Applications of Underwater Excavations and Underwater Concretings for Remedialing A Critical Instable Excavation and for Solving Excessive Inflowing Debit due to Very Porous Subsurface Layer

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ABSTRACT: This paper presents and discusses applications of underwater excavations and underwater concreting for solving two cases with two different problems. All plans and actions of both cases were backed up by comprehensive analyses and calculations. In the first case, due to an under-reinforced condition, an excavation had caused large lateral deformation of the steel sheet pile reinforced by steel struts. The remedial actions were consisted of backfilling inside excavated area, excavating several parts of outside excavated area and underwater excavation and underwater concrete for base of the excavation area. In the second case, several parts of sheet piles could not be driven to the planned depths of impervious layer leaving very porous layer beneath their tips, underwater excavation and underwater concreting for the base of excavation area were selected to increase the safety and to solve the problem of excessive inflowing sea water debit. This paper also shows that careful and thorough numerical analysis results were comparable with both field conditions during constructions.

Keywords: underwater excavation, underwater concreting, critical excavation

1 INTRODUCTION

The stability of braced cut excavation relies on lateral supports from struts in addition to the soil passive resistance. The presence of struts may reduce the flexural stresses in the wall, reduce the lateral movements and increase the stability (safety). However, in certain conditions, the application of braced cut excavation alone is not enough to satisfy the serviceability requirements. This paper presents and discusses two (2) cases with underwater excavation and underwater concreting to improve excavation performances due to their critical conditions.

Excavation on soft soil and porous subsurface layer are complex cases due to their potential excessive displacement and excessive inflowing water debit into excavated area, respectively. In several cases, improvements in excavation method are required to avoid slope failure on soft soil and to avoid excessive debit on porous subsurface layer.

To predict behaviors, we utilized a professional finite element software PLAXIS 2D with 15 node element.

One of the key features is constitutive model selection, this paper performed analysis using Mohr Coulomb and Hardening Soil constitutive model.

2 CASE 1 PROJECT

This part presents the excavation improvement by remediation of under construction Sea Water Intake (SWI) on soft soil. The excessive deformation on sheet pile and surrounding soils were occurred during 10 meters depth of excavation. As a result, it has been decided to improve the construction method to remediate the excessive deformation and increase the safety of excavation. An alternative method using backfilling combine with additional concrete slab at the base of excavation.

2.1 *Initial Condition*

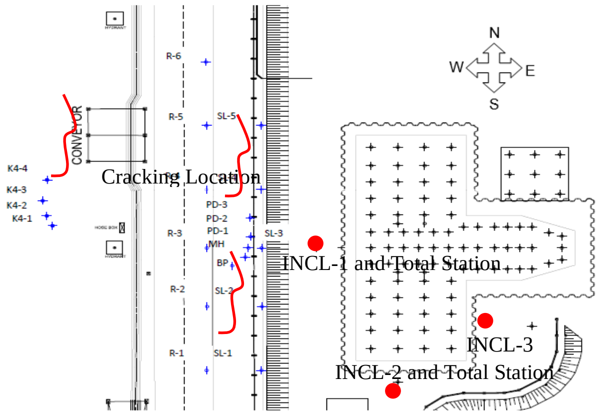

Figure 1. Site plan of Case 1 Project

Figure 2. Excavation of Case 1 Project

The Sea Water Intake (SWI) Case 1 Project was constructed on medium dense sand layer from ground surface to an approximate depth of 5 meters below the existing ground surface. Beneath the sand was soft to medium stiff clay layer to an approximate depth of 25 to 30 meters below the existing ground surface. Underlying the soft clays were very stiff clay that extended to the end of boring at a depth of 40 meters below the existing ground surface. According to laboratory test results, the clay layer in the intermittent is high plasticity clay with the values of plasticity index of (IP) range from 70% to 90%. The Atterberg limit and water contents for various depths are shown in Figure 4. From the UU Triaxial test results, the undrained shear strengths of the clay layer range from 7.3 kPa to 18.9 kPa. If compared to N-value data, the undrained shear strength (Su) from UU test have an approximate ratio of Nspt = Su/4. The average natural unit weight of the clays is 15.3 KN/m^3. The average value of compressibility index (cc) is 0.66.

