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"Geotechnical Engineering for Future Infrastructure Development in Indonesia"

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Underwater Excavations and Underwater Concretings for Remedialing A Critical Instable Excavation on Soft Soils and for Solving an Excavation with Excessive Inflowing Debit

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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 concreting for base of the excavation area. In the second case, at most locations, sheet piles could not be driven to the planned depths to impervious layer due to the compacted gravelly size material leaving very porous layer beneath their tips. With the conditions, the 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 numerical results of thorough analysis are comparable to (monitored) field conditions.

Keywords: underwater excavation, underwater concreting, critical excavation, soft soils, porous material.

1 INTRODUCTION

The stability of a 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 on the wall, reduce the lateral movements and increase the stability (safety). However, at 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 of underwater excavation and underwater concreting to improve excavation performances due to their critical conditions.

Excavations on soft soil and porous subsurface layers are complex due to their potential excessive displacement and excessive inflowing water debit into excavated area if cut off could not (difficult) be achieved, 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. The model was developed with the 15 node elements. Both the Mohr Coulomb and Hardening Soil constitutive models were selected for analyses.

2 CASE 1 PROJECT

This part presents and discusses the remediation of an under-construction Sea Water Intake (SWI) on soft soil which was in a very critical condition. Excessive deformations of installed sheet piles and surrounding soils occurred after the 10 meters depth of excavation. Both analysis result and visual observation concluded that immediate actions must be taken to remediate the excessive deformation and increase the safety of the excavated area. As engineers requested to solve the condition, we proposed a combination method of backfilling inside excavated and removal part of the outside area combined with installation concrete slab at the base of excavation by underwater concreting and underwater excavation the rest excavated material.

2.1 *Condition Before Remediation*

The Sea Water Intake (SWI) of the Case I Project (Figures 1 and 2) was constructed on a medium dense sand layer from ground surface to an approximate depth of 5 meters below the existing ground surface. Beneath the sands was a soft to medium stiff clay layer to an approximate depth of 25 to 30 meters. 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 clays with the values of plasticity index of (PI) range from 70% to 90%. The Atterberg limits 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 we compare to the N-values, the undrained shear strength (Su) from UU test have an approximate Nspt/Su ratio of 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.

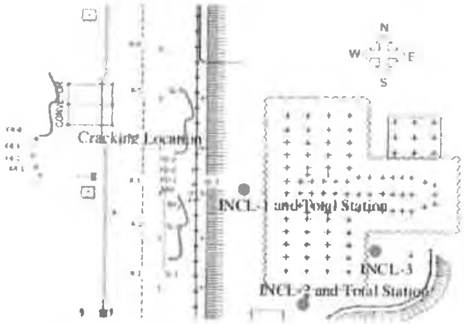

Figure 1. Site plan of Case I Project

Figure 2. Excavation of Case I Project

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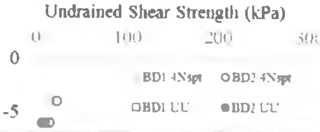
Figure 3. Undrained shear strength with depth

Originally, the design of excavation reinforcement was designed based on selected parameters which were most likely on the upper bound values. The upper very soft clay layer which was considered as Su=23 kPa will has a ratio of **N_{spt}= S_u/6.5.** The soil modulus (E) = 4550 kPa is also relatively large compared to the soil modulus tested by laboratory testing. Our back calculation analysis results confirmed this hypothesis.

Instrumentations were installed to monitor soil and structure deformations. Four (4) manual monitorings by total Station were also performed at specific critical locations, such as sheet piles, important structures, soil cracking, etc. Additional inclinometers were also installed in addition to the existing ones, since their reliability records were doubtful. In addition to monitor the stability of the excavation system, the monitoring were also to prevent disturbance to critical existing infrastructures surrounding the area which were consisted of: Pipe Pedestal, Main Hole and Bona Pipe. The locations instrumentation and soil cracking position are shown in Figure 1.



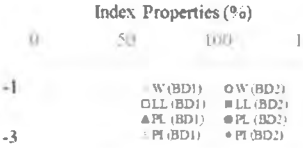


Figure 4. Index properties of various depths

Figures 5 and 6 illustrate the initial designs for SWI retaining structure Case 1 Project.

