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Analysis of Excavation Collapse at PM1 Thermal Power Plant – A Case Study

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Abstract Deep excavation in soft soil has many potential risks due to overall instability, basal heave instability, failures due to piping, boiling, large buckling of struts, yielding of a wall, excessive wall deflection, excessive ground settlement, and adverse effects on adjacent structures. In fact, incidents or collapses during the construction of deep excavations have been observed around the world, but only a few cases are reported due to contractual or other reasons. Those that are reported are usually of such a large scale and severity that they receive public attention, and even then, there is generally limited information available. Chan N. F. (2012) has documented 58 severe collapses or incidents during deep excavations in Hong Kong and worldwide. As the desired excavation depth increases, the above risks at all stages of excavation may occur individually or in combination. Therefore, this article analyzes the construction process and incidents that happened in Pond B, Phu My 1 Thermal Power Plant, Vietnam, to find the causes and draw lessons learned. For many different reasons, this item encountered many problems. However, this item was still completed. The article uses the Plaxis 2D ultimate to evaluate incidents and follow countermeasures. Analysis results show that the Pond B excavation process encountered problems due to the base heave instability, unsafe strut-wall system structure, excessive ground settlement, excessive lateral displacement of the ground, and poor weld quality. The contractor has successfully implemented countermeasures that have proven to be effective.

Keywords Excavation Analysis, Deep Excavation, Pond B, Excavation Collapse, Failure, Case Study

1. Introduction

Previous concepts for braced excavation design only considered the selection and calculation of the strut-wall system [1, 2]. Assuming the strut-wall system behaves like an elastic structure, the strut supports the wall, and all are subjected to lateral soil pressure. Therefore, the design only focuses on the deformation and load-bearing capacity of the strut-wall system and pays little attention to the settlement and movement of the soil both inside and outside the excavation. Many computer software, such as RIDO, WALLAP, FREW, and many others, have been proposed to ease the computational work. El-Nimr et al. [3] evaluated the normalized deflection of wall systems. Research results show that they are affected by the 3D shape of the excavation, the excavation sequence, and the time effects. Moorman [4] analyzed the relationship between the displacement of the wall system and the settlement of the ground, the strut-wall system used, and the excavation method employed. The author utilized a database of over 530 case studies to investigate the main parameters that influence the performance of deep excavation in soft soil. Tang and Tung-Chin [5] used nonlinear optimization techniques to analyze the deformation of the wall system under different soil conditions. Results indicate that the wall deflection can be accurately back-figured for all excavation cases, hypothetical or actual, at each stage. Jan et al. [6] proposed an Artificial Neural Network (ANN) model to predict the displacement of the wall system according to the excavation sequence. The calculation results of the previous excavation step are used as input data to predict the displacement of the next excavation step. Hsii-Sheng et al. [7] proposed jet grouting to reinforce the bottom of the excavation to limit the displacement of the wall system.

Many studies [8, 9] consider the failures or collapses of excavations during construction as a disaster. At worst, they endanger workers and adjacent structures. The area around the excavation may experience excessive ground settlement, so structures may be significantly damaged. Because the failures or collapses can be severe, prevention methods are necessary. Excavation stability analysis includes overall stability, piping and boiling, and bottom heave analysis in addition to the strut-wall design as per previous concepts. Burland et al. [8] observed a 29 m deep excavation in Oxford. They found that the ground settlement within 60 m around the pit was up to 100 mm, the inward displacement of the retaining wall was up to 200 mm, and the bottom heave was up to 100 mm. Chang-You [9] synthesized theories of deep excavation behavior and proposed methods to analyze overall stability, the influence of ground settlement on adjacent structures, and bottom heaving.

Bjerrum and Eide [10] and other researchers like Terzaghi (1943), Eide et al. (1972), and Goh (1994) described the basal heave mechanism that occurs at the bottom of the excavation. They proposed empirical formulas to calculate the safety factor for bottom heave stability of excavations to provide prevention solutions.