According to initial original design parameter are approximately the upper bound value. Theϕ = 35° on sand layer is an upper bond value. The Su=23 kPa compared to N_{spt} value has a ratio of $\mathbf{N_{spt}= S_u/6.5}$. Soil modulus (E) = 4550 kPa is too large compared to soil modulus by laboratory testing. This condition could be the reason of large values of undrained shear strength (Su) that were obtained by back calculation analysis.

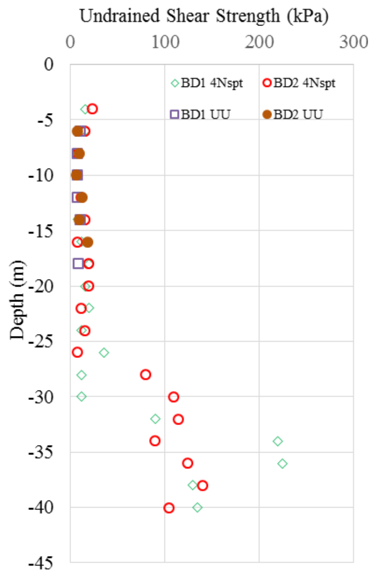

Figure 3. Undrained shear strength with depth

The instrumentation were installed to monitor soil deformation. Four manual monitorings by Total Station were performed at specific intended locations, such as sheet piles, important structures, soil cracking, etc. The inclinometer at existing excavation is located on West, East, and South side. The existing of infrastructures in this location were consisted of: Pipe Pedestal, Main Hole and Bona Pipe. The instrumentation location and soil cracking position are being continuously monitored as shown in Figure 1.

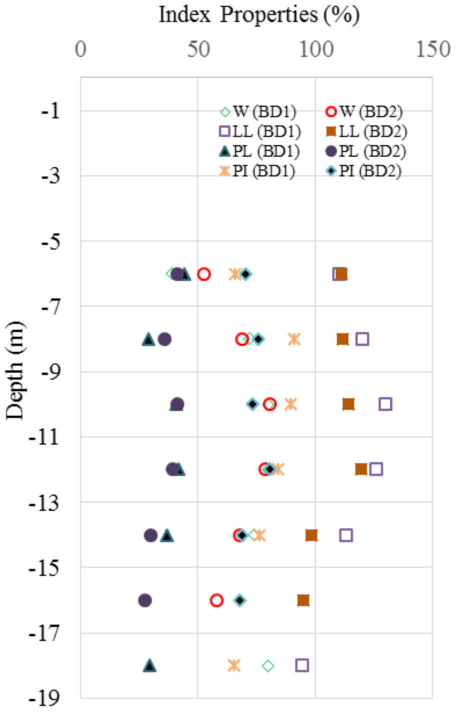

Figure 4. Index properties for various depths

Figures 5 and 6 are initial designs for SWI retaining structure. There are several monitoring systems conducted to monitor deformation around excavation area.

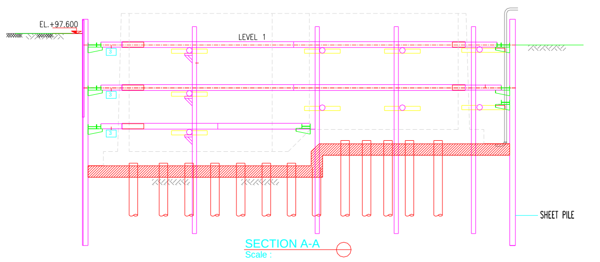

Figure 5. Cross section and structural configuration on East-West direction

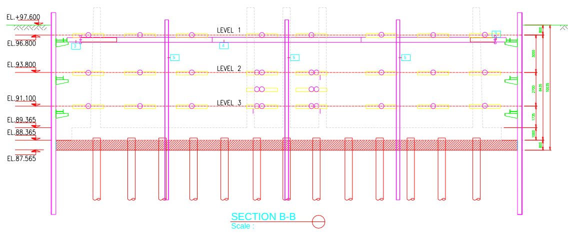

Figure 6. Cross section and structural configuration on North-South direction

According to the recorded data, existing infrastructures are generally deformed with relatively direction to the excavation area. For monitoring points that located on North site have deformed to South-East direction with magnitudes of deformation close to 10 cm to South and approximately 25 cm to East. In addition, monitoring points on South of SWI have deformed to North-East direction with magnitudes of close to 18 cm to North and approximately 28 cm to East. For whole monitored locations generally occurred settlements which range 15 cm to 50 cm.

