Two Case Studies of Collapsed Temporary Excavation using Contiguous Bored Pile Wall

Liew, Shaw-Shong & Loh, Yee-Eng G&P Geotechnics Sdn Bhd, Kuala Lumpur

ABSTRACT

Failure investigation of temporary excavation can be deployed in a systematic manner using the principles of forensic engineering. This paper focuses on the investigation works for two (2) temporary excavation failures in Klang Valley area, Malaysia. Both excavations utilised temporary steel struts propped against partially completed basement structure as lateral shoring supports along the peripheral retaining wall. The two investigations show that it is undoubtedly necessary to have appropriate and sufficient materials sampling and testing as part of the investigation processes if conclusive evidence regarding the failure cause is to be determined. The method, combined with the experience of the investigators, adequate evidences of material defects and numerical modelling of actual construction processes by finite element analyses, provide useful insight in exploring the probable causes of the collapse of temporary excavation and identify the major cause(s) accounted for the collapses. The excavation failure investigation methodology presented in this paper can serve as a guide for the investigation of similar failures and also as a lesson learnt for future excavation projects.

1 INTRODUCTION

Engineering failure investigation is a detective process of determining why and how things went wrong. Forensic engineering can be defined as the application of the engineering sciences to the investigation of failures or other performance problems, with a focus on uncovering causes so that improved facilities can be engineered.

The first stage in determining the failure causes is the investigative synthesis, where all the information gathered is recorded in a logical manner, typically in a report format and in chronological order. The following details are generally required from the Client for undertaking a excavation failure investigation study:

- i. Subsurface Investigation (SI) factual Report.
- ii. Geotechnical design report prepared by the consultant (if available)
- iii. Original topography of the site (before earthworks)
- iv. Earthworks plan and cross-sections
- v. Permanent retaining walls and temporary strutting design analyses and calculations by the consultant or/and contractor
- vi. Detailed construction drawings of the walls and its excavation and strutting sequences
- vii. As-built drawings of the basement walls
- viii. Construction sequences adopted at site
- ix. Weather records of the site
- x. Instrumentation monitoring reports
- xi. Site records and reports of the incident

From the listed information, it is then necessary to determine which information supports or refutes each of the possible failure hypotheses. This may be initially be done by considering general failure causes, such as those related to natural disasters, act of sabotage, material defects, design, construction sequences, or the environment.

Once the probable causes of failure are identified, the information and knowledge learned during the investigation should be listed, for inclusion in the final investigation report. This should include any unusual aspects of the failure, and also any recommendations as to how the failure could be prevented in the future, through improved design, construction controls or good maintenance practices, or through an increased knowledge of the materials used and their properties.

It is normally impossible to conclude with complete certainty what the cause of the excavation failure being investigated was. Instead, there are often multiple factors that contributed to and with one most probable factor triggered the failure, in which probable causes are normally stated in the context of failure investigation. The common probable causes of excavation failure could be one or more than one of the following factors:

- a. Natural Disasters : fire, earthquake, tsunami, tremor, wind, rainfall and flood
- b. Act of Sabotage: explosive substances
- c. Material Defects: reused steel strutting sections with poor conditions, concrete properties
- d. Design: modelling and design parameters, robustness and ductility
- e. Construction: sequences of works, excavation depth
- f. Maintenance: drainage system, no timely review of instrumentation results

In general, the investigation procedure in geotechnical failure are listed below but not limited to:

- i. Check safety factor of the original design
- ii. Check the as-built construction for any deviations from original design
- iii. Identify design shortcomings, material defects, workmanship deficiencies, if any
- iv. Interview design team, construction management team, site personnel and eye witnesses
- v. Consult other experts if required, for matters beyond the investigator's expertise or knowledge of the facts
- vi. Identify possible collapse scenarios and rationalise conflicting facts or evidences
- vii. Determine the major contributory and triggering factors that cause the collapse
- viii. Conduct advanced non-linear analysis /tests to ascertain the collapse mechanism
- ix. Confirm the collapse mechanism with those from facts and evidences
- x. Write report

This paper discusses the approach taken to investigate the two case studies of distressed open excavation with contiguous bored pile wall (CBP) and steel strutting system at Klang Valley area in Peninsular Malaysia. During the course of investigation, interesting processes of gathering factual evidences, verification of conflicting facts, analysis for the sequence of events and finally arriving at conclusive findings with convincing evidence will be discussed in the paper. Last but not least, it is extremely useful to learn from these two failure cases to improve construction safety of future projects.

