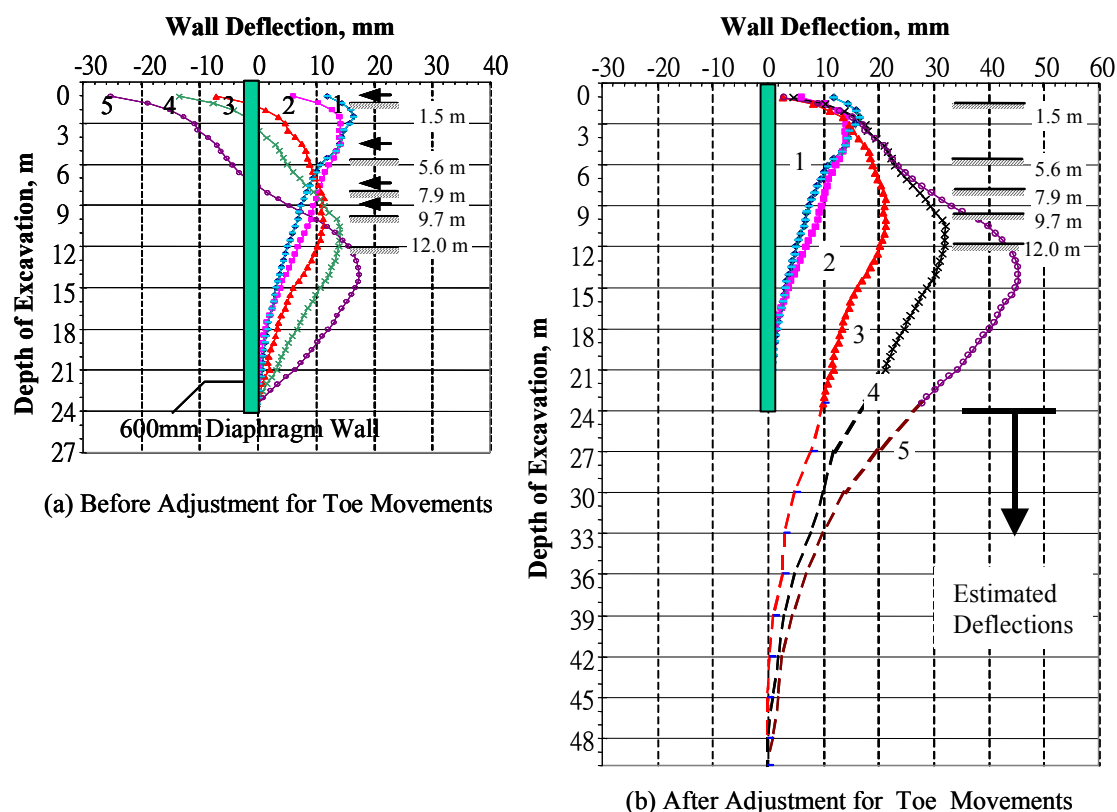(b) After Adjustment for Toe Movements

Figure 7   Correction of Inclinometer Readings for Toe Movements

If the wall is assumed to be unmoved at the connections of the first level of struts (at a depth of 1.35m) in Stages 2 to 5 excavations, the readings can be adjusted by the movements recorded at this level and the profiles so obtained, as shown in Figure 7(b), will be quite similar to what is shown in Figure 6.   It is thus suspected that the toe of the wall has indeed moved by nearly 30mm.   It is interesting to note that toe movements started even as early as in the Stage 3 excavation in which excavation was carried out to a depth of only 7.9m and wall deflections extended to a great depth.

The example given above illustrates the fact that inclinometer readings shall be interpreted with great care. This may appear trivial to many engineers, but there are indeed many studies carried out based on uncorrected data. It is thus suggested that inclinometers be anchored in competent strata to eliminate doubts whenever it is economical to do so.

