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Behavior and Performance of Diaphragm Walls under Unbalanced Lateral Loading along the Chao Phraya River

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ABSTRACT: This paper presents the behavior and performance of diaphragm walls constructed for basements of two buildings along the bank of the Chao Phraya River bank, Bangkok, Thailand. These buildings were constructed at different periods under one contract. A case history on basement excavation of the first building has been reported previously by Thasnanipan et al. (1998). For the second building, more numbers of instrumentation including VWSGs in a diaphragm wall panel and pressure gauges in struts to monitor stress in the wall and the performance of bracing system were used respectively. The construction sequence adopted for excavation, performance of the walls and bracing based on instrumentation results are discussed.

1 INTRODUCTION

Retaining structures and supported systems for deep excavation in urban areas, particularly in the vicinity of a river, are often subjected to unbalanced lateral load due to asymmetric loading and surcharges. The various publications and design guides by GCO no. 1/90 (1996), Williams & Waite (1993) and Padfield & Mair (1991) give special guidance on overall stability of retaining structures and strutting systems in such conditions. Case histories on braced excavations under unbalanced/non-symmetrical lateral loading also have been reported by Thasnanipan et al. (1998), De Rezende Lopes (1985) and Kotoda et al. (1990).

This paper presents two basement excavation works up to 12.7m below the ground level for two separate buildings using braced diaphragm walls along the bank of the Chao Phraya River, Bangkok. The first one had previously been reported by Thasnanipan et al. (1998). Construction of the second building commenced about one year after completion of the first one.

2 SITE AND SUBSOIL CONDITIONS

The building sites are located 40.0m apart, and separated by an existing 5 storey building between them. They are located along the river and surrounded by existing structures including a historical building (Fig. 1). The existing buildings were supported by pile foundation. The distance between diaphragm walls along the river and the existing old river wall buildings respectively. They are also situated in the heart of an old established culturally-significant zone. Space and building height limitations necessitate multi-level basements to increase usable floor areas although the planned buildings are right on the river bank.

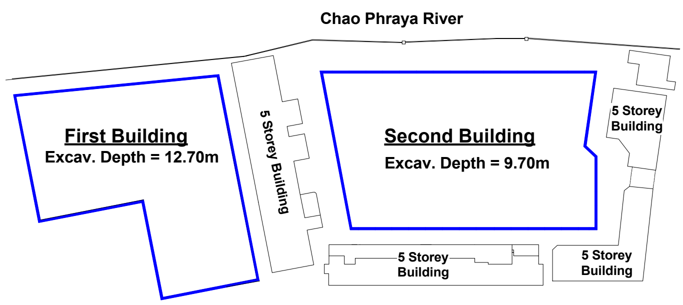

Figure 1. Layout of building sites

At the building sites, the river is about 205m wide and 10-12m deep mid-stream. The riverbed near the river wall is about 2.2-3.0m in depth with a gentle slope. The river water level in dry season is about 1.6m below ground level and in the rainy season sometimes rises above ground level, causing a flood.

Primary site investigations included one borehole 60m deep and one field vane shear test for each site. For the first building, prior to designing temporary bracing and basement excavation work, drilling of additional three boreholes 20m deep and two field vane shear tests were carried out to check subsoil variations. For the second building, two additional 60.0m deep boreholes were dug. The shear strength of soft clay in the planned second building site is higher than the soft clay in the first site. However, there is no significant variation in subsoil conditions within each site, particularly along the depth of planned diaphragm walls. Subsequently the profile of the sites and surcharges from surrounding buildings are of major concern for unbalanced lateral loading conditions in excavation works. Table 1 shows a summary of subsoil properties obtained from the boreholes and test data.

Table 1. Summary of soil properties

Soil Type	Layer Top in Depth m	W %	γ_s kN/m^3	C_u kPa	SPT N
Soft Clay	0-3.0	35-78	16-19	30	
Med. Clay	12.7	30	19	71	
Stiff Clay	14	22-34	19-21	43-300	14-52
Dense Sand	25	14-25	20-23	-	35-50
Silty Clay	36.5	17-21	20-23	175-240	30-45
Dense Sand*	42-45	20-26	21	-	>58

* Some 4-5m thick hard clay seams present at depth 48-52m

3 DIAPHRAGM WALLS

For both buildings, 800mm thick cast in-situ concrete diaphragm walls of 28m toe depth with two level temporary bracings were designed for basement excavation. The maximum excavation depths for the first and second buildings were 12.7m and 9.7m respectively. 28m deep walls were necessary for overall stability of excavation as they were located on the riverbank.