2 CASE HISTORY 1 (Liew & Khoo, 2008)

2.1 Project Background

This case history involved construction of a two-storey basement in an urban area. The scope of investigation was to evaluate the conditions of a distressed temporary shoring structure, investigate the probable causes and subsequently to propose remedial options.

Figure 1 shows the location of the project site and the adjacent land lot which had been affected due to ground distresses.

Temporary shoring structure consisting of CBP wall propped by raking struts against lower basement slab was proposed by the contractor to provide peripheral support for a 10.5m deep excavation at western boundary of the project site. The designed CBP wall was 16m long 750mm diameter bored pile with top cut off level at RL 54.6m. In order to facilitate the CBP installation,

12m long temporary steel sheet piles (type FSP IIIA) were installed at retained ground at RL59.0m with about 0.7m offset from project site boundary to provide sufficient working platform and as the temporary shoring support for the exposed 4.4m temporary excavation (from RL59.0m to RL54.6m). Figure 2 shows the details and cross-section of the proposed alternative temporary shoring system and permanent retaining wall.

After completion of CBP wall installation, excavation was continued from RL54.6m to RL52.0m. Ground distresses at the adjacent retained platform in the forms of ground subsidence, tension cracks and excessive deviation of the CBP wall were observed. From the site observation, deviation of CBP wall was likely caused by the over-excavation of the temporary passive berm with localized deep pile cap excavation in front of the wall without timely installation of the planned raking strut. The incidence had affected the adjacent property lot with considerable ground distresses and also resulted in structural damages to the CBP wall.



Figure 2. Cross-section of the proposed alternative retaining walls.

2.2 Site Conditions

From the earthworks as-built drawings, the ground level before excavation was relatively flat with levels ranging from RL54.8m to RL56.5m and a steep soil slope (1V:1H) of about 3m high sloping from adjacent lot (at RL 59.0m) towards the proposed site. Based on the topographical survey plan of adjacent lot, which was believed to be the condition before the earthworks, part of the proposed site (i.e. at north-western side) is of sloping terrain from approximately RL65m to RL56m. There was a 6m high cut slope existed within this sloping ground as indicated in Figure 3.



Figure 3. Topographical survey plan of adjacent lot (subjected to disturbance before reaching the finished level).

Based on the pre-development topographical condition as shown in Figure 4, the contour lines for both the adjacent lot and the project site range from RL54m to RL47m. As such, it was evidenced that earthworks had previously been carried out at these areas to raise the building platform level to RL 59.0m and RL 56.0m for the adjacent lot and the project site respectively. Both of the sites are on filled platforms. Particularly, the distressed area was primarily located at the valley where thicker fill was placed. High potential of saturation of fill due to perched groundwater seepage after filling in the previous valley terrain can be expected if subsurface drainage is not provided. In addition to the topographical map, assessment on the piling information was carried out to reveal the soil consistency profile within the site as shown in Figure 5. It was found that the distressed ground and retaining wall areas correspond well with the expected thicker fill and deeper weathering profile at the valley area.



Figure 4. Contours of original ground and subsequently proposed earthwork levels.



Figure 5. Interpreted contours of hard stratum from piling penetration information.

2.3 Site Inspection and Mapping

It was observed that continuous tension cracks appeared at varying distance away along the sheet pile wall of approximately 73m long. The tension cracks were more distant away from the sheet pile wall at the southern end and became closer to the sheet pile walls toward northern end. At the time of inspection, more extensive excavation was carried out at the southern portion than the northern portion. The platform for the backyard car park of the adjacent lot had shown ground subsidence with tension cracks as shown in Figure 6 and Figure 7.



Figure 6. Site conditions of adjacent lot after the incidence of wall movement and ground distresses (Southward view).



Figure 7. Site conditions of adjacent lot after the incidence of wall and ground distresses (Northward view).

Site mapping had been carried out on the observed tension cracks and tilted sheet pile wall. Much more tension cracks were observed at the southern region. In this particular location, the sheet pile wall was seriously deviated outward relatively to the initial wall alignment. Efforts were made to map the crack lines by using measuring tape and slope meter. Figure 8 shows the details of mapped tension cracks. Generally, the worst tension cracks were measured with crack width of up to 400mm and shear drop of 500mm between the two dislodged earth blocks. At the critical location (Gridline G), the major crack line was measured at approximately 6.2m from the original fence line. While, the furthest crack line was measured at approximately 12m from the original fence line. In addition, the overall tilt angle of subsided platform at this area was crudely measured to be about 12° as shown in Figure 6. Sheet pile wall had also moved outward about maximum 1.2m from the initial wall alignment (at Gridline G). On the other hand, the measured pile top deviation of contiguous bored pile wall is also shown in Figure 8.

