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NUMERICAL SIMULATION OF DEEP EXCAVATION IN GRAVEL LAYERS

Chien-Yi Wu and Shuh-Gi Chern

Key words: gravel, excavation, elastic modulus, numerical simulation.

ABSTRACT

This study analyzed gravel excavation cases in the Xindian District of Taiwan. Instead of the parameters typically employed in deep excavation analysis we used the deformations of diaphragm walls in a back analysis to achieve recommendations regarding the ideal range of the soil elastic modulus E_s in the analysis parameters of numerical simulations for the Xindian District stratum or similar strata. We used Plaxis 2D software to analyze the gravel layer. The results indicated that, when the fifth layer of gravel was set to N = 100 and the range of the soil elastic modulus E_s was set between 7.84N and 9.8N MPa, where N is the N value of the Standard Penetration Test, the maximum deformation of the wall at the final excavation stage was satisfactorily estimated. A few studies have investigated the gravel layer of the Xindian District; therefore, the current study used the gravel layer of Bagua Mountain in the Changhua Region of Taiwan for conducting a comparison. Previous studies on the gravel layer of Bagua Mountain have reported that the soil elastic modulus E_s of this layer ranged from 88.2 to 833 MPa, which is consistent with the empirically estimated range of the elastic modulus E_s (180.3-980 MPa) in our study.

I. INTRODUCTION

Most studies conducted on the topic of deep excavation have focused on soft soil layers and sandy layers. Only a few studies in Taiwan have investigated the engineering characteristics of gravel layers. Deep excavation engineering is a complex task that involves evaluating the soil-structure interaction. The design analysis of deep excavations can be conducted using relevant knowledge and practical experience in the geotechnical and structural fields. The concept of continuum mechanics is incorporated into the finite element method to simulate the behavior of soil and retaining structures during construction by defining the stress-strain relationship and the boundary conditions of the soil. On the basis of the parameters used in a design analysis for the deep excavation of a gravel layer in the Xindian District, this study used back analysis to evaluate the monitoring data collected in this excavation project, thereby exploring the deep excavation behavior of the gravel layer in this district and providing suggestions for analyzing and designing the deep excavations of similar gravel layers.

The characteristics of gravel layers are related to the particle size, shape, compactness, and content of the gravel as well as to the properties of the fine aggregate filler. Hung (1978) reported that, when the coarse aggregate content (larger than the No. 4 sieve) is 75% or higher, the engineering properties of the gravel layer are determined according to the properties of coarse aggregates; by contrast, when the coarse aggregate content is lower than 70%, the engineering properties of the gravel layer are determined according to fine aggregates. Das (1994) indicated that if coarse-grained soil contains more than 35% fine aggregates, enough fine aggregates occur between the separate coarse aggregates to enable the soil to behave more like a fine-grained material.

The geological condition of a gravel layer is related to the particle size of its gravel; thus, the properties of gravel layers cannot be simulated easily in a laboratory. The physical and mechanical properties of gravel layers are commonly obtained by performing on-site tests such as sieve analysis as well as unit weight, plate loading, and direct shear tests. The ground layer evaluated in this study was the upper layer of the Ching-Mei gravel layer in the Xindian District. Because the gravel layer is located in an urban area and because the investigation budget was limited, no suitable site was available for performing on-site tests. Thus, relevant test data was scant. To examine geotechnical engineering problems, the values of the parameters used are generally estimated through empirical formulae based on on-site tests or hypothesized using practical experience. Regarding the parameters of the gravel layer in this study, the monitoring data of actual cases and literature data were used as references in conducting a back analysis of reasonable parameters for the gravel layers in the evaluated cases. The results can serve as a reference for numerical analysis of similar parameter ranges of the soil elastic modulus (E_s) for deep excavations in the Xindian District.

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I. NUMERICAL ANALYSIS METHOD AND MODEL

Plaxis 2D software is used to address all types of soil-structure interaction problems encountered in geotechnical engineering. This software can be used to analyze various geotechnical problems including the behavior of deep foundation excavations, slope stability, reinforced retaining walls, soil nailing, ground anchors, internal bracing, raft foundations, pile foundations, seepage flow, and tunnels. Plaxis 2D is considered a powerful tool in solving geotechnical engineering problems and provides numerous constitutive models that enable users to simulate the stress–strain behavior of soil.

