PROTECTING ADJACENT BUILDINGS DURING UNDERGROUND CONSTRUCTION

by

Z. C. Moh, S. K. Kong and R. N. Hwang

Reprinted from Proceedings of Symposium on Innovative Solutions in Structural and Geotechnical Engineering
In Honor of Professor Seng-Lip Lee
May 14~15, 1999, Bangkok, Thailand

PROTECTING ADJACENT BUILDINGS DURING UNDERGROUND CONSTRUCTION

Za-Chieh Moh
President, Moh and Associates Group, Taipei, Taiwan

S. K. Kong
General Manager, Moh and Associates (S) Pte. Ltd., Singapore

Richard N. Hwang
Manager, Rapid Transit Project, Moh and Associates, Inc., Taipei, Taiwan

ABSTRACT: This paper discusses factors affecting lateral deflections of retaining walls in attempting to find effective means to reduce lateral deflections of walls during excavations, hence, ground movements and damaging potential to adjacent buildings. Based on observations, increase in rigidity of retaining system is effective in reducing wall deflections. Ground improvement is a viable alternative at sites where control of wall deflections by increasing wall thickness is not practically feasible.

1. INTRODUCTION

Damages to buildings adjacent to underground constructions are by no means rare. Protest from suffering neighbors frequently caused delays of projects and also resulted in large financial losses. Legal process may drag on for years after the completion of the project. To prevent such unhappy events from repeating requires considerable efforts from both designers and contractors.

Failures of retaining systems during excavations of course will impose risk or damaging effects on adjacent buildings. Nowadays, with advanced analytical tools and sufficient experience, structural designs of retaining systems are in general adequate and failures of excavation due to inadequacy of structural members are rather rare. Failures, if any, are most likely to be associated with either soil or groundwater problems. Based on observations, the top four major causes of failures are (a) basal heave, (b) piping at base, (c) blow-in and (d) leakage on retaining walls together with rupture of nearby water mains.

This paper outlines general principles in building protection with emphasis on limiting wall deflections and also discusses a few case histories in which the effectiveness of protective measures are studied.

2. BUILDING PROTECTION MEASURES

Failures due to soil and groundwater problems are likely to be far-reaching and are thus more serious than failures due to other causes. Even without failure, ground movements could still become intolerable to adjacent buildings, particularly the ones with poor structures/foundations. Although ground movements are inevitable whenever an excavation is made, their magnitudes can be much reduced if adequate preventive measures are taken. Such preventive measures include (Moh and Hwang, 1994):

(a) those reduce ground movements at sources,
(b) those limit ground movements from propagating,
(c) those strengthen structures, and

(d) those correct building movements.

Experience has shown that the most effective measures are those reducing ground movement at source. Other measures are costly, ineffective and/or difficult to be carried out. This is particularly true for measures aiming at correcting building movements. The old saying that "An ounce of prevention is worth of a pound of cure" holds true.

The major sources of ground movement include deflections of retaining walls and consolidation of soft clays. Because of the page limit, this paper will address only to wall deflections and measures to reduce wall deflections.

3. FACTORS AFFECTING WALL DEFLECTIONS

The factors which affect the magnitudes of wall deflections are, in general:

(a) depth of excavation
(b) ground conditions
(c) method of construction
(d) rigidity of retaining system
(e) ground improvement
(f) workmanship

Of these factors, the last four are fully in the hands of the designer and/or contractor. It is doubtless that a stiffer retaining system with extensive ground improvement will reduce wall deflections to any amount specified. However, cost is always a serious concern and frequently governs decisions. It is therefore desirable to establish a methodology to quantify the sensitivity of wall deflections to these factors. With modern technology, it is possible to perform parametric studies on individual factors to figure out the influence of each of these factors on wall deflections using numerical analyses. However, it is fairly risky to adopt the results of numerical analyses unless they are backed by field measurements. Theoretical development and observations shall go in parallel and shall go together.

4. CHARACTERISTIC CURVE OF WALL DEFLECTION

Of the factors listed above, depth of excavation is the most clearly definable. The following relationship was proposed by Moh and Hwang (1999a) :

$$\delta_{\max} = k_\theta H^\theta \tag{1}$$

in which

$\delta_{\max}$ = maximum wall deflection
H = depth of excavation
θ = empirical exponent, dimensionless
k_θ = empirical factor, with a dimension of ($\mathrm{m}^{1-\theta}$)

Since depth has been explicitly considered in the above equation, empirical coefficient k_θ is primarily a function of the rest of factors listed above. It can be deemed as an index of the performance of a wall.

