■Ngày nhận bài: 17/7/2021 ■Ngày sửa bài: 14/8/2021 ■Ngày chấp nhận đăng: 27/8/2021

Back analysis on deep excavations

Phân tích ngược các hố đào sâu

> TRAN HOANG TIN
Thu Dau Mot University
E-mail: tinth@tdmu.edu.vn

ABSTRACT:

Back analysis is necessary for further design and construction of deep basement projects in urban areas. This paper is to present two case studies that include 3 basements excavated up-to15m below current street level. Both Hardening Soil model (HSM) and Mohr Coulomb model (MCM) were applied for the analyses of geotechnical aspects. Inclinometers were installed to record diaphragm wall's deflections before construction. The analysis points out that the horizontal displacement results of the diaphragm wall as indicated by the HSM are closer to the observed data than those given by the calculation using the MCM in both cases. Therefore, using the HSM in analysis of deep excavations is more reliable than the MCM. Furthermore, a correlation between soil stiffness and SPT is strongly recommended for further designs.

Keywords: back analysis; deflection; deep excavations; HSM,;MCM

TÓM TẮT:

Phân tích ngược là phương pháp cần thiết cho thiết kế và thi công các dự án tầng hầm sâu ở các đô thị. Bài báo trình bày hai trường hợp nghiên cứu ở 3 tầng hầm đào sâu đến 15m dưới nền đường. Cả hai mô hình đất cứng (HSM) và mô hình Mohr Coulomb (MCM) đã được áp dụng để phân tích các khía cạnh địa kỹ thuật. Các thiết bị máy đo độ nghiêng đã được lắp trước khi thi công để ghi nhận độ lệch ngang của tường vây. Nghiên cứu chỉ ra rằng các chuyển vị ngang của tường vây cho bởi HSM gần dữ liệu quan trắc hơn các chuyển vị cho bởi MCM. Do đó, sử dụng HSM trong phân tích các hố đào sâu thì tin cậy hơn MCM. Hơn nữa, sự tương quan giữa độ cứng của đất và giá trị SPT được khuyến cáo mạnh mẽ cho các thiết kế tiếp theo.

Từ khóa: phân tích ngược; độ lệch ngang; hố đào sâu; mô hình đất cứng; mô hình Mohr Coulomb.

1. INTRODUCTION

High-rise projects in Ho Chi Minh city are usually adjacent to existing low-rise buildings. In specific, both projects analyzed in the case studies are adjoined by old structures with shallow foundations. Moreover, geotechnical profiles show weak clay and saturated sand below the surface. Previous projects could be negatively impacted by insufficient knowledge and lack of experience, which could result in collapse or partial failure during excavation. For example, some building projects failed during excavation. These include Saigon Residence (02 basements, 2007), Pacific Tower (05 basements, 2007), Saigon M&C Tower (05 basements, 2009), etc., thus demonstrating the need for investment in design and construction.

Selecting an appropriate soil model and using soil parameters correctly are vital Most soil data can be taken by conducting field and laboratory tests, while soil stiffness parameters are difficult to measure in that way. Therefore, estimating soil stiffness by using empirical formulas is the norm as can be seen in Table 1.

Table 1. Correlation equations for estimating of soils stiffness

	Equation	Reference	Regions of applicability
Eu,i=	(600 to 1200)Cu	Clough & Mana 1976	San Francisco clay (obtained
			from back analysis of excavat
Eu,i=	420Cu	Dames & Moore 1975	AGS CH Clay
Eu,i=	600Cu	Ladd & Edgers 1972	Maine organic CH-OH clay
Eu,i=	670Cu	Ladd & Edgers 1972	Bankok CH clay
Eu,i=	820Cu	Ladd & Edgers 1972	Boston CL clay
Eu,i=	(250 to 500)Cu	Bjerrum 1964	Normally consolidated
			Norwegian clays
E =	(200 to 400)Cu	C.G. Chinnaswamy 2011	none-clayey soils and clays
E =	(2500 to 3000)N	C.G. Chinnaswamy 2011	none-clayey soils and clays
E =	2000N	JamesWC Sze &	obtained from back analysis
		Jackie CY Yau 2011	of excavation in HCM city
E=	500(N+15)	Bowles 1998	normally consolidated sand
E=	250(N+15)	Bowles 1998	saturated sand
E=	750(N+24)	Bowles 1998	overconsolidated sand
E=	1200(N+6)	Bowles 1998	Gravel sand/Gravel
E=	1200(N+15);N<15	Bowles 1998	Gravel sand/Gravel
E=	600(N+6); N=<15	Bowles 1998	Gravel sand/Gravel
E=	300(N+6)	Bowles 1998	silty sand
E=	7500+800N	Papadopoulos (1992)	

