GEOTECHNICAL ISSUES IN UNDERGROUND CONSTRUCTIONS

by
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Geotechnical Issues in Underground Constructions

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SYNOPSIS: An analytical procedure is proposed herein for estimating lateral deflections of retaining walls. The performance of different retaining systems can be represented by a parameter α which is the slope of the plot of normalized wall deflection versus depth of excavation. The α values for different geological regions and different retaining systems are given. Also discussed in this paper are problems associated with groundwater and measures to overcome these problems.

1. INTRODUTION

Failures are not unusual during underground constructions, particularly, for rapid transit systems in cities in which the ground is usually soft and difficult to deal with and excavations are abnormally deep. It is fair to conclude that nearly a half of the failures were caused by the ignorance of the importance of geotechnical engineering either in design or during construction and the other half were a result of encounter of situations which were unforeseeable.

The most important element, from a geotechnical point of view, in underground constructions is certainly the retaining structure. In the past, design of retaining walls is based on the philosophy of balancing earthpressures on the two sides of walls. With more and more awareness of the importance of safety of structures adjacent to excavations, the emphasis is now shifted towards limiting ground movements. Since ground movements are closely related to deflections of retaining walls, it is desirable to limit wall deflections in the first place. Experience does prove that with sufficient efforts, wall deflections can be much reduced by increasing the rigidity of the retaining systems, however, there are cases in which the adoption of a stiffer retaining system will not be able to reduce wall deflections to acceptable values. In such cases, ground treatment is a valid option for the purpose.

Another geotechnical issue of equal importance, if not more, is groundwater control. Groundwater has been responsible for a great majority of failures in underground constructions. This is particularly true in the Taipei Basin where there exists an extremely permeable water-bearing gravel stratum at close proximity of the bottom of deep excavations.

The effectiveness of measures for reducing wall deflections is studied in detail herein. It is also the intention of this paper to address to the ways to solve groundwater problems with emphasis on the experience gained in the construction of the Initial Network of the Taipei Rapid Transit Systems (TRTS). Since wall deflection and groundwater are the two most important geotechnical issues, a safe construction can be expected once these two issues are properly taken care of.

2. GEOLOGY

A geological map of the Taipei Basin is presented in Fig. 1 for easy reference (Lee, 1996) and a system map of the TRTS Initial Network is shown in Fig. 2. An east-west section and a north-south cross-section of the

Basin are given in Fig. 3. As can be noted that at the surface is a thick layer of Sungshan Formation. Toward the east and the north, silty clay dominates while in the central city area, where the Taipei Main Station is located, the six-sublayer sequence is evident.Toward the west, the way the various strata are interbedded becomes rather complex. Toward the south, ground becomes gravelly. A typical CPT profile obtained in the central city area of Taipei is shown in Fig. 4 and the soil strengths obtained in laboratory tests are given in Fig. 5. The soft nature of subsoils in the Sungshan Formation is readily apparent.

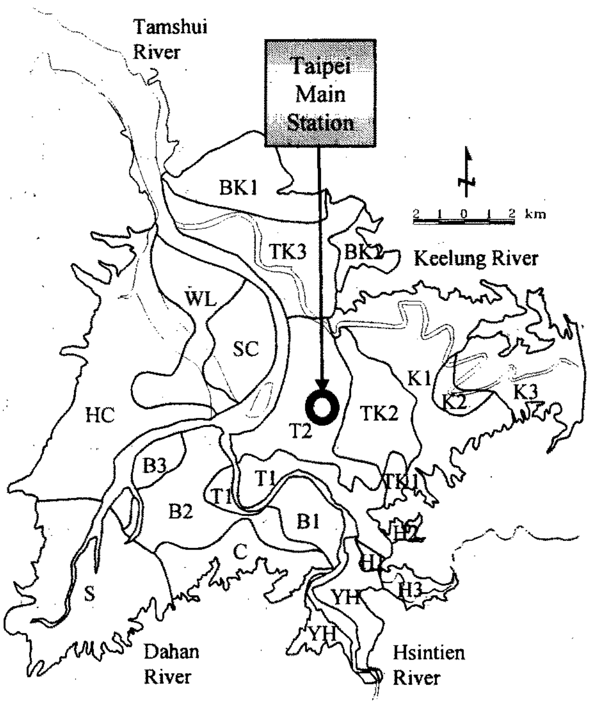


Fig. 1 Geological Zoning Map

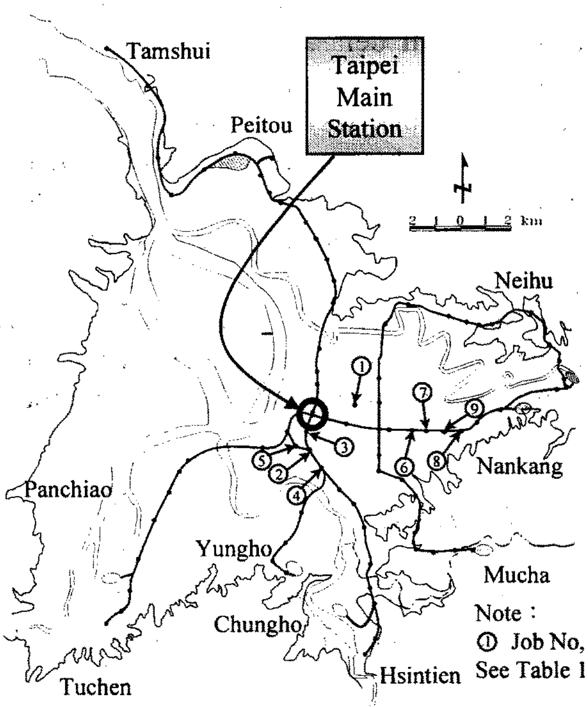


Fig. 2 TRTS Network of Taipei and Locations of Cases Studied

The Sungshan Formation is underlain by a water-bearing stratum, the so-called Chingmei Gravels, which contains gravels of various sizes and is extremely permeable. This