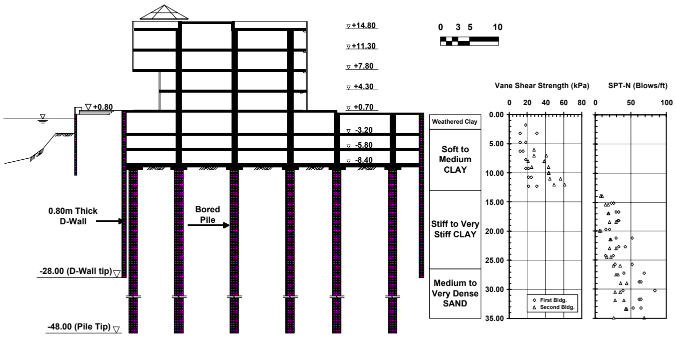

Figure 2. Basement section and soil profile (Second Building)

Different wall types were classified by design according to load conditions, slab supporting locations and geometry of ground profiles. The walls were designed generally as vertical beams to withstand bending moments up to 1100 kN-m/m. In the slab opening areas, additional horizontal bars were provided for secondary bending moments. A non-linear Beam-Column Analysis using a one-dimensional finite element program predicted maximum wall movements up to 24.2mm for the first building. Based on the performance of walls for the first building, a maximum wall movement of 51.0mm was allowed during excavation for the second building. The diaphragm walls for its basements were analyzed with elasto-plastic soil model using WALLAP program.

4 PILE FOUNDATIONS

Foundation piles having pile toe at 48m depth were constructed using bored piling under bentonite slurry. Quantities of piles are tabulated in Table 2.

Table 2. Quantity of foundation piles

Building	Pile Dia. 800mm	1000mm	1200mm	1500mm
First	32*	18*	31	6
Second	17*	33	5	23

* Some piles were incorporated in diaphragm wall panels.

5 INSTRUMENTATION

As construction sites were located in a very sensitive urban area and subjected to unbalanced lateral loading conditions, various types of instrumentation were installed and systematically monitored. Layouts of instrumentation for the first and second buildings are shown in Figures 3a and 3b respectively.

For the second building, 5 levels of vibrating wire strain gauges (VWSG) in pairs were installed in one diaphragm wall panel and 2 sets of earth pressure gauges were installed in struts to observe stress in the diaphragm wall and strut forces respectively. Types of instrumentation are presented in Table 3.

Table 3. Quantity of Instrumentation

Type of Instrumentation	First Building	Second Building	Measurements made
Inclinometer	8	6	Wall deflections
Settlement Plate	10	20	Ground settlements
Tiltmeter	10	10	Tilting of Buildings
Vertical Beam Sensor	5	5	Tilting of buildings
VWSG	-	1 location*	Strain in rebars
Earth Pressure Gauge	-	2 x 2	Sturt forces

* A pair of VWSG in 5 layers along the depth of wall panel

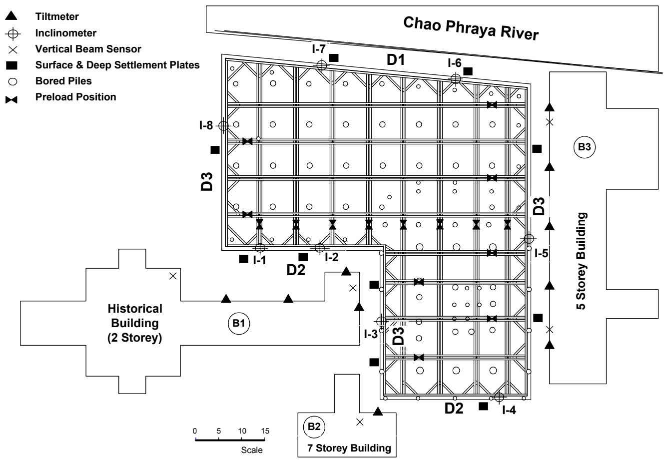

Figure 3a. Layout of temporary bracing and instrumentation (First Building)