$E_{u,i}$ is initial tangent undrained modulus;c_u is undrained shear strength; E is young's modulus of soil [kPa]; N is SPT value;

Another experience conducted by Vermeer and Meier (1998) asserted $E_{50} < E_{oed}$ for stiff over-consolidated clays, $E_{50} > E_{oed}$ for soft clays, and $E_{50} \approx E_{oed}$ for sands, especially for analysis with the FE-code PLAXIS should be considered.

The Mohr Coulomb Model (MCM) only requires Young's modulus of soil (E), while the Hardening Soil Model (HSM) needs three different stiffness parameters E_{50}^{ref}: Reference secant stiffness in standard drained tri-axial test [kN/m^2]; E_{oed}^{ref}: Reference tangent stiffness for primary oedometer loading [kN/m^2]; E_{ur}^{ref}: Reference un/reloading stiffness [kN/m^2] and in Figure 1.

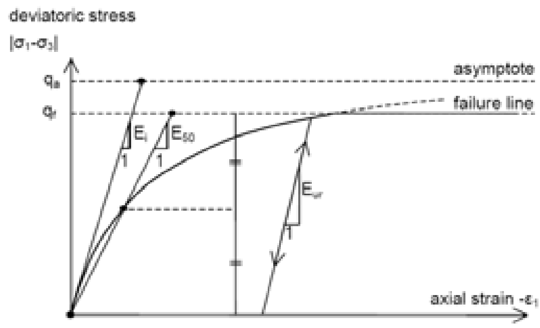

Figure 1. Hyperbolic stress-strain in primary loading for a standard drained tri-axial test

Figure 2. Location of case studies on map

2. A CASE STUDY OF REE TOWER

2.1. Initial condition

The building consists of 3 basements and a 24-storey super-structure. The site area is approximately 2000sqm and located in District 4, HCMC shown in Figure 2. There is an old 05-storey house on the right hand side and another 1-storey house adjoining on the left hand side shown in Figure 3.

The 800mm thick diaphragm wall was constructed at a depth of 25 to 30m. The diaphragm wall's deflection was observed by the installation of 10 Inclinometer points as shown in Figure 3.

The general excavation is 12.2m below ground level, while the excavated depth of the lift core is 14.8m. Moreover, section A-A in Figure 4 crossing the lift core is simulated and compared to the observed data (ICL2).

Parameters of soil layers are illustrated from ground surface to a depth of 50m as in Table 2. The groundwater table was measured at 6.5m in depth.