During the emergency repair work, the vibration effect of re-installing the temporary sheet piles wall by vibro-hammer had caused further tension cracks at the backyard car park platform. Therefore, it was suspected that the platform was of filled ground, which might not be well compacted as the effect of vibration might have caused soil densification and aggravated the creep movement.



Figure 8. Crack mapping layout and CBP wall movement.

2.4 Subsurface Investigation

Before the wall and ground distresses, two stages of subsurface investigation (SI) works were carried out at the proposed site. The second stage SI works is the additional SI conducted at the perimeter western boundary for the alternative basement wall design by the contractor. At that time, no much attention was given in identifying the weak deposits between the original ground and the platform backfill. The SI layout is shown in Figure 9.

After the incidence, additional three boreholes were sunk within the distressed wall area to investigate the subsurface profile and to install instruments for substructure construction monitoring. In particular, a layer of 6-9m thick of very soft to soft sandy/silty clay (SPT-N \leq 4) was encountered between RL52m and RL43m as detected in the few boreholes near to the distressed wall area.

Generally, SPT-N values of the subsoil range from 3 to 6 at the top 13m of the subsoil and gradually increase with depth thereafter. This implies that the top layer of subsoil is most probably of fill material underlain by the soft deposits in the valley. An interpreted cross-section of subsurface profile at the distressed wall area is shown in Figure 10. The groundwater levels recorded in the boreholes and observation wells were generally higher at the western side and lower towards the eastern side of the project site, which tallies with the suspected flow path in the valley during predevelopment condition.



Figure 9. Layout of subsurface investigation boreholes.



Figure 10. Interpreted subsurface profiles at the distressed wall area.

2.5 Back Analysis

In order to confirm the probable causes of ground distresses and wall movement, Finite Element (FE) analysis using computer software "PLAXIS" was performed independently based on the interpreted subsurface profile as shown in Figure 11. The construction sequences of excavation were simulated in the FE analysis.

At the analysis stage where over-excavation in front of the wall was carried out, the analysis results revealed that the retained earth platform displaced excessively in the horizontal and vertical (settlement) directions with the temporary sheet pile retaining wall moving forward. As part of the lateral resistance to the temporary retaining walls by the passive berm was removed before installation of raking strut, over-excavation of this passive berm had reduced the lateral resistance to the sheet pile wall and subsequently mobilised the structural strength of the retaining walls beyond serviceability state condition reaching towards the ultimate limit state condition. The excessively displaced temporary sheet pile wall had induced additional lateral force to the installed contiguous bored piles (CBP) walls. The high induced flexural stress unavoidably damaged the CBP pile. The results of FE analyses (see Figure 11) reasonably well agreed with the measured wall movements and ground deformations (e.g. tension cracks, settlement and depression).



Figure 11. Results of finite element analysis.

2.6 Remedial Design

The immediate remedial measure was to temporarily backfill the excavation adjoining the distressed area to the top of CBP wall with temporary stabilising berm (1V:1H). A variety of permanent remedial options have been explored. Finally, internal strutting against permanent basement structures was adopted to provide a safe and cost effective solution.

The remedial works carried out included installation of additional row of 18m long sheet piles behind the deviated CBP area to stabilise the deteriorating retained ground. Temporary strutted cofferdam was constructed to facilitate the localized lift pit excavation. Two layers of temporary horizontal strut were used to prop the sheet pile wall against the partially completed permanent basement structures as shown in Figure 12. Excavation was only allowed to be carried out in stages after the struts were put in place. Once the final excavation level (B2) was reached, the deviated CBP wall was cut off at that level and verified with integrity testing using (both low and high strain dynamic pile tests) to confirm the structural integrity. Fortunately, most of the damages of the deviated CBP wall were well above the B2 level. Cast in-situ reinforced concrete wall was constructed with the exposed starter bars from the intact CBP wall. The finished permanent basement structure is shown in Figure 13.



Figure 12. Cross-section of remedial works.



Figure 13. Cross-section of permanent basement structure.