This study used the Mohr-Coulomb model in the Plaxis 2D program for simulating the soil behavior. The Mohr-Coulomb model employs an elastic-perfectly plastic failure mode. A soil model that is based on the elastic-plastic theory must conform to Hook's law and consider factors such as the yielding criterion and the flow rule at the elastic stage. The following soil parameters are required:

(1) Elastic modulus E: Most geotechnical materials exhibit linear behavior when bearing small loads. Thus, when a material demonstrates a high range of linear elasticity, the initial tangent modulus (E_0) should be adopted. However, in general situations, a secant modulus with a 50% ultimate strength (E_{50}) can be adopted for the soil.
(2) Poisson's ratio ν: When the Mohr-Coulomb model or the elastic model is adopted, the coefficient of earth pressure at rest K_0 can be used to determine the ν value as follows: $\nu = K_0/(1 + K_0)$.
(3) Cohesion c: Parameters can be selected based on the actual cohesion c of different types of soil. In the Mohr-Coulomb model, the c value controls the soil failure criteria. In addition to inputting the c value according to the actual cohesion, Plaxis 2D can be used to analyze sand without cohesion ($c = 0$). However, when the value of c is set to zero in the program, errors may occur in the analysis and the calculation. Thus, to facilitate the calculation process, a value of $c > 0.2$ kPa can be input according to the suggestion of the Plaxis 2D user manual (2006).
(4) Angle of internal friction ϕ: The angle of internal friction can be determined according to the soil type and general on-site or laboratory tests of shear strength.
(5) Dilatancy angle ψ: The dilatancy angle of cohesive soil (i.e., clay) can be assumed to be zero (excluding highly overconsolidated clay layers). The ψ value of sandy soil is determined according to the soil density and angle of internal friction ϕ. When the ϕ value of sandy soil containing quartz exceeds 30°, $\psi = \phi$ - 30°. When $\phi < 30°$, $\psi \fallingdotseq 0°$. Because the ψ of sandy soil is so low that it sometimes becomes zero or a negative value ($\psi \fallingdotseq 0°$ or $\psi < 0°$), this value can be assumed to be 0° when the Mohr-Coulomb model or the elastic model is adopted.

III. EXCAVATION CASE ANALYSIS

1. Case 1

1) Site Situation

This excavation case was located on the southern side of Minquan Road in the Xindian District of New Taipei City in Taiwan. The site was irregularly shaped with an area of approximately 8522 m^2. The elevation difference of the terrain was within 1 m (Kenkul Corp., 2005a).

2) Foundation Ground Layer

The foundation ground layer was divided into five layers from top to bottom (C & M Hi-Tech Engineering, 2003; Lin, 2011), and the properties of each layer were described subsequently. Table 1 shows the simplified engineering parameters of the ground layer. The groundwater level was approximately 10.7-11.3 m below the ground level. In addition, the pressure of the groundwater level in the gravel layer (i.e., the fifth layer) was determined to be approximately 11.4-11.6 m below the ground surface, which was nearly equal to that of the free water aquifer on the ground surface.

(1) First layer: The backfill layer gradually changed to yellowish-brown clayey silt and gray-mottled silty sand containing some gravel. The average thickness of this layer was approximately 4.0 m, and the suggested N value was approximately 5.
(2) Second layer: This coarse gravel layer comprised yellowish-brown coarse, medium, and fine silty sand. The average thickness of this layer was approximately 11.7 m, and the suggested N value was approximately 40.
(3) Third layer: This gravel layer comprised yellowish-brown and gray coarse, medium, and fine silty sand. The average thickness of this layer was approximately 4.6 m, and the suggested N value was approximately 35.
(4) Fourth layer: The fourth layer comprised gray sand and clayey silt containing thin layers of sand and clay. The average thickness of this layer was approximately 4.1 m, and the suggested N value was approximately 16.
(5) Fifth layer: The fifth layer comprised coarse gravel and yellowish-brown coarse, medium, and fine silty sand. The thickness of this layer was greater than 10 m, and the suggested N value exceeded 50.

3) Foundation Excavation Planning

(1) Geotechnical facility description: The foundation excavation depth was 17.30 m. The basic pattern included a raft foundation with a retaining structure of diaphragm walls with a thickness of 70 cm. The depth of the walls was 27 m.
(2) Internal bracing system: The bottom-up excavation method was adopted in this case. After the various excavation stages, horizontal braces were established in a system of five layers. H-shaped steel beams were used to create the bracing frame. Fig. 1 illustrates the excavation profile.