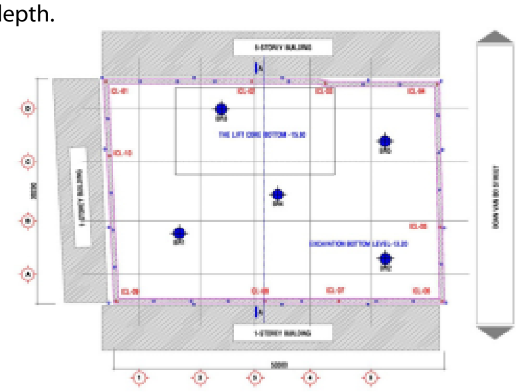

Figure 3. Site condition of REETOWER

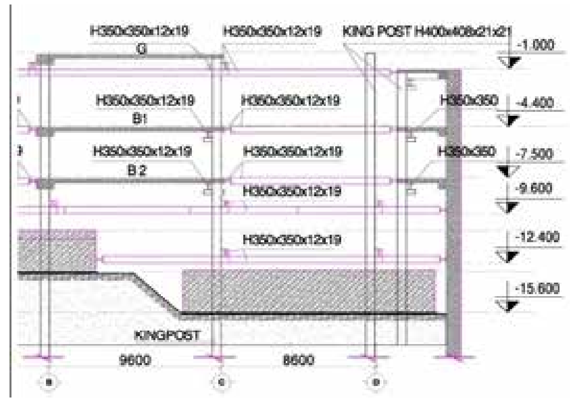

Figure 4. Section A-A

Table 2. Soils properties

Depth	Name	SPT	g_unsat	g_sat	c_ref	phi
[m]		[Blow]	[kN/m^3]	[kN/m^3]	[kN/m^2]	[°]
0-10	<Layer 1> Organic CLay	1	15	15.4	16.7	13
10-12	<Layer 1B>sandy CLay	5	19.4	19.8	18.1	16
12-15	<Layer 2A>clayey sand/sand	9	20.1	20.9	8.0	27
15-20	<Layer 2B> clayeySand/Sand	13	20.1	20.9	8.0	27
20-24	<Layer 2C> clayey Sand/Sand	8	20.1	20.9	8.0	27
24-35	<Layer 2D>clayey Sand/Sand	16	20.1	20.9	8.0	27
35-41	<Layer 3>sandy CLAY	22	19.3	19.9	14.3	26.5
41-50	<Layer 4>CLay	45	19.6	20.4	49	22

2.2. Construction Sequence

Semi-top-down method was applied for basement construction. The following construction sequence from the current street level -1.0m to the bottom level.

- Phase 1: Install the diaphragm wall
- Phase 2: Install bored piles and place in kingposts
- Phase 3: Excavate the 1st layer down to level -3.1m
- Phase 4: Cast 200mm thick G floor slab at level -1.0m
- Phase 5: Excavate the 2nd layer down to level -4.7m
- Phase 6: Cast 200mm thick B1 floor slab at level -4.4m
- Phase 7: Excavate the 3rd layer down to level -7.7m, dewater down to level -8.7m
- Phase 8: Cast 200mm thick of the B2 floor slab at level -7.5m
- Phase 9: Excavate the 4th layer down to level -11.2m, dewater down to level -12.2m
- Phase 10: Install the 1st additional struts H350x350x12x19 at level -9.6m
- Phase 11: Excavate the 5th layer down to level -13.2m, dewater to level -14.2m
- Phase 12: Cast 2500mm thick raft caps and a part of B3 floor at level -10.6m.
- Phase 13: Install the 2nd additional struts 2H350x350x12x19 at level -12.4m
- Phase 14: Excavate the 6th layer down to the lift core at level -15.8m, dewater down to level -16.8m
- Phase 15: Remove struts and cast unfinished floor slabs.

2.3. Back analyses

In the original design, the problem was simulated with the HSM. Soil stiffness parameters were estimated based on the empirical equations. In the back analysis, the soil stiffness of layer 2D is re-estimated at a level 5 times greater than the initial estimated in terms of both HSM and MCM, as indicated in the following tables:

Table 4. Soil stiffness parameters using for HSM in the original design

Depth	Name	Type	E50ref	Eoedref	Eurref
[m]			[kN/m^2]	[kN/m^2]	[kN/m^2]
0-10	<Layer 1> Organic CLay	UnDr	12000	12000	36000
10-12	<Layer 1B>sandy CLay	UnDr	10000	10921	30000
12-15	<Layer 2A>clayey sand/sand	Dr	9000	9000	27000
15-20	<Layer 2B> clayeySand/Sand	Dr	25000	25000	75000
20-24	<Layer 2C> clayey Sand/Sanc	Dr	24000	24000	72000
24-35	<Layer 2D>clayey Sand/Sand	Dr	30000	30000	90000
35-41	<Layer 3>sandy CLAY	UnDr	34500	34500	103500
41-50	<Layer 4>CLay	UnDr	51750	51750	155300

Table 5. Soil stiffness parameters using for MCM

Depth	Name	Type	E_ref
[m]			[kN/m^2]
0-10	<Layer 1> Organic CLay	UnDr	12000
10-12	<Layer 1B>sandy CLay	UnDr	10000
12-15	<Layer 2A>clayey sand/sand	Dr	9000
15-20	<Layer 2B> clayeySand/Sand	Dr	25000
20-24	<Layer 2C> clayey Sand/Sand	Dr	24000
24-35	<Layer 2D>clayey Sand/Sand	Dr	150000
35-41	<Layer 3>sandy CLAY	UnDr	34500
41-50	<Layer 4>CLay	UnDr	51750