Deformation monitoring's of sheet pile were also conducted manually by total station. Monitoring points were measured on each strutting level and top of sheet pile. As shown in Figure 7, on the West side of sheet pile, the maximum deformation to East is approximately 44 cm at the bottom of excavation and to West direction is approximately 20 cm located at the top of sheet pile. On South side of excavation, the maximum deformation of sheet pile to North direction is close to 145 cm located at the bottom of excavation and to South direction is approximately 20 cm located at the top of sheet pile.

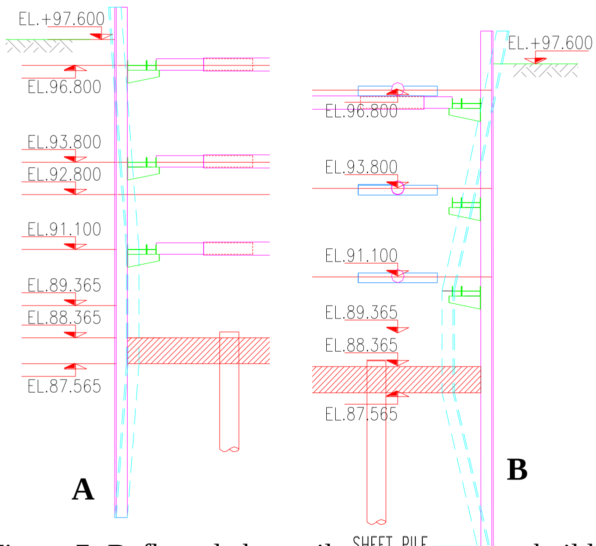

Figure 7. Deflected sheet pile compare to as build drawing A) on South site near SW-2, B) on West site near SW-1

2.2 *Analysis Model*

Excavation analysis were conducted to investigate the stability of earth retaining structures based on the most current soil condition and structural condition after deformation has occurred. Figures 5 and 6 shows the design of temporary structure and excavation. The figures are used as geometry model on back calculate analysis.

Table 1. Soil parameters for BH1

Type	GL	Su	ϕ	E_{50}	E_{ur}	Eoed
	M	kN/m2	°	kN/m^3	kN/m2	kN/m2
Sand			30	15000	15000	45000
Clay		5 to 20		1250	1000	4500
Clay		10		1250	1000	4500
Clay		90 to 140		18000	18000	54000

Table 2. Soil Parameters for BH2

Type	GL	Su	ϕ	E_{50}	E_{ur}	Eoed
	m	kN/m2	°	kN/m^3	kN/m2	kN/m2
Sand			30	15000	15000	45000
Clay		5 to 20		1250	1000	4500
Clay		10		1250	1000	4500

Table 3. Initial soil parameters

Type	GL	Su	ϕ	E_{ur}	ν
	m	kN/m2	°	kN/m2	
Gravelly Sand			35	14000	0.25
Gravelly Sand			27	2800	0.25
Sandy Silt		23		4550	0.35
Clay		117		46800	0.25
Silty Clay		280		11180	0.35
Silty Clay		169		67600	0.35

Table 4. Sheet pile OZ 20A properties

OZ 20 A	Sym	Value	Unit
Axial Stiffness	EA	3.32E+08	kN/m
Flexural Rigidity	EI	8.74E+04	kN/m2/m
Allowable Force	d	5.46E+03	kN/m
Allowable Moment	w	7.14E+08	kN.m/m