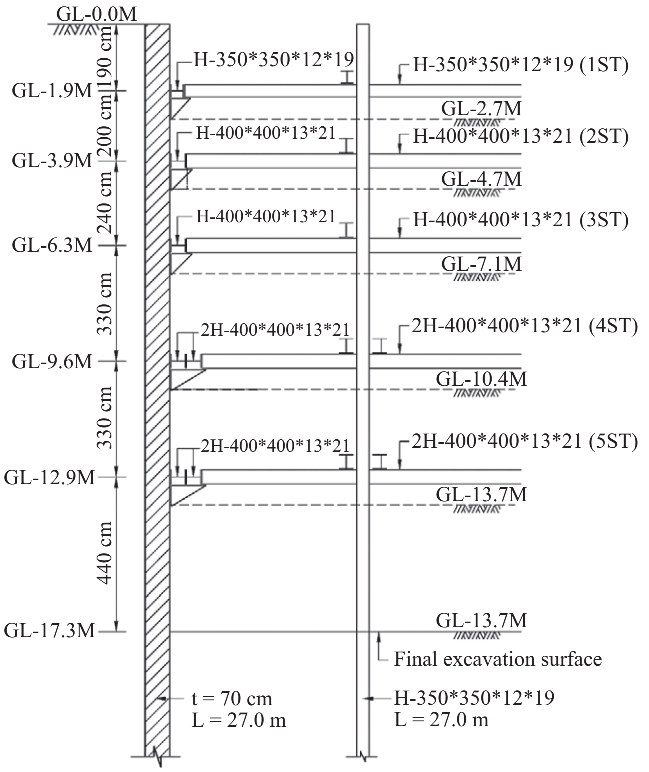

Fig. 1. Bracing profile for Case 1.

(3) Excavation steps: After the excavation reached Stage 1 (ground level (GL.) = -2.7 m), the first brace layer was installed. Stage 2 involved excavating to GL. -4.7 m before installing the second brace layer. The third, fourth, and fifth brace layers were installed at GL. -7.1, GL. -10.4, and GL. -13.7 m at Stages 3, 4, and 5, respectively. Finally, Stage 6 involved excavating to GL. -17.3 m.

Table 1. Simplified ground layer engineering parameters for Case 1 (C & M Hi-Tech., 2003).

Ground layer description / Bottom depth of each layer	N value	γ_t kN/m^3	Total stress c kPa	Total stress ϕ °	Effective stress c' kPa	Effective stress ϕ °
1 The backfill layer gradually changed to yellowish brown clayey silt and silty sand mottled with gray including some gravel. Average thickness 4.0 m	1.5-19 (5)	19.3	* 9.8	* 21	0	30
2 The coarse gravel layer comprised yellowish brown coarse, medium, and fine silty sand. Average thickness 11.7 m	15 to > 50 (40)	21.6	—	—	* 4.9	* 38
3 The gravel layer comprised yellowish brown and gray coarse, medium, and fine silty sand. Average thickness 4.6 m	20-58 (35)	21.1	—	—	* 0	* 38
4 The composition of this layer was gray sand and clayey silt containing thin layers of sand and clay Average thickness 4.1 m	12-26 (16)	19.4	9.8	24	0	32
5 The coarse gravel layer comprised yellowish brown coarse, medium, and fine silty sand. Average thickness 10.0 m, hole bottom	> 50 (50)	* 22.1	—	—	* 9.8	* 40

Note:
1 *represents an estimate, ( ) represents suggested N value, and σ' represents effective vertical earth pressure.
2 An excerpt from the "Geological Survey and Analysis Report for Parcel Number 34 and 110 at Dafeng Road, Xindian District, Taipei County" by C & M Hi-Tech Engineering Co. Ltd. (2003).

2. Case 2

1) Site Situation

Case 2 was located on the eastern side of Zhongzheng Road (near Minquan Road) in the Xindian District of New Taipei City in Taiwan. The site was irregularly shaped with an area of approximately 4560 m^2. The difference in the terrain elevation was within 1 m (Kenkul Corp., 2006).

2) Foundation Ground Layer

The foundation ground layer was divided into five layers from top to bottom (Kenkul Corp., 2005b; Lin, 2011), and the general properties of each layer were summarized subsequently. Table 2 shows the simplified engineering parameters of the ground layer. The groundwater investigation data indicated that the groundwater level of the foundation was at a depth of approximately -11 m below the ground level.