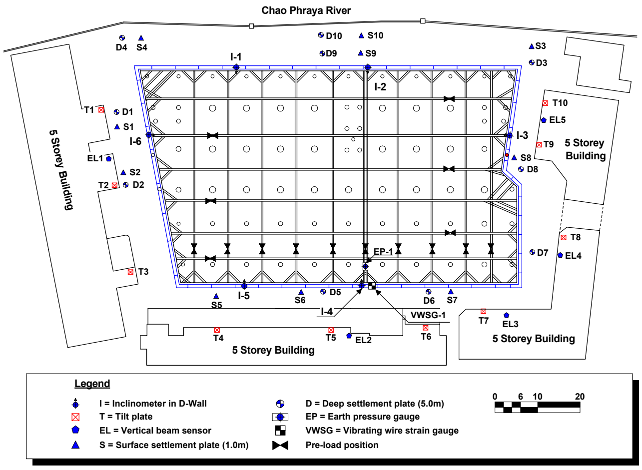

Figure 3b. Layout of temporary bracing and instrumentation (Second Building)

6 EXCAVATION WORK

Conventional bottom-up method with two levels of temporary bracing was adopted for excavation to construct foundation and basement floors of both buildings. Prior to excavation work for the first building, a two-dimensional computer analysis was carried out using PLAXIS finite element computer program to study possible behavior of the wall system under unbalanced lateral loading conditions. Such conditions were considered to be resulting from (1) full depth of the earth, (2) a step sloping river bed and (3) full depth of earth with possible surcharges from the adjacent buildings.

A major concern was that the diaphragm walls alongside the river would be thrust from the opposite walls. These bore a higher lateral load through axial force of struts as excavation progressed in stages. This wall behavior was indicated by computer modeling (Fig. 4), and previous reports by De Rezende Lopes (1985) and Kotoda et al. (1990). To prevent any adverse wall behavior alongside the river, the following measures were taken;

1. Using a simple and efficient temporary bracing system
2. Pre-loading on one end of struts on the opposite walls
3. Excavating first soil in front of the wall closest to the river at any excavation stage
4. Frequently monitoring of wall movements
5. Minimizing construction time

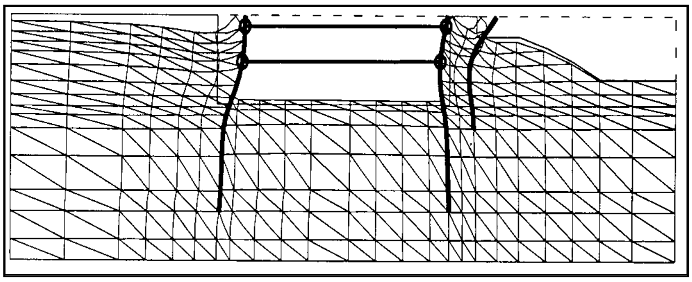

Figure 4. Behavior of walls shown by defromed mesh of two-dimensional model

Moreover, an observational construction approach (Ikuta et al. 1994), using the "most probable" conditions and parameters in the design with a contingency plan for "most unfavorable" conditions was employed. Firstly a monitoring plan was established before excavation. Secondly the instrumentation monitoring data were used to trigger the contigency plan. Generally predicted wall movements were set as primary trigger values for the contingency plan. After successful completion of the first building, the same approach was also adopted in excavation work for the second building. For the first level bracing of the second building, single strut arrangement (less rigid but economical compared to that of the first building) was adopted with provision of strut-force monitoring.

A simple cross-lot bracing system with continuous wale beams was used for both buildings. 20m and 19m-long, steel king posts of H300 x 300 in section were used to support the working platform and bracing system for the first and second buildings respectively.