Factors Affecting Wall Deflections
It has been reported that wall deflections obtained in TRTS constructions were, in general, one-third of those obtained previously (Moh and Hwang, 1999).   The superior performance of TRTS constructions are primarily a result of greater stiffness of the retaining systems and better workmanship.   The factors affecting wall deflections can be summarized as follows

    (1)   depth of excavation
    (2)   width of excavation
    (3)   ground conditions, e.g., soil stiffness, groundwater table

(4)   depth to competent base stratum
(5)   method of construction, e.g., top-down, bottom-up, or semi top-down
(6)   rigidity of wall system, represented by wall thickness, including buttress, if any
(7)   length of wall
(8)   stiffness of the strutting system, including spacing and member size
(9)   preloading of struts
(10)   corner effects, i.e., proximity to boundaries of pits (3D effects)
(11)   ground treatment, i.e., grouting
(12)   foundation piles (or tension piles)
(13)   adjacent structures, e.g., surcharge loads and basement effects
(14)   workmanship, e.g., over-excavation, promptness of strutting and preloading

It will be desirable to quantify the effects of these factors individually, but this will be a mission impossible.   For each factor listed above, there are a few possible variations to be considered.   For example, ground conditions can be classified into several categories, there are several ways to treat the ground, etc.   If a minimum of three cases are required for each combination of these variations for statistical analysis to be meaningful, the number of cases required will be enormously large.   Therefore, analysis can only be performed for specific combinations and the conclusions obtained are hoped to be applicable to other combinations.

Wall Deflection Paths and Reference Envelops

Figure 8 shows the wall deflection paths, which are plots of the maximum deflections in the deflection profiles versus depths of excavations in a log-log scale (Moh and Hwang, 2005), obtained for bottom-up constructions in the T2 Zone of the Taipei.   The envelops, designated as "reference envelops" herein, of the data points can be considered as site characteristic curves for diaphragm walls and can be used for evaluating the performance of individual walls.   The performance of a diaphragm wall can be judged by comparing its deflection path with relevant reference envelop for the site as illustrated in Figure 9:

Path A: The presence of basements, retaining walls and foundation piles in the vicinity is likely to reduce wall deflections in the early stage of excavation.
Path B: On the other hand, surcharge loads in the vicinity of excavation, if any, will increase wall deflections in the early stage of excavation.
Path C: Because the influence of adjacent structures and/or surcharges diminishes as depth of excavation increases, deflection paths tends to converge toward the reference envelop.
Path D: As excavation exceeds a certain depths, the performance of the wall is affected by the stability of the toe of the wall.   For walls with sufficient lengths beyond the formation levels and/or with their toes properly embedded in competent strata, wall deflections will increase at diminishing rates (in a log-log scale) and their deflection paths are expected to bend downward.
          Ground treatment will have similar effects.
Path E: On the other hand, if the deflection path for a certain wall becomes flatter than the reference envelop, it is most likely that the toe of the wall has become unstable.
          Soft strutting system and poor workmanship will have similar effects.

As shown in Figure 10, reference envelops can be defined by: (a) wall deflections for shallow excavations, represented by deflections at depths of excavation up to 4m, i.e., δ_4,   (b) wall deflections projected to a depth of excavation of 100m, δ_{100}, and (c) depth to the competent

base stratum, D_{max}, and (d) depth below which the deflection increases at a diminishing rate, i.e., D_{bend}. The reference envelop below D_{bend} can be simulated by an arc tangent to the upper portion of the envelop and perpendicular to the competent base stratum.

Reference Envelops for Bottom-Up Construction in the T2 Zone

Figure 9(d) shows that wall deflections increased at diminishing rates for excavations deeper than 15m i.e., D_{bend} = 15m, which corresponds to one-third of the depths to competent base strata for the cases shown. In general, the Sungshan Formation can be as deep as 60m and it is therefore assumed D_{bend} = 20m just to be on the safe side. Accordingly, the reference envelops for walls with different thicknessess can be constructed by using the data shown in Figure 8 and the procedures outlined in Figure 10 and the results obtained are shown in Figure 11.