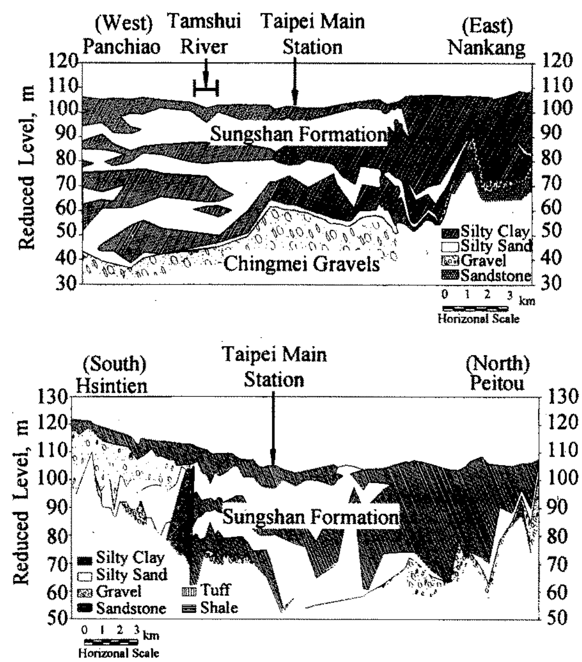


Fig. 3 Geological Profiles of the Taipei Basin

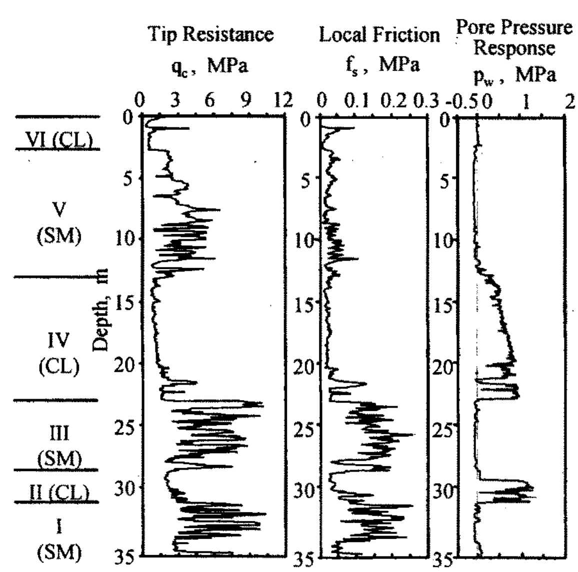


Fig. 4 CPT Profile in Central Taipei

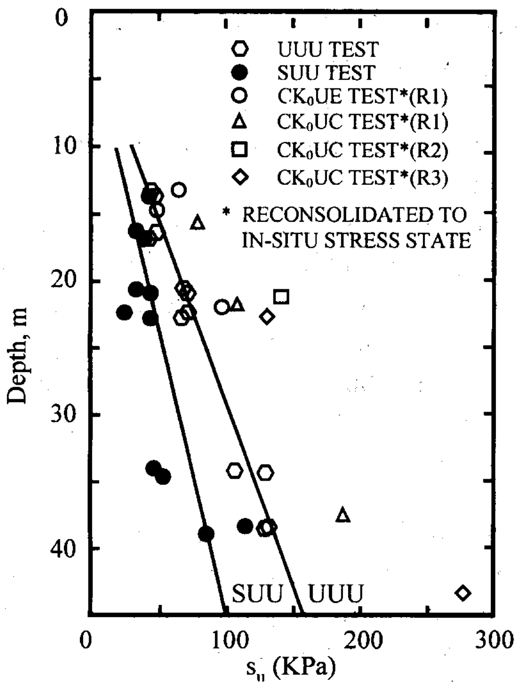


Fig. 5 Undrained Shearing Strength of Clays in the Sungshan Formation

gravelly layer was the sole water supply for the entire Taipei City prior to the 70's. It was responsible for several major failures during the underground construction of the Taipei Rapid Transit Systems (Moh, Ju and Hwang, 1997; Ju, Duann and Tsai, 1998).

3 WALL DEFLECTIONS

Table 1 summarizes some of the observations made in recent years. Wall deflections obtained in the TRTS constructions in the central city area (T2 Zone), i.e., Cases 2 to 5, are compared to those obtained previously (Woo and Moh, 1991) in Fig. 6. It is readily apparent that TRTS constructions out-perform the constructions carried out in the past as can be noted from the far less wall deflections observed. The factors affecting wall deflections can be summarized as follows:

a) depth of excavation

b) ground conditions, i.e., soil stiffness and ground water table

c) method of construction, e.g., top-down, bottom-up, or semi top-down

d) rigidity of retaining systems, including wall elements and strutting members

Table 1 Cases Studied

Job Number	Site Contract Location	Geology	Method of Construction	Case No: Inclinometer	Wall Thick Thickness (m)	Wall Depth (m)	Wall Movement (mm)	Depth of Excavation (m)	δmax / Depth (%)	α Eq. 1	Ground Treatment
1	Site B (Chang, et. al)	TK2	top-down	1a: SID1	1.20	55.00	110.0	26.6	0.41	155	
				1b: SID8	1.20	55.00	120.0	26.6	0.45	170	
				1c: SID2	1.20	55.00	83.0	26.6	0.31	117	buttress
				1d: SID7	1.00	55.00	98.0	26.6	0.37	139	buttress
				1e: SID3	1.20	55.00	45.0	26.6	0.17	64	cross panel
				1f: SID5	1.20	55.00	70.0	26.6	0.26	99	cross panel
2	CH219 G11	T2	semi top-down	2a: SID2	1.20	41.00	26.5	23.4	0.11	48	
				2b: SID3	1.20	41.00	36.1	23.4	0.15	66	
				2c: SID4	1.20	41.00	34.6	23.4	0.15	63	
3	CH218 R12	T2	bottom-up	3a: SID1	1.00	30.50	26.6	16.4	0.16	99	
				3b: SID2	1.00	30.50	24.7	16.4	0.15	92	
				3c: SID3	1.00	30.50	24.7	16.4	0.15	92	
4	CH220 G10	T2	semi top-down	4a: SID9	1.20	42.00	24.2	23.7	0.10	43	
5	CN251 CCT	T2	bottom-up	5a: SID2	0.80	30.00	8.4	20.1	0.04	21	cross panel
				5b: SID3	0.80	30.00	21.4	20.1	0.11	53	
				5c: SID6	0.80	30.00	29.6	16.4	0.18	110	
				5d: SID8	0.80	30.00	41.5	16.4	0.25	154	
6	CN256	K1	semi top-down	6a: SID1	1.00	37.00	49.0	16.2	0.30	187	
				6b: SID3	1.00	37.00	47.0	16.2	0.29	179	
				6c: SID4	1.00	37.00	53.0	16.2	0.33	202	
7	CN257 BL13	K1	bottom-up	7a: SID4	1.20	44.00	32.0	18.8	0.17	91	jet-grouted slab
				7b: SID6	1.20	44.00	24.0	18.8	0.13	68	jet-grouted slab
				7c: SID7	1.20	44.00	40.0	18.8	0.21	113	jet-grouted slab
8	CN258 BL14	K1	top-down	8a: SID5	1.20	38.00	23.0	16.7	0.14	82	jet-grouted slab
				8b: SID19	1.20	38.00	27.0	16.7	0.16	97	jet-grouted slab
9	CN259C	K1	bottom-up	9a: SID3	1.00	32.00	25.0	15.0	0.17	111	
				9b: SID2	1.00	31.50	22.0	15.0	0.15	98	cross-beams

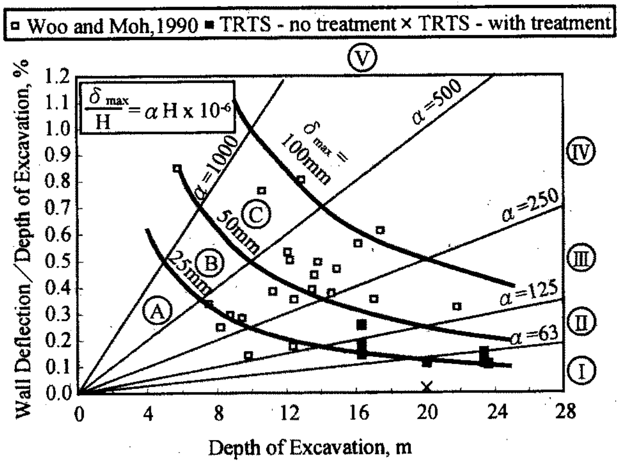


Fig. 6 Wall Deflections in the T2 Zone of the Taipei Basin

e) ground treatment, e.g., grouting

f) preloading of struts

g) workmanship, e.g., promptness of strutting

Because all these factors are affecting the magnitudes of wall deflections at the same time, a methodology is desirable for quantifying their effects individually.