Liyanapathirana and Nishanthan [11] observed the impact of deep excavation on ground settlement in adjacent areas. They used the finite element method to analyze the behavior of a single pile affected by ground movement outside the excavation. This method can simulate the excavation process, including excavation steps, lowering the water table, and installing struts. Analysis results indicate that increasing axial load has a negligible effect on pile behavior. However, the conditions of pile head fixation, stiffness, and distance of the diaphragm wall system have a significant influence on the behavior of piles adjacent to the excavation. In their study, Hu et al. [12] examined the design and construction process of an excavation adjacent to the Shanghai Metro line. Based on their analysis, they proposed several solutions, which included the use of a diaphragm wall combined with an internal support system, consolidation pumping, soil cement pile, and a reasonable excavation process. As a result, the excavation and high-rise building were completed, and the impact on the subway line was effectively controlled. Mitew [13] used the finite element method to assess the impact of deep excavation on adjacent structures, including the extent of the impact area and the settlement value of the surrounding ground surface. Bhatkar et al. [14] analyzed the behavior of soft ground surrounding the East Indian metro tunnel on the banks of the Hooghly River—the analysis aimed to determine the most suitable tunneling method. Vertical and horizontal deformations in the vicinity were monitored during construction to ensure that nearby structures were safe. The predicted strain was to be relatively high when compared to the measured strain. By back-calculating from the measured deformation, the study found a conservative estimation of stiffness during the research and anisotropic soil behavior. This research demonstrates that numerical analysis can be an effective tool for predicting soil behavior during excavation and installation of support systems. Rybak et al. [15] analyzed failures during strut-wall construction, tunneling, soil unloading caused by excavation steps, and horizontal loading during the pre-stressing of struts or anchors.

Hashash [16] uses the nonlinear finite element method to determine ground deformation based on excavation depth. In a study conducted by Bin-Chen et al. [17], they use the finite element (FE) method to analyze the three-dimensional effects of a deep excavation on wall displacements in loose to medium-dense sands. The study found that using the Mohr-Coulomb model and the soil modulus obtained from in-situ dilatometers provides reasonable predictions of wall displacements due to excavation.

Hashash [18] compared two back-analysis methods to analyze an excavation in Chicago. The first method is a genetic algorithm (GA), a parameter optimization approach. GA is a stochastic global search technique for optimizing an objective function with linear or nonlinear constraints. The second approach is the Self-learning Simulations (SelfSim) method, a back analysis technique. It combines finite element methods, continuously evolving material models, and field measurements. Research findings indicate that the surface settlement calculated using the SelfSim method is consistent with the observed settlement, making it a more reliable approach than the GA optimization method.

In a study conducted by Lim et al. [19], the performance of different constitutive soil models was evaluated for analyzing deep excavations under undrained conditions. The models tested were the modified Cam clay model, the hardening soil model, the hardening soil with small strain model, the Mohr-Coulomb ($\varphi = 0$), and the undrained soft clay model. The results showed that the modified Cam clay model, the hardening soil model, the hardening soil small strain model, and the Mohr-Coulomb ($\varphi = 0$) with $E_u / s_u = 500$ gave accurate predictions for wall deflections, which were in line with field measurements during the final stage of excavation. However, none of these models could accurately predict ground settlement. On the other hand, with parameters directly obtained from tests, the undrained soft clay model could predict both wall deflections and surface settlements with high accuracy. A study by Hashash et al. [20] proposed a numerical simulation model called Artificial Neural Network (ANN). This model is based on a constitutive soil model established from the given lateral wall deflection and surface settlement. The ANN model extracts the soil's constitutive behavior directly from field measurements of excavation response. The resulting soil model is then used in numerical analysis, which provides accurate ground deformation and can be used to predict future excavations or later excavation stages. In addition, the model can be continuously improved by incorporating additional field information.

As per the above discussion, five types of failures can occur during the construction of deep excavations. These failures are related to four specific problems that must be controlled, namely: (1) overall stability of the excavation, (2) deformation and bearing capacity of the strut-wall system, (3) excessive settlement of the ground around the excavation, (4) piping and boiling, and (5) basal heaving. The authors have highlighted the importance of selecting the correct analysis method and constitutive soil model, as well as considering various factors that may influence the excavation behavior. Carrying out deep excavation in soft soil is becoming increasingly complex as construction sites are often situated in densely populated urban areas, surrounded by numerous ongoing construction projects.

2. Materials and Methods

2.1. Pond B Excavation

The PM1 Combined Thermal Power Plant is a large-scale construction project that was built in Vietnam back in 1997. During that time, advanced construction technology was not widely implemented in Vietnam, especially regarding deep excavation in soft soil. The construction area is situated along the banks of the Thi Vai River in Baria-Vung Tau province, known for its riverside soft soil that gets flooded during high tide. One of the items of the project is the Pond B excavation located at the end of the cooling water discharge channel. Its primary purpose is to receive water from gas turbines, which flow through four reinforced concrete culverts, then distribute the water into six steel pipes placed under the water and discharged to the Thi Vai River. Unfortunately, Pond B excavation experienced multiple failures and had to be built for a third time before completion.