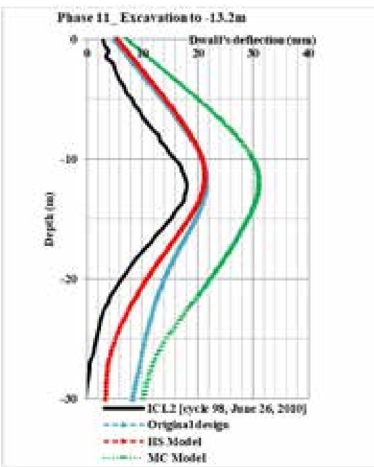

Figure 5. Diaphragm wall deflection profiles in terms of HSM, MCM, and ICL2

The results of wall deflection in Figure 5 show the horizontal displacement of the wall in term of MCM is larger than that given from the HSM at the excavated depth of 12m. Furthermore, the wall deflection indicated by HSM is closer to the observed data than that given by the MCM. Therefore, the HSM is more suitable than the MCM for the purposes of analyzing deep excavations.

3. A CASE STUDY OF MB SUNNY TOWER

3.1. Initial condition

MB Sunny Tower is a development consisting of 1428sqm. In terms of dimensions, the project is 65m long and 22m wide. It is located in District 1, HCM city. There is a 03-storey house on the rear side, a 1-storey house on the right hand side, and a 02-storey house adjoining on the left side. All details are shown in Figure 6.

The building consists of a 22-storey super structure with 03 basements. The maximum depth of basement excavation is 16m at the lift core zone, while the general excavation depth is only 14.5m.

The diaphragm wall consists of 600mm in thickness and 27m in depth. It was constructed in a segmented manner. It was built with concrete grade B25, rebar strength type of SD390. The main rebar layout of the inner and outer walls was equipped with φ25@200+φ28@200 and 2φ25@200, respectively.

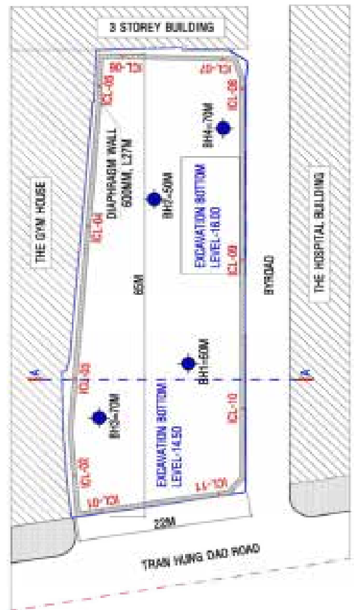

Figure 6. Site condition of MB Sunny Tower

3.2. Ground conditions

Soil data was taken from 4 boring holes shown in Figure 6. Groundwater level was observed at 3.0m below the surface. The soil parameters are presented in Table 6.

Table 6. Soils properties

Depth	Name	SPT	g_unsat	g_sat	c_ref	phi
[m]		[blows]	[kN/m^3]	[kN/m^3]	kN/m^2	[°]
0-2.7	<Layer 1> Soft organic clay	4	16	16.2	15	4
2.7-9	<Layer 2>Silty/Clayey Sand	10	19.9	20.3	10.1	20.3
9-17.5	<Layer 3>clayey Sand	14	20.1	20.7	3.7	29.7
17.5-25	<Layer 4> clayey Sand/Sand	13	20.1	20.6	9	25
25-35	<Layer 4B> clayey Sand/Sand	18	20.1	20.6	9	25
35-43	<Layer 5> clayey Sand/Sand	15	20.4	20.8	3.5	30
43-47	<Layer 5B>silty/clayey Sand	11	19.8	20.1	9.9	24
47-51	<Layer 6> Clay	29	20	20.3	46	16

3.3. Construction sequence

The bottom-up method was applied in this basement construction. Three propping levels were designed. The first propping level using single strut type H400x400x13x21 was installed at 2.0m below surface level. The second level at a depth of 6.3m was propped by both single strut type H400x400x13x21 and double strut type 2H400x400x13x21. The double strut type 2H400x400x13x21 was propped at 10.5m in depth at its lowest level. Waling beams at three levels were installed with H400x400x13x21.