By and large, it is expected that wall deflections can be related to depth of excavation as follows:

$$\frac{\delta_{max}}{H} = \alpha H \bullet 10^{-6} \quad \text{.........................} \quad Eq.\ 1$$

where δ max = maximum wall deflection, H = depth of excavation. Lines corresponding to α = 62.5, 125, 250, 500, and 1000 (with an implicit dimension of m^{-1}), with H expressed in meters, are presented in Fig. 6. These lines divide the plot into various zones, i.e., Zones I, II, III, etc, with wall deflections in each zone equal to twice of that for the preceding zone.

The majority of data points obtained previously in the T2 Zone in the Taipei Basin, as depicted in Fig. 6, fall in Zone IV and 300 will be a reasonable average of α values by inspecting the data visually. For TRTS excavations, the α values for individual data points are given in Table 1 and the average is worked out to be 82, which is only 27% of the average value for previous excavations. In other words, wall deflections in TRTS excavations are, on an average, less than a third of what were obtained previously. It is difficult to single out any factor which was primarily responsible for the difference. Since depth of excavation and ground conditions have been explicitly considered in the plot, the difference in the two sets of data must be a result of the combination of the rest of factors listed above.

All the TRTS cases presented in Fig. 6 are for bottom-up or semi to-down excavations without any form of ground treatment. It is difficult to figure out the details associated with the previous excavations. In any case, it is believed that the superior performance of TRTS constructions are primarily a result of higher stiffness of the retaining systems and better workmanship. In the TRTS constructions, diaphragm walls are usually thicker than what were previously adopted by 200mm to 300mm. This, together with preloading of struts, gives much greater stiffness of the retaining systems. Furthermore, stringent quality control and quality assurance procedures are followed, therefore, works were carried out more orderly and more carefully.

3.1 Wall Thickness

The average α values for cases with diaphragm walls of 0.8m (Cases 5b, 5c and 5d), 1m (Cases 3a, 3b and 3c) and 1.2m (Cases 2a, 2b, 2c and 4a) are 106, 94 and 55, respectively, and data in each of these three sets are fairly consistent. The influence of diaphragm wall thickness is readily apparent.

3.2 Ground Conditions

A similar trend can be observed in Fig. 7 for K1 Zone. Previous data would indicate an average α value of 300. The probable reason for the fact that the α values for the

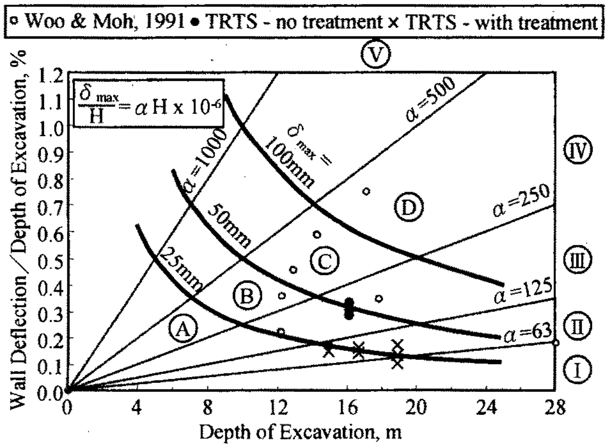


Fig. 7 Wall Deflections in the K1 Zone of the Taipei Basin

previous data are both 300 for T2 and K1 Zones is that the diaphragm walls used in the K1 Zone tended to be thicker than those used in the T2 Zone.

The average α value for TRTS excavations is 170, or about a half of what was obtained previously. Because the subsoils in the K1 Zone are too soft for deep excavations, ground treatment was carried out at most of sites. Therefore, data points for TRTS constructions are rather limited in number. In all these 4 cases analyzed (Cases 6a, 6b, 6c and 9a), the walls were 1m in thickness. The average α value of 170 is nearly 1.8 times of the value of 94 for the TRTS excavations in the T2 Zone of the same diaphragm wall thickness. The comparison does shed light on the influence of ground conditions on wall deflections.

3.3 Method of Construction

All the cases mentioned above were bottom-up constructions. Job 1 in Table 1 was carried out in a top-down sequence. Although it is located in TK2 Zone, the ground conditions are expected to be pretty close to those of T2 Zone. In fact, it is located in the T2 Zone in the old geological map of the Taipei Basin (Moh and Associates, 1987). Furthermore, although it is a non-TRTS excavation, it was carried out in recent years and was carried out, more or less, to TRTS standards. Therefore, it is fair to compare the results with what was observed in TRTS constructions.

The two inclinometers, 1a and 1b, gave an average α value of 163 which is 3 times of the α value for TRTS excavations in T2 Zone with diaphragm walls with the same thickness of 1.2m. It is believed that ground conditions and methods of construction have equal contribution to this difference in performance.

3.4 Singapore Marine Clay

Regarding Singapore experience, the 6 cases listed in Table 2 (Kong, 1999) and plotted in Fig. 8 indicate that the α values for sheet-piled excavations in Singapore marine clay range from 1000 to 7000, with an average of 3872 which is very close to the upper bound of Zone VII. Singapore marine clay is probably the softest materials the authors have ever experienced. When excavation was

Table 2 Sheet-Piled Excavations in Singapore Marine Clay (Kong, 1999)

Sources	Excavation depth, H m	Max Deflection δ_{max}, mm	δ_{max}/H %	Nos. of Strut Levels	α value
Lee (1986)	8.80	130	1.48	4	1679
Chin (1986)	7.50	400	5.33	3	7111
Yang (1985), Case I	9.00	352	3.90	3	4346
Yang (1985), Case II	8.00	100	1.25	3	1563
Yang (1985), Case IIII	5.00	120	2.40	1	4800
Moh and Associates (1987)	6.30	148	2.35	3	3729

carried out at MRT Novena Station, wall deflections had already exceeded 100mm before the first level of struts were placed. One could even feel the "quake" when a lorry passed nearby.

It is hypothesized that the cases presented in Table 2 and Fig. 8 are those with problems and are not necessarily representative of the general excavations in marine clay. It is the authors' opinion that an α value of 2500 would be more appropriate. This α value is 8 times of that for the non-TRTS cases in the K1 Zone in which clay dominates. The clay in K1 Zone is roughly three times in strength in comparison with young Singapore marine clay. Furthermore, in all the non-TRTS excavations in the K1 Zone, diaphragm walls were used. Ground conditions and rigidity of wall may have equal contributions to this difference in performance.

3.5 Discussions

The results of the above regression analyses are summarized in Table 3 and Fig. 9. The primary factors affecting α values are: a) ground conditions, b) wall type, c) method of construction, and d) preloading of struts, and e) ground treatment. Based on the limited data available, the multipliers proposed in Table 4 can be used to extrapolate the α value from one case to another. As always, a spread of data of plus and minus 50% is expected.