(1) First layer: This layer was a backfill layer containing yellowish-brown clayey silt and sandy silt. The average thickness of this layer was approximately 3.7 m, and the suggested N value was approximately 7.
(2) Second layer: This coarse gravel layer comprised yellowish-brown coarse, medium, and fine silty sand. The average thickness of this layer was approximately 12.4 m, and the suggested N value was approximately 40.
(3) Third layer: This gravel layer comprised yellowish-brown coarse, medium, and fine silty sand containing some pebbles. The average thickness of this layer was approximately 3.4 m, and the suggested N value was approximately 23.
(4) Fourth layer: This gray clayey silt comprised silty clay and thin layers of fine sand. The average thickness of this layer was approximately 4.9 m, and the suggested N value was approximately 13.
(5) Fifth layer: The fifth layer comprised coarse gravel and yellowish-brown coarse, medium, and fine silty sand. The

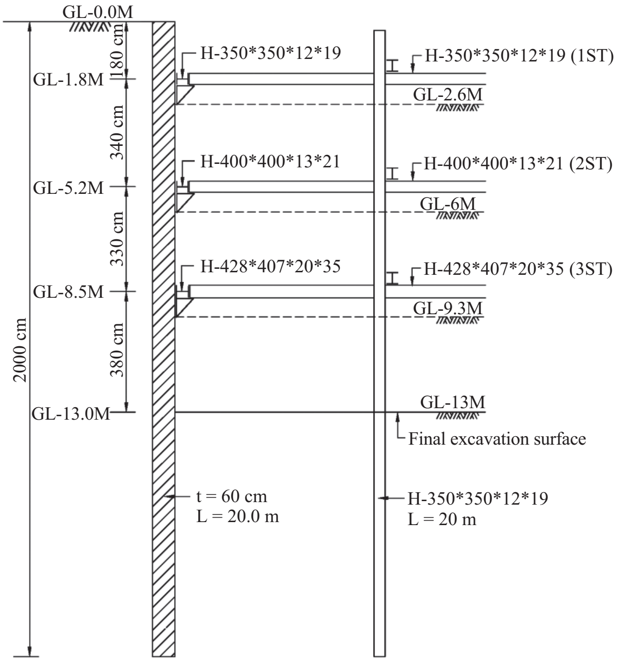

Fig. 2. Bracing profile for Case 2.

thickness of this layer was greater than 10 m, and the suggested N value exceeded 50.

3) Foundation Excavation Planning

(1) Geotechnical facility description: The foundation excavation depth was 13 m. The basic pattern included a raft foundation with a retaining structure of diaphragm walls with a thickness of 60 cm. The depth of the walls was 20 m.

(2) Internal bracing system: The bottom-up excavation method was adopted in this case. After the various excavation stages, horizontal braces were established in a system of three layers. H-shaped steel beams were used to create the bracing frame. Fig. 2 depicts the excavation profile.

(3) Excavation steps: The foundation was excavated using a conventional method. At Stage 1, the excavation was conducted at GL. -2.6 m to establish the first brace layer. Stage 2 involved excavating to GL. -6.0 m and installing the second brace layer. The third brace layer was installed at GL. -9.3 m at Stage 3. Finally, Stage 4 involved excavating to the bottom surface of GL. -13.0 m. Because the groundwater level of the case location was low, coarse gravel layers were the primary composition involved in the excavation; thus, a lower lateral pressure was exerted on the retaining wall structure than was common.

Table 2. Simplified ground layer engineering parameters for Case 2 (Kenkul Corp., 2005b).

Ground layer description Bottom depth of each layer	N value	γ_t kN/m^3	Total stress c kPa	Total stress ϕ °	Effective stress c' kPa	Effective stress ϕ °
1 Backfill layer containing yellowish brown clayey silt and sandy silt. Average thickness 3.7 m	5-17 (7)	19.4	9.8	22	* 0	* 30
2 The coarse gravel layer comprised yellowish brown coarse, medium, and fine silty sand. Average thickness 12.4 m	22 to > 50 (40)	21.6	—	—	* 4.9	* 38
3 The gravel layer comprised yellowish brown coarse, medium, and fine silty sand containing some pebbles. Average thickness 3.4 m	16-31 (23)	21.1	—	—	* 0	* 34
4 The gray clayey silt comprised silty clay and thin layers of fine sand. Average thickness 4.9 m	11-20 (13)	19.5	9.8	24	0	31
5 The coarse gravel layer comprised yellowish brown coarse, medium, and fine silty sand. Average thickness 11.1 m, hole bottom	> 50 (50)	* 22.1	—	—	* 9.8	* 40

Note:

1 *indicates the estimate and ( ) represents the suggested N value.