(a) Analysis for 600mm Walls

(b) Analysis for 800mm Walls

(c) Analysis for 1000mm Walls
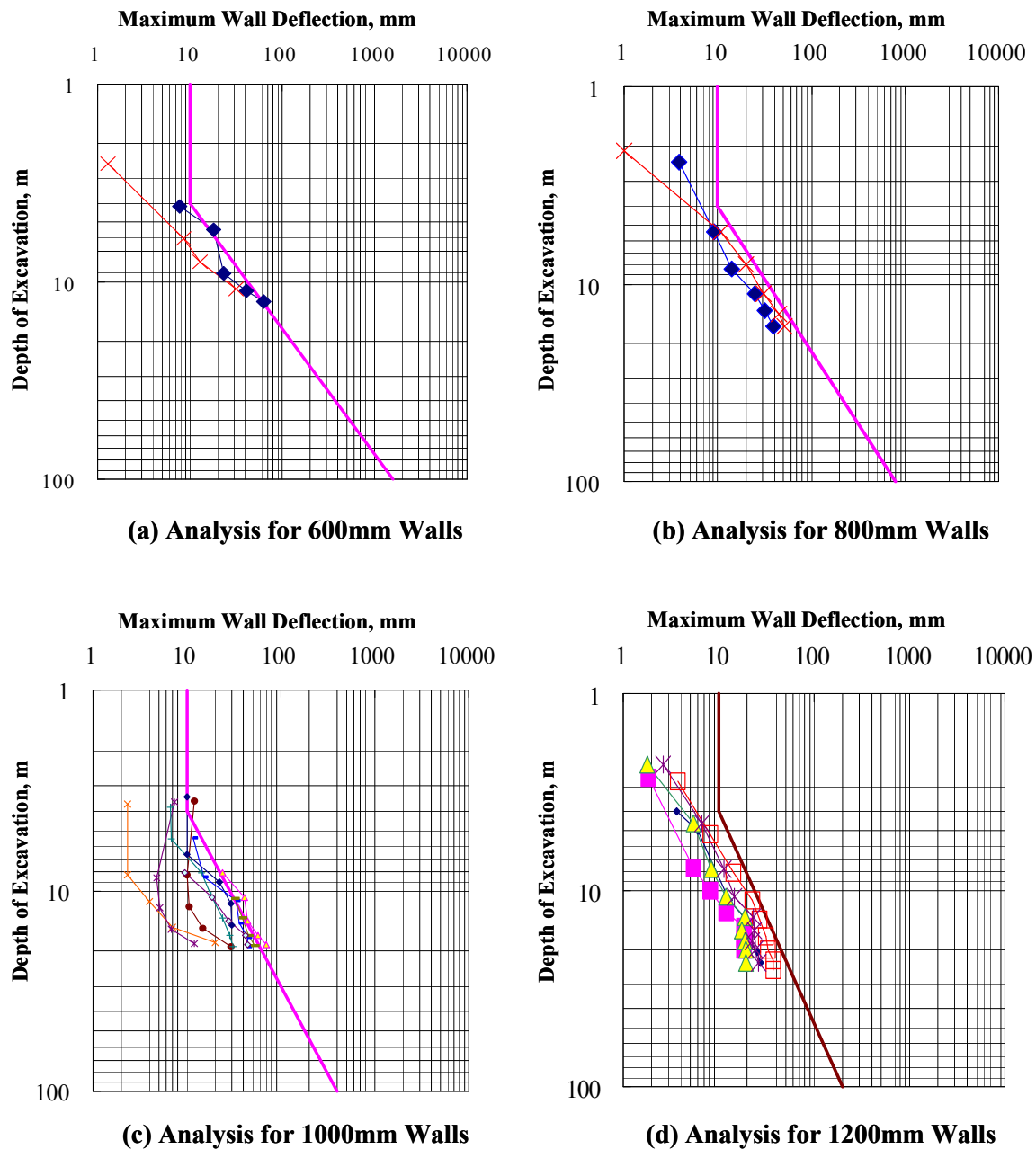
(d) Analysis for 1200mm Walls