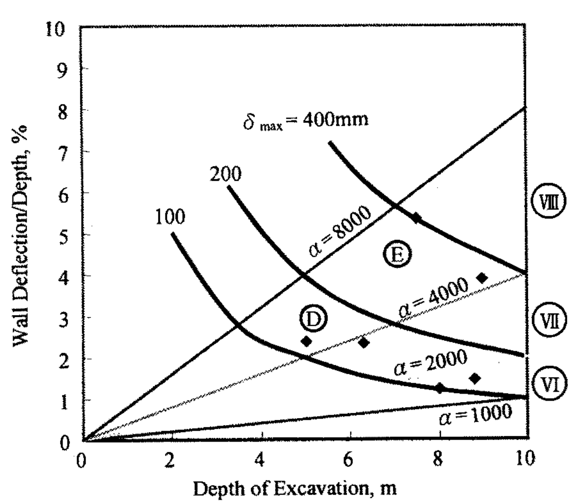


Fig. 8 Wall Deflections in Singapore Marine Clay

Table 3 α Values for Different Sets of Conditions

Data Set	Geology	Case	Region of Deflection	α value
Bottom-up Constructions				
1	T2	previous data	IV	300
2	T2	TRTS– 0.8m wall	II	106
3	T2	TRTS– 1m wall	II	94
4	T2	TRTS– 1.2m wall	I	55
5	K1	previous data	IV	300
6	K1	TRTS– 1m wall	III	170
7	Singapore	marine clay	VII	2500
Top-down Constructions				
8	TK2	Recent - 1.2m wall	III	160

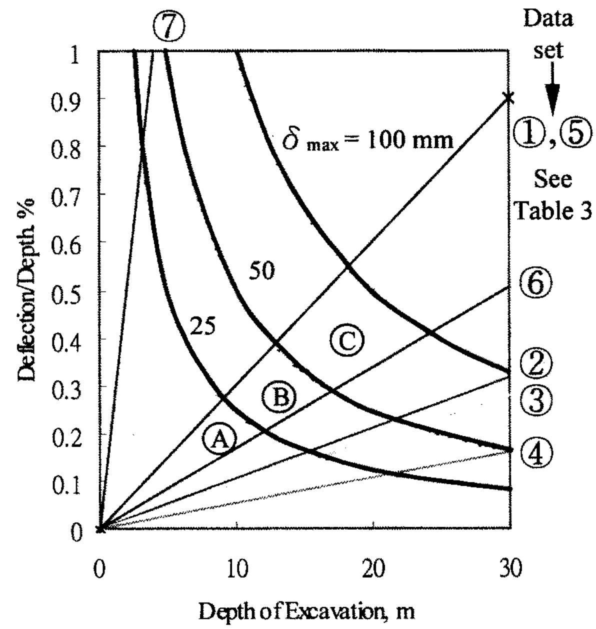


Fig. 9 Estimated Wall Deflections for Different Sets of Ground Conditions

It is believed that this increase in wall thickness is cost-effective in the sense that in TRTS constructions none of the problems were caused by lateral deflections of walls. This eliminates the necessity of protective measures

Table 4 Proposed Multipliers for Estimating α Values

Parameter	Representative Condition	Multiplier
Ground Conditions	T2: K1: marine clay	1: 1.5: 5
Retaining Structures	1.2m wall: 1.0m wall: 0.8m wall: sheet pile	1: 1.5: 2: 4
Strut	preloaded: without preloading	1: 2
Method of Construction	bottom up: top down	1: 2
Ground Treatment	treated: untreated	1: 2

which are both expensive and ineffective. Most importantly, construction can be progressed smoothly without having to worry about the safety of adjacent structures or utilities and engineers do not have to diverge their attention to non-technical nuisances, such as negotiation, compensation, or even law suits. For diaphragm walls of 1.2m or thinner, the costs for the installation would be about the same, regardless of wall thickness. The costs for the extra concrete are really minimal.

4 EFFECTS OF GROUND TREATMENT

The plots shown in Figs. 6 to 9 are divided into Zones A, B, C and so on in accordance with maximum wall deflections. From a building protection point of view, the following are rules of thumb regarding wall deflections,

a) deflections of 25mm or less, i.e., Zone A, are acceptable and building protection measures are usually not required;

b) deflections in the range of 25mm to 50mm, i.e., Zone B, may cause damages to poor structures within 10m or so;

c) deflections in the range of 50mm to 100mm, i.e., Zone C, may cause damages to structures with individual footings within 10m or so;

d) deflections in the range of 100mm and 200mm, i.e., Zone D, even piled foundations within 10m or so may be damaged;

e) deflections exceeding 200mm, i.e., Zone E, are unacceptable.

The above "rules of thumb" are certainly subject to debate as some people are more tolerant than others and conditions of structures are difficult to assess.

As can be noted from Fig. 9 that even with TRTS standards (Sets 2, 3, 4 and 6), wall deflections are likely to exceed 25mm if excavations exceed 15m or so in depth and to exceed 50mm if excavations exceed 22m. Extra efforts are required if adjacent structures are likely to be damaged and it is necessary to reduce wall deflections. In such cases, ground improvement may be considered to:

a) increase the passive soil resistance below the bottom of excavation, and/or

b) reduce active earthpressures acting on the outer face of the wall.

Experience has shown that it is not cost effective at all to perform ground treatment behind the wall. Furthermore, no matter what grouting method is used, side effects are inevitable. For example, jet grouting has been found to induce significant heave in clays and settlement in sands. Heaves and settlement of an order of 100mm to 300mm were frequently observed and case histories of damages to adjacent structures or utilities due to jet grouting are not uncommon. Therefore, treatment outside excavation is highly discouraged. It is much safer to carry out improvement inside the excavation.

Basically, jet grouting can be made to form:

a) individual columns sparsely spaced simply to increase soil stiffness,

b) buttresses to increase the stiffness of wall,

c) cross beams as buried struts,

d) grid beams,

e) continuous slab, or,

f) cross panels.

Figure 10 shows a case (Job 1 in Table 1) in which two different schemes were adopted to treat the ground in an attempt to reduce wall deflections. This provides an opportunity to directly compare their effectiveness (Chang, Wang and Huang, 1998). The site has a L shape made of two interconnected blocks, ie.,