Table 5.Strut SPP 406 properties

Strut Type	Behavior	EA
		kN/m
SPP 406	Elastic	3.42E+06

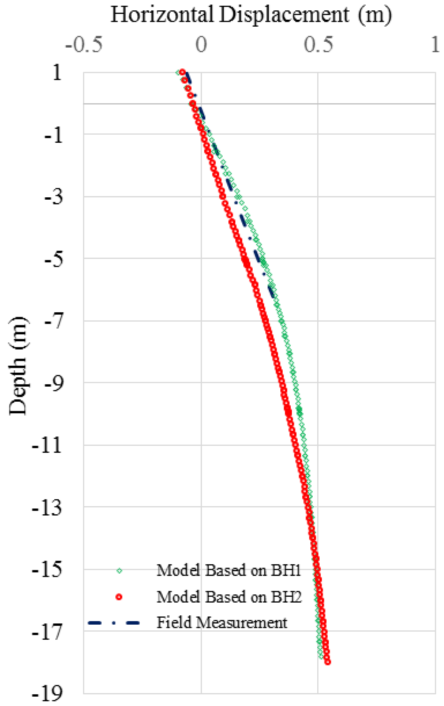

Figure 8. Horizontal displacements on East-West direction

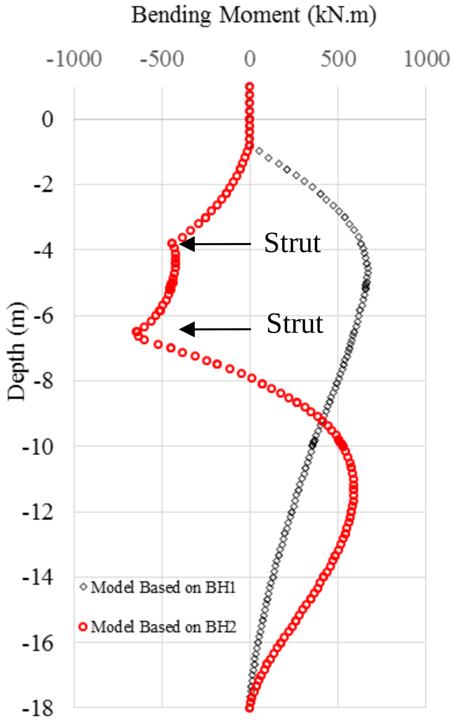

Figure 9. Bending moments on East-West direction

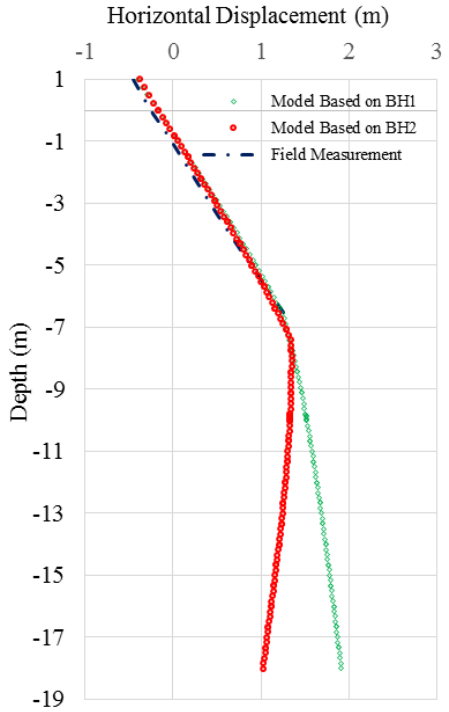

Figure 10. Horizontal displacements on North-South direction

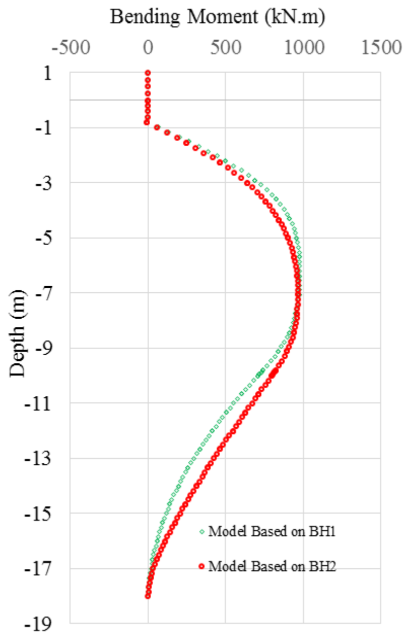

Figure 11. Bending moments on North-South direction

Figures 8 and 9 show the results of back calculation analysis in East-West direction. From result on BH1, excavation has caused soil in plastic condition during excavation to a depth of -7.5 meters, caused by excessive soil deformation (heaving at excavated SWI). Maximum calculated sheet pile deformation is approximately 54 cm. On the other hand, from result on BH2, final cut of excavation could successfully be reached with a safety factor number of 1.2. Extreme calculated deformation of soil after final cut was approximately 55 cm.