The exponent θ is also a function of these factors. It may even be, to a certain extent, a function of depth of excavation as well. Equation (1) can be written in a logarithm form as follows:

$$\log(\delta_{\max}) = \theta \log(H) + \log(k_\theta) \tag{2}$$

It can then be noted that θ is the slope of deflection-depth relationship in a logarithm plot. Figure 1 shows such a plot for readings obtained by individual inclinometers at excavations carried out in the Taipei Basin (Moh and Hwang, 1999a; 1999b). The data for excavations shallower than 5m are widely scattering without a clear trend. However, the slopes of curves for excavation depths of 5m to 30m fall in the narrow range of $1<\theta<2$. The cases shown therein are identified by geological zone (T2, K1 and TK2), method of excavation (B: bottom-up; T: top-down; S: semi top-down) and wall thickness (070: 700mm; 080: 800mm, etc) as denoted. A similar plot is given in Fig. 2 for Singapore soils and the same range was observed for the θ values.

The curves shown in Figs. 1 and 2 can be viewed as "characteristic curves" for wall deflections. It is believed that with efforts it will be possible to isolate factors affecting the shapes of these curves and to evaluate the sensitivity of the θ values to these factors. For the time being, only the cases corresponding to the upper and the lower bounds of the θ values are studied. The following two equations, correspond to the upper and the lower bounds of θ, respectively (Moh and Hwang, 1999a):

$$\frac{\delta_{\max}}{H} = k_2 H = \alpha H \times 10^{-6} \tag{3}$$

and

$$\delta_{\max} = k_1 H = \beta H \tag{4}$$

The factor 10^{-6} was introduced in Eq. (3) so α, with a dimension of (m^{-1}), would be whole numbers. The same treatment was not applied to the factor β, which is dimensionless and is a meaningful factor in the sense that it is the ratio of wall deflection to depth of excavation.

4.1 TAIPEI EXPERIENCE

The geology and ground conditions in the Taipei Basin are well documented (Woo and Moh, 1990; Chin, Crooks and Moh, 1994) and will not be repeated herein. Wall deflections observed during TRTS (Taipei Rapid Transit Systems) constructions, normalized to depth of excavation, are compared to those previously obtained (Woo and Moh, 1990) in the T2 Zone of the Taipei Basin in Fig. 3 (Moh and Hwang, 1999a; 1999b). The figure is divided into Zones I, II, III... by lines corresponding to α = 62.5, 125, 250, 500...etc. Wall defections in each zone are, on an average, twice as much as those in the preceding zone. For example, wall deflections in Zone II are in general twice as much as those in Zone I, wall deflections in Zone III are twice as much as those in Zone II, so on and so forth. It is readily apparent that the TRTS works outperform the previous excavations. A similar plot is given in Fig. 4 for information obtained in the K1 Zone and the same conclusion can be reached. The probable reasons for the better performance for TRTS works include higher rigidity of retaining systems, preloading of struts, and better workmanship. It should be noted that the data points shown in Figs. 3 and 4 are the ending points in the characteristic curves and correspond to the final stage of excavations.

The effects of ground conditions, method of construction, rigidity of retaining systems, preloading, ground improvement, etc. can be studied by evaluating the changes in the α and β values. Based on the data available in the Taipei Basin, the α and β values for different combinations of conditions are given in Table 1. With considerable engineering judgment, scaling factors were derived for varies parameters and are given in Table 2 for information. They can be used to obtain the α and β values for combinations of conditions for which data are unavailable (Moh and Hwang, 1999a; 1999b).