2.7 Summary of Findings and Lessons Learnt

The investigation results and lessons learnt from this case study revealed the following findings:-

- a. The distressed ground was located over a natural valley where thicker fill was placed over the previous soft deposits without proper engineering treatment to form building platform. Soft deposits at the lower part of the valley and potential concentrated underground seepage are common in hilly terrain and should not be overlooked. Desk study of pre-development ground contours to identify potential geotechnical problems is highly recommended.
- b. Filling over valleys without proper site clearing, removal of unsuitable soft deposits and compaction could result in highly unstable backfill for any open excavation. It is important to thoroughly investigate the subsoil condition beneath the fill. Otherwise, proper treatment to the low-lying ground before the development earthworks shall not always be assumed.
- c. Occurrence of tension cracks during initial open excavation and installation of sheet piles suggested that the underlying subsoil and at the valley area are inherently vulnerable to ground disturbance and hence are prompted to distressing.
- d. The existence of soft compressible material at the valley area was further confirmed during additional subsurface investigation and localized deep pile cap excavation when reaching the final excavation level.
- e. Original topographical features are the important design consideration for excavation stability and remedial strategy. In these case studies, natural valley with soft deposits was not detected during design stage causing cost escalation as a result of costly remedial work.
- f.Perched groundwater regime can occur in backfilling over natural valley leading to unfavourable behaviour of backfill.
- g. Excessive removal of passive berm with localized pile cap excavation without installing the planned strutting resulted in loss of wall support and led to excessive ground movement.

3 CASE HISTORY 2

3.1 Project Background

This case history involved construction of two-level basement in city area using CBP wall with temporary strutting. About 40m stretch of CBP wall collapsed, after the water pipe burst incident at the back lane reported 3 hours ago as shown in Figure 14.



Figure 14. Collapsed CBP between Gridlines C and H.

Figure 15 shows the overall retaining wall section for the above-mentioned development. The affected walls in this investigation were 1000mm diameter CBP wall installed at 1075mm centre-to-centre (c/c) and unreinforced CBP wall of 1200mm diameter installed at 1500mm c/c at a distance of 2.4m behind this 1000mm diameter perimeter CBP wall. Another row of CBP wall was installed for localised lift core pit excavation.



Figure 15. Section of CBP Walls.

3.2 Review of Information and Technical Clarification Meeting

A technical clarification meeting was held between the contractor and consultant (Professional Engineer engaged by the contractor for temporary works) with the presence of Loss Adjuster to discuss, clarify and confirm on the supplied information, technical aspects of the design and discrepancies identified in the information provided.

During the site visit, it was observed that central portion (Gridlines C to H) of CBP wall had collapsed; single inclined struts at Gridlines G and H were observed buckled and disconnected from steel corbels while steel corbels at Gridlines D, E and F mounted at Basement 1 slab were completely sheared off. The unreinforced CBP wall and perimeter CBP wall fell rotationally and leaning towards the excavation face as shown in Figure 16.

Based on the photographs taken before the wall failure and information provided by the contractor, the as-built strutting layout was re-constructed and presented in Figure 17. Figure 17 shows that horizontal tie beams were installed to provide lateral restraint for single inclined strut but no bracing system was provided to properly transfer and balance the restraint force. The lateral restraint system provided by horizontal tie-beam was incomplete in accordance with BS5950-1:2000 where Clause 4.7.1.2 in the BS5950-1:2000 states that a restraint should be connected to an appropriate triangulated bracing system for the overall internal stability of the strutting system.

Figure 18 shows post-installed bolts were adopted for steel corbels at Gridlines D, E and F and cast-in bolts were used for steel corbels at Gridlines G and H. The steel corbel was lifted-up with snapped post-installed bolting connection after wall collapse indicating that the steel corbel corbel connections for struts at Gridlines D, E and F are the weakest link of the entire strutting system.

Other than that, Figure 19 also shows that the cut holes at steel corbel for Strut F were not complied with the allowable dimension permitted by Table 33 of BS 5950-1:2000. This again would lead to uneven distribution of strut loads onto bolting connection possibly resulting into progressive failure of bolts which may further reduce the overall capacity of the strut.



Figure 16. Site condition and strutting details after CBP wall failure.



Figure 17. As-built strutting layout.