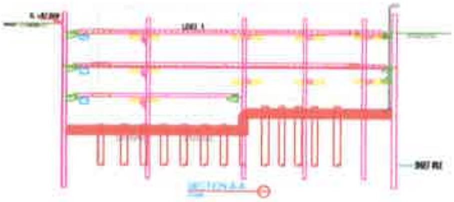


Figure 5. Cross section and structural configuration on East-West direction

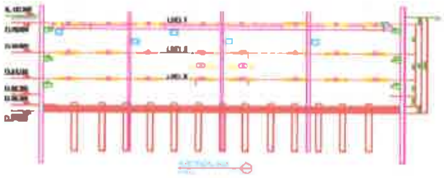


Figure 6. Cross section and structural configuration on North-South direction

According to the recorded data, existing infrastructures generally deformed toward the excavated area. The monitoring points at North site have deformed to South-East direction with magnitudes of 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. All monitored points generally had settled which ranges magnitudes of 15 cm to 50 cm.

Deformation monitoring of sheet pile was 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 was approximately 44 cm at the bottom of excavation and to West direction was approximately 20 cm located at the top of sheet pile. On South side of excavation, the maximum deformation of sheet pile to North direction was approximately 145 cm located at the bottom of excavation and to South direction was approximately 20 cm located at the top of sheet pile.

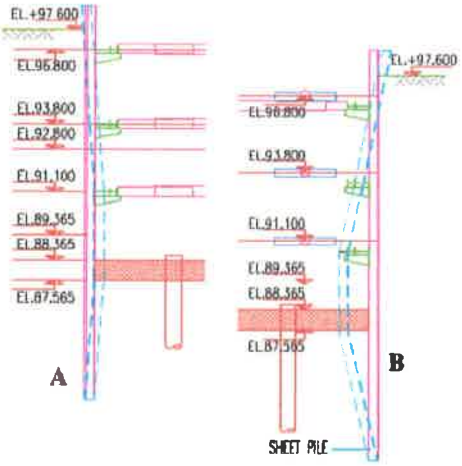


Figure 7. Deflected sheet pile compare to as build drawing A) on South site near SW-2, B) on West site near SW-1

2.2 *Analysis Model*

Excavation analysis was conducted to investigate the stability of earth retaining structures based on the most current soil and structural conditions after deformation has occurred. Figures 5 and 6 show the design of temporary structure and excavation. The figures are used as geometry model for back calculation 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}	v
	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

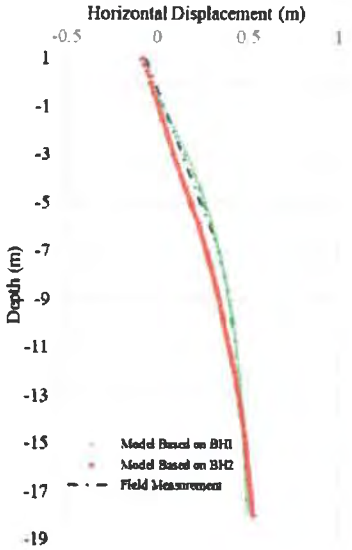


Figure 8. Horizontal displacements on East-West direction: calculated vs. measured

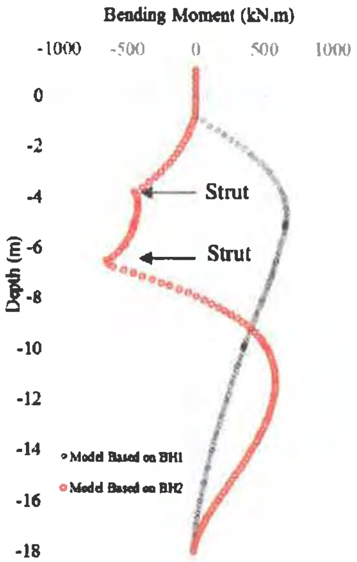


Figure 9. Calculated bending moments on East-West direction

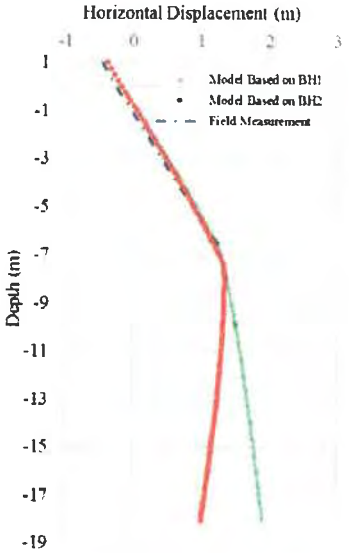


Figure 10. Horizontal displacements on North-South direction : calculated vs. measured

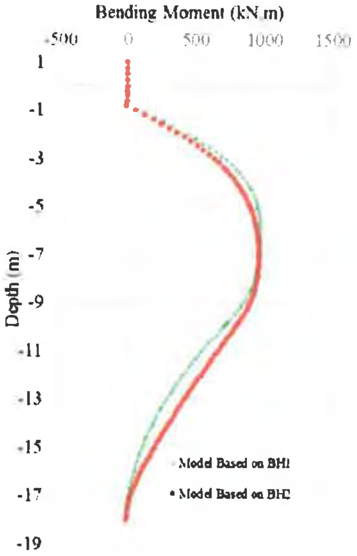


Figure 11. Calculated bending moments on North-South direction

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 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 calculated sheet pile deformation is approximately 197 cm. Analysis results based on BH-2 show a minimum calculated safety factor of 1.02 which is close to instability. Extreme calculated deformation of sheet pile is approximately 139 cm. From both analysis results, 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.