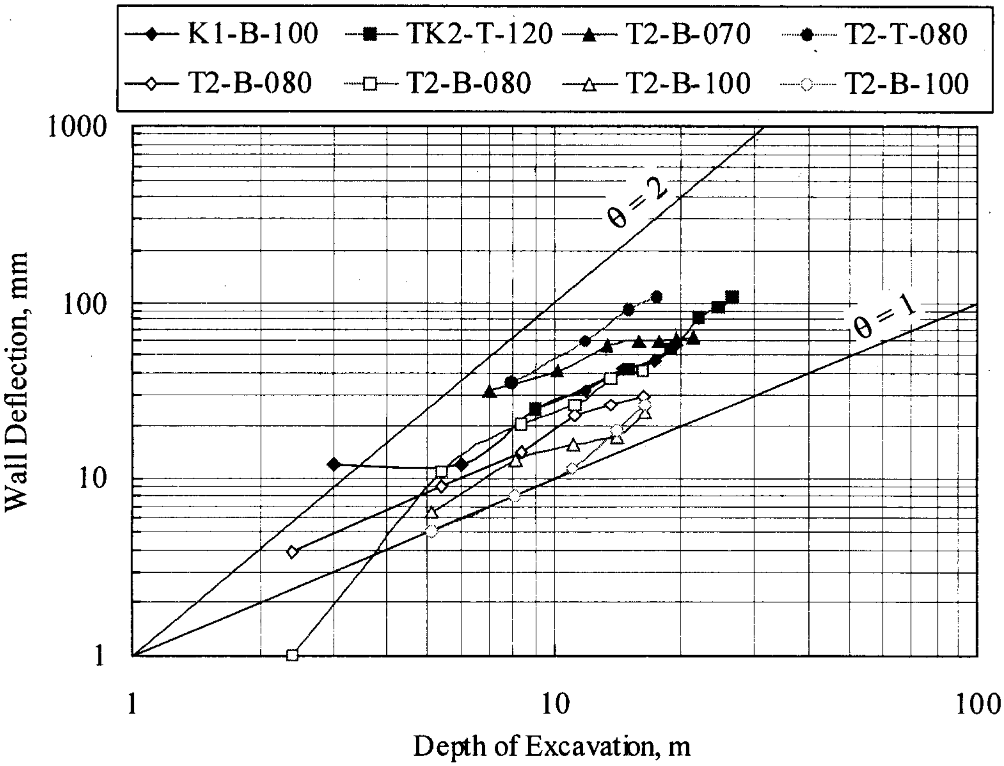


Fig. 1 Wall deflections in Taipei Basin

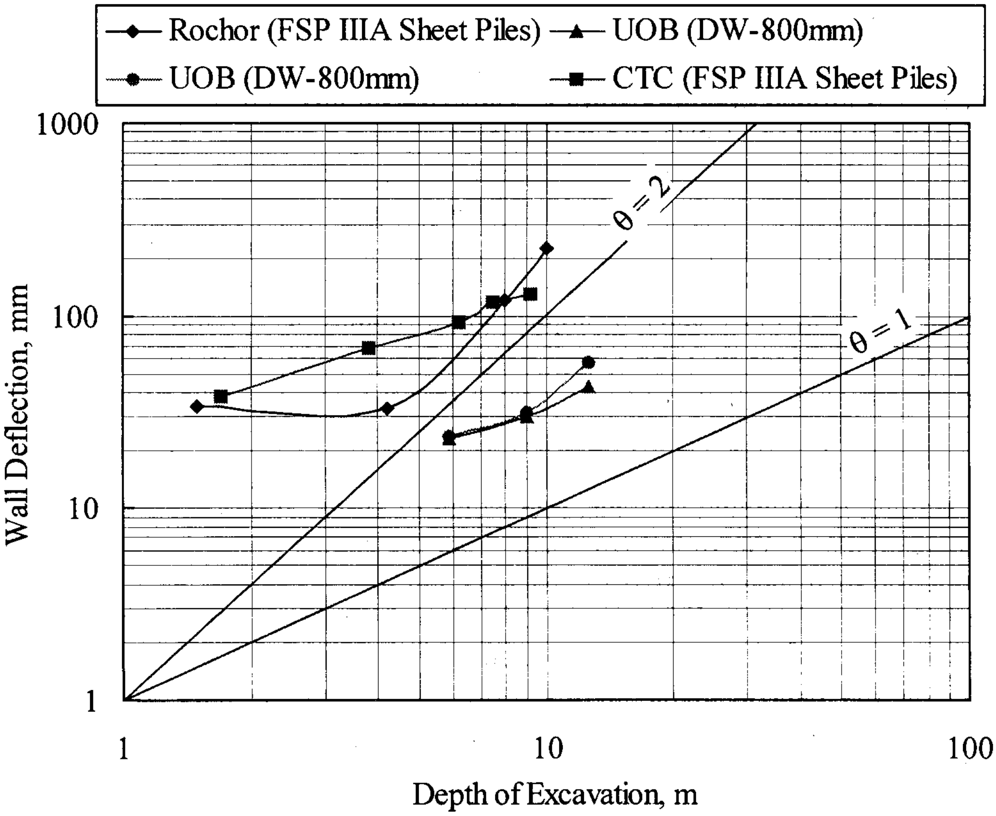


Fig. 2 Wall deflections in Singapore marine clay

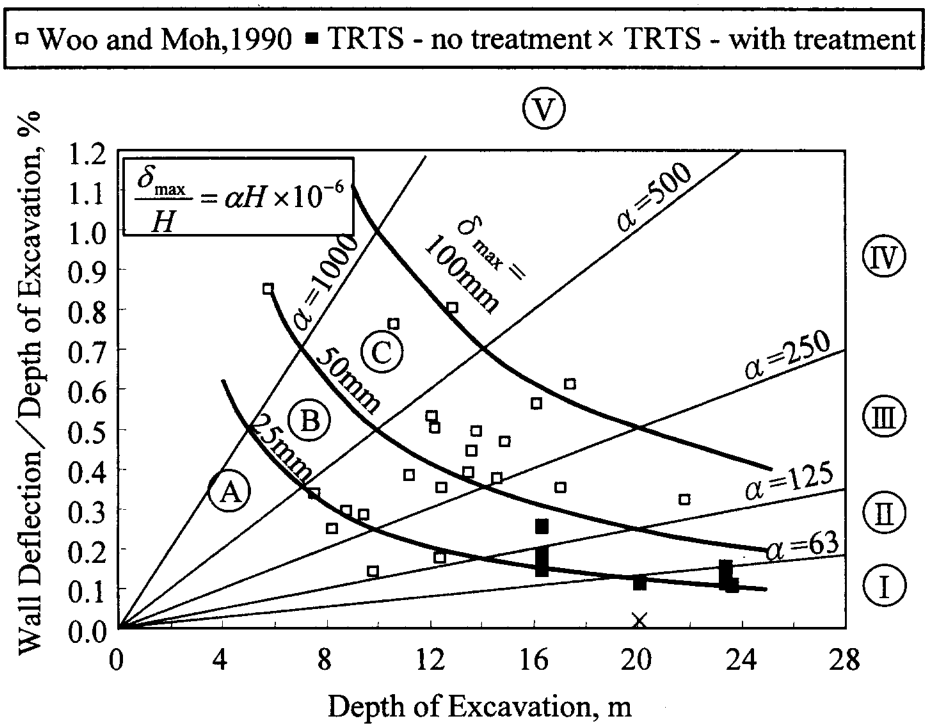


Fig. 3 Wall deflections in the T2 Zone of the Taipei Basin

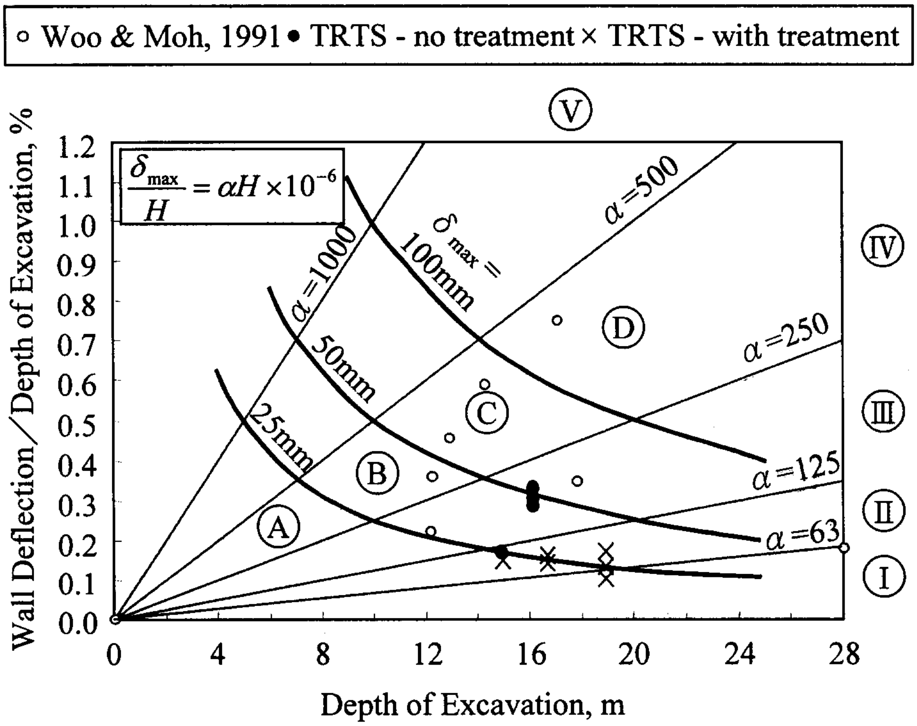


Fig. 4 Wall deflections in the K1 Zone of the Taipei Basin

Table 1 Alpha and beta values for deep excavations in the Taipei Basin

	α (Moh and Hwang, 1999b)			β, % (Moh and Hwang, 1999a)		
Set	T2 Zone	TK2 Zone	K1 Zone	T2 Zone	TK2 Zone	K1 Zone
B-120 *						
B-100	94(3)		160(7)	0.15(3)		0.30 (7)
B-090						
B-080	106(3)			0.18(3)		
B-070		140(2)			0.30(2)	
S-120	55(4)			0.13(4)		
S-100			189(3)			0.31(3)
S-090						
S-080						
S-070						
T-120		163(2)			0.43(2)	
T-100						
T-090			265(3)			0.52(3)
T-080	352(2)			0.61(2)		
T-070						

Notes: S: seimi-top down construction method
B: bottom-up construction method
T: top-down construction method
* The numbers after "-", e.g., -120, -100, etc denote thickness of diaphragm walls in centimeters
The numbers in parentheses are number of cases in the category