Figure 8   Wall Deflection Paths for Bottom-Up Constructions in the T2 Zone
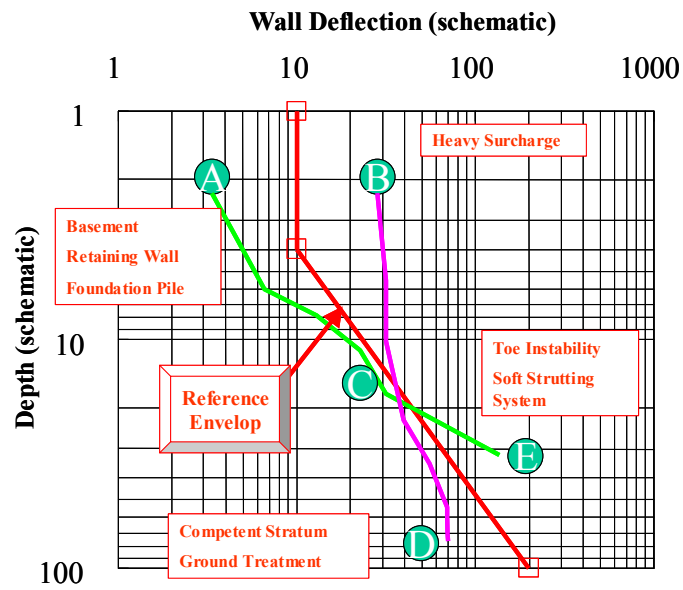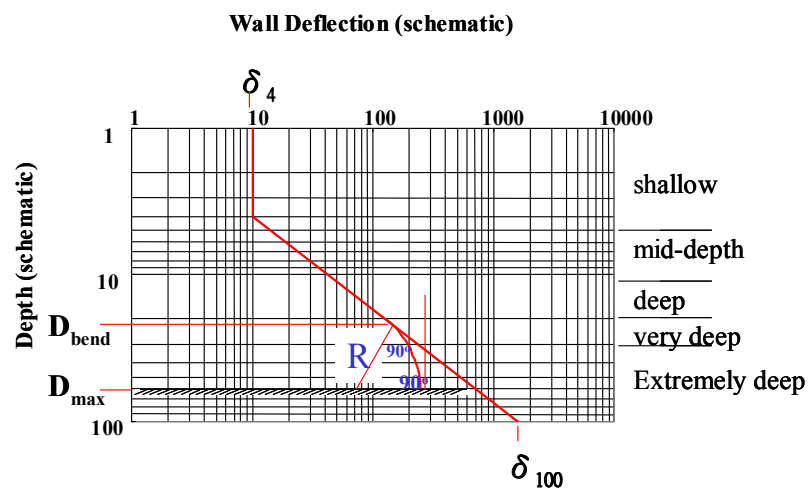
Figure 9   Patterns of Wall Deflection Paths

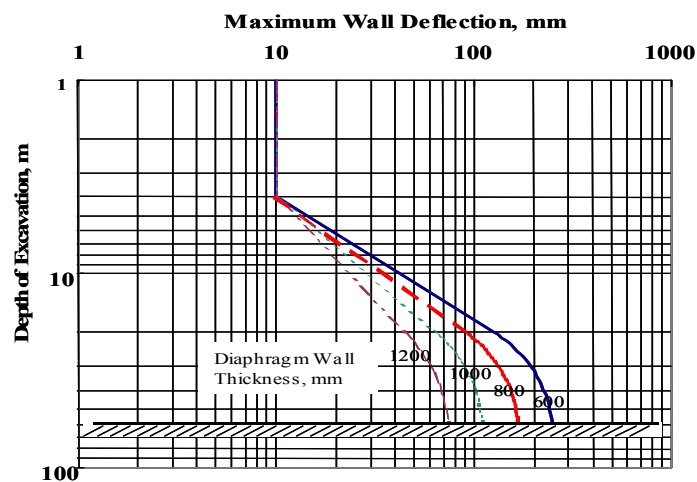Figure 10   Effects of Proximity to Competent Base Stratum