The most critical time for this case is when excavation was at the final stage (before being backfilled) and water was completely pumped out. In the condition, the safety factor is 1.02. The condition was selected for the evaluation of the selected soil parameters. With the parameters and initial safety factor (1.02) a remedial method was designed, evaluated and analyzed to gain additional an acceptable safety factor.

Based on analysis result, 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 performing the followings:

a. Back fill inside SWI at North and South side with granular material (bagged) sands.
b. Reduce weight of soil outside of SWI by excavating portions of North and South side 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 were performed as recommended improvement by a combination of three actions (Figure 12). First, backfilling 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 the combination of backfilling and excavating the soil outside the excavated area. This method has increased the calculated factor of safety from 1.02 to 1.22. The third action is the 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.

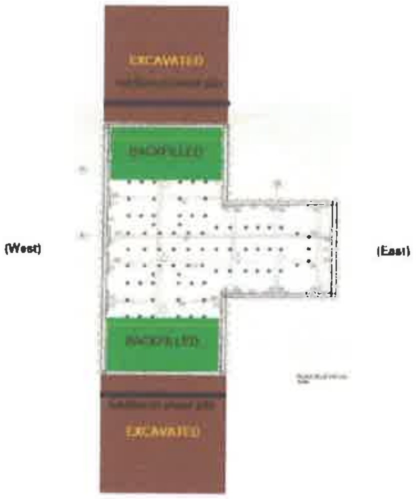


Figure 12. Remediating Method using Backfilling and Additional Retaining Sheet Pile Combination.

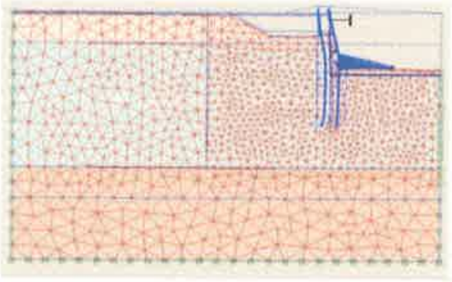


Figure 13. Analysis Result for the Selected Remediation Method.

3 CASE 2 PROJECT

3.1 *Condition Before Remediation*

In this Case 2 project, 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

Soil layer and soil parameters for analyses were developed and determined according to soil data. The soil investigation typically found the top layer of soil at SWI area slag material from elevations ranging from +4.53 to -9.48 meters. Beneath the slags, 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 found ground water table at an 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.

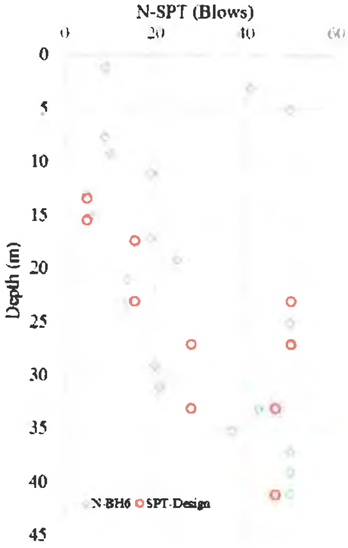


Figure 15. N_{SPT} values for various depths

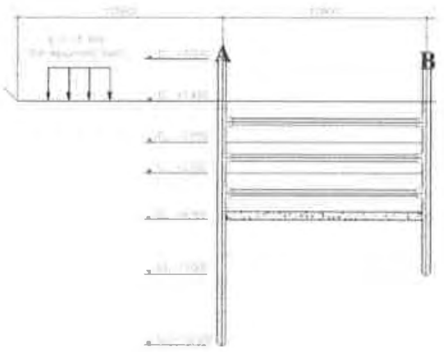


Figure 16. Cross section and structural components of Case 2 Project

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. Selected soil parameters for analysis

Type	GL	Su	ϕ	E_{50}
	M	kN/m2	°	kN/m³
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