Figures 10 and 11 show the results of back calculation analysis in North-South direction. According to analysis results based on BH-1, excavation has caused plastic condition on during excavation after strut installation. Extreme sheet pile deformation is approximately 197 cm. Analysis results based on BH-2was successfully reached with a safety factor number is 1.02 (closed to instability condition). Extreme calculated deformation of sheet pile is approximately 139 cm. From both analysis result, the developed bending moment on sheet pile was unfortunately already larger than its maximum allowable capacity. Nevertheless, the acting/developed forces on struts are still smaller than its allowable capacity.

To obtain a conservative approach, the safety factor of the most critical time are considered, when excavation was at the final stage (before being backfilled) and water was completely pumped out, the lowest safety factor is occurred with 1.02.The condition was selected to evaluate the selected soil parameters based on laboratory test and N-SPT results. With the parameters and initial safety factor (1.02) selected remedial actions were developed, evaluated and analyzed to gain additional safety factor which is acceptable.

Based on analysis, for further stage(s) of construction, to improve the stability of system of excavation area of SWI additional reinforcing system are presented, especially the North and South sides of SWI by perform the followings:

a. Back fill the North sides and South sides of inside SWI with granular material. Backfilling can be performed with bagged sands.
b. Reduce weight of soil outside of SWI by excavate portions of North and South sides with 3 meters of depth and 10 meters long.
c. Install additional sheet piles to strengthen the earth retaining structural system.
d. Pump water out of SWI area.

e. Concrete installation at the center cap of SWI along with the walls.
f. Install additional struts of North of SWI.
g. Concrete installation on North and South of pile caps.

The analyses of the SWI are performed as recommended improvement by a combination of three actions. First, backfilled the excavation area with granular material on North and South sides. This remedial action has increased the calculated factor of safety from 1.02 to 1.09. Second, is combination of backfilled and excavated the soil in the outside portion of SWI. This method has increased the calculated factor of safety from 1.02 to 1.22. The third action is combination of backfilling, excavating the soil in the outside portion of SWI and installing additional sheet pile has increased the calculated factor of safety from 1.02 to 1.28. Analysis results from the recommended remedial actions are shown in Figures 12 and 13.

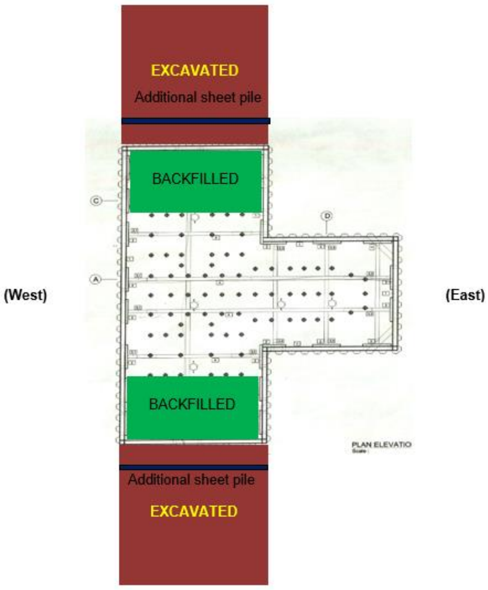

Figure 12. Remediating Method using Backfilling and Additional Retaining Sheet Pile Combination.

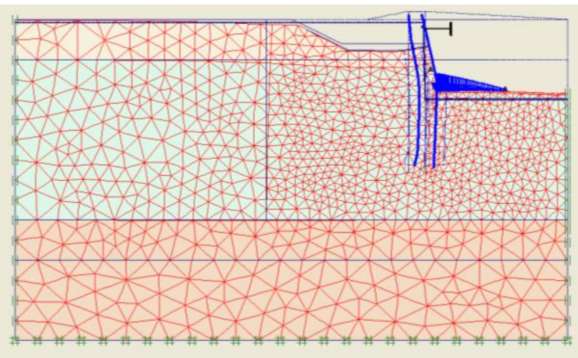

Figure 13. Finite Element Model for Remediation Method.