Figure 11   Reference Envelops for Bottom-Up Constructions in the T2 Zone

Effects of Stiffness of Walls on Wall Deflections

In the early stage of excavation, walls behave as cantilevers and wall deflections are governed mainly by the soil stiffness and the stiffness of the walls has little influence on wall deflections.   Data are scattering and a deflection of 10mm can be assumed as the envelop for the design purpose regardless of wall thickness. Once the first and second levels of struts are installed and preloaded, the stiffness of walls becomes more and more important as excavations go deeper and deeper. It can be shown that the slope of a reference envelop can be mathematically expressed as follows:

$$\delta = a \bullet d^{\theta} \tag{1}$$

in which δ is the wall deflection at depth d, a=constant, θ = slope of reference envelop and

$$\theta = 0.715 \bullet \log\left(\frac{\delta_{100}}{\delta_4}\right) \tag{2}$$

The effects of stiffness of walls on wall deflection paths can be judged by the slopes of the reference envelops.   Since the deflections at a depth of 4m are assumed to be the same for the four cases shown in Figure 8, the effects of wall stiffness can be evaluated by comparing the deflections projected to a depth of 100m, i.e., δ_{100}, which are 1600mm, 800mm, 400mm and 200mm for walls of 600mm, 800mm, 1000mm and 1200mm in thickness, respectively.

Effects of Ground Conditions on Wall Deflections

The quantity of data is insufficient to establish envelops for other zones in the Taipei Basin. Furthermore, the differences in ground conditions in different zones are not large enough to show the difference in performance of walls. To test whether the above-mentioned approach is applicable to ground conditions elsewhere, Figure 12 shows the wall deflection paths for two inclinometers (hollow symbols and dashed envelop) installed at a MRT construction site near Nicoll Highway in Singapore (COI, 2005; Wong, 2005; Goh, 2005) as compared to those obtained for the T2 Zone (solid symbols and solid reference envelop) in the Taipei Basin. The site is located in a reclaimed land and a representative soil profile is shown in Figure 13. Excavation was supposed to be carried out to a depth of 33.5m and diaphragm walls with a thickness of 800mm (locally, 1000mm) were used.   As can be noted from Figure 12 that inclinometer I-65 on the north side of excavation showed a wall deflection of 16mm at the top in the first stage of excavation to a depth of 1.6m and a maximum deflection of 200mm was recorded as excavation reached a depth of 24.6m (the 8th dig) at the end of March, 2004. Wall deflections did not increase further as excavation proceeded below this depth presumably because of the presence of the competent Old Alluvium at a depth of 32m (RL 70m) or so. In fact, wall deflections decreased by 20mm at the 10th dig (excavation depth of 30.5m) from its maximum. As can be noted from Figure 13 that soil deposits are indeed thicker on the south side than the north side and the Old Alluvium is at a depth of 40m (RL 63m), instead of 32m. The outward movements of the north wall could well be due to imbalance of earthpressures on the two sides of the excavation.

Inclinometer I-104 on the south side of excavation was installed after excavation had already started. The first set of readings were taken on 14 October 14 of 2004 while excavation was commenced on 26 September.   Therefore the wall deflection recorded was only 11mm, versus 16mm for I-65, at a depth of excavation of 1.6m.   Despite this fact, the deflection