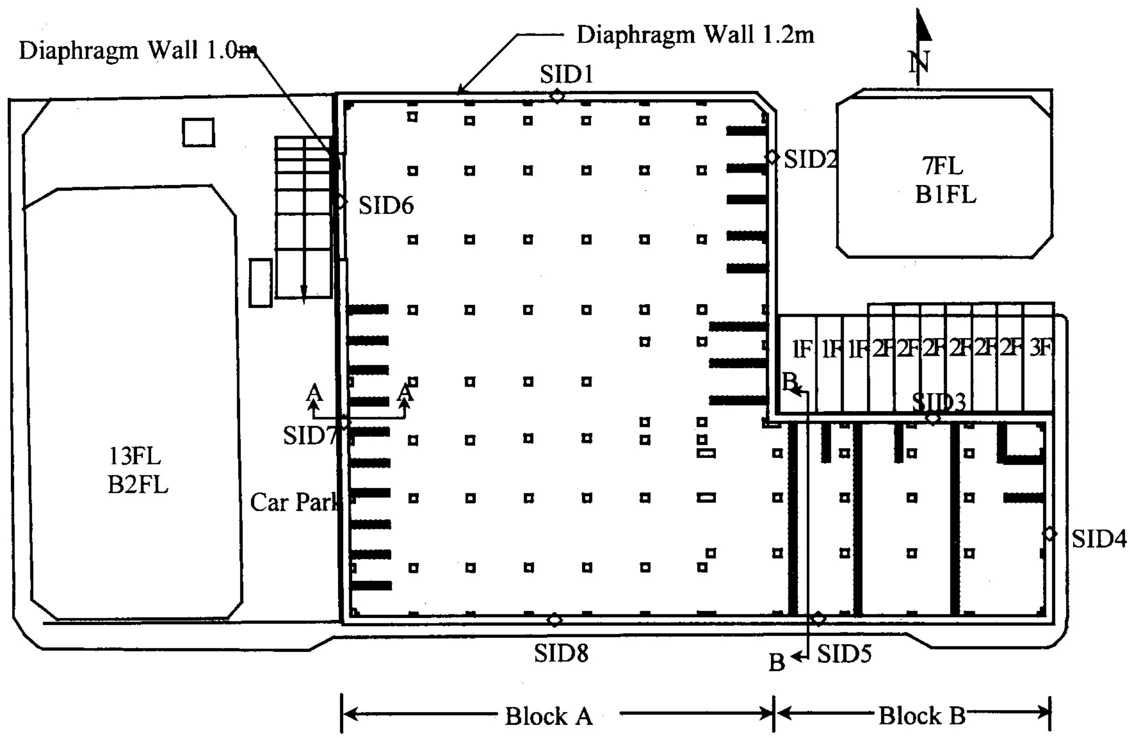


Fig. 10 Plan and Layout of Ground Treatment for Job 1

Block A which is a 60m x 74m square, and Block B which is a 39m x 28m rectangle. Because the top-down construction method was used, foundation piles were installed before the commencement of excavation. Diaphragm wall of 1m in thickness was used on the western side of Block A where earthpressures were expected to be less because the wall was right next to an underground parking lot. Elsewhere, the diaphragm walls were 1.2m in thickness.

No treatment of any kind was made on the northern and the southern sides of Block A and the wall deflections obtained therein serve as the basis for comparison. At the end of excavation to a depth of 26.6m, Inclinometers SID1 and SID8, refer to Fig. 11, showed maximum lateral deflections of 110mm and 120mm, respectively, which, refer to Table 1, correspond to α values of 155 and 170 with an average of 163.

4.1 Buttress

Figure 12 shows the fact that, as can be noted from the results obtained by Inclinometers SID2 and SID7, the use of buttresses, installed by using the diaphragm walling technique on the eastern and the western walls of Block A reduced wall deflections to 83mm and 98mm, respectively, with an average α value of 128 which is 22% less than that for the cases without buttress. The effects of the thinner wall at the location of SID7 is believed to have been balanced by the presence of the underground parking lot right next to the wall, and comparison is therefore fair. In this particular case, the buttresses were not structurally connected to the diaphragm wall and grouting was carried out in-between the two.

4.2 Cross Panel

On the other hand, the results obtained by Inclinometers SID3 and SID5 shown in Fig. 13 indicate that the use of lean-concrete panels across the width of excavation using the diaphragm walling technique, reduced wall deflections to 45mm and 70mm, respectively, with an average α value of 82 which is exactly

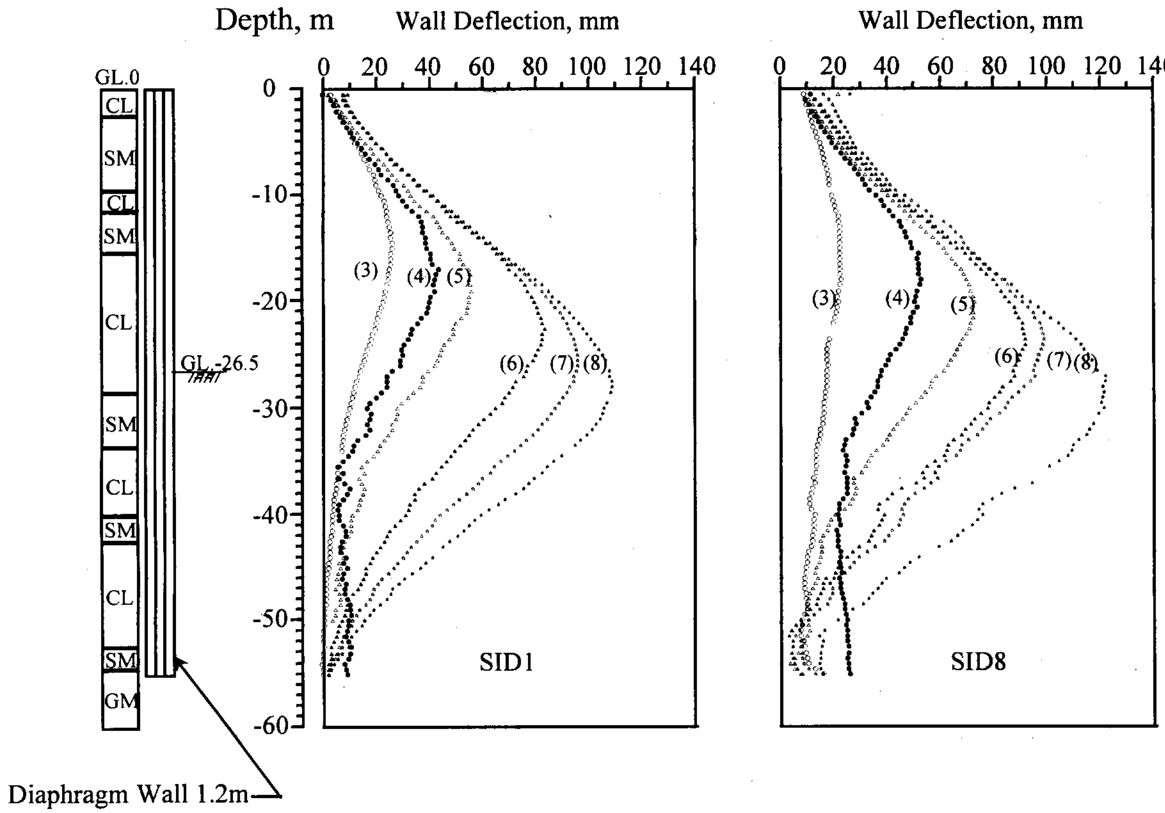
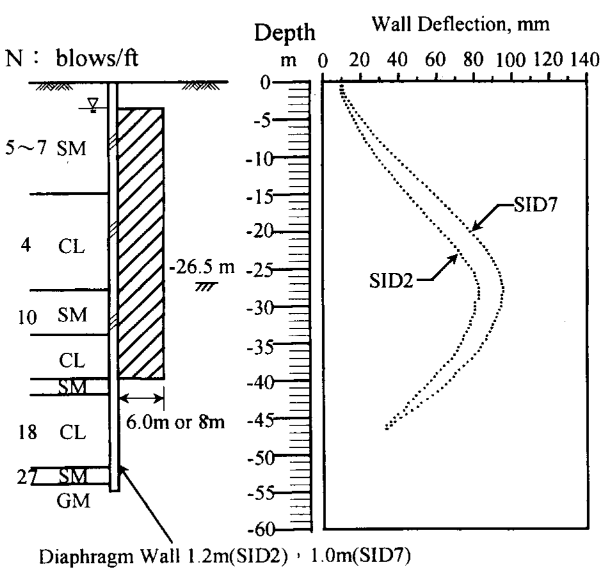
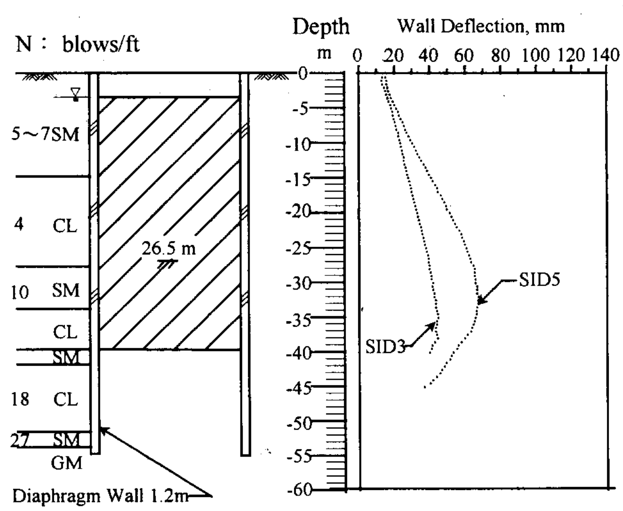


Fig. 11 Wall Deflections - Job 1 Without Ground Treatment



Section Ⓐ-Ⓐ , Fig. 10

Fig. 12 Wall Deflections - Job 1 With Buttresses



Section Ⓑ-Ⓑ , Fig. 10

Fig. 13 Wall Deflections - Job 1 With Cross Panels

a half of what was obtained for the cases without any treatment. The cross panel in Case 5a reduced the α value for this 20.1m excavation to 21 which is 5 times smaller than the α value for cases in Set 2 shown in Table 3. It is therefore concluded that cross panels are effective in reducing wall deflections.