The Pond B excavation, as shown in Figure 1, measures 30 x 12 meters with a depth of 9 meters from the ground level. It is situated right on the flooded river bank, which requires the entire area to be filled above the tide level at an elevation of +2.3 meters. The underground water level is currently at the ground level, which is 1.03 meters below the filling level.

The Geological Survey and Construction Company 4, having an office at 65 bis, Mac Dinh Chi Street, District 1, Ho Chi Minh City, carried out field investigations and laboratory tests. In December 1998, they prepared and submitted an investigation results report to Mitsubishi Heavy Industries Company. Table 1 summarizes the soil properties of boreholes BH43, BH 44, and BH 45 near the construction site. These soil data were also back-analyzed and evaluated to ensure accuracy during the modeling and calculation process.

Table 1. Soil data from the site investigation

Soil Layer	Symbol	N	D	W	γ	Δ	ε_0	W_L	W_P	I_P	I_L	Direct shear test	
				(%)	(g/cm^3)	(g/cm^3)						φ^0	C Kg/cm^2
1	MH	0	0-12	94.6	1.44	2.65	2.58	97.6	51.7	45.9	0.93	$4^0 42'$	0.070
2	SM	5	12- 14.2	72.2	1.54	2.7	2.035	73.4	36.2	37.3	0.97	$6^0 14'$	0.123
3	CL	5-15	14.2-18	18.8	2.05	2.68	0.549	16.8	16.2	0.6	4.33	$28^0 35'$	0.25
4	SM	20	18-23.5	15.4	2.06	2.66	0.486	15.4	14.1	1.3	1.0	$31^0 39'$	0.09
5	CL	25	23.5-28	37.2	1.76	2.71	1.17	44.7	21.1	23.6	0.68	$18^0 21'$	0.113
6	SC	25-35	28-37	23.9	1.99	2.67	0.658	22.8	20	2.8	1.39	$29^0 33'$	0.18

2.2. Experiences Recorded during the Construction at the PM1 Combined Thermal Power Plant Project and Other Projects of the Contractor

2.2.1 Overall Stability

In July 2000, during the construction of blocks 12+13 at discharge culverts, the contractor observed continuous displacement of the steel sheet piles. The contractor's immediate response was to remove soil from the backside of the piles. After the failure was recovered, the construction works continued until completion.

Prestressed concrete sheet piling work started in October 1999, and excavation began in March 2000 for the cooling water intake canal. On May 6, 2000, after digging about 150 m of the channel from the intake mouth to a depth of -6.8 m and after heavy rain, the contractor noticed a displacement of the piles and the anchor wall in the range from pile number 30 to 105. The most significant displacement measured at the top of pile 75 was 394mm. The anchor wall at the corresponding position also moves in the same direction as the pile wall. Through monitoring that the pile wall was constantly moving, to prevent the risk of significant landslides, the contractor filled with rock in the front side of the pile wall to an elevation of +1.0 m. After filling with gravel, the piles tend to move back. However, to ensure long-term operation, the displaced piles were pulled out and re-drived using longer piles.

This phenomenon has been recorded by many authors when one or all retaining walls slip on a sliding arc because removing the soil during excavation causes the loss of balance between the inside and outside of the pit.

2.2.2. Basal Heave

The excavation of the valve pit measuring 2 x 2 meters in size and 4 m deep, built on the water supply pipeline project in Thu Duc, Ho Chi Minh City, was stopped during excavation due to design issues. One month later, the construction work was allowed to resume. However, upon inspection, the contractor found that the bottom of the pit had heaved by about 1 meter. Further investigation revealed that the heave was caused by soft soil, prolonged suspension of work, and the 3-meter-high surcharge of a nearby road project about 5 m away.

This phenomenon has been previously recorded by experts in the field, including Terzaghi (1943), Bjerrum & Eide (1956), Eide et al. (1972), and Goh (1996). They have described simplified heave mechanisms and provided empirical formulas to determine the safety factor for bottom stability.

Nowadays, geotechnical engineers use various software to analyze bottom heave. With these tools, they can provide solutions before the excavation work begins. By addressing the issue of checking the safety of the basal heave of the excavation in advance, the risk of failure can be minimized.