paths for the two inclinometers were amazingly similar till a depth of excavation of 24.6m or so at the 8th dig. Contrary to the case of I-65, there was no sign of reduction in the rate of increase of wall deflections with depth of excavation (in a log-log scale) below this level. In fact, wall deflection increased by 90mm, from 350mm to 440mm, in the 3-day period immediately before the collapse of the retaining system. The collapse occurred on 20 April while the 10 dig was completed and excavation reached the depth of 30.5m on 16 April, 2004. A Committee of Inquiry (COI) was formed by the Singapore government to investigate the causes of failure. Although, it was concluded that the collapse was triggered by the buckling of walings and was not directly linked to wall deflections (COI, 2005), COI is of the opinion that the retaining system was seriously under-designed and wall deflections and bending moments were seriously under-estimated.
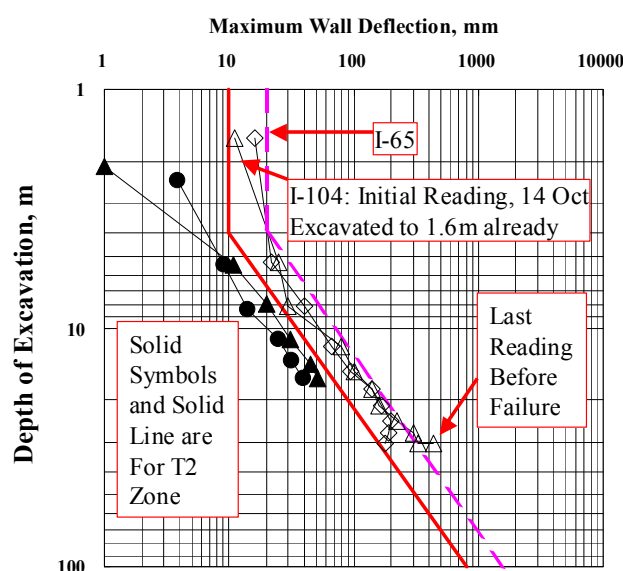
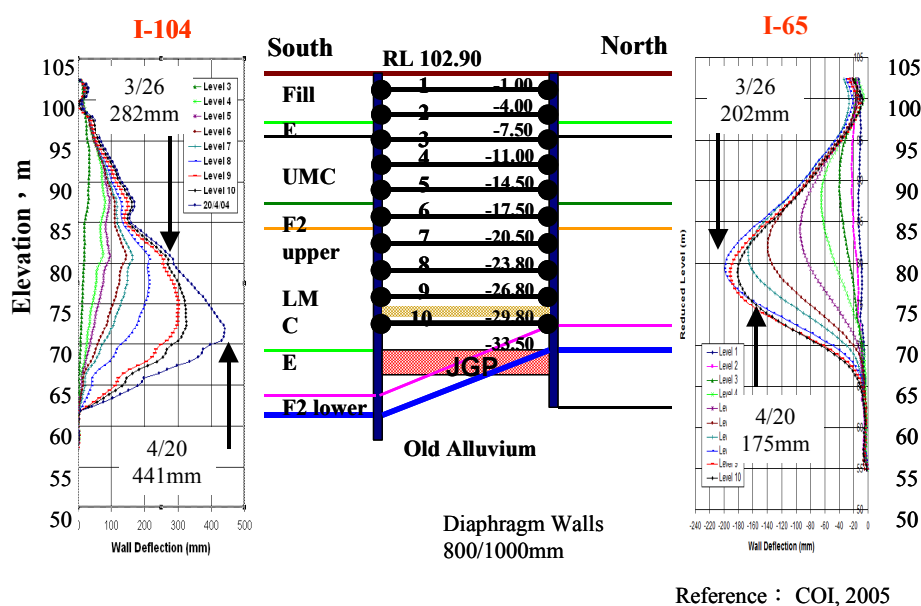Figure 12   Deflection Paths in Singapore Marine Clays as Compared with T2 Zone