Table 2 Multipliers for extrapolating α and β values from observed values

Parameter	Representative Condition	Multiplier on α (Moh and Hwang, 1999b)	Multiplier on β (Moh and Hwang, 1999a)
Ground Conditions	T2	1	1
	K1	2	2
	TK2		1.5
	Singapore marine clay	5	
Retaining Structures	1.2m diaphragm wall	1	1
	1.0m diaphragm wall	1.5	1.1
	0.8m diaphragm wall	2	1.3
	0.7m diaphragm wall		1.5
	sheet pile	4	
Strut	preloaded	1	1
	without preloading	2	1.5
Method of Construction	bottom up	1	1
	top down	2	2
Ground Treatment	treated	1	
	untreated	2	

4.2 SINGAPORE EXPERIENCE

The geology and ground condition in Singapore have been well documented also (Choa, et. al, 1996; Hulme, Potter and Shirlaw, 1989). Although Singapore marine clay is well known for its weak strength, sheet piles were commonly used as retaining walls for excavations to 9m or so. Figure 5 shows a plot of normalized wall deflections versus depth of excavations for the 6 cases listed in Table 3 (Moh and Hwang, 1999a). It can be noted that most of data points scatter widely in Zones VI to VIII. The α values vary from 1500 to 7000 with an average of 3872 which fits in Zone VII. The wide scatter of data may partly attribute to the different types of sheet piles used. It was hypothesized that these cases were intentionally quoted in the literatures for their excessive deflections, therefore, a reduced α value of 2500 was recommended in Moh and Hwang (1999a) as a representative value for sheet-pile walls in marine clay. Attempts are being made to collect more data with different types of walls, including diaphragm walls with different thickness, so the influence of rigidity of the retaining systems can be quantified.

5. EFFECTS OF GROUND IMPROVEMENT

The plots shown in Figs. 3 to 5 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 generally 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 located 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 located within 10m or so;
(d) deflections in the range of 100mm and 200mm, i.e., Zone D, even structures supported on piled foundations or mat foundations located 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 structures are more tolerant than others in the same category and conditions of structures are difficult to assess.