In both cases, the initial excavation to 2.5m for installing the first level bracing revealed historic foundations. Temporary bracing and excavation works

Table 4. Summary of steel sections for the temporary bracing

Building	Bracing Level	Bracing Elevation	Strut Sections	Design Strut Force kN/m
First	I	-2.0	2 x WF350 x 350	484.0
	II	-6.5	2 x WF400 x 400	789.0
Second	I	-2.0	1 x WF400 x 400	279.4
	II	-7.0	2 x WF350 x 350	332.1

were delayed for about 3 and 2 months for the first and second buildings pending permission by the archaeological department for further excavation.

7 INSTRUMENTATION RESULTS

Construction activities and the corresponding results of instrumentation monitoring with construction time are presented in Figures 5a and 5b. These figures, particularly for the second building indicate a good relationship between construction activities on one hand and responses of the walls, existing buildings and ground on the other hand. It can be observed that significant changes in these responses generally occured during the initial excavation stage in which walls were unsupported.

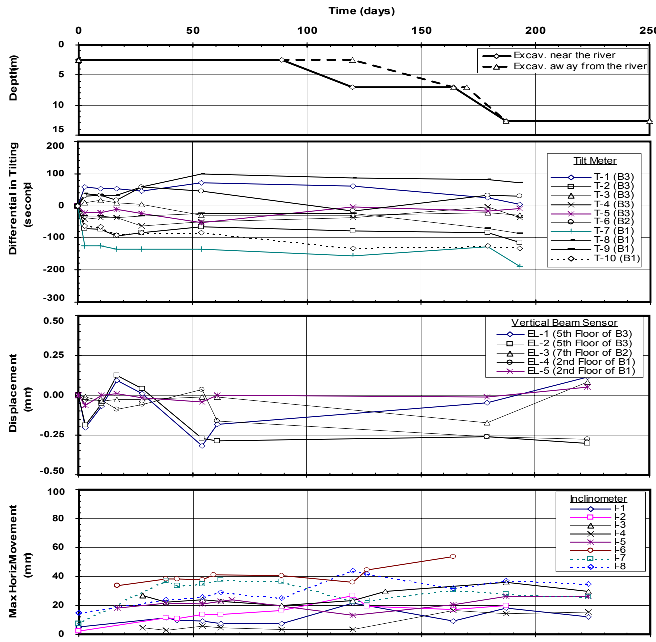

Figure 5a. Instrumentation results with construction time (First Building)

During the period of delay, monitoring of instrumentation was frequently carried out to compare the monitoring data and the trigger values for planning a contingency plan against possible risks to adjacent structures from unsupported excavations to 2.5m depth.

7.1 *Inclinometer monitoring*

For the first building, generally the predicted and measured lateral wall movements were in good agreement, except for those at the top portion of walls (Fig. 6a). The wall-top movements exceeded predicted movements in the case of both buildings due to delay in installing the first level bracing, leaving the walls in cantilever condition for about 2 months or more.