2 An excerpt from the "Geological Survey and Analysis Report for Seven Parcel Numbers Including Number 62-1 on Dafeng Road, Xindian District, Taipei County" by Kenkul Corp. (2005b).

3. Analysis of Basic Hypotheses

(1) The excavation construction was assumed to demonstrate plane strain behavior.

(2) Reference Fan (2005) finite element analysis was performed to substantiate the empirical approach proposed, in which the horizontal range (R) was determined to be at least four times the excavation depth (H2) behind the diaphragm wall, the vertical range (D) was determined to be at least three times the penetration depth (H1) plus the excavation depths (H2) (i.e., D = 3H1 + H2), and a uniformly distributed load of 14.7 kPa (A-A) was assumed to be exerted on the ground surface.

(3) According to the site situation, the foundation excavation depth, excavation shape, bracing system allocation, and soil layer boundaries were considered. Complete analysis grids were also established. The boundary elements of the grids were assumed to exhibit no horizontal or lateral displacement outside the influence range.

(4) The stiffness of the diaphragm wall was reduced to 70% based on practical experience.

(5) The diaphragm wall and bracing were simulated using beam elements.

(6) A 15-node triangular element was used for the analysis.

(7) When the wall bottom pierced a rock stratum or gravel layer to a certain depth (1.5 m or higher), the wall bottom was not displaced horizontally; this assumption was established based on empirical monitoring data of relevant cases. Thus, the horizontal displacement of the wall bottom was constrained during the analysis.

Table 3. Input soil layer strength parameters for Case 1.

Depth (m)	Soil type	N	c' (kPa)	ϕ' (°)	γ_{unsat} (kN/m^3)	γ_{sat} (kN/m^3)	E_s (kPa)	ν
0.0~4.0	SF, ML, SM	5	0	30	19.3	19.5	15000	0.333
4.0~15.7	GW	40	4.9	38	21.8	22.0	7840N-9800N	0.278
15.7~20.3	GW, SW	35	0	38	21.1	21.4	7840N-9800N	0.278
20.3~24.4	ML	16	0	32	19.4	19.7	100000	0.320
24.4~33.0	GW	100	9.8	40	22.1	22.3	7840N-9800N	0.263

Table 4. Input soil layer strength parameters for Case 2.

Depth (m)	Soil type	N	c' (kPa)	ϕ' (°)	γ_{unsat} (kN/m^3)	γ_{sat} (kN/m^3)	E_s (kPa)	ν
0.0~3.7	SF, ML	7	0	30	19.4	19.5	24000	0.333
3.7~16.1	GW	40	4.9	38	21.6	22.0	7840N-9800N	0.278
16.1~19.5	GM, SM	23	0	34	21.1	21.4	7840N-9800N	0.306
19.5~24.4	ML	13	0	31	19.5	19.7	100000	0.327
24.4~35.5	GP, GM	100	9.8	40	22.1	22.3	7840N-9800N	0.263

Table 5. Input diaphragm wall strength parameters for Case 1.

Thickness (m)	E (kPa)	I (m^4/m)	Reduction coefficient	0.7EA (kN/m)	0.7EI (kNm^2/m)
0.7	2.3E+07	0.028583	0.7	1.13E+07	4.60E+05

Table 6. Bracing parameter input table for Case 1.

Bracing layer	Bracing location	Model	A (cm^2)/pole	0.7EA (kN)	Preload pressure (kN/m)
1ST	GL. -1.9 m	1× H 350	173.9	2.51E+06	65
2ST	GL. -3.9 m	1× H 400	218.7	3.15E+06	131
3ST	GL. -6.3 m	1× H 400	218.7	3.15E+06	196
4ST	GL. -9.6 m	2× H 400	218.7	6.30E+06	245
5ST	GL. -12.9 m	2× H 400	218.7	6.30E+06	245

4. Ground Layer and Structural Parameter Decision

This study used the elastic modulus E_s of the gravel layer as the variable; after trial and error, E_s = 7840N~9800N is used in this study. Tables 1 and 3 (Case 1) and Tables 2 and 4 (Case 2) show the other soil and setting parameters. The diaphragm wall and bracing structures in the cases were simulated using beam elements. The primary input data included cross-sectional area (A), Young's modulus (E), and moment of inertia (I). The stiffness of the diaphragm wall was reduced to 70% based on practical experience. Tables 5 and 6 (Case 1) as well as Tables 7 and 8 (Case 2) indicate the basic parameters of the structural elements.