Figure 18. As-built corbel and anchored bolts details



Bolt shear off

Steel Corbel with Post-installed Bolting Connection was Lifted-up After Wall Failure

Post-installed Bolts After Wall Failure



As-built Details of Steel Corbel for Strut-F

Figure 19. Steel corbel with post-installed bolting connection after wall failure.

It was noticed that the sequences of works modelled by the consultant were not strictly followed by the contractor at site as presented in **Error! Not a valid bookmark self-reference.** and **Error! Not a valid bookmark self-reference.**



Figure 20. Modelled sequences of works in FE analysis by the consultant (same as FE analyses Cases O and P1 as discussed in Table 2).



Figure 21. Sequences of works executed by the contractor at site (Same as FE analyses Cases P2 and P3 as discussed

in Table 2)

3.3 Subsurface Investigation

Locations and subsoil profiles for the relevant boreholes located near the collapsed CBP wall are shown in Figure 22 and Figure 23.



Figure 23. Subsoil profiles.

No laboratory strength test was specified for determination of effective stress parameters in first, second and third SI works. As such, laboratory testing was performed in fourth SI work to determine

the effective stress parameters of subsoil and the interpreted results of the relevant engineering parameters were summarised in Table 1.

Borehole (Soil Sample)	Depth of sample	Soil Type (Based on BS 5950)	Shear Box Test		Consolidated Undrained Triaxial Test (C.I.U.)		
			c' (kPa)	□' (°)	c' (kPa)	□' (°)	
BH-G1 (UD1)	2.50m b.g.l	Silt	4	35	Not enough soil sample for C.I.U. test		
BH-G1 (UD2)	9.00m b.g.l	Silt	4	25	1	32	
BH-G3 (UD1)	1.50m b.g.l	Sandy Clay	4	28	Not enough soil sample for C.I.U. test		
BH-G3 (UD3)	4.50m b.g.l	Silt	3	31	Not enough soil sample for C.I.U. test		

Table 1. Summary of laboratory test results for shear box and C.I.U. test

Figure 24 shows upper and lower bound envelopes of shear strength parameters from drained shear box were c' =4kPa and ϕ' = 35° for former while c' =4kPa and ϕ' = 27° for the latter. For this investigation, moderate strength parameters of c' = 4kPa and ϕ' = 30° were selected for the overburden soil layer. Same effective friction angle of 30° was adopted in the consultant's design except for higher apparent cohesion c' value of 10kPa was adopted.



Figure 24. Interpreted shear strength parameters from drained shear box test.

It is evidenced that the adopted high apparent cohesion is not justified from the interpreted strength test results above. The apparent cohesion has significant impact to the assessment of lateral earth pressure. A small increase in the apparent cohesion will reduce significantly the lateral earth pressure. Hence, it would be necessary to be prudent in assessing the cohesion parameter.

From the available water standpipe readings, the groundwater table fluctuated between RL 33.16m and RL 33.67m. For design check and back analysis purposes, initial groundwater is adopted at RL33m which is same as consultant's design assumption for normal design groundwater table and within the reasonable range of the measured data.

3.4 Finite Element (FE) Results

For fair assessment on the consultant's design, the adopted parameters such as shear strengths (c', ϕ '), deformation modulus (E), permeability (k) and initial groundwater table in the first checking (Case O) were similar to the consultant's analysis. In order to study the effect of soil properties (shear strength and deformation modulus) and construction sequences to the performance of retaining wall system, the FE analysis cases as shown in Table 2 are performed.

Case		*Strength Parameters	**Groundwater Level	Remarks
ses of works in FE nsultant (Figure 20)	0	c' = 10kPa, ϕ '= 30° Note: Consultant's adopted strength parameters	RL 33m	Checking on base design where passive berm of 4m wide and 5m high was not modelled
Modelled sequenc analysis by the Cor	⁻ 1	c' = $4kPa$, ϕ '= 30° Note: Moderately strength parameters interpreted by G&P	RL 33m	Parametric study 1 adopting moderately strength parameters where passive berm of 4m wide and 5m high was not modelled
orks executed by the te (Figure 21)	2	c' = 10kPa, ¢'= 30° Note: Consultant's adopted strength parameters	RL 33m	Parametric study 2 adopting Consultant's strength parameters where passive berm of 4m wide and 5m high was modelled according to actual construction sequences implemented by the Contractor at site
Actual sequences of w Contractor at si	-3	c' = $4kPa$, ϕ '= 30° Note: Moderately strength parameters interpreted by G&P	RL 33m	Parametric study 3 adopting moderately strength parameters where passive berm of 4m wide and 5m high was modelled according to actual construction sequences implemented by the Contractor at site

* For overburden soil only. Properties for rock layer remained unchanged.