4.3 Jet-Grouted Slab

In K1 Zone, as mentioned above, the average α value obtained in the 4 cases in TRTS constructions is 170 which is used as the reference value for the following comparisons. A jet-grouted slab of 4m in thickness were used for constructing Station BL13 in Contract CN257 of the Nankang Line (Job 7 in Table 1) and for constructing the cut-and-cover tunnels next to this station. Excavation was carried out to a depth of 22.8m by using the bottom-up constructio method. Wall deflections of 32mm, 24mm and 40mm, refer to Fig. 14, were obtained corresponding to α values of 91, 68 and 113 with an average of 91 which is far smaller than the reference value of 170 for the cases without treatment, indicating the significant effectiveness of the scheme.

Jet-grouted slabs were also used for constructing Station BL14 in Contract CN258 of the Nankang Line (Job 8 in Table 1). Excavation was carried out to a depth of 16.7m and, refer to Fig. 15, wall deflections were 23mm and 27mm, corresponding to α values of 82 and 97, giving an average of 90. The effectiveness of the scheme is again proved. Unlike the case for Station BL13 (Job 7 in Table 1), this excavation was carried out by using the top-down method. In this case, it appears that these two methods of construction did not make any difference. This is somehow inconsistent with the previous findings and shall be confirmed by more data.

4.4 Cross Beam

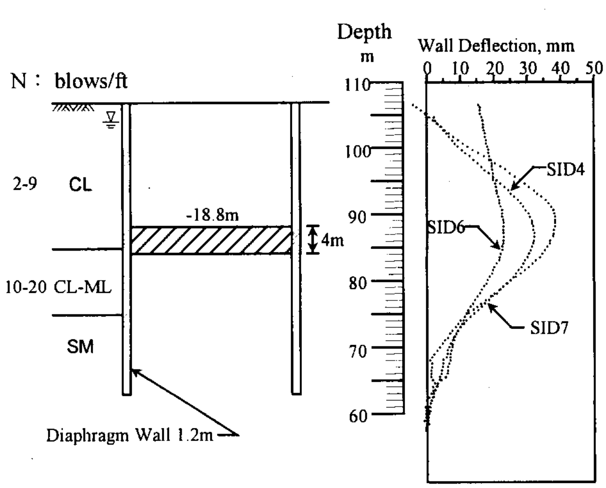


Fig. 14 Wall Deflections - Job 7 With Grouted Slab

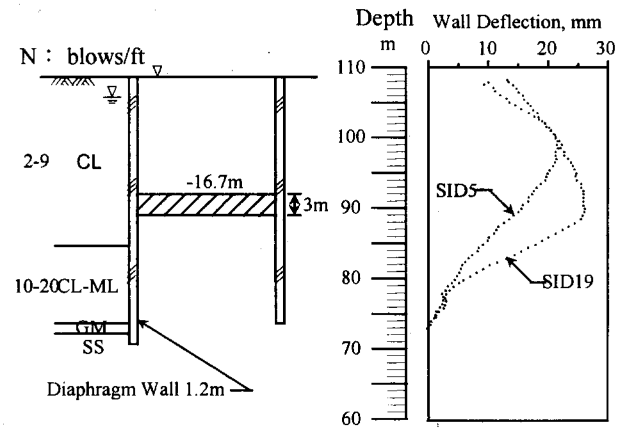


Fig. 15 Wall Deflections - Job 8 With Grouted Slab

It is often questioned whether it is necessary to have to form a solid slab across the entire excavation or to form solid cross panels with the full height. Will a partial treatment be as effective? Figure 16 shows the results obtained in the excavation carried out in Contract CN259C (Job 9 in Table 1) to a depth of 15m with the bottom supported by reinforced concrete cross beams of only 1m in width and only 3m in height. Beams were installed at 5m spacings. Wall deflections of 22mm to 27mm were observed, giving an average value of α is 98 which is about the same as the average value for Job 7 in which jet-grouted slab was used. It is therefore concluded that concrete cross beams are as effective as grouted slab.

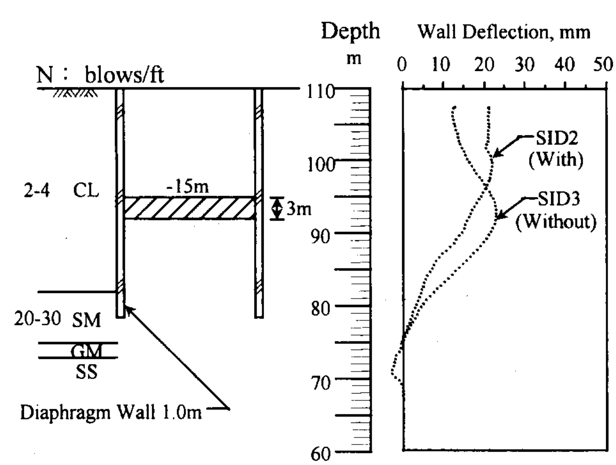


Fig. 16 Wall Deflections - Job 9 With and Without Cross Beams

4.5 Summary

All the cases studied above are plotted in Fig. 17 with the four major factors, which affect wall deflections the most, i.e., wall thickness, method of treatment, geological zone, and method of construction, denoted. Such a figure can be used to quantify the effects of individual factors if sufficient data are available.

5 GROUNDWATER CONTROL

Experience indicates that a great majority of failures in underground constructions were associated with groundwater problems and this is particularly true in the Taipei Basin in which there exists an extremely permeable water-bearing gravel stratum, i.e., the Chingmei Gravels. A sufficient factor of safety is always necessary for maintaining the integrity of soil plug below the bottom of excavation against piping. A natural solution is to lengthen the soil plug by extending the diaphragm wall to a sufficient length so the hydraulic gradient will be small enough to avoid piping. However, there are cases in which such a method is impractical or uneconomical. Three alternatives are available to deal with the problems, i.e.,