2.2.3. Piping and Boiling

The bottom of the pump pit at the PM1 project was located at a depth of - 16.0 meters. The pit was situated in a fine sand layer with a groundwater level of - 4.0 meters. When excavation work for the concrete lining began, boiling occurred. To resolve this, the contractor drilled wells to collect underground water. This allowed excavation work to proceed without any problems.

However, due to the high cost of lowering the underground water level, the contractor decided to dry the excavation pit using a surface water pump after the bottom slab was concreted. Unfortunately, this causes erosion of the sand beneath the slab, leading to poor contact between the slab and the soil base. To remedy this, the contractor pumped sand-cement mixture to fill the gap.

Similar boiling occurred at the bottom of the cooling water intake channel, which was also an item of the PM1 project and had similar geology. After the underground water level was lowered, the canal bottom work was completed successfully.

2.2.4. Deformation and Bearing Capacity of a Strut–Wall System

The braced excavation comprises a wall system and a strut system. The materials used in the wall and strut systems are elastic, so their deformation and load-bearing capacity can be easily controlled. Although failures of strut-wall systems are rare, they can be severe when they do occur. These structures are damaged due to excavation instability, piping and boiling, excessive soil movement, workmanship, etc.

Several examples of such incidents include the collapse of the N2 tunnel, a subway station in Hangzhou, China (2008), the Singapore MRT excavation (2004), the Taegu Metro Barrett wall incident in Korea (2000), etc.

To prevent such incidents, it is crucial to have experienced construction engineers and supervision organizations. Monitoring, inspection, and supervision work should be analyzed statistically and handled promptly.

2.2.5. Excessive Settlement of Ground around Excavation and Related Effects on Adjacent Structures

Excessive movement of the PC corrugated sheet pile wall in the cooling water intake channel, as mentioned in section 2.1.1, causes the ground behind the pile wall to settle firmly, further causing movement of the anchor wall at the end. The reason was that the soft soil layer in this area was thicker than reported by the investigation, increasing the pressure on the pile wall. Although the pile wall and anchor bars were not damaged, the anchor wall was greatly deformed and had to be reconstructed.

The phenomenon of settlement of the ground outside the excavation was also recorded in the water discharge culvert. Because the items were built in an open land area with no surrounding structures, settlement outside the excavation was monitored but not recorded.

This experience was applied for deep excavations, including Pond B, to warn of incidents.

2.3. Research Method

The article uses Plaxis 2D software combined with actual observations to evaluate the behavior of the excavation.

Due to the shape of the excavation, the author chose the Plaxis 2 D Ultimate version 22.01.00.452 for research. The five soil models, Camclay, Hardening soil, Hardening soil with small strain, Mohr-Coulomb, and Soft clay, are respectively applied for analysis. Ultimately, the Hardening Soil model, drained or undrained, was selected for this study because of the best simulation of excavation behavior.

3. Analysis of the Construction Process

3.1. First Failure

The construction of this item began in April 1999, using an open-cut method. The strut-wall system was made using type IV steel sheet piles 24 m long, along with steel beam H300 strut and waller. The driving work for the steel sheet piles was completed after 1.5 months of construction. The excavation was performed simultaneously, using a dredging excavator and installing a strut system. The layout of the steel sheet piles and strut system for the excavation is shown in Figure 1.

While excavating for installing the fourth strut layer at an elevation of about -5.7 m, a failure occurred. The third layer of the strut system at the offshore end suddenly slid up with a loud noise. Cracks appear on the ground around the excavation. To ensure safety, workers immediately evacuated, and the pit was filled with water. Monitoring results over the next eight months were conducted as usual and did not detect additional significant pile head displacements or ground settlement.