** The initial groundwater table behind the retaining wall is assumed to be constant at RL33m. After every stage of excavation, the ground water level within the excavation site is re-defined to the surface of the new formation level to model the drawdown of water table within the excavated area. Hence the steady stage seepage on the active and passive sides of the retaining wall at each construction sequence is considered.

Table 2. FE analysis cases.

Summary of design checks based on the normal design water level at RL33m for all the analysis cases is tabulated in Table 3 and reveals the following findings:

- a. Design of unreinforced CBP wall for all cases of the section is adequate for moment resistance. However, the moment capacity of unreinforced CBP wall will drop drastically if any cracking was formed when any of the structural thresholds are exceeded and hence the back calculated moment of unreinforced CBP wall can be unreliable.
- b. Design of perimeter CBP wall for all cases is adequate except Case P3 in which the design is inadequate for shear resistance.
- c. Design of lift core CBP wall for all cases is adequate except Case P3 in which the design is inadequate when the actual construction sequences at site was used, which is significantly deviated from the assumed construction sequence in FE model and the apparent cohesion for overburden soil is reduced from 10kPa to 4kPa as justified from the interpreted strength parameters.
- d. Design of strutting members is adequate based on extreme strut force predicted by the consultant. However, design of strutting members for Cases O, P1, P2 and P3 is either inadequate or unsafe based on extreme strut forces predicted in the independent analyses by the investigator.
- e. There are an increase of approximately 24% in strut force and seven times larger in the maximum predicted wall deflection for perimeter CBP when the apparent cohesion for overburden soil is reduced from 10kPa to 4kPa and actual construction sequences are deviated from site execution. These findings reveal the apparent cohesion is a very sensitive factor on the performance of CBP wall.
- f. Adoption of high apparent cohesion in FE analysis by the consultant had caused the underestimation of strut force and virtually led to insufficient provision of strutting members (as per findings from Cases P1 and P3).
- g. Large wall movement predicted in Case P3 reveals the adoption of moderately strength parameters in retaining wall design analysis would be able to identify the potential distress of existing utilities due to large ground movements which had happened in this project.

Wall Type	Max. ultimate Capacity	Max. Results from FEM (Note: normal groundwater level was RL33)	Consultant's Analysis c' = 10kPa, ¢'= 30° (Without passive berm)		Case O (Independent Analysis). c' = 10kPa, φ'= 30° (Without passive berm)		Case P1 (Independent Analysis)- c' = 4kPa, ¢'= 30° (Without passive berm)		Case P2 (Independent Analysis). c' = 10kPa, \$'= 30° (With passive berm)		Case P3 (Independent Analysis). c' = 4kPa, ¢'= 30° (With passive berm)	
			Extreme Value from FEM	Remark	Extreme Value from FEM	Remark	Extreme Value from FEM	Remark	Extreme Value from FEM	Remark	Extreme Value from FEM	Remark
Unreinforc ed CBP wall (1200mm size @ 1.5m c/c)	-	Deflection	40.8mm	- 42 -	36mm	<u>.</u>	53mm) w	77mm	<u> </u>	814mm	
	256kNm	Service Bending Moment per pile.	132kNm	Adequate	93kNm	Adequate	50kNm	Adequate	61kNm	Adequate	80kNm	Adequate
Perimeter CBP wall (1000mm size @ 1.075m c/c)	•	Deflection	30.4mm	- 55	36mm	. et	41mm	1 et	81mm		619mm	
	1700kNm	Service Bending Moment per pile	640kNm	Adequate	856kNm	Adequate	851kNm	Adequate	626kNm	Adequate	1122kNm	Adequate
	1233kN	Service Shear Force per pile	700kN	Adequate	391kN	Adequate	428kN	Adequate	462kN	Adequate	1069kN	Inadequate
Liftcore CBP wall (1000mm size @ 1. 5m c/c)	+	Deflection	2.6mm	58	9mm	24	9mm	<u>a</u>	12mm		17mm	×.
	1260kNm (Note: 16T25)	Service Bending Moment per pile	135kNm	Adequate	247kNm	Adequate	217kNm	Adequate	626kNm	Adequate	1257kNm	Inadequate
Strut Force (For strut spacing of 8.4m)	3375kN	Maximum induced force per strut	1856kN	Adequate	2890kN	Inadequate	3452kN	Unsafe	2654kN	Inadequate	3301kN	Inadequate

Notes:

The extreme induced force is compared with ultimate capacity and service (safe working load) capacity on checking the adequacy of design. In brief, the adequacy of design is classified as follows:

(1) Adequate Design: Extreme induced force \leq Service capacity;

(2) Inadequate Design: Service capacity < Extreme induced force \leq Ultimate capacity and;

(3) **Unsafe** Design: Extreme induced force > Ultimate capacity;

Where service capacity = Ultimate capacity / 1.4

Table 3. FE analysis results.