a) to seal the soil plug at the bottom,

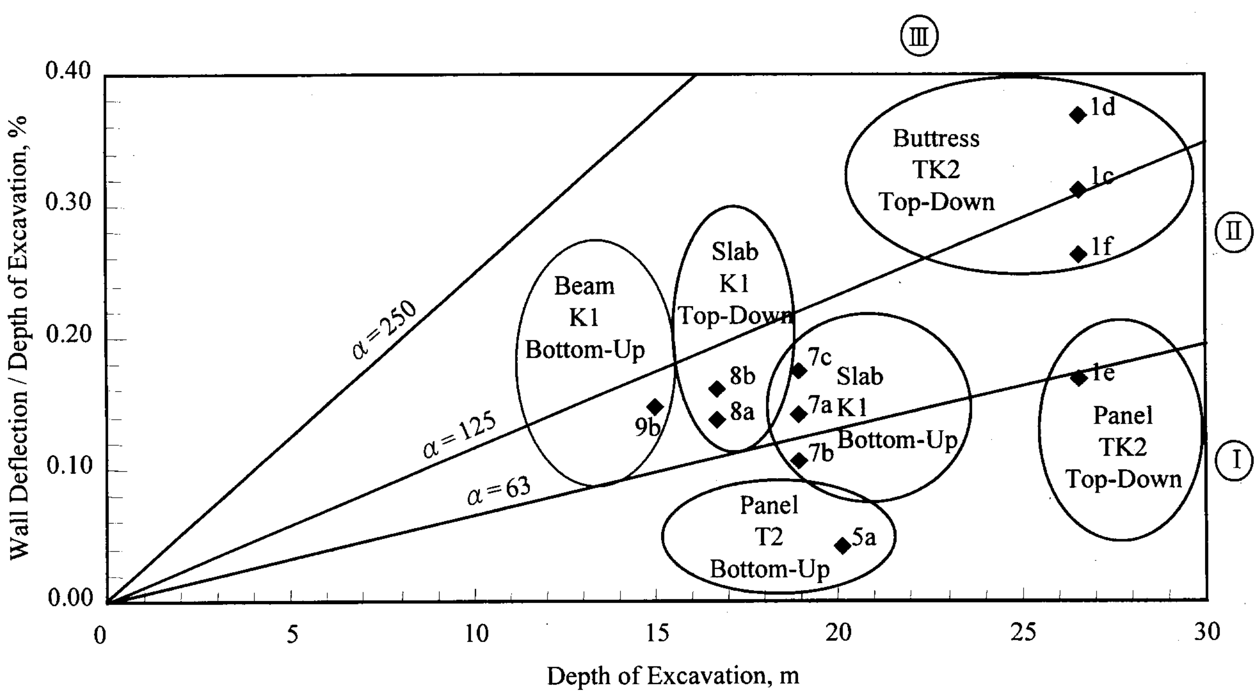
b) to lower the groundwater level, and

c) to fill up the pit and carry out excavation in water.

Case histories for all these alternative are introduced as follows.

5.1 Sealing Soil Plug

Figure 18 shows the scheme adopted for constructing the ventilation shaft in the Chungho Line of the TRTS. Because immediately below the bottom of excavation exists the Chingmei Gravels which is extremely permeable and very difficult to excavate, it is impractical to extend the diaphragm wall to a sufficient length to reduce hydraulic gradient to



Note : The circled descriptions are in the sequence of appearing as follows : a) wall thickness, b) method of treatment, c) geological zone and d) method of construction

Fig. 17 Wall Deflections for Different Schemes of Treatment

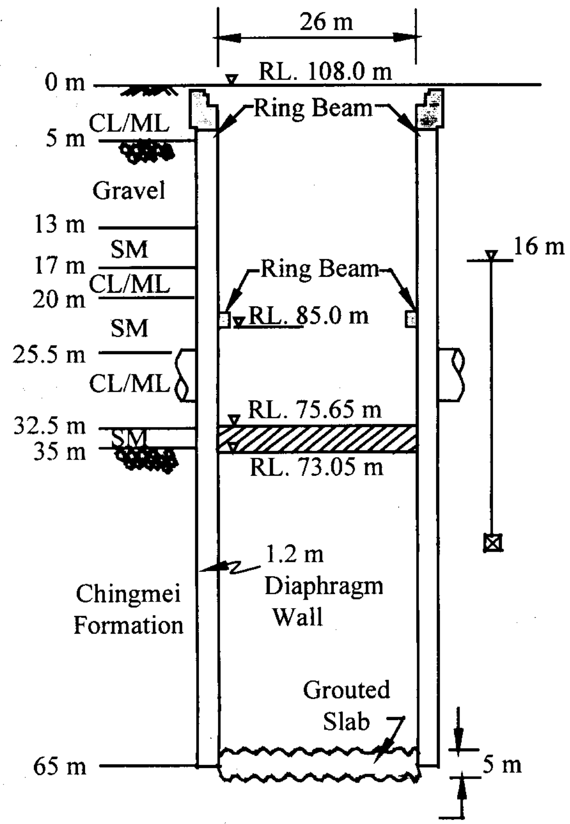


Fig. 18 Grouted plug at Ventilation Shaft in Contract CH221

the acceptable value. An impermeable pad of 5m in thickness was formed at the toe level of diaphragm wall by grouting to completely seal off the soil plug. This was effective and excavation was able to proceed successfully.

This shaft is 26m in its inner diameter and was retained by diaphragm wall of 1.2m in thickness. There were no internal bracings other than the two ring beams. The diaphragm wall essentially became a compression ring resisting earthpressures all around its outer face. Wall deflections were within 10mm which corresponds to an α value of only 10. This value is extremely small for an excavation of 32.4m in depth in the T2 Zone in which the shaft is located. It is thus evident that circular excavation by far out-perform rectangular ones.

5.2 Pumping

Pumping was carried out at maximum rates of 3600, 4170, and 2450 tons per hour in Contracts CP261, CP262 and CT201T of TRTS for excavations to depths of 34m, 36.6m, 28.9m, respectively (Hwang, et. al., 1996; Moh, Chuay and Hwang, 1996). Figure 19 shows the drawdowns of piezometric pressures in the Chingmei Gravels observed at various distances in the period in which pumping was carried out in Contract 261. As can be noted that even at a distance of, say, 5 km away from where pumping was carried out, the drawdowns could still be as much 2m, indicating the great hydraulic conductivity of the Chingmei Formation. The lowering of piezometric pressures in the Chingmei Gravels, however, did not result in much ground settlements, if any, in the central city area because of the fact that:

a) refer to Fig. 4, the impervious clayey Sublayers II and IV effectively retained the water pressures in the Sungshan Formation. and

b) the ground had settled by more than 2m prior to the 70's because of the lowering of

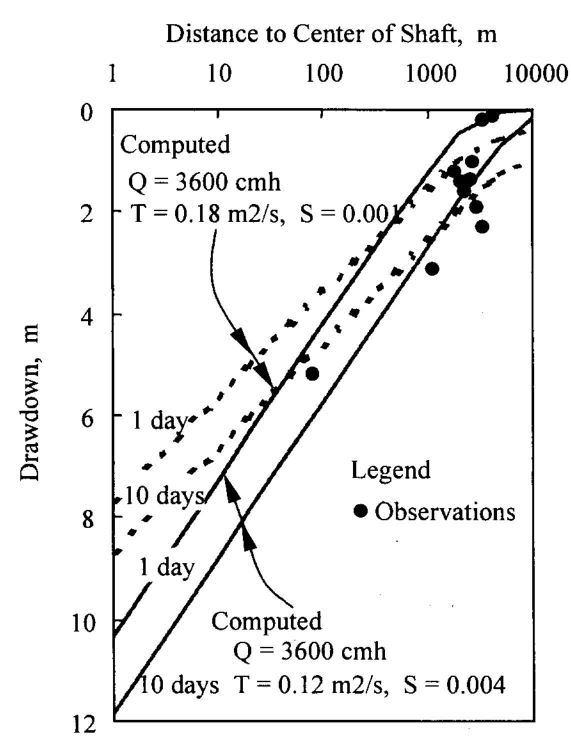


Fig. 19 Groundwater Drawdown during CP261 Pumping

piezometric pressures in the Chingmei gravels by as much as 40m.