As can be noted from Table 2 that wall deflections can be reduced by increasing wall thickness and/or applying preloads to struts. However, there is a limit on how far this can go. Take the case of TK2-T-120 (semi top-down excavations retained by diaphragm wall of 1.2m in thickness) as an example, with an α value of 163 and a β value of 0.43, wall deflection would exceed 50mm if excavation exceed 17.5m and 11.6m, respectively, and poor building within 10m are likely to be damaged. In K1 Zone, the situation is expected to be more critical. To further increase wall thickness may not be justifiable because:

(a) the cost for walls exceeding 1.2m in thickness increase drastically with thickness, and
(b) space may not be available, particularly for underground stations beneath narrow roads.

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 improvement 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.

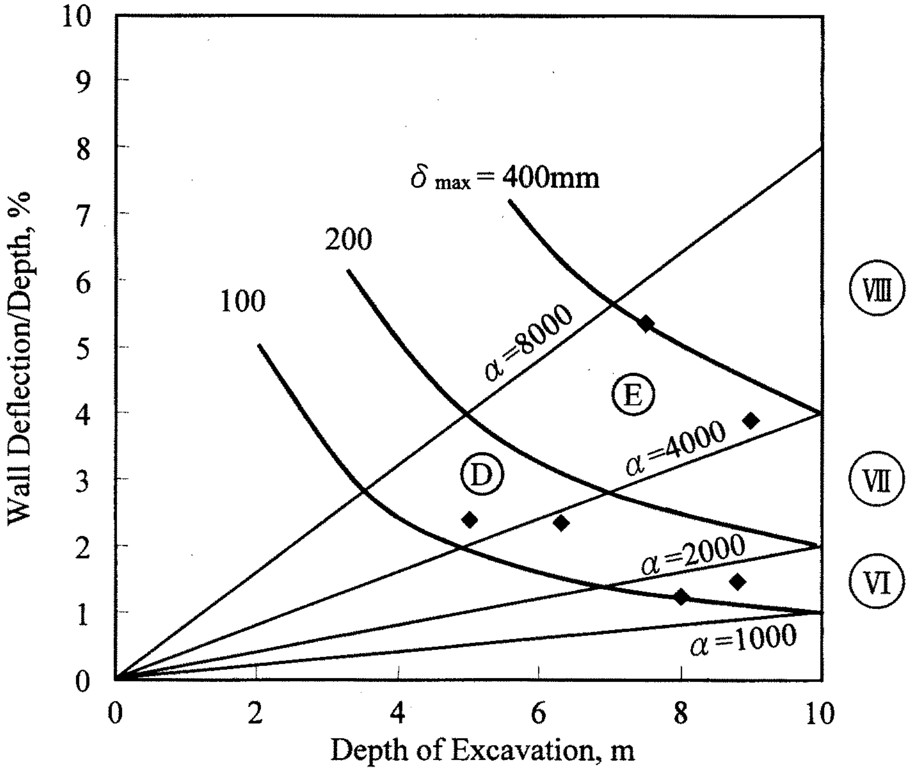


Fig. 5 Deflections of sheet pile walls in Singapore marine clay

Table 3 Sheet-piled excavations in Singapore marine clay

Sources	Excavation depth, H m	Max Deflection δ_{max}, mm	δ_{max}/H %	No. of Strut Levels	α value
Lee (1986)	9.23	130	1.41	4	1526
Chin (1986)	7.50	400	5.33	3	7111
Yang (1985), Case I	9.00	350	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

Figure 6 shows the results obtained in 6 cases in which different treatment schemes, including buttress, cross beam, cross panel, and grouted slab, were adopted (Moh and Hwang, 1999b). The data are insufficient in quantity for drawing conclusions. Based on the comparison given in Figs. 3 and 4, it is simply "felt" that a reduction factor of 2 can be achieved if ground treatment is applied.