3.5 Back Analysis

In the second part of this investigation, the construction sequences were modelled based on furnished information which was clarified and confirmed by the consultant and contractor during several rounds of clarification meeting. Back analysis had been carried out based on the instrumentation data for Inclinometer INC-4 installed inside the collapsed CBP wall. Attempt was made to calibrate the predicted wall deflection with instrumentation data of Inclinometer INC-4 to assess the behavior of the CBP walls and investigate the possible mechanisms of strut failure and/or wall collapse.

From Figure 16, it was sufficiently evident that all the post-installed bolts at struts D, E and F were completely sheared off. Hence, these struts were modelled in FEM back analysis as elasto-plastic anchor element.

Repetitive water pipe burst incidents at the back lane had contributed to massive movement of CBP wall prior to the CBP wall collapse incident as highlighted in Figure 25. Therefore, water pipe burst incidents happened on Day 206 and Day 227 were taken into consideration in FEM analysis for best fitting the inclinometer readings. Figure 26 and Figure 27 show the FE mesh and predicted CBP wall deflections for back-analysis.







Figure 26. FE mesh of back-analysis.



Figure 27. Modelled CBP wall deflections Vs monitoring results for INC-4.

From the assessments mentioned in the preceding section, it would be possible to deduce the following sequence of events and development of wall collapse mechanism as below:

- a. The original excavation design with optimistic strength parameters has suffered excessive ground movements, which subsequently led to potential distresses of water pipe after experiencing the intolerable deformation.
- b. The water leakage caused increase of lateral wall pressure and induced more lateral wall movement and more leakage, hence the same for the deformation of the retained ground.
- c. The strut forces also increased as a result of the increase in lateral wall pressure.
- d. The steel corbel connections at Struts D, E and F, especially the large hole cutting for accommodating the poorly positioned bolts, would have resulted in uneven distribution of shear force from the struts. Hence, the total shear resistance with probable progressive failure of bolts would be lower than the simple summation of the shear resistance of individual bolts (3375kN). With the increasing strut load, the bolts at the steel corbels sheared off and led to total loss of support for the CBP wall.
- e. It is likely that Struts E and F had given way first and affected the Struts D (bolts at steel corbel of Struts D, E and F were sheared off) and the other two longer Struts G and H (Struts G and F buckled and dislodged from the corbels).

Figure 28 and Figure 29 illustrate the important chronological events related to the CBP wall collapse.







Figure 29. Flow of important events related to CBP wall collapse (Part 2 of 2).

3.7 Summary of Findings

The findings of the independent investigation for the CBP wall collapse are summarised as follows:

- a. The apparent cohesion adopted by the consultant is on the optimistic side without verification from laboratory test which led to unexpected large wall displacement and increase in strut forces when experiencing increase of water pressure behind the wall as a result of water pipe burst incident.
- b. The actual executed construction sequences at site by the contractor were not conformed to consultant's modelled construction sequences.
- c. Failure investigation of temporary excavation can be deployed in a systematic manner using the principles of forensic engineering. This paper focuses on the investigation works for two (2) temporary excavation failures in Klang Valley area, Malaysia. Both excavations utilised temporary steel struts propped against partially completed basement structure as lateral shoring supports along the peripheral retaining wall. The two investigations show that it is undoubtedly necessary to have appropriate and sufficient materials sampling and testing as part of the investigation processes if conclusive evidence regarding the failure cause is to be determined. The method, combined with the experience of the investigators, adequate evidences of material defects and numerical modelling of actual construction processes by finite element analyses, provide useful insight in exploring the probable causes of the collapse of temporary excavation and identify the major cause(s) accounted for the collapses. The excavation failure investigation methodology presented in this paper can serve as a guide for the investigation of similar failures and also as a lesson learnt for future excavation projects.