Ground settlements in suburbs where ground has not been pre-consolidated previously, however, were of an order of 10mm during the period in which pumping was carried out at these three sites.

Settlement due to lowering of groundwater table could be extremely serious and should never be overlooked. Settlements exceeding 200mm were frequently observed in Singapore, for example, along Scotts Road during the construction of the North-East Line of the Singapore MRT as a result of leaking of groundwater into tunnels. Recharging is naturally one of the solutions to the problem. However, unless the water is pressurized, the scheme is unlikely to be effective. It is to the authors' knowledge that recharging was carried out at Gallang Gas Works during the construction of the East-West Line of the Singapore MRT and was effective in maintaining the groundwater table thereat. Unfortunately, little information is available in literature.

5.3 Wet Excavation

For constructing Marina Bay Station of the Singapore MRT System and the cut-and-cover tunnels extending from this station across Telok Ayer Basin, excavation was carried out in flooded cofferdams as illustrated in Fig. 20

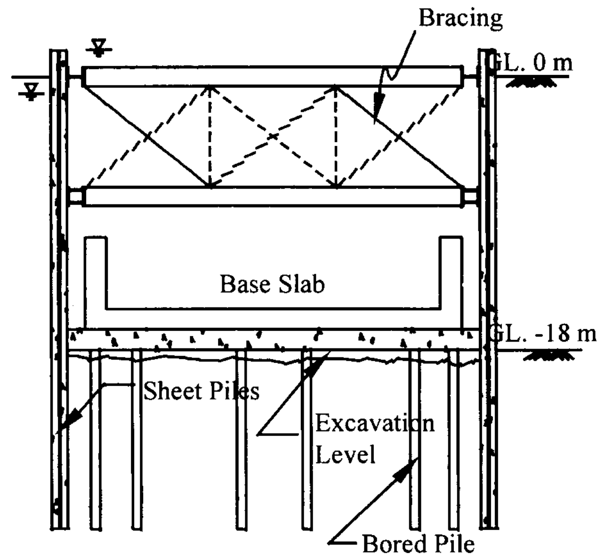


Fig. 20 Wet Excavation at Marina Bay in Contract 310 of SMRT (Modified from Denman et. al., 1987)

(Denman, et. el., 1987). The site was in a piece of reclaimed land and was undergoing settlement of a rate of 30mm per year when construction was carried out in 1986. Excavation was carried out to a depth of 7m first with two levels of struts placed. The pit was flooded and the remaining excavation was carried out by dredging. Upon reaching the final depth of excavation, a concrete slab was cast and tension piles installed. The pit was then drained and the permanent structure was constructed.

A very innovative scheme, refer to Fig. 21, was adopted for constructing the sewerage systems in Mexico City which is well known

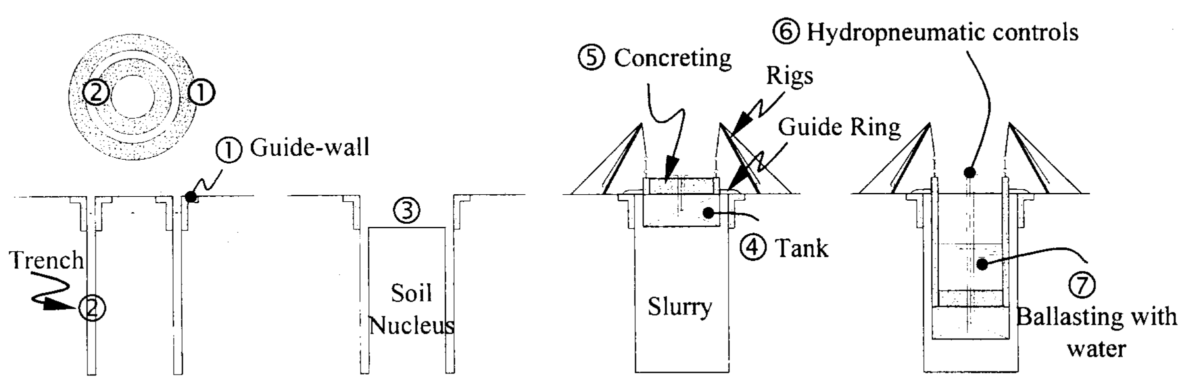


Fig. 21 Wet Excavation and Floating Construction Mexico City (Modified from Auvinet and Organista, 1998)

for its poor ground conditions (Auvinet and Organista, 1998). Water contents of subsoils were as much as 400% at places and ground settlements were frequently of an order of meters. First of all, a guide trench is made with two RC rings as guide walls. The width of this trench is unimportant and a width of 0.5m will be adequate. Excavation is then carried out in the trench to a depth of a couple of meters below the bottom of the permanent structure. The stability of the trench is maintained by slurry. The inner guide wall is then demolished and the central soil nucleus is removed by dredging. A cylindrical steel tank is placed on the surface to serve as a platform for casting the permanent structure, which is essentially a floating cylindrical tank. The tank sinks progressively into the slurry while the construction of the permanent structure proceeds. It may be necessary to charge water into the tank or add weight to it to enable it to sink. Maintaining the verticality of the tank all the times is crucial. The tank is constrained at surface and the verticality of the structure is maintained by hydro-pneumatic control. Finally, as the tank sinks to the desired level, the space between the tank and the surrounding soil is grouted from the bottom up. Since 1969, a total of 35 circular shafts, of which the largest is 17m in diameter, have been excavated using this scheme and it is intended to construct shafts with diameters as large as 25m in the future.

A similar scheme was worked out years ago when the methods for constructing the three deep ventilation shafts in Contract CH221, CP261 and CP262 of TRTS were reviewed by the authors on behalf of the project owner, Department of Rapid Transit Systems of the Taipei Municipal Government. Its application can best be illustrated in the case shown in Fig. 18. In the case shown, diaphragm wall penetrate into the Chingmei Gravels by 30m. This is a very difficult task and it took nearly a full year to complete the diaphragm wall. If the scheme depicted in Fig. 21 is adopted, diaphragm wall could be totally omitted. The authors did not have the courage of omitting the diaphragm wall entirely when they proposed the scheme but recommended that diaphragm wall penetrate into the Chingmei Gravels by only a meter or so. The proposal was not followed up because of the lack of precedents and the policy that any methods adopted must be substantiated by successful precedents.

Treatment of slurry and dumping of watery spoil are problems to be solved. Therefore, this scheme is most suitable at sites with sufficient space for treatment plant to be set up. Sites close to rivers will be ideal because it is easier to pump water from, or discharge water into, the rivers.

6. CONCLUSIONS

The above discussions lead to the following conclusions:

a) Performance of retaining systems can be represented by wall deflections which in turn can be expressed by a new parameter, α, introduced herein.

b) The values of α depend on ground conditions, rigidity of retaining systems, method of construction, preloading of struts, ground improvement and workmanship, etc., and shall be established based on local experience.

c) The α values for the T2 and K1 Zones of the Taipei Basin and Singapore marine clay were proposed for different retaining systems.

d) In the cases analyzed, the use of jet-grouted slab, cross panels and cross beams reduces wall deflections to a half and the use of buttresses reduced wall deflections by a quarter.

e) Groundwater plays an important role in underground constructions and shall be properly taken care of in both the design and the construction. In case dewatering is likely to lead to settlement problems, wet excavation is a viable alternative.

ACKNOWLEMENTS

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