3 CASE 2 PROJECT

3.1 *Initial Condition*

In this case, very porous subsurface affect the inflowing debit into excavation area due to high permeability, in addition there is several parts of sheet piles could not be driven to the planned depths of impervious layer leaving very porous layer beneath their tips. To continue the work, underwater excavation to the planned excavation depths and underwater concreting for the base of excavation area were selected due to its most cost effective solution.

The underwater excavation method increase the lateral resistance by hydrostatic pressure on critical condition, this method increase the safety reduce the lateral deformation at critical stage of excavation. The underwater concrete are placed through the base of excavation to reduce excessive inflowing debit to excavation area, in addition, underwater concrete increase the lateral stiffness to the retaining structure.

Figure 14. Excavation of Case 2 Project

Stratigraphic condition at the site and soil parameters for the proposed project were developed and determined according to the site investigation, typically the top layer of soil at SWI area is dominated by slag material from elevations ranging from +4.53 to -9.48 meters. The soil investigation found silty clay at approximate elevations of -9.48 to -20.48 meters and -24.48 to 30.48 meters, clayey silts with hard consistency found at approximate elevations of -20.48 to -24.8 meters and silty sands with sandy silts at approximate elevations of -30.48 to -36.72 meters. The Groundwater monitoring during investigation was found at approximate elevation of +2 meters.

Figure 16 shows the actual depth of penetration, there are several sheet pile could only driven to approximate depths of 10 meters. This condition implies the lower safety factor and debit of water flow in excavation area become larger, ground where the permeability is so high that dewatering activity need a high pumping capacity and very costly. Underwater excavation are decided to reduce the lateral forces during excavation on critical moment (final excavation) and underwater concrete installed on the base of excavation as barrier for water flow in to excavation area.

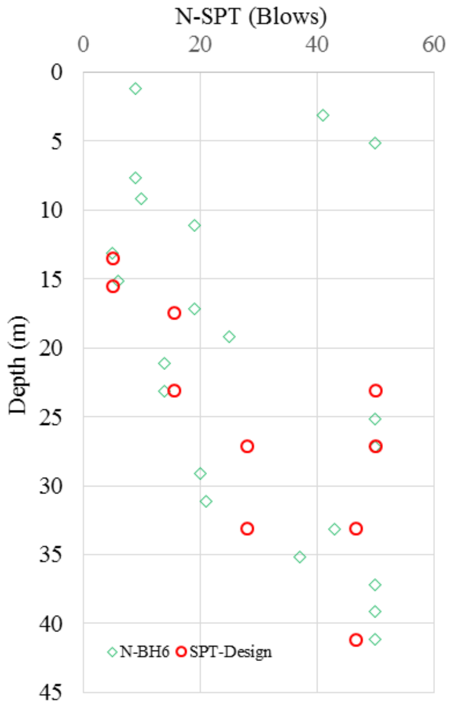

Figure 15. Standard Penetration Test results with depth

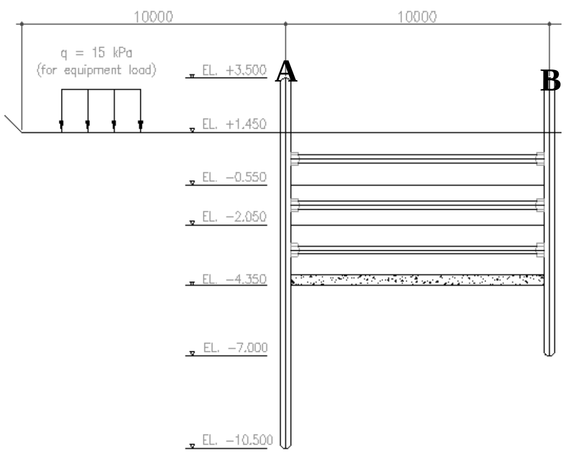

Figure 16. Cross section and structural component of Case 2 Project

3.1 *Analysis Model*

The analysis of excavation was performed to investigate the stability of earth retaining structures using underwater excavation and underwater concrete on very porous sub-soil layer. Figure 16 show the design of temporary structure and excavation model.