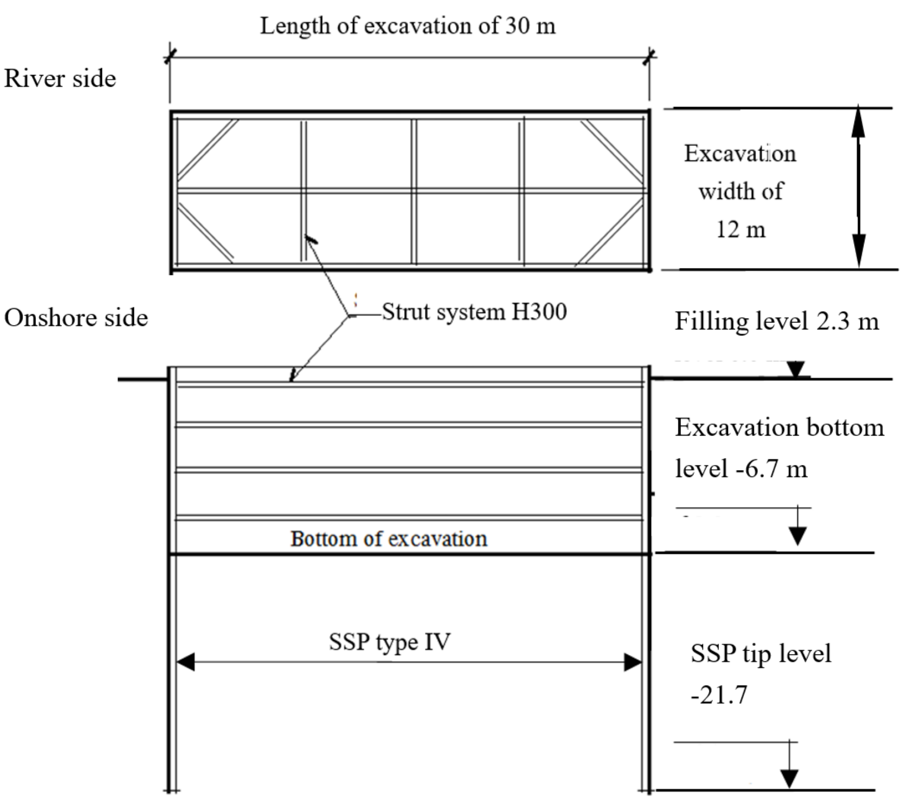

Figure 1. Plan and section of the excavation pit

Calculated results using Plaxis 2D software show that the excavation is considered stable, with an overall stability safety factor of about 9.8. This is consistent with the actual development of the incident. Figure 2 shows the change in internal force in the support system, indicating that the force acting in the third layer of the strut when failure occurs is about 1.5 MN/m. This force is quite large compared to the compression capacity of a steel beam H300. However, the internal force was insufficient to deform the strut and cause damage. A combination of many factors may have caused the failure.

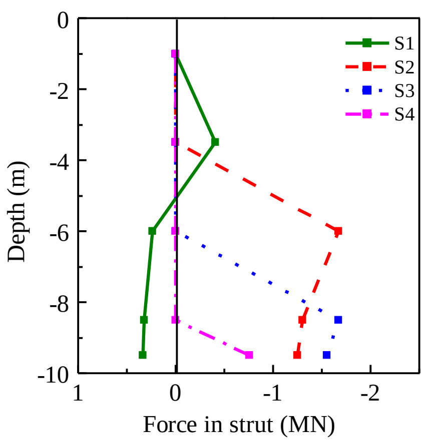

Figure 2. Internal forces in strut layers S1, S2, S3, S4 versus excavation depth

Based on Figure 3, it is evident that the offshore row of the pile wall had the most significant lateral displacement at the bottom elevation (at el. -6.7 m) at about - 300 mm, and the lateral displacement at the top of the pile wall (at el. +2.3 m) was +100 mm. The inclination of the pile wall can be determined to be approximately 4.5%. Additionally, it is worth noting that the displacement of the inner pile wall row is negligible.

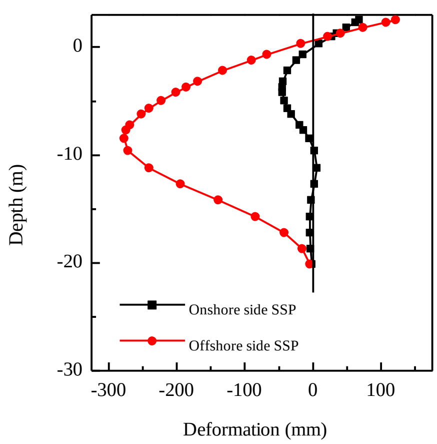

Figure 3. Pile wall deformation versus depth

Furthermore, it is noticeable that the weld that connects the struts and the waller of the third layer of struts on the outer row of the pile wall is under compression at the bottom surface and under tension at the top surface. A large force in the strut, the inclination of the pile wall, and poorly conducted welds (as identified after recovering the strut layers) caused damage to the connection between the strut and the waller.

As depicted in Figure 4, the bottom heave near the outer pile wall (at a distance of 90 m) reaches up to 300 mm, which is significantly larger than the heave near the inner pile wall (at a distance of 80 m), which is at about 70 mm. Substantial bottom heaving near the outer pile wall causes the wall system to deform, leading to a potential failure.