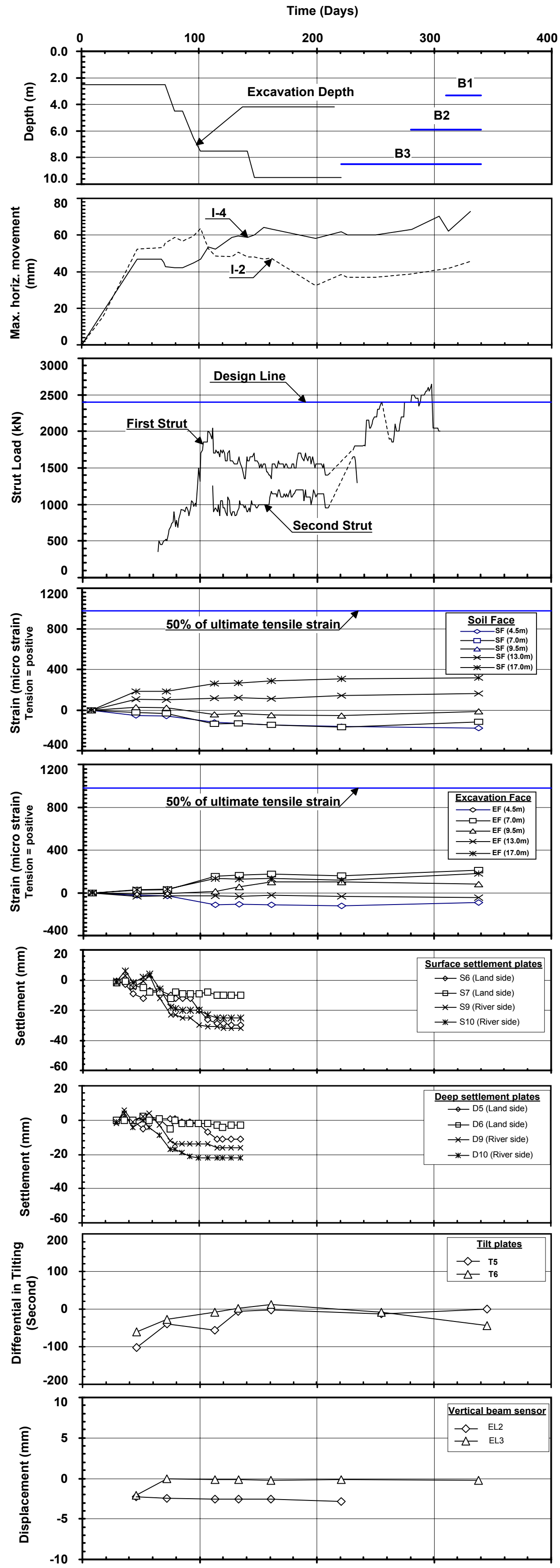

Figure 5b. Instrumentation results with construction time (Second Building)

For the second building, wall movements were monitored weekly, sometimes every 2-3 days when necessary. At I-6 location, wall movements were found to reach the trigger/predicted value, especially at the top of wall when further excavation reached 7.5m depth after installation of the first level bracing.

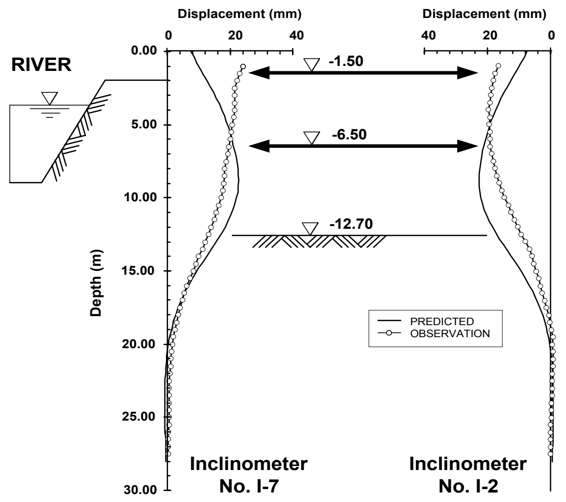

Figure 6a. Lateral wall movements (First Building)

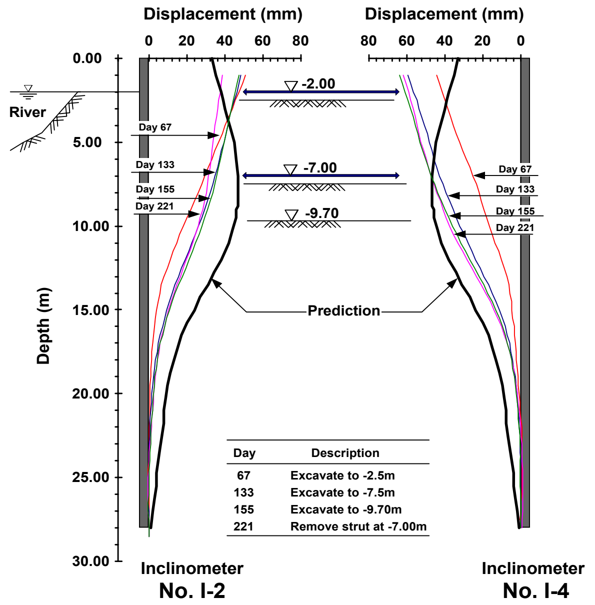

Figure 6b. Lateral wall movements (Second Building)

Lateral wall movement reached a maximum rate of 6.5mm/day. Then a tension crack on the ground surface appeared about 8.0m away from the wall. A close examination of the bracing system indicated that one strut had swayed slightly about 130mm at a distance of 15.0m from the wall. Moreover, the sections of the strut had been connected at an angle. An additional strut was then immediately stacked on the defective one and then wall movements were found to cease. Installation of the second level bracing was immediately carried out for this area.