5. Analysis Process

Because the on-site excavation process is extremely com plex,

Table 7. Input diaphragm wall strength parameters for Case 2.

Thickness (m)	E (kPa)	I (m^4/m)	Reduction coefficient	0.7EA (kN/m)	0.7EI (kNm^2/m)
0.6	2.3E+07	0.018	0.7	9.66E+06	2.90E+05

Table 8. Bracing parameter for Case 2.

Bracing layer	Bracing location	Model	A (cm^2)/pole	0.7EA (kN)	Preload pressure (kN/m)
1ST	GL. -1.8 m	1× H 350	173.9	2.51E+06	82
2ST	GL. -5.2 m	1× H 400	218.7	3.15E+06	131
3ST	GL. - 8.5 m	1× H 428	360.65	5.20E+06	163

the actual excavation construction steps were simplified in the general numerical simulation according to influential conditions such as the on-site monitoring data and construction situations. The excavation simulation assumed that the construction of the diaphragm wall had been completed. Thus, the effect of the process of constructing the diaphragm wall on the ground layers was not considered. Regarding the internal side of the foundation, pumping was used to lower the groundwater level to facilitate the excavation operation. The groundwater level was maintained at 1.0 m below the excavation surface. The following construction procedures were used for each case:

1) Case 1

(1) Stage 1 of excavation, GL. -2.7 m.
(2) First level of strut installation, GL. -1.9 m
(3) Stage 2 of excavation, GL. -4.7 m
(4) Second level of strut installation, GL. -3.9 m
(5) Stage 3 of excavation, GL. -7.1 m
(6) Third level of strut installation, GL. -6.3 m
(7) Stage 4 of excavation, GL. -10.4 m
(8) Fourth level of strut installation, GL. -9.6 m
(9) Stage 5 of excavation, GL. -13.7 m
(10) Fifth level of strut installation, GL. -12.9 m
(11) Stage 6 of excavation, GL. -17.3 m (excavated to the bottom; end of analysis mode)

2) Case 2

(1) Stage 1 of excavation, GL. -2.6 m.
(2) First level of strut installation, GL. -1.8 m
(3) Stage 2 of excavation, GL. -6.0 m
(4) Second level of strut installation, GL. -5.2 m
(5) Stage 3 of excavation, GL. -9.3 m
(6) Third level of strut installation, GL. -8.5 m
(7) Stage 4 of excavation, GL. -13.0 m (excavated to the bottom; end of analysis mode)

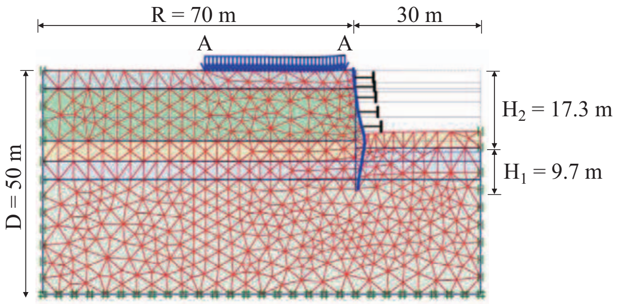

Fig. 3. Numerical grid deformation during the final excavation stage in Case 1.

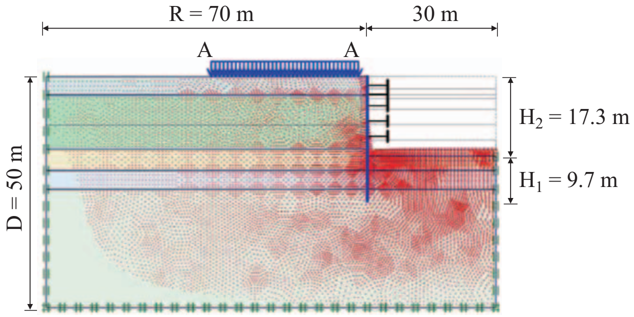

Fig. 4. Overall displacement vector during the final excavation stage in Case 1.