The failure mechanism at Pond B can be summarized as follows: (1) for the outer pile wall, the displacement of the pile wall is +100 mm at the top and -300 mm at the bottom of the excavation. Therefore, the pile wall inclines at an angle compared to the vertical; (2) The inclination generates additional moment force in the joint between the strut system and the wall system. This moment worsens the poorly welded joint; (3) The poor welds, the inclination of the pile wall, the bottom heave, and the significant force in the strut system all cause the strut to slip upward. That is the reason for the failure that occurred at Pond B.

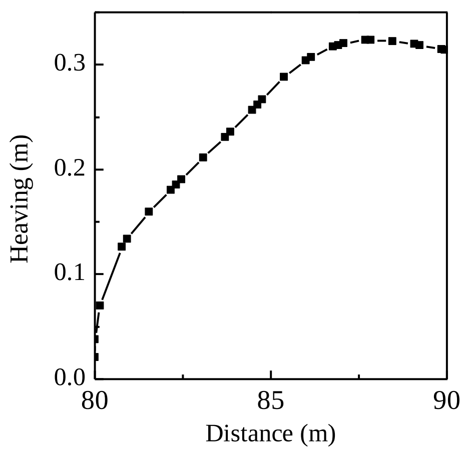

Figure 4. Bottom heaving of Pond B excavation in a direction perpendicular to the river bank

As indicated in Figure 5, the bottom heave value along the cross-section of the excavation can reach up to 300 mm. Additionally, after three days, the displacement value of the pile wall at the bottom of the pit is around 300 mm, which is the significant and unsafe amount as demonstrated in Figure 3. Figure 5 highlights that the bottom heave is likely to increase over time. For this reason, it is reasonable that the contractor decided to backfill the pit, relocate it about 30 m away from the riverbank, and reconstruct the pit to ensure safety.

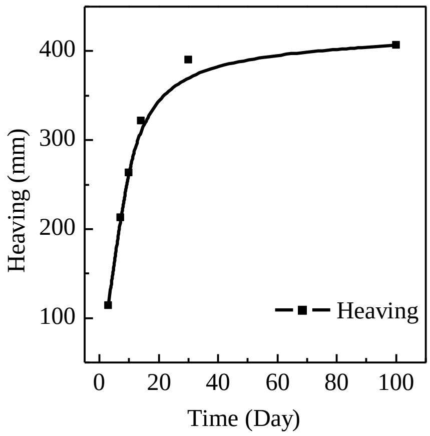

Figure 5. Bottom heaving over time

3.2. Second Failure

Eight months after the initial failure, the EPC contractor adjusted the design. Pond B was moved to a location about 30 meters from the shore but remained unchanged in design. Figure 6 displays a panoramic view of the excavation, while Figure 7 shows the construction method for the new Pond B. The pile wall was designed to be fixed, and two layers of steel sheet piles were driven as a retaining wall. The construction sequence followed these five steps below:

(1) Step 1: The outer sheet pile wall layer is driven from the filling level + 2.3 m, using type IV piles that are 12 m long and located 30 m away from new Pond B. The outer pile layer was only driven on three sides, as the onshore side was not piled due to the discharge canal being under construction at an elevation of - 3.7 m.

(2) Step 2: The first phase of excavation is to dig to an elevation of - 3.7 m to create a working platform for the second phase of excavation. This platform had an area of about 20 m by 46 m, including Pond B area and service road. Excavation with the 3:1 slope starts from the outer pile layer at elevation + 2.3 m inward to the working platform at an elevation of - 3.7 m;

(3) Step 3: After creating the working platform at an elevation of - 3.7 m, the inner layer of steel sheet piles was driven using type IV piles that are 12 m long;

(4) Step 4: The second phase of excavation from the working platform at an elevation of -3.7 m to the designated elevation. This step includes pouring lining concrete. The top surface of the lining concrete is at an elevation of -6.7 m)

(5) Step 5: This final step includes RC concrete slab and walls.

However, after excavating to the designed level (- 3.7 m), the contractor noted many broken piles. Most of the piles were broken at the joints about 1 - 2 m under the excavation bottom elevation of -6.7 m. Later, when the fractured pile sections were pulled up for inspection, it was discovered that the welds connecting the two pile sections were damaged.