Regarding unbalanced lateral loading condition, the wall movements of the first building were found unaffected by this condition (Fig. 6a). However, for the second building, monitoring results from I-2 and I-4 suggested that the wall alongside the river had been pushed against the retaining soil by the opposite wall (Fig. 6b).

All inclinometer readings for both buildings indicated that the walls were in fixed-end condition with fixity at depths of about 20.0m. Maximum wall movements after installation of bracing ranged from 0.14% to 0.64% of the corresponding excavation depth for the first building, and 0.27% to 0.83% for the second building. These were also within the range (about 0.1% to 1.2% of excavation depth) of wall movements in other projects completed in Bangkok area.

7.2 *Pressure gauges*

Readings from the pressure gauges installed on struts of the second building were recorded daily. After removal of second level bracing upon completion of base slab and during construction of basement 2 floor, measured strut force of the first level bracing reached the trigger value. However, in general the readings indicated that the bracing system used was adequate.

Table 5. Predicted, allowed and measured strut forces (Second Building).

Item	First level kN/m	Second level kN/m
Predicted by WALLAP	354.5	307.3
Allowed	279.4	332.1
Measured	331.3	206.2

7.3 *Vibrating Wire Strain Gauges*

Strains developed in the wall of the second building were measured by using VWSGs attached to main reinforcement bars of the wall panel where I-4 was also located. A comparison between the measured strains and the allowed strains of the wall was made as shown in Figure 5. The comparison indicated that strains induced in the wall were well below the allowed values.

7.4 *Settlement Plates*

For the first building, a maximum settlement of 16mm was recorded. For the second building, in most cases, ground settlements (surface and deep settlements at 1.0m and 5.0m depth) had been found to stop after installation of the first bracing. In some cases, the surface settlements stopped only after installation of the second level bracing, resulting in maximum settlements up to 30-50mm. The large settlements were found to be associated with surcharges from stockpiles of construction materials and in one case, from unsupported concrete steps, under demolition, which had become detached from the adjacent buildings.

7.5 *Tiltmeter and Beam sensor*

Readings are also shown in Figure 5. Results from monitoring and typical values for maximum building slope or settlement for damage risk assessment are compared in Table 6.

Table 6. 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*	-	
	1/2008**	-	
		-	
Vertical	1/3165*	-	
Beam Sensor	1/985**	-	
Risk Category 2	1/500	10 - 50	Slight: possible superficial damage which is unlikely to have structural significant

* monitoring results for the first building,
** for the second building presented in this paper

8 DISCUSSION

The planned excavation sequence and deep embedment (about 18m and 8m below the excavation and the fixity respectively) of the walls are considered to have contributed to minimizing the effect of unbalanced load conditions, particularly for the walls alongside the river.

Instrumentation results, particularly from the monitoring of wall movements have given effective warning against inproper installation of bracing system and excavation sequences. This allowed improvement or modification of the bracing system where necessary.

No damage was found in adjacent buildings, as confirmed by instrumentation results, particularly those from vertical beam sensors and tiltmeters.

In comparing bracing systems for the first and second building works, stiffness of the first level bracing used for the second building is estimated to be only 50% of that for the first building while expected strut force is 25% less. Moreover single strut arrangement in the first level bracing for the second building is less rigid compared to dual strut arrangement which can act as a composite beam for the first building. Instrumentation on struts confirmed that bracing system used was adequate.

In both cases, early installation of bracing, particularly for the first level is very important for minimizing wall top movements, subsequently reducing accumulated wall movements for further excavation.

9 CONCLUSION

With proper instrumentation, two basement excavations using braced diaphragm walls subjected to unbalanced lateral loading from the river and adjacent buildings were completed.

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

For underground construction work, monitoring with instrumentation plays a major role, especially when delay occurs during excavation. Moreover, overall construcion cost, time and risk can be minimised with the use of instrumentation.

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