Table 6. Soil parameters for analysis

Type	GL	Su	ϕ	E_{50}
	m	kN/m2	°	kN/m^3
Slag Layer	-2	1 to 5	30	12000-15000
Silty Clay		27		5400
Clay		80		16000
Clayey Silt		150		40000

Table 7. Sheet pile OT 22 properties

OT 22	Sym	Value	Unit
Axial Stiffness	EA	2.34E+06	kN/m
Flexural Rigidity	EI	1.12E+05	kN/m2/m
Allowable Force	F_{all}	2138	kN/m
Allowable Moment	M_{all}	417.45	kN.m/m

Table 8. Strut SPP 406 properties

Strut Type	Behavior	EA
		kN/m
SPP 406	Elastic	2.61E+06

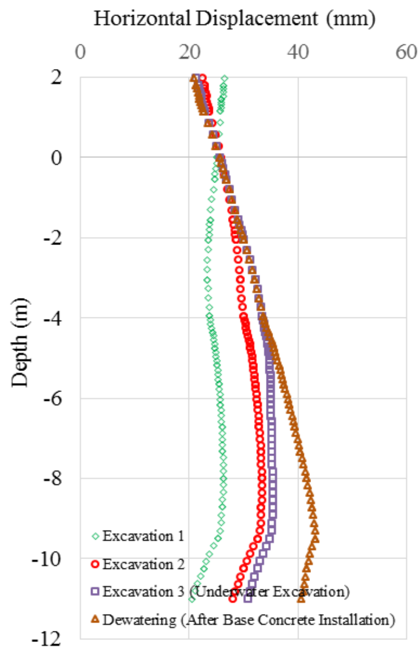

Figure 17. Horizontal displacement of Cross Section A

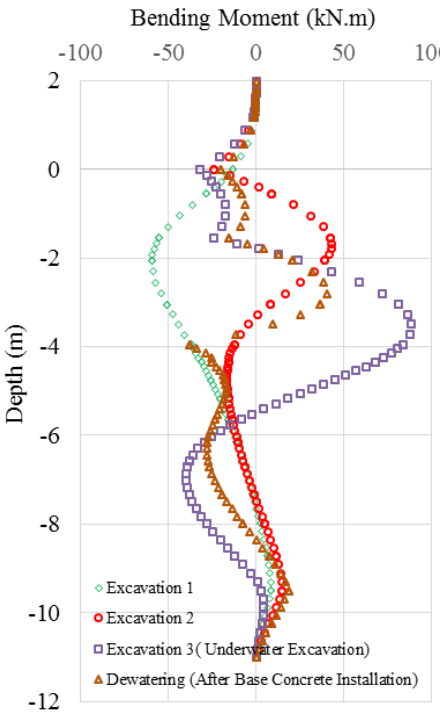

Figure 18. Bending moments Cross Section A

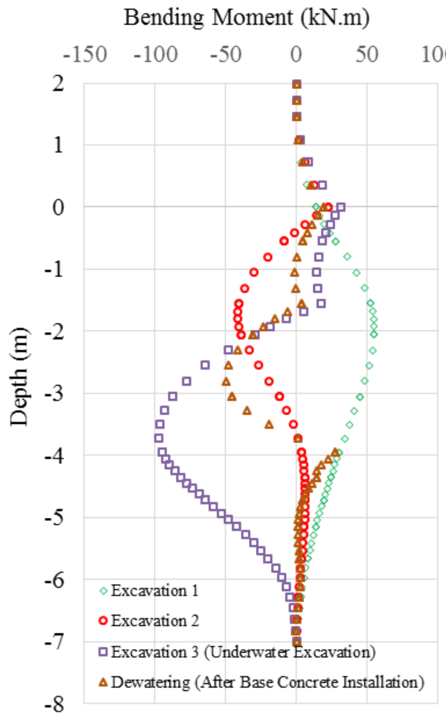

Figure 20. Bending moments Cross Section B

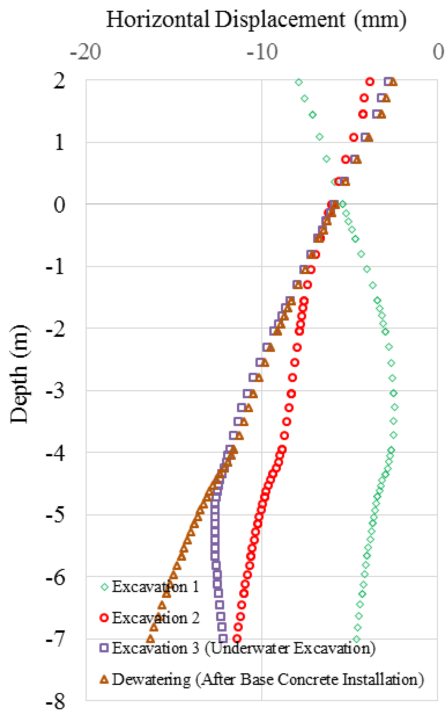

Figure 19. Horizontal displacement of Cross Section B

Table 9. Output of maximum bending moments and horizontals displacements on Cross Section A