Figure 6. Photo of Pond B from the left bank to the right bank of the drainage culvert. (Left of photo is the Thivai River)

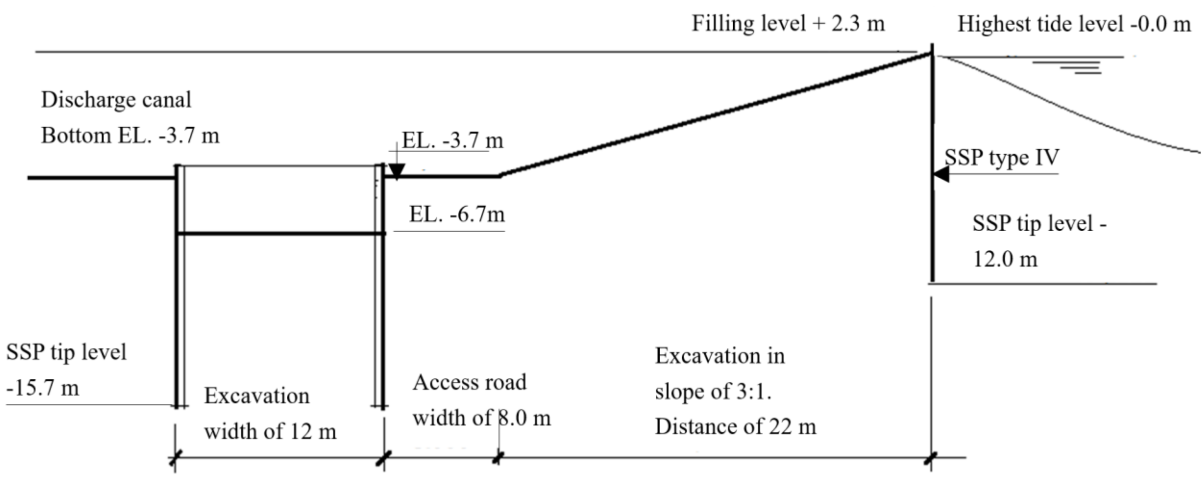

Figure 7. Longitudinal section of the drainage culvert and Pond B (second excavation)

After a few days of handling broken piles, the contractor noted that the bottom heave of the excavation was increasing. Mud was extruding around the perimeter of the unbroken piles. Figure 6 shows at least three broken piles, and workers are working on them. The method of handling broken piles is to dig up the soil, remove the damaged sections, and replace them with a reinforced concrete column poured on-site. Realizing that the time to remedy all broken piles could be extended, the contractor decided to backfill the pit with sand up to the working platform elevation of -3.7 m to replace it with new piles.

According to the analysis results obtained through Plaxis, it has been found that the continuous remedying of the damaged piles leads to uncontrollable bottom heave of the excavation. As shown in Figure 8, the bottom heave can increase to 150 mm within seven days and increase further over time. Such a high level of bottom heave makes it challenging to construct the bottom slab at the intended design elevation.

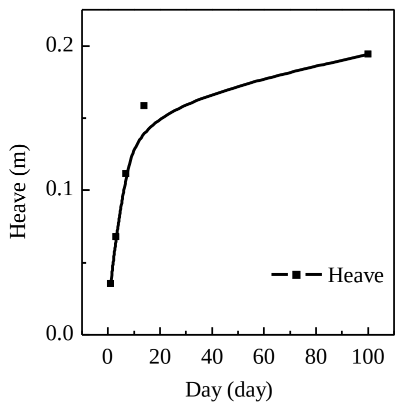

Figure 8. Bottom heaving over the time

Figure 9 indicates that the horizontal displacement of the pile wall is negligible (only about 30 mm). Hence, the failure of the excavation is mainly caused by the excessive heaving. The prolonged processing time of the piles is identified as the main reason behind the second failure.

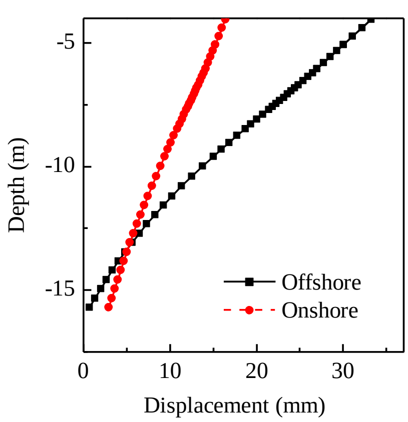

Figure 9. Displacement of steel sheet piles versus depth

Based on both reality and calculation results, the contractor's decision seems reasonable. From Figure 6, it can be seen that the soil condition at the bottom level of the pit is not stable enough to pour a lining concrete with a thickness of 100 mm. Therefore, it is necessary to pour a lining concrete with a thickness of 0.5 m simultaneously during excavation. This concrete can limit the bottom heave and ensure a clean platform for constructing the reinforced concrete bottom slab.