Figure 13   Soil Profile and Wall Deflections at a MRT Site near Nicoll Highway

Figure 14 compares the results of cone penetrometer tests (CPT) obtained at this site with those obtained in the T2 Zone of the Taipei Basin. Despite the drastic differences in soil properties at these two locations, the wall deflection paths shown are quite similar in pattern as illustrated in Figure 12. The wall deflection paths are all parallel and the wall deflections obtained in soft marine deposits in Singapore are about 2 times of those obtained in the T2 Zone. It is thus suspected that the slopes of reference envelops are influenced predominantly by the stiffness of walls and are relatively insensitive to ground conditions. On the other hand, wall deflections in the early stage of excavation, i.e., the deflections of walls as cantilevers, however, are strongly dependent on the stiffness of soils and are relatively insensitive to the stiffness of walls.
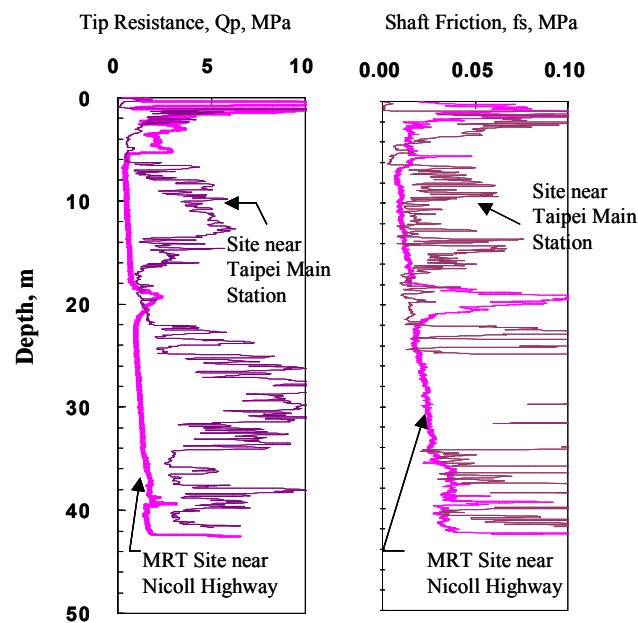

Figure 14   CPT Results Obtained at Sites near Nicoll Highway and Taipei Main Station

Effects of Ground Treatment on Wall Deflections

Ground treatment was frequently adopted in the K1 Zone for deep excavations because of the poor strengths of clays. Figure 15 shows the wall deflections obtained during excavation for constructing Yung Chun Station (BL14) of TRTS and Houshanpi Station (BL15) by using the semi-top-down construction method. Excavation was carried out to a maximum depth of 16.7m in the former case and to a maximum depth of 20.4m in the latter. Both pits were retained by diaphragm walls of 1200mm in thickness. Because of the poor ground conditions, grouted slabs of 3m in thickness were installed beneath the formation levels by using the SWING method for the purpose of reducing wall deflections.

As can be noted from Figure 15 that the maximum wall deflections remained more or less constant below a depth of 9m or so in the case of BL14 and below a depth of 13m or so in the case of BL15. These depths correspond to 60% of the depths to the grouted slabs. This phenomenon is similar to what is shown in Figure 9(d) and it is therefore concluded that the grouted slabs, if properly constructed, can be considered as the competent base strata for predicting wall deflections. It, however, should be noted that wall deflections were 15mm

and 20mm, respectively, at the levels of the grouted slabs for the two cases due to the contraction of the slabs. These deflections correspond to compressive strains of 0.15% to 0.2% in the slabs and are within the allowable level.

It is interesting to note that 2 levels of grouted slabs were indeed used at the MRT site near Nicoll Highway shown in Figure 13 by using the high pressure jet grouting technique. The upper one was installed between the 9th level and the 10th level of struts and served as a temporary strut. It was removed after the 9th level of struts were installed and preloaded for excavation to proceed to the 10th level of struts. The lower one was a permanent slab installed beneath the formation level. These two slabs did not serve the purpose of reducing wall deflections (refer to Figure 12) as indicated by the fact large wall deflections of 300mm on the south and 40mm on the north, giving an average strain of 1.7% for a span of 20m while an allowable strain of only 1% was assumed in design. It was revealed in the Committee of Inquiry that these slabs had diagonal gaps of at least a couple of meters in width because of the presence of a culvert for 66kv cable across the width of the excavation making grouting difficult.
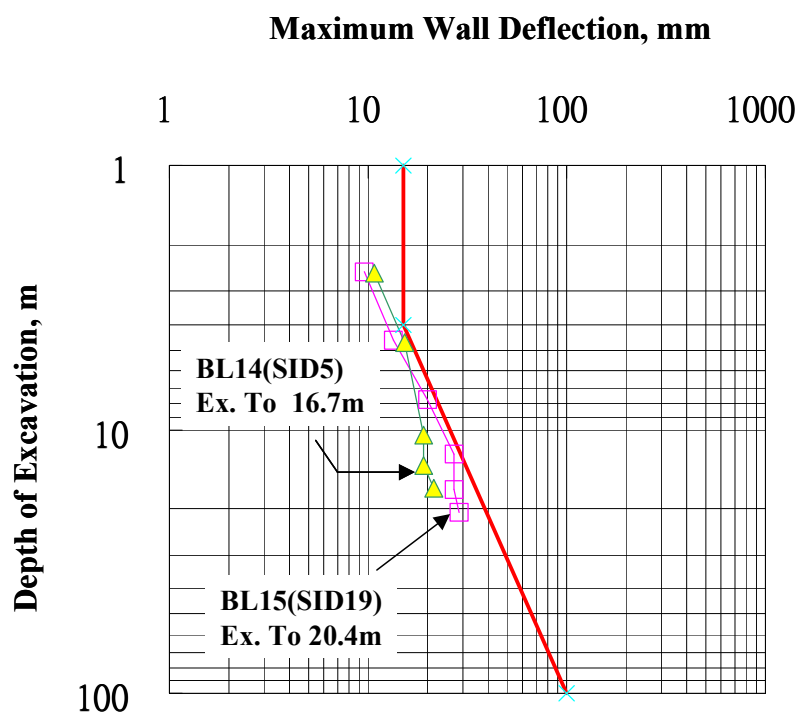
Maximum Wall Deflection, mm