Phase	Maximum	
	Bending Moment (kNm/m)	Displacement (mm)
Excavation 1	59.14	26.91
Excavation 2	43.09	33.32
Excavation 3	88.31	35.35
Dewatering	45.19	43.11
Global SF Excavation 3	1.83	
Global SF Dewatering	2.05	

Table 10. Output of maximum bending moments and horizontal displacements on Cross Section B

Phase	Maximum	
	Bending Moment (kNm/m)	Displacement (mm)
Excavation 1	55.25	8.6
Excavation 2	41.87	11.44
Excavation 3	97.29	12.69
Dewatering	49.53	17.55
Global SF Excavation 3	1.83	
Global SF Dewatering	2.05	

The results of analysis without underwater excavation shows the slope stability failure at the 7 meters depth with horizontal displacement 165 mm. Tables 9 and 10 for underwater excavation shows the less displacement with maximum horizontal displacement 43. 11 mm with safety number is 1.83 at critical condition. According to ground water monitoring and sump pumping test data, the inflowing debit in to excavation area were relatively large with 18000.56 m^3/day. This condition according to the actual soil condition is dominated by slag layer above the shortest sheet pile tips.

Numerical analysis model are consist of inflowing debit calculation for each stage construction. In this model slag permeability is calculated based on pumping test data by back calculate the inflowing debit per hour. Dewatering analysis results with the finite element analysis has resulted in a relatively close prediction to sump pumping test data. The numerical analysis result is 18582.27 m^3/day, this value was relatively close with actual field condition with a debit of 18000.56 m^3/day for 1.80 meters head difference. Underwater concrete were performed after third excavation, this numerical analysis was reduced inflowing debit about 80% to 90% as shown in Figure 21.

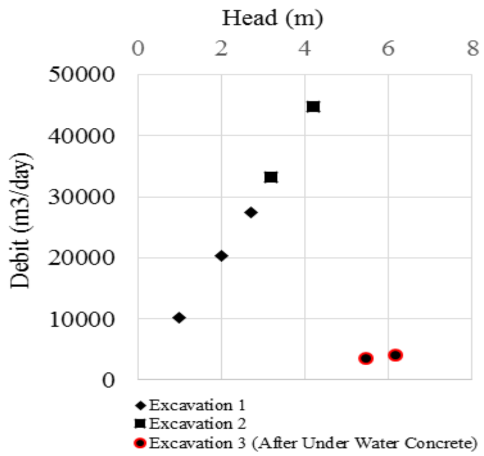

Figure 21. Inflowing debit with head during dewatering

4 CONCLUSION

This paper presented the results of remediation method using numerical experiments based on finite element analyses. Reinforcing the excavation by backfilling method on critical excavation on soft clay and underwater excavation combine with underwater concrete on very porous sub-soil layer. Both of this method has revealed (technically and economically) to improve excavation stability and reduce inflowing debit into excavation area. Based on analysis results, safety of factor of excavation were increase significantly due to additional support from slab concrete. On the other hand, the selection of input parameter for the analysis plays crucial role.

REFERENCES

Brinkgreve, R.B.J. 2011. *Plaxis Material Models Manual*. Netherland.

Duncan, J.M. and Chang, C.Y. 1970. Non-Linear Analysis of Stress And Strain in Soils. *Journal of The Soil Mechanics and Foundation Division, Proceeding of the American Society of Civil Engineers, September 1970*. USA.

Burland, J. B., et. al. 2004. Design and Construction of Deep Basements Including Cut-and-Cover Structures. London: IStructE.

U.S. Department of Transportation Federal Highway Administration. 2005e. Earth Retaining Structure. *FHWA-NHI-05-046*. United States of America.