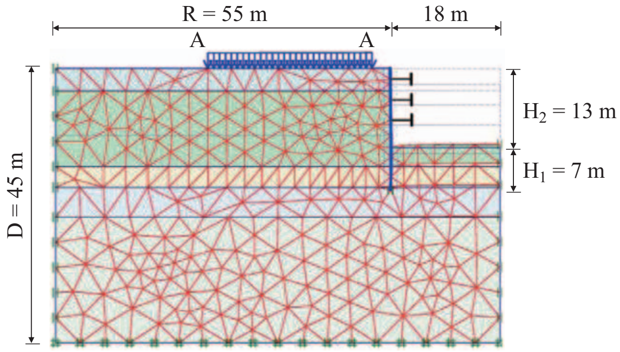

Fig. 5. Numerical grid deformation during the final excavation stage in Case 2.

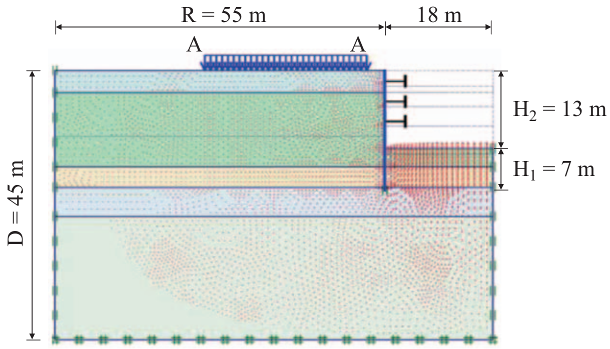

Fig. 6. Overall displacement vector during the final excavation stage in Case 2.

6. Back Analysis

Back analysis comprises inverse and direct approaches. The inverse approach involves using a reasonable hypothesis of the composition mode of the soil material and mathematical methods to represent the displacement as on-site stress and deformation functions. The displacement value is used to infer the on-site stress and the deformation modulus. The direct approach involves continually modifying the input parameters of analysis and comparing the analyzed and measured values until the difference between these values falls within the permitted error range. Although the inverse approach offers rapid and direct execution, the materials must be simplified and represented as homogenous and elastic materials, which is therefore ineffective for analyzing nonlinear materials. In the direct approach, a nonlinear analysis can be performed, and the nonlinear and elastic-plastic behavior of the materials can be evaluated. Thus, the direct approach was adopted in this study, nd Plaxis 2D was used as the analysis tool for simulating the stress-strain behavior of the foundation excavation.

The analysis simulation and comparison were conducted based on the final excavation stage. Because the coarse gravel layer in the fifth layer exhibited N values exceeding 50, the N value was assumed to be 100. The soil elastic modulus E_s was used as a variable. The analysis was conducted by gradually increasing or decreasing the soil elastic modulus E_s. Figs. 3 and 4 (Case 1) and Figs. 5 and 6 (Case 2) illustrate the resulting images of the final excavation stage analyzed using Plaxis 2D.

7. Case Results and Discussion

The results of the Plaxis 2D analysis for the final excavation stage were combined through overlay mapping with the data of the actual wall deformation curve to determine whether the wall deformation curve was within the reasonable estimation range, which was ±10% of the maximum deformation of the actual monitoring data. Figs. 7 and 8 depict the results obtained. The analysis yielded the following results:

(1) This study compared the evaluated cases with numerous monitoring data and observed that, when the wall bottom pierced into a rock stratum or gravel layer to a certain depth (1.5 m or higher), the wall bottom was not displaced horizontally. Thus, the horizontal displacement of the wall bottom was constrained during the analysis. The analysis results showed that the maximum deformation location and the tendencies of the wall deformation curve were consistent with the actual monitoring results, indicating that the basic hypotheses of the analysis were reasonable.

(2) The back results of the two cases indicated that, when the N value of the fifth layer of the coarse gravel layer was set to 100 and the soil elastic modulus E_s was within 10% of the maximum deformation of the actual monitoring data,