Figure 15   Effects of Grouted Slabs on Wall Deflections

There are quite a few ways to treat soft ground for the purpose of maintaining the integrity of retaining structures and reducing wall deflections. For example, instead of slabs, grouted beams were frequently used. They can be considered as slabs with reduced stiffness. Buttress, for another example, can be considered as stiffener to diaphragm wall and will have a similar effect as increase wall thickness.

USE OF REFERENCE ENVELOPS IN DIAPHRAGM WALL DESIGNS

In the old days, excavations were generally shallow and sheet piles were commonly used as retaining structures. Design of walls and struts was normally based on the so-called Peck's diagrams (Peck, 1969). As excavations go deeper and deeper, and also as computer technology advances, it has become more and more popular to design walls and struts by using commercial software packages. This is a healthy development as complicated ground conditions and complicated configurations and procedures of excavations can be accounted for in the analyses. Different software packages, however, tend to give drastically different results. Furthermore, there are just too many uncertainties associated with soils and even experts of world class differ in opinions on however soils shall be modeled. It will be very difficult for practicing engineers to manage all these uncertainties. Therefore, till numerical analyses are proved to be reliable and can be used by engineers with confidence, empirical approaches remain to be useful aids complementing numerical analyses.

As mentioned above that performance-base design is currently adopted in all major projects, diaphragm walls are designed on the principle of limiting their deflections for the purpose of minimizing their influences on adjacent buildings and properties. The concept of reference envelop will be very useful as a guide to the design of walls. For example, in the T2 Zone of the Taipei Base, if wall deflections are to be limited to 30mm, walls with thicknesses of 600mm, 800mm, 1,000mm and 1,200mm will be appropriate for bottom-up excavations up to 8m, 9m, 10m and 13m, respectively, based on Figure 11. If a maximum wall deflection of 50mm is allowed, the corresponding depths of excavation can be increased to 11m, 12m, 18m and 22m, respectively. It, however, shall be cautioned that the structural capacity of walls and stability of ground below formation levels always have to be checked by rigorous analyses.

CONCLUSIONS

The concept of reference envelop presented herein is useful for evaluating the performance of diaphragm walls. The effectiveness of various types of ground treatment on wall deflections can be evaluated by comparing wall deflection paths with reference envelops. It can also be used to predict subsequent wall deflections based on the data obtained in the early stages of excavation.

It appears, based on limited data available, wall deflections in shallow excavations are influenced mainly by ground conditions and are relatively insensitive to wall thickness and, on the other hand, the slopes of reference envelops in subsequent excavation are functions of wall thickness and are relatively insensitive to ground conditions. If this indeed holds true, reference envelops for different ground conditions and different methods of constructions can be obtained with only limited data available by following the procedures outlined herein.

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