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SYNOPSIS 0.8m thick cast-in-situ diaphragm wall having toe depth of 28m with two level temporary bracing was used to construct the basements of a structure which is located near the river and surrounded by buildings, including a historical one in Bangkok, Thailand. Due to the site condition, unbalanced lateral loading on the wall was expected and an excavation down to -12.7m below the existing ground level was carried out with instrumentation, consisting of (8) inclinometer tubes installed in the wall panels, settlement plates around excavation zone and tiltmeters and beam sensors on the existing structures. This paper presents computer model analysis and performance of the wall including results of instrumentation. Behaviour and performance of the wall is compared with those of other projects in Bangkok area.

INTRODUCTION

In Bangkok, the growing land price and need for space has necessitated deeper and larger basement excavations, even in some unfavorable subsoil and site conditions and in limited spaces. Subsoil conditions in Bangkok is generally a very soft clay of 12m to 18m thick layer underlain by stiff to hard clay and series of sand layers. Excavation in such soft soil requires efficient retaining structures and cast-in-situ diaphragm walls have therefore come in use frequently. This paper presents performance of a bracing excavation with diaphragm wall adjacent to the river and surrounding structures, including a historical building. This historical building, having archeological and cultural values, not only limited the height of the building but also influenced the construction time. Since the location of planned building is in a current limited height zone in which up to 4 storey height is permitted, 3 level basement was included to increase the usable floor area.

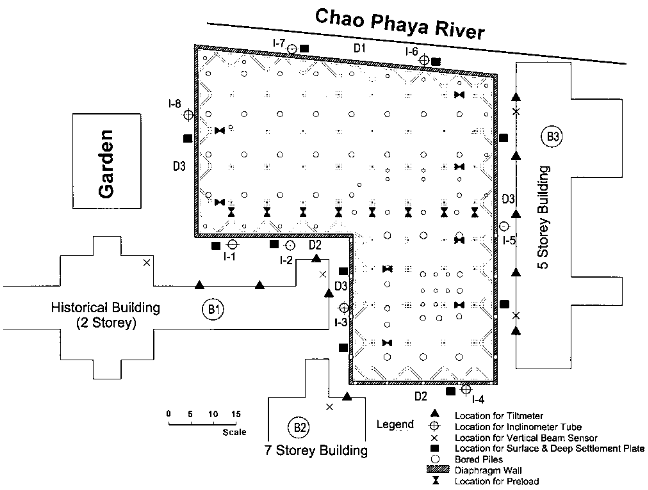

Fig. 1 Layout of project site and instrumentation

SITE CONDITION AND SUBSOIL

The building site is located nearby the Chao Phaya River and surrounded by a historical building and other existing structures supported by pile foundation (Fig. 1). Accurate information on foundation of these existing buildings was not available. The diaphragm wall along the river (D1) was constructed about only 4.0m away from the existing old river wall. The river bed near the river wall is about 2.2m-3.0m in depth, sloping towards the mid-stream to a depth of about 10m to 12m. River water level is about 1.6m below the ground level during dry season and sometimes in the rainy season is above the ground level, causing inundation.

A primary site investigation was carried out by drilling two boreholes and two field vane shear tests. Prior to design for bracing and basement excavation, drilling of additional two boreholes and two field vane shear tests were performed to check variability of subsoil conditions. The subsoil

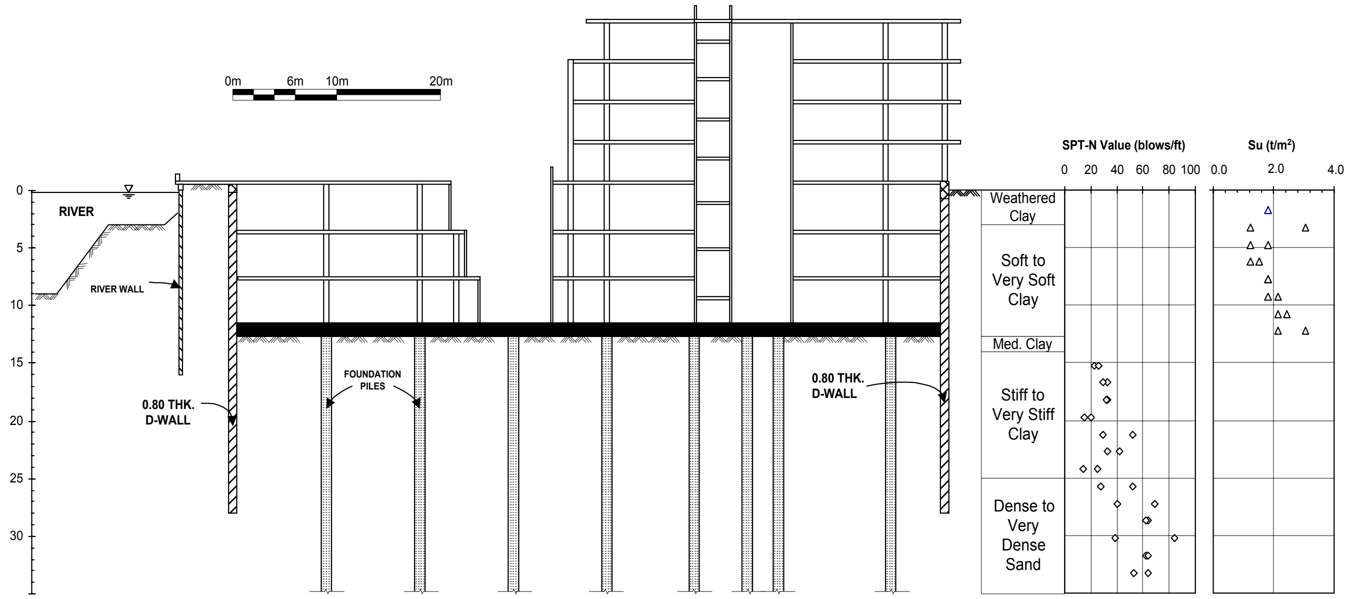

Fig. 2 Basement section and soil profile

properties obtained from the boreholes and test data were summarized in Table 1.

Table 1. Summary of subsoil properties

Soil Type	Layer Top in Depth m	W %	?s kN/m3	cu kPa	SPT N
Soft Clay	0.-3.0	35-78	16-19	30	
Medium Clay	12.7	30	19	71	
Stiff Clay	14	22-34	19-21	43-300	14-52
Dense Sand	25	14-25	20-23	-	>39

DIAPHRAGM WALL

A 800mm thick diaphragm wall was designed for excavation down to 12.7m below the ground with two levels of temporary bracing. The diaphragm wall toe was embedded down to 28m to achieve the overall stability of the excavation which is right on the river bank (Fig. 2).

Originally excavation stages were modeled by using a one-dimensional finite element computer program (Nonlinear Beam Column Analysis). Soil elements were modeled as spring and wall elements were modeled as beam. Four types of wall were designed to suit the temporary construction stages and permanent conditions. The walls were reinforced to withstand bending stresses up to 1000 kN.m/m in vertical direction. The maximum movement of the wall was expected to be 24.2mm.

For the wall sections adjacent to floor slab openings, especially in the water storage area, additional reinforcements were provided for bending stress in horizontal direction.

PILE FOUNDATIONS

A total of 165 bored piles consisting of 0.8m, 1.0m, 1.2m and 1.5m in diameter being founded in depth of 48m was constructed to support the structure. Out of 165 piles, 5 numbers of 0.8m diameter piles and 8 numbers of 1.0m diameter piles were incorporated with diaphragm wall panels as a leg to carry the load transferred through the wall.

INSTRUMENTATION

Due to the locality and particular conditions of the site, more instrumentation than those used by other projects in Bangkok were installed and monitored. Three types of instrumentation - Eight inclinometer tubes were installed in diaphragm wall panels to monitor displacement of wall, ten settlement plates of 1.0m and 5.0m in depth around excavation zone, and ten tiltmeters and four vertical beam sensors on surrounding buildings were installed to monitor tilting. Layout of instrumentation is presented in Fig. 1 .

BASEMENT EXCAVATION

The principal steps of basement excavation and construction sequence are described below;

1. Construct capping beam and excavation to 2.5m below ground level.
2. Install first level bracing at -1.5m and pre-load the struts
3. Excavate down to -7.0m
4. Install second level bracing at -6.5m and pre-load the struts
5. Continue excavation down to the final depth at -12.7m
6. Construct mat foundation (Basement 3), Basement 2 and remove second level bracing
7. Construct Basement 1 and remove the first level bracing

Since the excavation work is located adjacent to the river and surrounded by old buildings, the diaphragm walls are subject to three different lateral load conditions resulting from (1) full depth of the earth, (2) steep downward slope of riverbed, and (3) full depth of earth with possible surcharges from the adjacent buildings. In particular, the walls alongside the river (D1) and the opposite walls (D2), were expected to undergo an unbalanced loading condition. During temporary bracing design, two dimensional computer modeling were carried out to study the effects of unbalanced lateral loading on the wall and bracing. An additional two dimensional model analysis was carried out prior to designing a temporary bracing. The model analysis indicated relatively less movement of Wall D1 towards excavation (Fig. 3). However, the following measures were adopted in excavation work to prevent potentially adverse behavior of wall D1 and to keep the lateral wall movements within tolerable limits;

1. Using a simple, but efficient temporary bracing system
2. Pre-loading of the struts (200kN/m and 400kN/m for first and second levels respectively) on the one end of the strut on Wall D2 only (Fig. 1).
3. Excavating the soil in front of Wall D1 side first at any excavation stage
4. Frequent monitoring of wall movements
5. Minimizing construction time

During the initial excavation to 2.5m for installation of first level bracing, a historic foundation was unexpectively discovered. Excavation was suspended for about 3 months and resumed after further excavation was permitted by the archaeological department.

Excavation for final depth was made in the rainy season and a berm made of sand bags for flood protection was constructed around the perimeter of the wall. After lean mixed blinding concrete had been cast at the final excavation level, the river water level rose to a maximum level about 0.5m above the ground level.

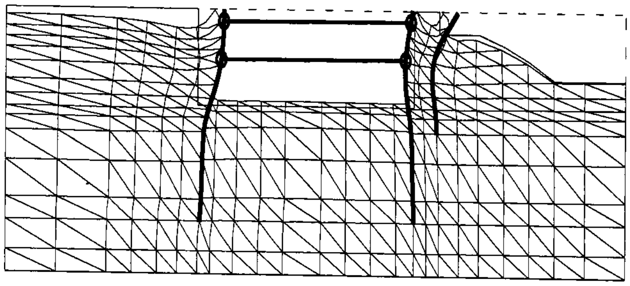

Fig. 3 Deformed mesh of two-dimensional model

TEMPORARY BRACING SYSTEM

A simple cross-lot bracing system with continuous wale beams was used to support the wall during basement construction stage (Fig. 1). 20m long steel king posts of H300x300 sections were driven into stiff clay to support the bracing system and temporary deck for excavation and construction equipment. A summary of steel sections used in bracing system is presented in Table 2.

Table 2. Summary of temporary bracing system

Bracing Level	Wale Beam	Strut	Expected Force (kN/m)
First	1xWF400x400	WF350x350	484.0
Second	2xWf400x400	WF400x400	789.0

Between wale beams and wall, lean mixed concrete were poured into the gaps between wale beams and wall to achieve a good load transfer. Pre-loading was carried out using hydraulic jacks. During pre-loading, the movements of the strut were measured. Horizontal movements of 0.82mm to 9.06mm and 0.15mm to 29.83mm in the direction of jacking for first level struts and second level struts respectively were recorded. For second level struts, the vertical movement of up to 1.68mm was measured at the jacking position.

INSTRUMENTATION RESULTS

Inclinometer Monitoring - Wall movements measured from inclinometers and predicted displacements by one-dimensional finite element analysis are shown in Fig. 4. Movements of Wall D1 and a section of Wall D3 close to the river, near the garden are found to be cantilever shape indicated by I-6, I-7 and I-8 respectively while movements of Walls D2 and D3 have inward bulging shape indicated by I-1 to I-5.

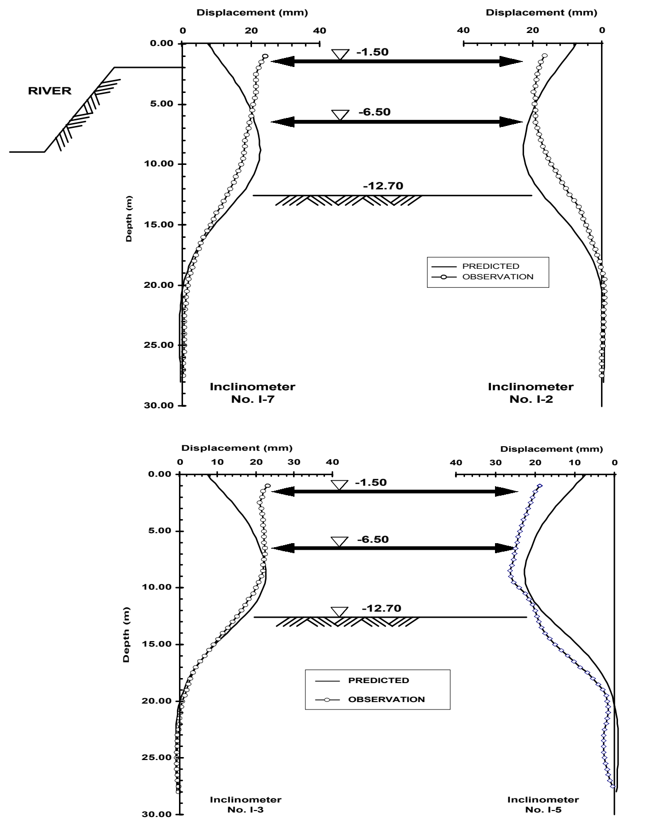

Fig. 4 Predicted and measured lateral wall movements

Generally the predicted and measured lateral wall movements are in a good agreement, except for the top portion of wall in which the measured wall movements exceed the predicted movements. The differences in wall movement were found to be caused by the following;

1. Advancement of excavation in front of Wall D1 further than other walls, allowing Wall D1 to stand longer prior to strut installation than the others
2. Wall D1 was in cantilever condition for about 3 months after initial excavation for first level bracing.
3. No direct nor immediate pre-loading on Wall D1.
4. Slight over excavation to install first level strut for all Walls.

The walls are in fix end conditions as fixity of walls indicated by all inclinometer reading are found to be at depth of about 20.0m which is 8m above the walls' toe depth.

Maximum lateral wall movements after installation of struts were in the range of 0.14% to 0.64% of the corresponding excavation depths (Fig. 5) and were also within the range (about 0.1% to 1.2% of excavation depth) of projects completed in Bangkok area.

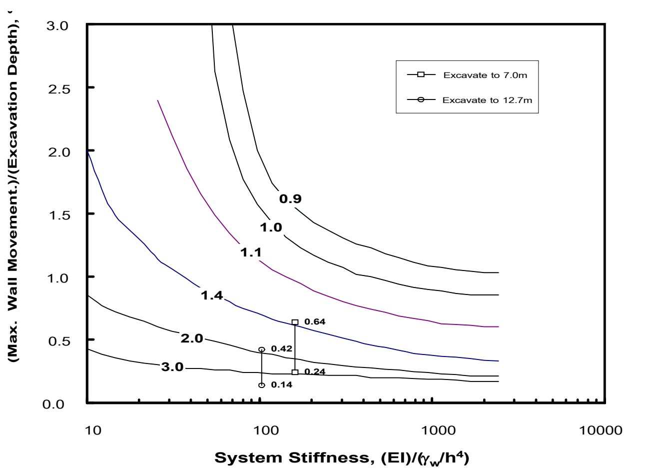

Fig. 5 Wall movement and excavation depth ratio

Settlement Monitoring - Ground settlement of up to 16mm was observed after completion of excavation works. The maximum settlement was found to be near the location of the inclinometer I-6 which indicates the largest lateral movement at the top of the wall.

Tiltmeter and Beam Sensor Monitoring - Readings of tiltmeter and beam sensors installed in the surrounding structures are shown with the progress of excavation in Fig. 6. A comparison between results from tiltmeter and beam sensor and typical values for maximum building slope or settlement for damage risk assessment is presented in Table 3.

Table 3. Comparison between observed values and maximum damage risk assessment (Lake et. al.))

	Max. Slope of Building	Max. Settlement of building (mm)	Description
Risk Category 1	<1/500	<10	Negligible : superficial damage unlikely
Tiltmeter	1/2082*	-	
Vertical Beam Sensor	1/3165*	-	
Risk Category 2	1/500	10 - 50	Slight: possible superficial damage which is unlikely to have structural significant

* monitoring results for the project presented in this paper

No building settlements nor cracks on the buildings were observed till completion of the basement construction. The buildings were evaluated to be in risk category 1. The risk assessment and monitoring results confirm a good performance of the braced excavation.

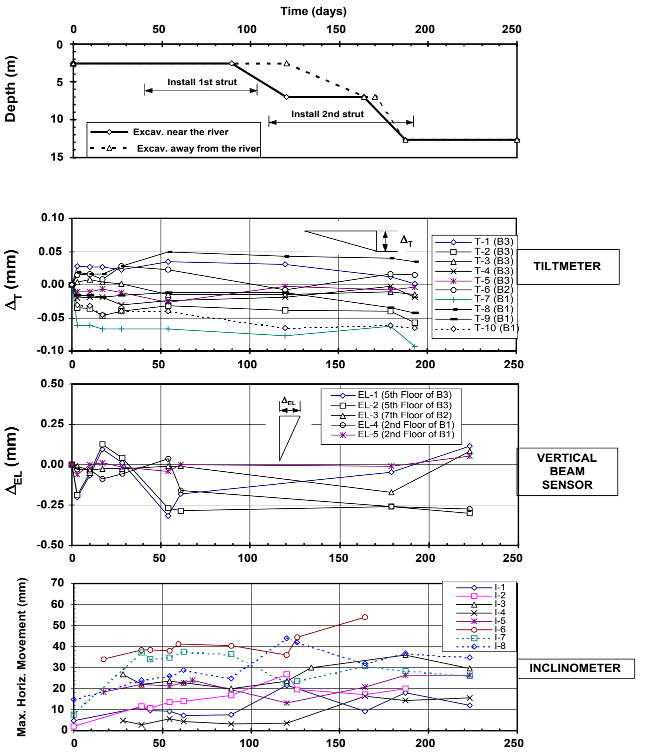

Fig. 6 Tilting of adjacent buildings and horizontal movement of diaphragm wall with construction time

PLANE STRAIN BACK ANALYSIS

A two-dimensional analysis was carried out by using finite element program named PLAXIS (Veermeer and Brinkgreve, 1995). A trial and error procedure in which the values of undrained Young's modulus (Eu) were adjusted iteratively was adopted to obtain the best fit between calculated and measured lateral wall movements. The excavation stages were modeled by using plane strain analysis. Mohr-Coulomb constitutive model was used for clay and sand. Walls and strut were modeled as plate element and elastic-perfectly plastic spring element respectively. Pre-loading in struts are also included in the analysis.

Results of analysis show that modulus value of soft clay is about 500Su and for stiff clay modulus value is about 2,000Su. For one-dimensional analysis, the spring stiffness used is equivalent to Eu=800Su for both soft clay and stiff clay.

DISCUSSION

Pre-loading is an effective way to reduce further wall movement after strut installation by providing a good intact between wall and supporting system. This is clearly seen in measured wall movements compared between I-7 and I-2, and I-3 and I-5 which are located in the walls generally facing each other (Fig. 4 and 1)

Delay in strut installation and construction time affect the wall movements, especially at initial excavation stage before installation of first level bracing.

Wall D1 which is alongside the river was found to be minimally affected by the unbalanced load conditions due to deep embedment (about 8m below the fixity) of the wall and presence of the old river wall. This was also shown by the two dimensional analysis (Fig. 3).

The predicted movements of the wall in cantilever condition at initial excavation stage in soft clay layer is considerably small due to the high modulus of soft clay adopted. In this case, construction time and sequence of excavation needed to be strictly controlled to keep the wall within the movement limit predicted by the one dimensional analysis.

Two dimensional analysis with facilities to model the geometry of site is required for the diaphragm wall subject to unbalanced lateral loading conditions.

CONCLUSION

Braced excavation using diaphragm wall subject to unbalanced loading due to adjacent river and surrounding buildings, was successfully achieved with proper instrumentation and monitoring.

Performance of the wall based on the instrumentation results are presented and discussed.

Back analysis was carried out with two dimensional modeling to determine the soil modulus. The soil modulus obtained from back analysis and the modulus adopted in wall designed were compared.

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