During basement construction, ground water level was designed to dewater 1.0m below excavation level. Two snap shots taken under construction are shown in Figure 7.

Figure 7. (a) under excavation of B3; (b) The 3rd level of walling beam & struts status

Diaphragm wall deflection was monitored based on installation of 11 inclinometer points. The horizontal displacement of the diaphragm wall was regularly reported twice a week during construction. The bottom-up construction sequence was started from the GL +0.0 as follows:

- Phase 1: Install the Diaphragm wall
- Phase 2: Install piles of 1000mm in diameter and 50m in length.
- Phase 3: Excavate the 1st layer down to level -2.5m
- Phase 4: Install the 1st strut layer with preload 40T/strut
- Phase 5: Excavate the 2nd layer down to level -6.8m and dewater to level -7.8m
- Phase 6: Install the 2nd strut layer with preload 60T/strut
- Phase 7: Excavate the 3rd layer down to level -11.0m and dewater to level -12.0m
- Phase 8: Install the 3rd strut layer with preload 80T/strut
- Phase 9: Excavate the 4th layer down to the B3 bottom at level -14.5m and down to the lift core zone at level -16.0m and dewater to level -15.5m, and level -17m at the lift core zone.
- Phase 10: Cast the B3 floor slab
- Phase 11: Dismantle the 3rd strut layer
- Phase 12: Cast the B2 floor slab
- Phase 13: Dismantle the 2nd strut layer
- Phase 14: Cast the B1 floor slab
- Phase 15: Dismantle the 1st strut layer
- Phase 16: Cast the G floor slab

3.4. Back analyses

In the back analysis, both the HSM and MCM are applied in the calculation of the section A-A which is shown in Figure 8. Soil parameters are inputted as those in the original design, except for the soil stiffness parameter of layer (4B). In back analysis, it is applied more than 5 times (E'=5xE). The simulation in Figure 9 completely follows the construction sequence. Analyzed results are compared with the observed data (ICL03).

- The observed result in cycle 3 on 28-9-2011 was shown in Fig. 10a that matches with phase 5 - excavation down to level -6.8m.
- The observed result in cycle 6 on 11-10-2011 was shown in Fig. 10b that matches with phase 7 - excavation down to level -11m.
- The observed result in cycle 16 on 7-11-2011 was shown in Fig. 10c that matches with phase 9 - excavation down to level -14.5m.

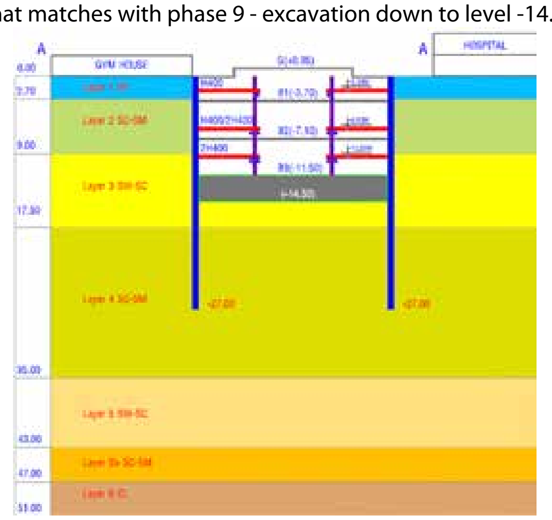

Figure 8. Section A-A

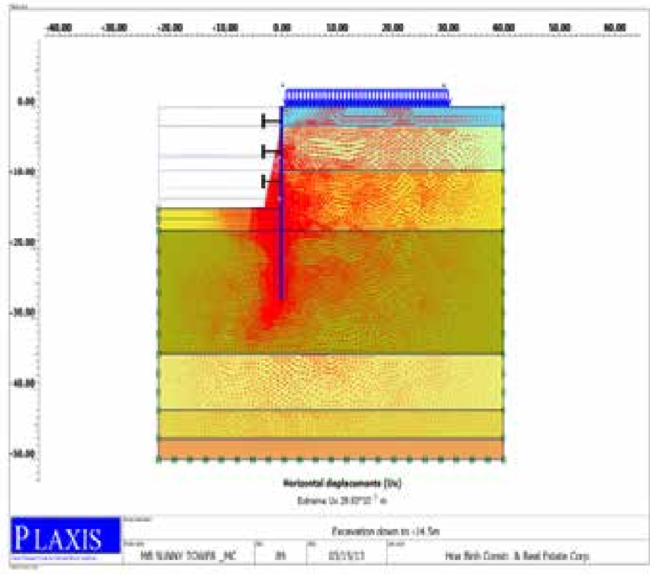

Figure 9. Geometry of the problem

Table 7. Soils stiffness parameters used in HSM

Depth (m)	Layer	Type	*E50ref [kN/m^2]*	Eoedref [kN/m^2]	Eurref [kN/m^2]
0-2.7	<Layer 1> Soft Clay	UnDr.	10000	18797	30000
2.7-9.0	<Layer 2>Silty/Clayey Sand	Dr.	25000	25000	75000
9.0-17.5	<Layer 3>clayey Sand	Dr.	30000	30000	90000
17.5-25	<Layer 4> clayey Sand	Dr.	30000	30000	90000
25-35	<Layer 4B> clayey Sand	Dr.	30000	30000	90000
35-43	<Layer 5> clayey Sand	Dr.	29000	29000	87000
43-47	<Layer 5B>silty/clayey Sand	Dr.	26000	26000	78000
47-51	<Layer 6> Clay	UnDr.	40000	40000	120000

Table 8. Soil stiffness parameters used in the MCM

Depth [m]	Name	Type	E_ref [kN/m^2]
0-2.7	<Layer 1> Soft organic clay	UnDrained	10000
2.7-9	<Layer 2>Silty/Clayey Sand	Drained	25000
9-17.5	<Layer 3>clayey Sand	Drained	30000
17.5-25	<Layer 4> clayey Sand/Sand	Drained	30000
25-35	<Layer 4B> clayey Sand/Sand	Drained	150000
35-43	<Layer 5> clayey Sand/Sand	UnDrained	29000
43-47	<Layer 5B>silty/clayey Sand	Drained	26000
47-51	<Layer 6> Clay	UnDrained	40000

Table 9. Comparison on struts forces

Fixed-end Anchor	Y [m]	Original design [kN/m]	HSM [kN/m]	MCM [kN/m]	Max. Capacity [kN/m]
1	-2.0	-112.9	-116.0	-70.8	-393.66
2	-6.3	-532.3	-522.0	-428.7	-590.49
3	-10.5	-782.1	-709.5	-613.3	-787.32

Bending moment of the diaphragm wall and axial force in struts in back analysis are compared with those given from the original design shown in Table 9 and Figure 10d. The maximum bending moment of 556kNm/m given from the calculation using the MCM is smaller than that outputted from the HSM by approximately 5%.

Moreover, the axial force results of struts provided from the MCM are less than that outputted from the HSM, ranging from 15%-20%. Generally, wall deflection profiles in Phases 5 & 7 provided by the MCM should not be used to compare as can be observed from the data in (ICL03). However, the profile of diaphragm wall deflection given by the MCM can be compared with the results given from the observed data and the HSM in phase 9, excavation down to level -14.5m. Otherwise, deflection profiles of the diaphragm wall as indicated by the HSM can be accurately compared to the observed data shown in Figure 10(a, b, c).

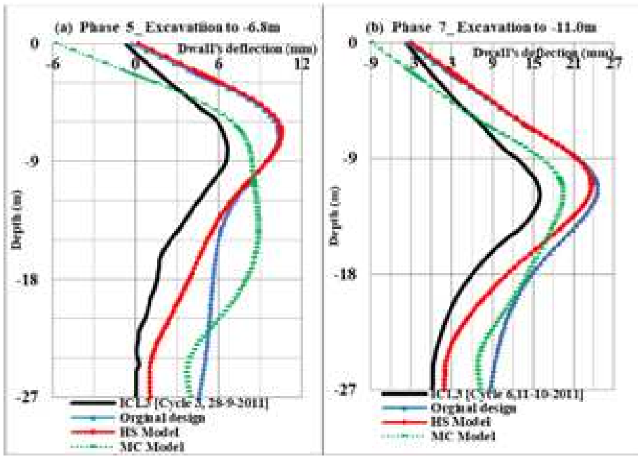

Figure 10. (a,b) Comparison of Dwall deflection given from the HSM and the MCM;

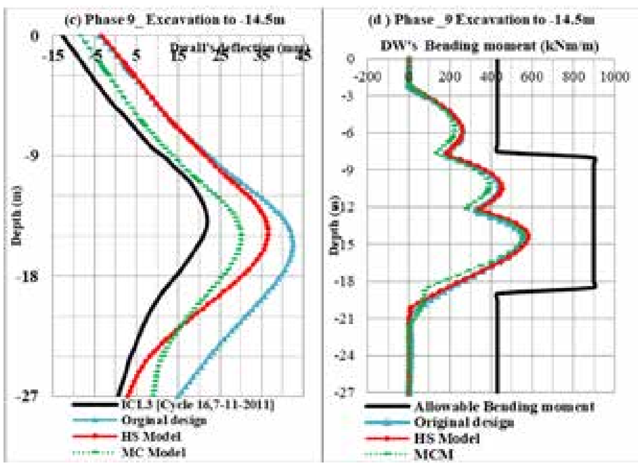

Figure 10. (c) Comparison of Dwall deflection given between ICL3, HSM, MCM;
(d) Comparison of Dwall bending moment given between HSM and MCM

4. DISCUSSION AND RECOMMENDATION

Firstly, back analysis on the diaphragm wall deflections in both cases given from the HSM are closer to the observed data and more reliable than those given by the MCM.

Secondly, back analysis on the axial forces in struts in both cases indicated by the HSM are larger and safer than those given by the MCM. Specifically, in the case of MB Sunny Tower, struts started bending under construction, which warns that the axial force in struts surpasses the critical strength of the strut.

Thirdly, axial forces in the struts provided by the HSM in both the original design and back analysis nearly reach the maximum limit. In contrast those given by the MCM are still 15 to 20 percent smaller. That is why it is recommendable to use the HSM for purposes of safety and reliability.

Lastly, another point relating to the stiffness of soil in Table 10; It is highly recommendable to use the following soil equation highlighted below in designs of deep excavation.

Table 10. soils stiffness recommended

Type of soil	Empirical equation
Soft organic clay	E=(500 to 700)Cu
loose-medium sand/sandy soils (N=<12)	E=(800 to 1200)N
medium dense sand/ sandy soil/stiff clay (12<N<15)	E=(2000 to 2500)N
medium dense to dense sand/ sandy soil (over-consolidated, N>15)	E=(5000 to 10000)N

Cu is undrained shear strength [kPa];
E is young's modulus of soil [kPa]; *N* is SPT value.

5. ACKNOWLEDGEMENT

The author would like to take this opportunity to thank HBC withouth whom this analysis would not have been possible. Lastly, he expresses his acknowledgement to HBC's Engineer, Mr. Pham Quan Dang.

REFERENCE

[1] Bjerrum L., *Observed versus computed settlements of structures on clay and sand*, Massachusetts Inst.of Tech. Cambridge mass, USA,1964

[2] Bowles, 1998, *Theory & practice of foundation design*, N.N Sorn & S.C. Das, India, 2006

[3] Chinnaswamy. C.G., Issues in numerical analysis of deep excavations for practicing engineers, *Intl.* of *conf. on Geotechics for sustainable development*, Hanoi, 783-789, 2011

[4] Clough, GW. and Mana, AI., Lessons learned in F.E analysis of temporary in soft clay. *Proc. 2nd Intl. Conference on Numerical methods in geomechanics*, Blacksburg, 496 – 510, 1976

[5] Dames and Moore, *Atlantic generating station, engineering properties of foundation soils*. AGS-PSAR Amendment 19, 1975

[6] James, WC. S. and Jackie, CY. Y., Challenges of deep basement excavation works in Vietnam, *Intl.* of *conf. on Geotechics for sustainable development*, Hanoi, 499-505, 2011

[7] Ladd, CC. and Edgers, L. *Consolidated-undrained direct-simple shear tests on saturated clays*, Dept. of Civil Eng., Mass. Inst. Of Technology, Cambridge, 354, 1972

[8] Papadopoulos, 1992, *Theory & practice of foundation design*, N.N Sorn & S.C. Das, India, 2006

[9] Vermeer P.A., Meier C-P., Stability and deformations in deep excavations in cohesive soils. *Instl. Conf. on soil structure interaction in urban civil engineering*, Darmstadt Geotechnics 1-4, 177 -192, 1998