6. DISCUSSIONS

Lateral deflections of walls are routinely monitored by using inclinometers. Inclinometers 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. In quite a few cases, inclinometers were not anchored in a stable stratum and toe movements were large. This is particularly true for inclinometers which are cast in diaphragm walls. In such cases, usually, inclinometers are only installed to the toe levels of the walls. The toe of an inclinometer is normally assumed to be fixed and the movements at all other depths are computed in relation to the toe. Although sometimes specifications do require the movement at the top of casing be measured for calibration. In reality, this is difficult to be done due to site constraints, particularly for underground stations to be constructed underneath busy streets. Figure 7 shows the readings obtained for two inclinometers, installed in diaphragm walls, one to the toe level and one to the bearing stratum. Because of the different excavation depths in the two cases, 16.2m and 11.1m, respectively, the diaphragm walls were different in thickness, i.e., 1,000mm and 800mm, and in length, i.e., 53m and 26m. When excavation reached a depth of 11.1m, Inclinometer SID4 which was anchored in the bearing stratum showed a maximum deflection of 45mm while Inclinometer SID6 which was installed only to the diaphragm wall toe showed a maximum deflection of only 20mm. At a depth of 26m, which corresponds to the toe level of Inclinometer SID6, a movement of 30mm was observed by Inclinometer SID4. It is thus suspected that the toe of SID6 had moved by a similar amount, or even larger. After correcting the readings of Inclinometer SID6 for this anticipated toe movement, the two sets of readings were very close. This illustrates the fact that inclinometer readings shall be interpreted with great care. As a general rule, it is suggested that inclinometers shall always be adequately anchored in a bearing stratum to eliminate doubts and to ensure accurate measurement of lateral wall deflection.

The various factors affecting wall deflections are discussed herein and a mathematical model is proposed for evaluating their influences on wall deflections. The procedures proposed are applicable to strutted excavation with competent bearing strata at depths greater than twice of the depths of excavation. Their applicability to anchored walls or walls braced by inclined struts is yet to be investigated. These procedures will not work if bearing strata are at shallow depths below the bottom of excavation.

7. REFERENCES

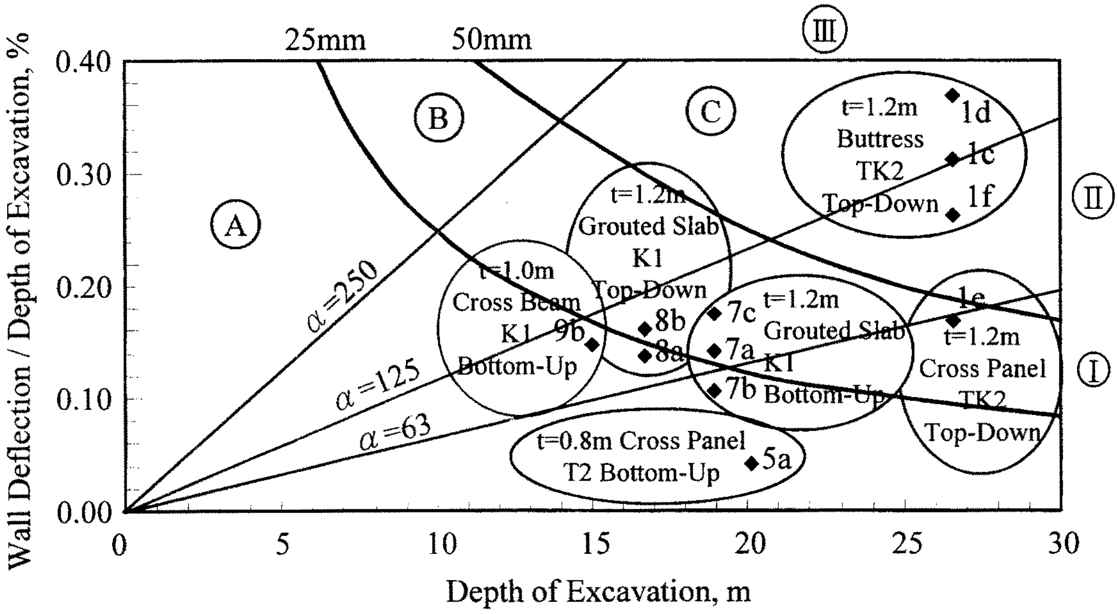
Chin, C. T., Crooks, J. H. A., Moh, Z. C., 1994, Geotechnical properties of the cohesive Sungshan deposits, *Geotechnical Engineering Journal,* December, Taipei, pp.77~103

Chin, Yong Kok, 1986, The MOE Building at Scotts Road, *2nd PWD Technical Seminar*, Singapore

