

Article

Estimation of Excavation Wall Displacements Using the Optimized Zoning Method for Soil Deformation Modulus in 2D FEM [†]

Hiroto Kumagai ^{1,2}, Kazuhiro Kaneda ³ and Toshiro Hata ^{2,*} 

¹ Takenaka Corporation, 1-5-1 Ohtsuka, Inzai 270-1395, Chiba, Japan; kumagai.hiroto@takenaka.co.jp or hkuma10621007@gmail.com

² Department of Civil and Environmental Engineering, Hiroshima University, 1-4-1 Kagamiyama, Higashi-Hiroshima 739-8527, Hiroshima, Japan

³ Department of Civil and Environmental Engineering, Chiba Institute of Technology, 2-17-1 Tsudanuma, Narashino 275-0016, Chiba, Japan; kazuhiro.kaneda@chibatech.ac.jp

* Correspondence: thata@hiroshima-u.ac.jp

[†] This paper is an extended version of the conference paper entitled “Evaluation of Retaining Wall and Surrounding Ground Behavior in Soft Cohesive Soil by FEM Analysis”, published in the Proceedings of GEOTECH HANOI 2023.

Abstract

Accurate prediction of retaining wall displacements is essential to ensure safety and protect adjacent structures in large-scale urban excavations. However, inherent uncertainties in soil parameters often limit prediction accuracy, and cumulative deviations may result in significant overestimation or underestimation of wall displacements at the final excavation stage. Although observational construction methods and inverse analysis approaches have been proposed to sequentially update parameters using early-stage monitoring data, these studies have primarily focused on numerical values, such as deformation modulus. However, limited attention has been paid to the spatial extent over which these parameters are assigned. This study introduces a method for optimizing the zoning width of the soil deformation modulus by incorporating monitoring data from the initial excavation stages into finite element analysis. A Mohr–Coulomb constitutive model was adopted, and the maximum wall displacement was used as the primary evaluation index. The optimal zoning width was defined as the condition that minimized the difference between measured and simulated displacements. The results demonstrate that the proposed optimization approach improves the predictive accuracy of wall displacements in later excavation stages. The proposed method was validated using a single excavation case; therefore, further verification using multiple case histories is required to confirm its broader applicability.

Keywords: finite element method; deep excavation; observational method; zoning width; deformation modulus



Academic Editor: Honggui Di

Received: 3 June 2026

Revised: 9 July 2026

Accepted: 30 July 2026

Published: 4 August 2026

Copyright: © 2026 by the authors.

Licensee MDPI, Basel, Switzerland.

This article is an open access article distributed under the terms and conditions of the [Creative Commons Attribution \(CC BY\)](https://creativecommons.org/licenses/by/4.0/) license.

1. Introduction

In large-scale urban excavation projects, accurate prediction of retaining wall and surrounding ground deformation behaviour is essential to ensure safety and protect adjacent structures. However, soil parameters used in excavation analyses are subject to significant uncertainty, and predictions based solely on laboratory tests or standard design values often fail to achieve sufficient accuracy. As construction progresses, prediction errors may accumulate, leading to overestimation or underestimation of displacements at the final

excavation stage. This issue has been widely recognized as a critical challenge in excavation engineering. To address this problem, previous studies have proposed methods for sequentially updating soil parameters using early-stage measurement data within the framework of observational construction methods and back analysis [1,2]. Recent studies have further improved prediction accuracy through inverse analysis using field monitoring data and intelligent optimization techniques [3,4]. In addition, advanced constitutive models considering small-strain stiffness have been successfully applied to improve the prediction of wall deformation and ground movements around deep excavations [5]. While these approaches have demonstrated effectiveness, most existing studies have primarily focused on numerical parameters, such as deformation modulus [6,7]. However, soil stiffness is not uniformly distributed in excavation problems because the stress state and strain level vary significantly with distance from the retaining wall and excavation depth. As excavation progresses, changes in confining pressure and strain levels lead to spatial variations in soil stiffness. If a single deformation modulus is assigned to a wide region, the numerical model may fail to capture these spatial variations, resulting in discrepancies between predicted and observed displacements. Therefore, appropriate zoning of the deformation modulus is important to represent the spatial variability in soil stiffness and to improve the predictive capability of numerical simulations. