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ABSTRACT

Geotechnical aspects in construction of diaphragm-wall-support 2 level underground car park building, located in the historically and culturally significant area of Bangkok is presented in this paper. Results of the preliminary analyses showed that the deflection of the thin diaphragm wall of 0.60 m width would be large if it was to be fully cantilevered to fulfill the architectural and utility aspects of the car park structure. It was therefore decided to use buttress to minimize the diaphragm wall deflection. Performance of buttressed-support diaphragm wall is demonstrated based on the inclinometer monitoring results. Intensive modification of construction sequence in actual work execution with "value engineering options" different from tender stage design is demonstrated along with application of observational method.

INTRODUCTION

The project is a two-level underground car park located in the center of Rattanakosin Island, the heart of an old established, historically and culturally significant area of Bangkok. The project owner, Bangkok Metropolitan Authority (BMA) awarded the semi-turnkey basis construction contract "Lam Kon Muang Underground Car Park" to SEAFCO Co., Ltd. as a contractor. The contract consists of 3 major scope of works : (1) Construction of building foundation and retaining structure - diaphragm wall, barrette and bored piles (2) Excavation works including temporary bracing design and installation (3) Construction of the entire two-level underground car park building having car park area of 18,552 m2 and roof-level park of 10,936m2 plus cut-and-cover tunnel, underpass access to the City Hall. Geotechnical aspects highlighting the performance of buttress-support diaphragm wall of 0.60m width for two level underground car park building is discussed in this paper.

PROJECT REQUIREMENT AND MAJOR CONSTRAINTS

Since there is a limited availability of car parking space in the surrounding congested neighborhood and the project site was being used as a grade-level car parking space prior to the award of the contract, the key requirement was to construct the underground car park in two phases – to construct Phase 1 while leaving space for car parking in Phase 2, and to utilize semi-finished underground car park of Phase 1 during construction of Phase 2. This requirement posed the need of temporary retaining wall between Phase 1 and 2.

Construction site is surrounded by numbers of sensitive structures as shown in Fig. 1, - in the south, Wat Suthat, one of Thailand's most important temples and the Historical Giant-swing, in the north, the City Hall and at South-east corner, the Historical Brahmin Temple. Rows of old shop-house buildings are closely located in the east and west boundaries of the project. Location of the project site itself in the vicinity of sensitive structures and buildings therefore posed some constraints, which called for the need of careful consideration in establishing the design principles and sequence of construction.

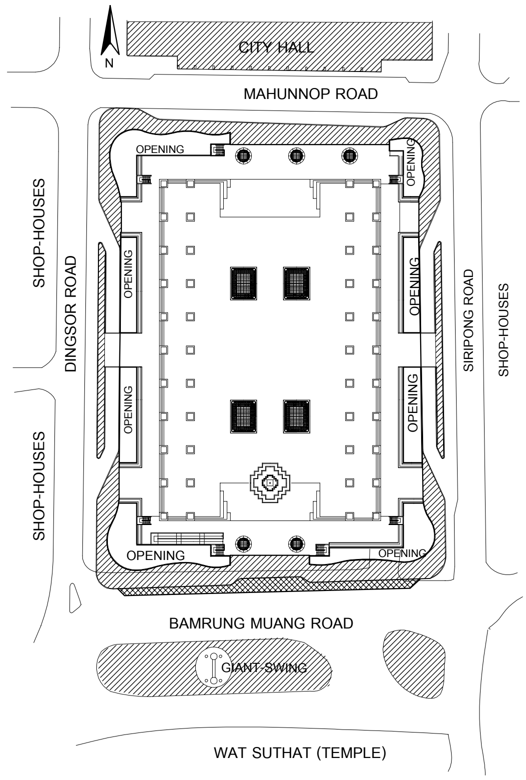

Fig. 1. Layout plan of the project showing adjacent buildings

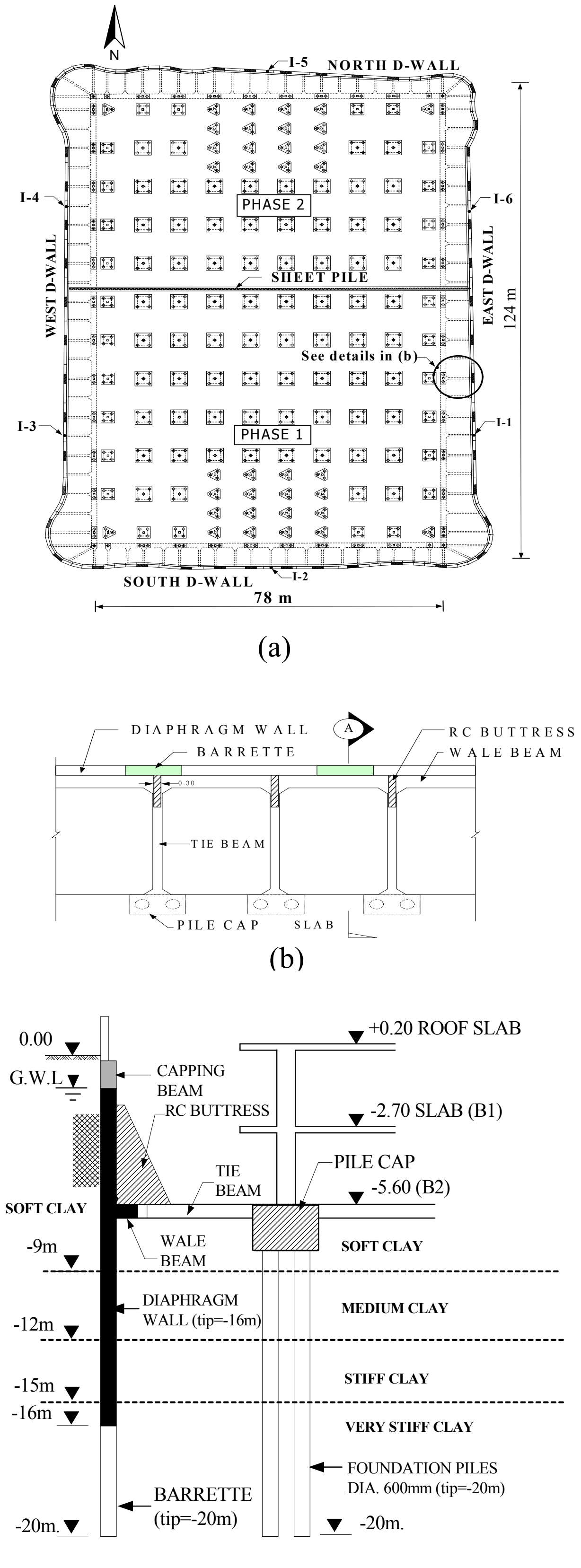

Fig. 2. (a) Layout of diaphragm wall and piles (b) plan of diaphragm wall and buttress (c) sectional view of the structures

The architectural and utility aspects of the project called for the design of the basement with a number of openings from the ground surface to the final basement slab level to facilitate the ventilation system as shown in Fig. 1. Hence roof slab cannot be physically utilized as bracing in most of the area where the diaphragm wall is to be acting as a cantilever retaining wall in the permanent stage. It was analyzed in the preliminary analyses that the deflection of the diaphragm wall of 0.60 m width would be large if it was to be fully cantilevered. As the project is located in a sensitive area, ground movement induced by large deflection was unfavorable. It was therefore decided to use buttress to minimize the diaphragm wall deflection as shown in Fig. 2.

DIAPHRAGM WALL, BARRETTES AND BORED PILE CONSTRUCTION

Diaphragm wall having 600mm width founded at 16m below ground level (B.G.L) was constructed simultaneously with dry-processed bored piles of diameter 600mm with toe depth 20m below ground level. Barrettes having same toe depth as bored piles were installed at 8m spacing along with diaphragm wall panels. Sheet pile wall (14m deep) was used as a temporary retaining wall at the boundary of Phase 1 and 2 as shown in Fig. 2.

SUBSOIL CONDITION AND DESIGN PARAMETERS

Typical subsoil profile at the site is characterized by thick Bangkok soft clay layer at the top followed by thin layer of medium clay, and stiff clay layers. Undrained shear strength (S_u) and SPT N-value obtained from 3 SI boreholes were plotted and design line was derived as shown in Fig. 3. The design soil parameters are tabulated in Table 1.

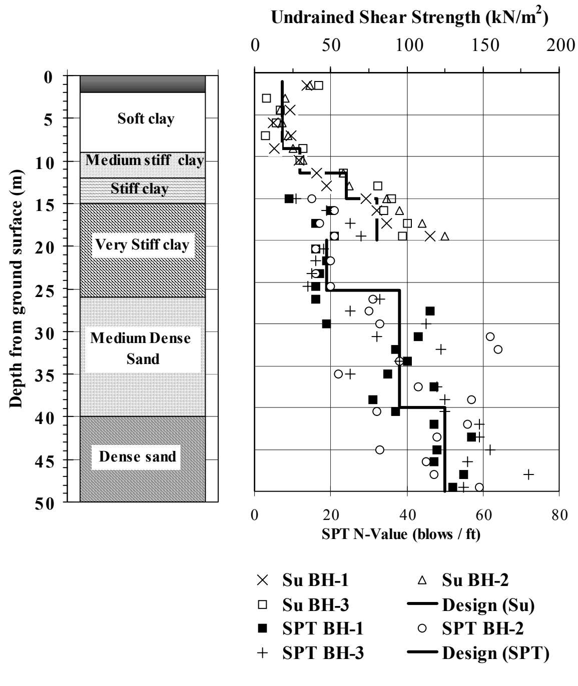

Fig. 3. Plotted Su and SPT N-value with design lines

Table 1. Design soil parameters

Soil Type	Depth (m)	γ (kN/m^3)	S_u (kN/m^2)	E_u (kN/m^2)
Soft clay	0 - 9	16.50	18	7000
Medium clay	9 – 12	17.50	30	19250
Stiff Clay	12 – 15	19.00	60	45000
Very stiff clay	15 - 26	19.50	80	60000

BRACING SYSTEM IN TENDER STAGE DESIGN

The designers involved in the tender stage design made a fairly conservative design with two levels temporary bracing as shown in Fig. 4. Uncertainty of the performance of a thin diaphragm wall in soft clay layer was the likely reason to adopt the conservative design in the tender stage. It is not unreasonable to adopt the conservative design considering time-dependent consolidation property of soft marine clay and likely long elapsed time of un-strutted diaphragm wall (relatively long un-strutted span of about 6.0 m between first strut and final excavation level for 600mm diaphragm wall) due to large volume of excavation work involved.

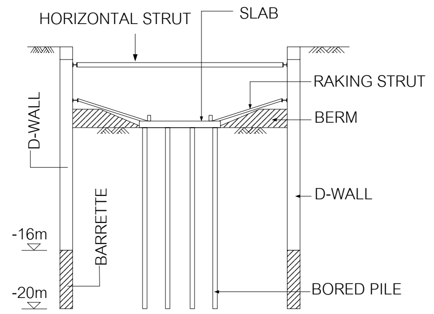

Fig. 4. Tender stage bracing system – diaphragm wall was designed with soil-berm and 2 struts support (horizontal bracing and raker)

VALUE ENGINEERING OPTION FOR PHASE 1

Value engineering review of the temporary works was undertaken by the contractor's new in-house design engineering team prior to the commencement of Phase 1 excavation works. Rigorous attention to detail of the design concept and constructability was made in the pre-construction discussions between design engineers and construction team.

The main objectives for value engineering options were to minimize the material and construction sequence involved in temporary works so as to accelerate the excavation time thereby saving overall costs. without compromising the safety aspect. After conducting a series of re-analyses with different conditions major modifications were made : (1) To lower the first strut level to –1.8m from the original tender stage design level –1.0m (2) To use only 1 temporary strut, omitting second level raking strut with the provision of sloping soil berm against diaphragm walls. Soil berm was to remove after completion of base slab construction in the majority of area – minimizing the elapsed time of partially un-strutted diaphragm wall between temporary strut and the final excavation level. Modified bracing system of Phase 1 as an outcome of value engineering review is illustrated in Fig. 5

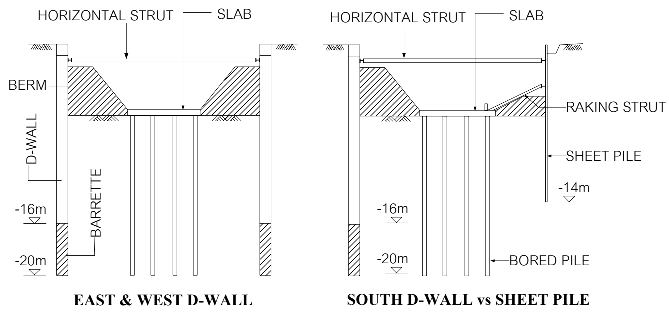

Fig. 5. Value engineering option - Modified Bracing system for Phase 1

IMPLIMENTATOIN OF THE OBSERVATIONAL METHOD

Professor R. B. Peck set out procedures for the observational method (OM) as applied in soil mechanics in the Ninth Rankine Lecture (Peck, 1969). Peck described the limitation and drawbacks of observational method. Powderham (1996) reviewed the main features of the observational method of Peck and summarized the key requirements as follows;

(1) It must be possible to alter the design during construction
(2) The contractual condition must be compatible and allow design to be directly related to actual construction method
(3) An acceptable level of risk must be identified and controlled. In particular this requires a planned course of action for every foreseeable eventuality
(4) Critical observation must be identified and obtained

During the review of tender stage design for Phase 1, it was recognized that two-phase excavation works in this project was ideally suitable for application of the observation method. The flexibility of the contractual requirement in temporary design which allowed the contractor to modify the design and construction method also provided the favor for the observational method.

In order to assess the most probable condition assumed in "value engineering design" and to take necessary actions if monitoring results reveal most unfavorable conditions (i.e. actual deflection of diaphragm wall reaches maximum acceptable limit), the observational method was implemented on the followings basis.

- Reviewed the design parameters together with critical conditions posed on sites as well as most critical stage in excavation and basement construction work

- Predicted the performance of diaphragm wall with "most probable" as well as "most unfavorable"conditions and parameters.
- Established the trigger criteria based on predicted diaphragm wall deflection
- Predefined the practical contingency plan for "most unfavorable" conditions where wall deflection reaches trigger levels
- Set out the instrumentation program with the consideration of above factors
- Monitored the performance of diaphragm wall. Compared the monitoring results with the predicted and trigger values and reassessed
- Implemented the contingency measures if monitoring result reaches action level of trigger values

Figure 6 shows the predicted diaphragm wall lateral displacement or deflection of Phase 1 (east, west and south diaphragm wall) at two conditions together with trigger levels and tender stage prediction. It should be noted that the diaphragm wall deflection was predicted to be maximum or most critical after removing the horizontal temporary strut.

Most probable condition was established for the predicted deflection of diaphragm wall with full influence of buttress support – assuming buttress effectively supports as permanent strut in diaphragm wall analysis model. Most unfavorable condition was set out for the predicted deflection of diaphragm wall without considering influence of buttress – buttress was excluded in the model. Diaphragm wall reinforcement was designed based on the most unfavorable condition.

In establishing the criteria for trigger values, it was necessary to consider the broad context in which diaphragm wall exists, design assumption and concept, likely behaviour of the wall itself or its predicted performance and effectiveness of selected temporary bracing system. Trigger levels were established to provide the design team and construction team, an opportunity for early review and resetting of the monitoring frequency as well as for implementing the contingency measure as necessary.

In general terms, exceeding alert trigger levels must initiate a review of design data, construction progress and monitoring frequencies with the consideration of possible measures to limit further deflection. Exceeding action trigger levels must initiate further review of above mentioned points and if necessary to initiate a planned course of action or contingency measures.

Effective and good communications between the design team and construction crew were made all along the excavation stages with clear responsibilities in construction control. The contingency measures included immediate backing filling of excavated soil and installing of temporary struts in the critical area.

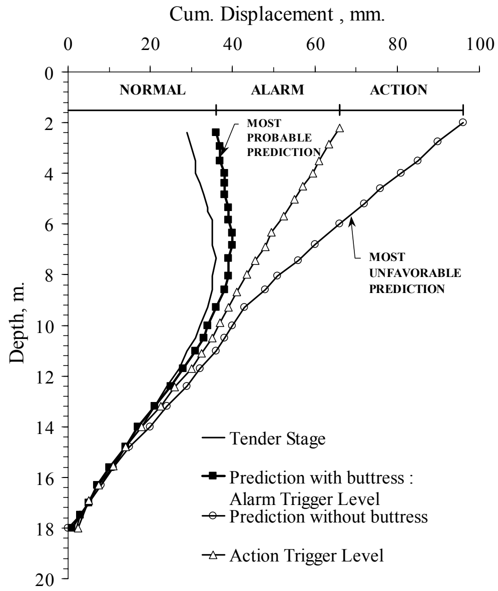

Fig. 6. Prediction of diaphragm wall performance in pre-construction stage with trigger criteria in comparison with tender stage prediction

MONITORING PROGRAM

In planning the monitoring system and program, it is important to consider the parameters to be measured which reflect the actual performance of the diaphragm wall support excavation. It is also necessary to take account the practical measurement applicable for established trigger criteria, number and frequency of measurement required to carry out a meaningful interpretation of wall behaviour which would be integrated in the implementation of observational method.

Comprehensive and robust monitoring program was set up as a key element in application of the observational method and to ensure that modified construction sequence would not have adverse effect in temporary stage and on permanent design. A total of 6 inclinometers (3 in each phase) were installed in diaphragm wall together with some survey points. In order to make effective use of the established trigger levels, an adequate number of measurement were carried out at appropriate frequency. Typical monitoring frequencies for inclinometer set up as guideline for the project is outlined below.

- Measured immediately before commencing excavation in the vicinity of instrument
- Minimum readings of 2 times a week while excavation in progress
- Minimum readings of 1 time a week when no excavation the vicinity of instrument
- Minimum readings of 3 times a week when measured deflection values exceeded alert trigger levels
- Minimum reading 1 time a day when measured deflection values reached action trigger levels

As the most critical stage was predicted at the time horizontal temporary bracings were removed, a full attention was paid to the inclinometer monitoring with the following special criteria and frequencies.

- First temporary strut removal was to carry out at the diaphragm wall panel where inclinometer was located
- Measured immediately before removal of temporary strut at the closest distance to the instrument.
- Measured every 6-8 hours immediately after removing the first strut
- Second strut to be removed must be the one located immediately adjacent to first strut which had removed
- Not to remove the second strut until inclinometer measured deflection values had stabilized
- Not to remove more struts unless measured deflection values were stabilized and within alarm trigger levels

In addition to inclinometer measurement, diaphragm wall movement was also monitored by the survey points strategically marked on the wall panels. Ground settlement and surface cracks behind the diaphragm wall were also visually checked by the construction team as daily basic.

PERFORMANCE OF PHASE 1 DIAPHRAGM WALL

After carrying out the comprehensive desk studies and establishement of systematc monitoring program presented above, Phase 1 excavation work was carefully commenced. Figure 7 shows the maximum accumulated diaphragm wall deflection at different stages of excavation monitored by inclinometer No.1 (I-1 at East wall of Phase 1) together with trigger levels and predicted maximum deflection profile of 3 different conditions - tender stage design (2 temporary struts), modified design with buttress and modified design without buttress. It can be observed from figure that measured lateral movement pattern of diaphragm wall agreed well with that of prediction for modified design with buttress - meaning buttress-support has significant influence on wall deflection.

Deflection profile of South diaphragm wall which braced against temporary sheet pile wall at Phase 1 and 2 boundaries is presented in Fig. 8. As can be seen in Fig. 8, South diaphragm wall deflection is significantly higher than that of east wall, which is likely to be caused by the fact that South diaphragm wall is braced with more flexible sheet pile wall. Description of stages shown in the legend of Fig. 7 and 8 is summarized in Table 2.

Table 2. Description of stages shown in Fig. 7 & 8

Stage	Description
1	Excavate to –2.2m and installed temporary strut
2	Excavate to –4m
3	Excavate to –6.6m with berm
4	Removal of berm
5	Removal of temporary strut after completion of buttress

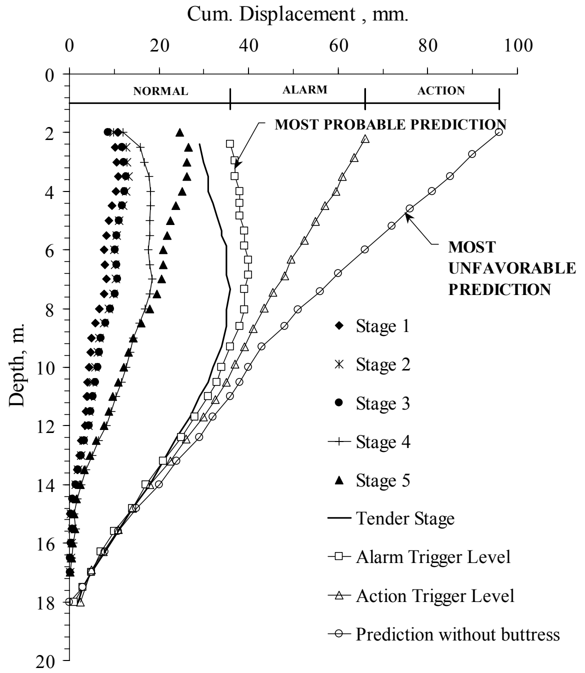

Fig. 7. Phase 1 East Wall - Monitored diaphragm wall deflection at different stages with trigger levels

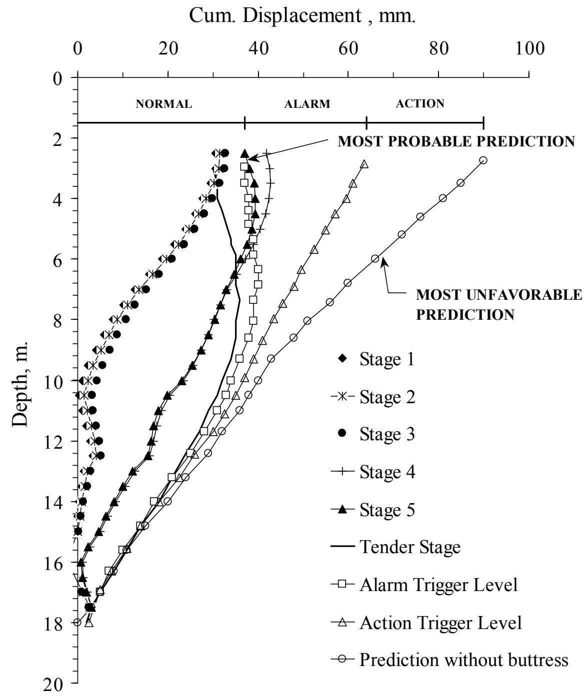

Fig. 8. Phase 1 South Wall - Monitored diaphragm wall deflection at different stages with trigger levels

Fig. 9. Phase 1 excavation work in progress with historical Buddhist temple Wat Suthat in background (Phase 2 area was being used as car parking space)

MODIFICATION OF PHASE 2 BRACING SYSTEM

Monitoring results of the Phase 1 excavation work provided an ample opportunity to review the design assumption, fine tune the parameters used in the analysis of the diaphragm wall for the Phase 2 and made modification of construction sequence. The major modifications are : (1) Removal of soil-berm at East and West diaphragm wall in shorter duration than that of Phase 1, and (2) Using raking struts instead of horizontal strut for North diaphragm wall a shown in Fig. 10.

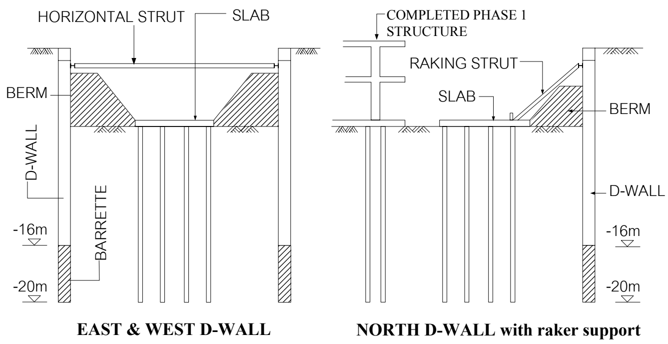

Fig. 10. Bracing system of Phase 2 diaphragm walls

PERFORMANCE OF PHASE 2 DIAPHRAGM WALL

Figure 11 depicts the measured deflection of east diaphragm wall. As can be observed in Fig. 11 in comparison with Fig. 7, the maximum deflection of east diaphragm wall in Phase 2 is larger than that of Phase 1. The likely reasons of this observation are;

- Un-strutted elapsed time for first temporary bracing in Phase 2 was longer than that of Phase 1.
- In Phase 1, horizontal struts were installed in north-south direction which temporary kingpost columns and strut were integrated in crisscross pattern with east-west struts - providing complete-support more rigid bracing system. Whereas in Phase 2, horizontal struts were installed only in east-west direction without having crisscross pattern with north wall – having less rigid bracing system than that of Phase 1.

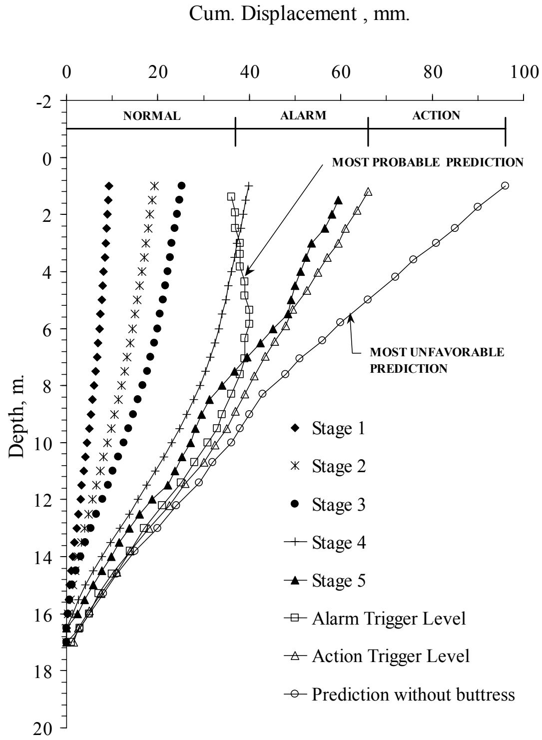

Fig. 11 Phase 2 East Wall - Monitored diaphragm wall deflection at different stages with trigger levels

With assurance of diaphragm wall performance from monitoring results of Phase 1, original plan of using horizontal struts for North diaphragm wall was modified by using raking struts instead. As can be seen in Fig. 12, deflection of North diaphragm wall (with raking strut support) is significantly higher than that of East diaphragm wall (with horizontal strut support). The main reason of larger movement of north diaphragm wall is due to the fact that it was supported only by the berm for the long period (about 52 days) before completion of raking struts so that soil-berm became soften during the long elapsed un-strutted period. Time-dependent deflection pattern due to softening and deformation of soft clay can be observed in North diaphragm wall as illustrated in Fig. 13. North diaphragm wall moved progressively toward excavation before completion of raking struts (at 52 days) as can be seen in Fig. 14.

Table 3. Description of stages shown in Fig. 11 & 12

Stage	East Diaphragm wall	North Diaphragm wall
1	Excavate to –2.2m and installed temporary strut	Excavate to –2.2m at d-wall, and to –6.6m with sloping berm
2	Excavate to –4m	Installed raking strut
3	Excavate to –6.6m with berm	Removal of berm
4	Removal of berm	Removal of raker after completion of buttress
5	Removal of temporary strut after completion of buttress	

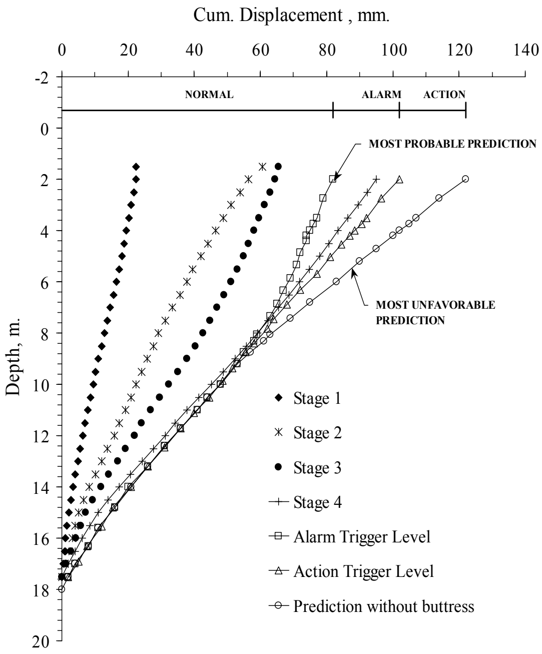

Fig. 12. Phase 2 North Wall - Monitored diaphragm wall deflection at different stages with trigger levels

Fig. 13. View of buttress-support diaphragm wall prior to temporary bracing removal

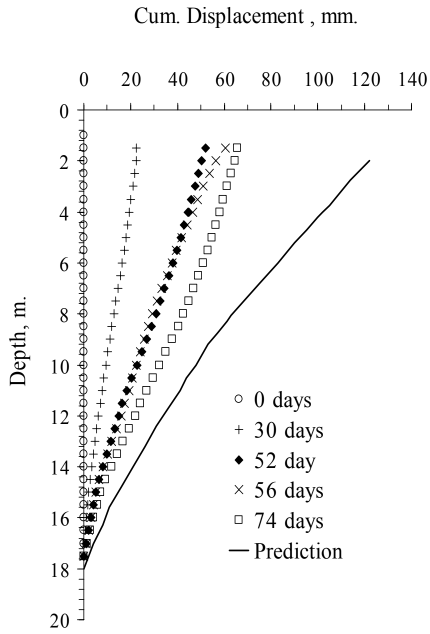

Fig. 14. Phase 2 – Time dependent wall deflection of North diaphragm wall – diaphragm wall was supported only by soil-berm for 52 days before completion of raker installation.

IMPLEMENTATION OF CONTINGENCY PLAN

Since deflection of North diaphragm wall (Phase 2) approached action trigger levels, monitoring frequency was increased and the following contingency measures were implemented on site.

- Poured 15cm thick 1m wide lean concrete on the top of the berm along North diaphragm wall to provide bearing-effect
- Installed additional king-post and diagonal struts attached to the raking struts to provide more rigid support against diaphragm wall
- Soil-berm was removed locally in bays followed by construction of wale beam, tie beam and buttress as shown in Fig. 15.

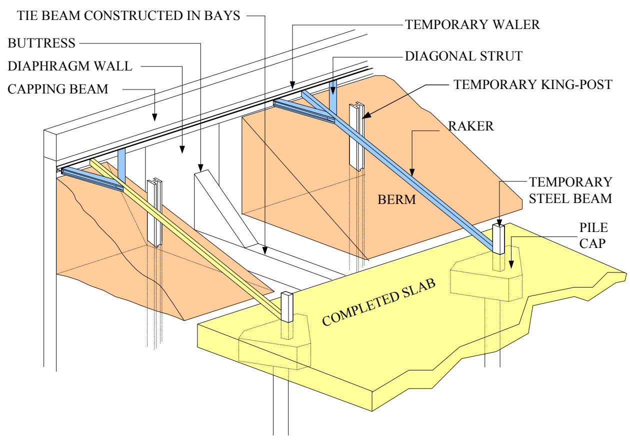

Fig. 15. Perspective of North diaphragm wall constructed in bay

Movement of diaphragm wall was observed to be decreased and eventually stabilized by the above actions. No significant ground settlement was observed in the vicinity of the North diaphragm wall.

Fig. 16. View of raker and soil-berm support – soil-berm was removed locally in bay

Fig. 17. View of buttress-support diaphragm wall after removal of temporary bracing

TIME AND COST SAVING FROM VALUE ENGINEERING OPTIONS AND THE OBSERVATIONAL METHOD

Significant cost and time saving were achieved from the value engineering option coupled with observational method implemented for both Phase 1 and 2. The major savings were achieved by less operation and material utilized in the following elements of temporary works.

- Cancellation of 2nd level raking struts against diaphragm wall for both phases
- Modification of bracing system – using raking struts with soil-berm support instead of horizontal struts for North diaphragm wall in Phase 2

CONCLUSION

Outcome of a through desk study at post-tender stage provided an effective value engineering option which offered significant cost and time saving for overall construction program. Effective and good communications between the design team and construction crew played a key role in successful completion of the project. Systematic monitoring program with clear defined trigger criteria was also the important element in implementing the observational method. This research study reveals that a thin permanent diaphragm wall coupled with effective design and construction method supplemented by the observational method and robust monitoring program could offer a logistically and financially attractive solution in construction of underground car park without disturbing the environment in the prominent historical area of Bangkok.

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