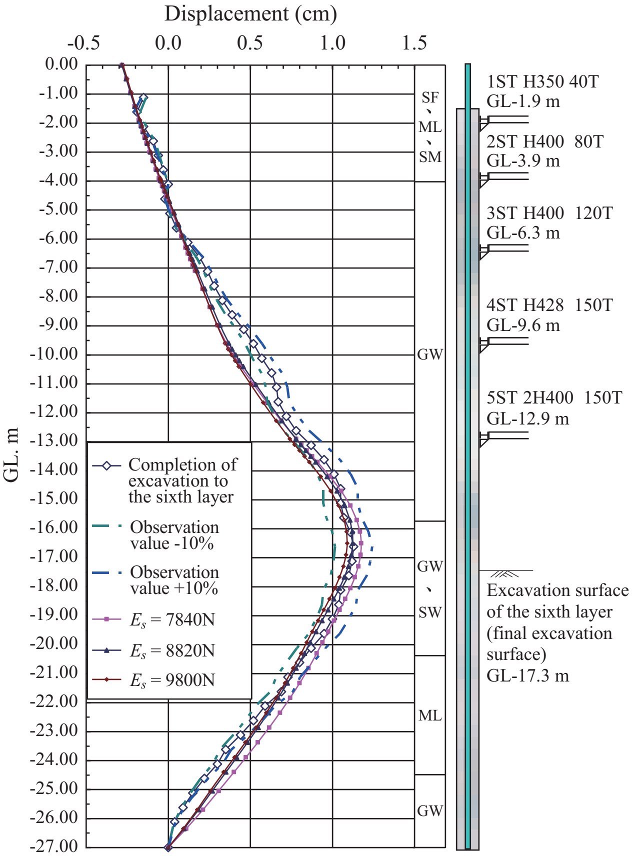

Fig. 7. Comparison between observation and simulation of lateral displacement for the final excavation stage of Case 1.

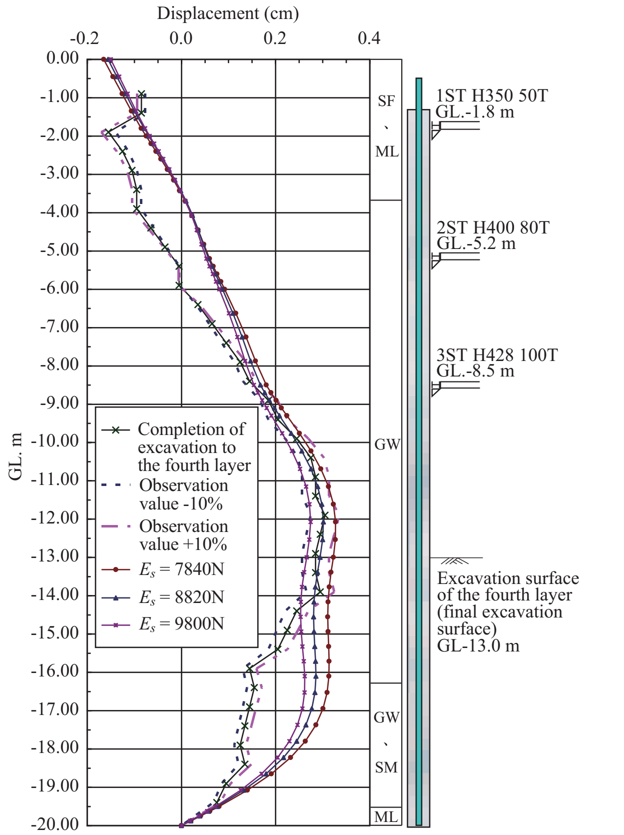

Fig. 8. Comparison between observation and simulation of lateral displacement for the final excavation stage of Case 2.

the soil elastic modulus E_s ranged between 7.84N and 9.8N MPa. The maximum deformations and deformation locations at the final excavation stage could be reasonably estimated in the evaluated stratum environment.

IV. CONCLUSION

This study used Plaxis 2D software to perform a back analysis of the parameters of two cases involving deep excavation; the analysis results were compared with the monitoring data of the actual wall deformations. The analysis results indicated that the simulations of the tendencies and locations of the maximum wall deformations produced satisfactory outcomes. Consequently, this simulation method was deemed to be satisfactory. The study yielded the following research conclusions:

(1) The analysis results of the actual situations indicated that the soil elastic modulus E_s of the gravel layer in the Xindian District ranged between 7.84N and 9.8N MPa. The maximum deformations at the final excavation stage could thus be satisfactorily estimated. The estimated results were similar to the elastic modulus E_s ranging from 88.2 to 833 MPa that was empirically determined by Kuo (1999) and Hou (2001), who performed studies on the gravel layers in Bagua Mountain.

(2) The actual situations in the Xindian District were used to perform the back analysis of the parameters. Future studies can investigate gravel layers in different regions by using the procedures presented in this study to attain progressively extensive research results that could serve as a reference for excavation designers.

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