Choa, V., Chu, J., Bawajee, R., Win, B.M. and Arulrajah, A., 1996, The strength and consolidation behaviour of Singapore marine clay at Changi, *Proc., 12th SEAGC,* May 6~10, Kuala Lumpur, Malaysia, Vol. 1, pp.81~85

Ho, C. E. and Wallace, J. C., 1993, Design and performance of diaphragm walls for a deep basement in Singapore, *Proc. 11th SEAGC,* May 4~8, Singapore, Vol. 1, pp.715~720

Hulme, T. W., Potter, L.A.C. and Shirlaw, J. N., 1989, Singapore Mass Rapid Transit System: Construction, *Proc., Instn Civil Engineering,* Part 1, August, London



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. 6 Wall deflections for different schemes of treatment

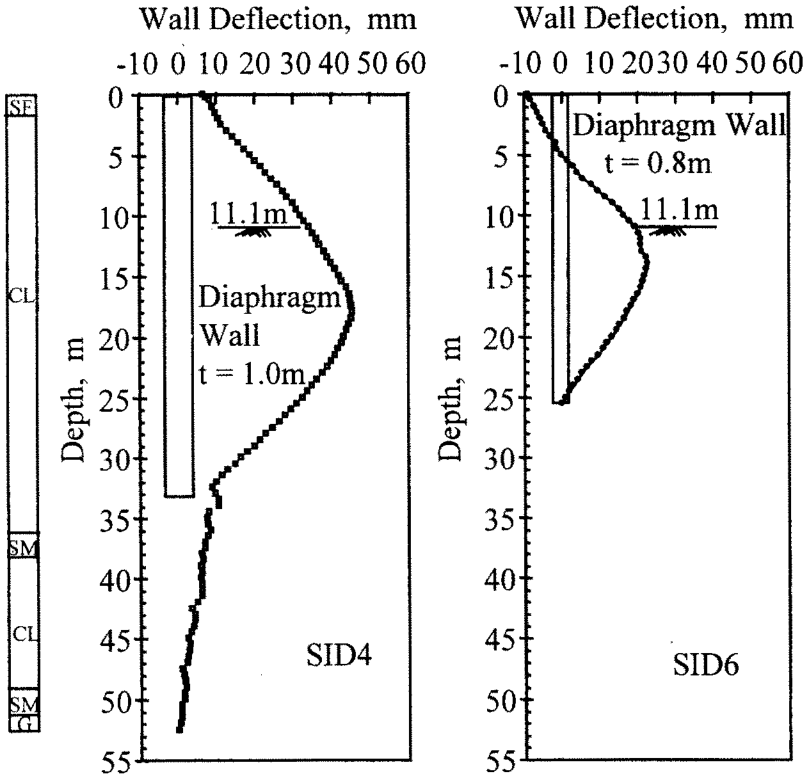


Fig. 7 Effects of toe stability on inclinometer readings

Lee, S. L., Yong, Y. K., Karunaratne, G. P., Chua, C. H., 1986, Field instrumentation for a strutted deep excavation in soft clay, *Proc., 4th Int. Geotechnical Seminar on Field Instrumentation and In-situ Measurement*, Singapore

Moh, Z. C. and Hwang, R. N., 1994, Building protection for underground works, *Keynote Speech, Regional Conference in Geotechnical Engineering*, (GEOTROPIKA 94), August 22-24, Malacca, Malaysia

Moh, Z. C. and Hwang, R. N., 1999a, Geotechnical Problems related to design and construction of the Taipei MRT, *Proc., Commemoration of Dr. Sang-Kyu Kim's Retirement Symposium*, April 17, Seoul, Korea

Moh, Z. C. and Hwang, R. N., 1999b, Geotechnical issues in underground constructions, *Proc., International Conference on Rapid Transits (ICRT)*, March 11~13, Singapore

Woo, S. M. and Moh, Z. C., 1990, Geotechnical Characteristics of Soils in the Taipei Basin, *Proc., 10th Southeast Asian Geotechnical Conf.*, Taipei, Taiwan, Vol. 2, pp.51~65

MAA, 1987, Proposed URA Rochor Complex Project/Basement Excavation and Substructure Construction - Final & Summary Report on Instrumentation Works

Yang, K. S., 1985, Three cases of deep excavations in soft clay in Singapore, *2nd PWD Technical Seminar*, Singapore