3.3. The Final Excavation

To achieve the final level of the excavation work, the contractor has proposed a set of countermeasures. These include filling the pit with sand up to an elevation of - 3.7 m, replacing the broken piles with the new ones, digging the soil deeper than the design level (more than - 6.7 m), during excavation, the concrete lining of about 0.5 m thick should be poured to ensure the stability of the bottom of the pit. The contractor poured the reinforced concrete bottom slab in as short a time as possible, within about one week.

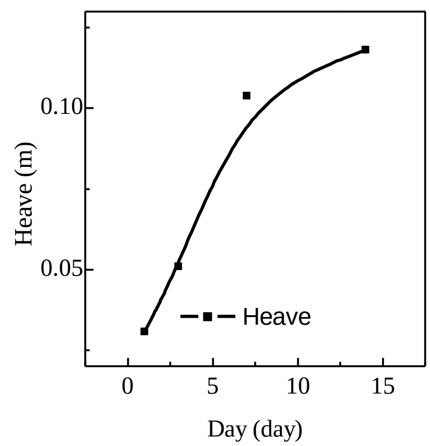

Figure 10. Bottom heaving over time

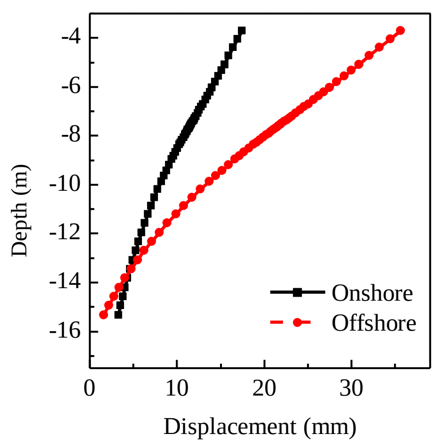

Figure 11. Displacement of steel sheet piles versus depth

This countermeasure has proven effective. The excavation and pouring of concrete lining, installing reinforcement, and construction of the bottom slab were completed successfully within one week. The bottom heave (see Figure 10) and the displacement of the steel sheet pile wall (see Figure 11) one day after excavation were 20 mm and 30 mm, respectively, which were negligible.

However, over time, bottom heave may increase after 14 days by about 130 mm (see Figure 10). To mitigate this risk, the contractor requested overtime to shorten the construction time of the 0.9 m thick reinforced concrete bottom slab within one week. Subsequent actual monitoring data after completion of the bottom slab did not record an increase in heave value or displacement of the steel sheet pile wall.

4. Conclusions

Based on the above analysis, some experiences can be drawn as follows:

1. When designing and constructing a braced excavation in soft soil, it is necessary to check the following issues simultaneously: (1) Overall stability of the excavation; (2) Stability of the bottom of the excavation due to heaving; (3) Stability due to underground erosion and boiling; (4) Deformation and load bearing capacity of the strut - wall system; (5) Excessive settlement of the ground around the excavation;
2. The first failure at Pond B excavation occurred mainly due to poorly conducted welds and/or stiffness of the steel sheet pile wall and/or bottom heaving. The stiffness of a type IV steel sheet pile that is 24 m long is small. Hence, the deformation/displacement of the piles at the bottom elevation of the pit was significant at about 300 mm, even though the displacement at the tip of the pile was insignificant. In addition, if the strut system is firmly connected to the waller and steel sheet pile wall, movement can be minimized.
3. The analysis results of the second failure show that the bottom heave is not significant (20 mm) immediately after digging. Still, over time, the heave will increase (130 mm after seven days), making it difficult for Pond B excavation.
4. The combination of excavation and simultaneously pouring a 0.5 m thick lining concrete is a reasonable countermeasure. This concrete can act as a concrete lining for subsequent construction work and contribute to counterbalancing against the bottom heaving.
5. The time factor when excavating in soft soil plays a vital role in ensuring construction safety. Therefore, the contractor needs to establish a method statement to respond promptly to all construction developments on site.

The incident occurred due to a lack of experience and knowledge of soft soil behavior and insufficient consideration of bottom heaving and supporting structures. Designing and constructing deep excavations in soft soil requires a step-by-step approach where data from previous construction phases is used to predict soil behavior in subsequent steps.

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