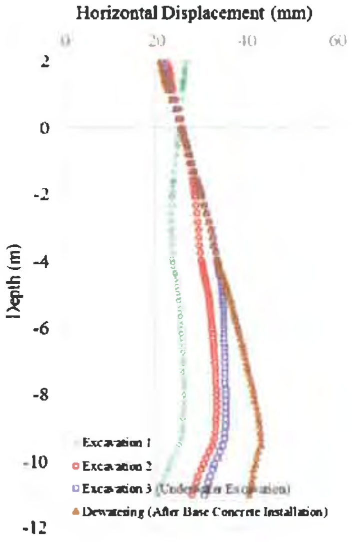


Figure 17. Calculated horizontal displacements of excavation stages of Cross Section A

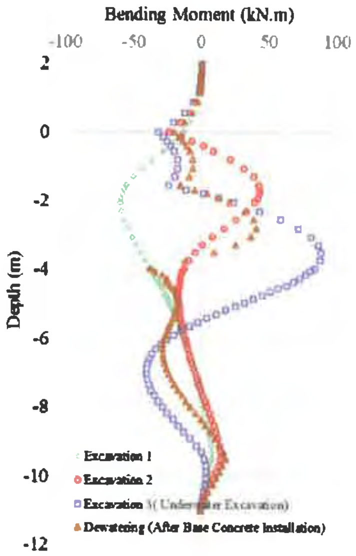


Figure 18. Calculated bending moments of excavation stages of Cross Section A

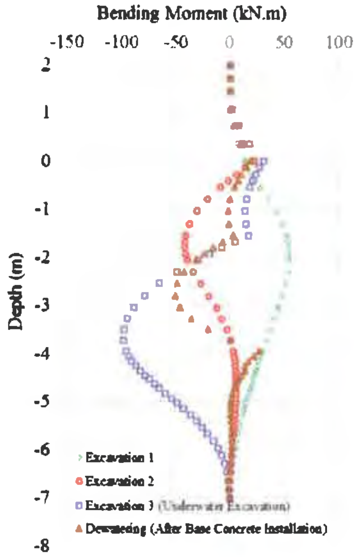


Figure 20. Calculated bending moments of excavation stages of Cross Section B

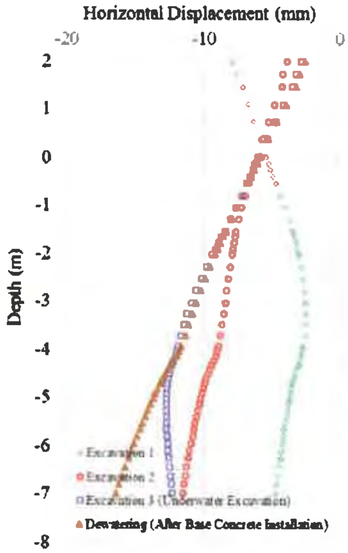


Figure 19. Calculated horizontal displacements of excavation stages of Cross Section B

Table 9. Calculated 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. Calculated 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	

Results of analysis without underwater excavation show that the slope stability failure at the 7 meters depth. Tables 9 and 10 for underwater excavation show the safety factor of has increased to 1.83 with a relatively limited displacement with a maximum horizontal displacement of 43.11 mm. According to ground water monitoring and sump pumping test data, the inflowing debit in to excavation area was relatively large, approximately 18,000 m^3/day. This condition according to the actual soil condition was 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 was calculated based on pumping test data by back calculating the inflowing debit per hour. Dewatering analysis results are relatively close to sump pumping test data. The numerical analysis result is 18,582 m^3/day. This value was relatively close to the measured debit of 18,000 m^3/day for a 1.8 meter head difference. Underwater concrete were performed after third excavation, this numerical analysis was reduced inflowing debit approximately 80% to 90% as shown in Figure 21.

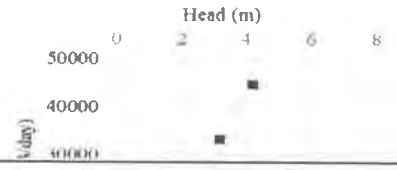


Figure 21. Calculated inflowing debits for various heads during dewatering

4 SUMMARY AND CONCLUSION

This paper presents and discusses 2 cases of projects which were remediated by the method of underwater excavation combined with underwater concreting on unstable excavation on very soft clays and on very porous sub-soil layer. This paper also presents results of numerical analysis by utilizing the finite element method to back up the executions of the 2 project cases. The cases show and prove that the selected combination of underground concreting and underground excavation method have successfully (technically and economically) improved excavation stability and reduced inflowing debit into excavation area. The presented cases also show that careful and comprehensive analyses have played crucial roles in the success of the remediation of the project cases.

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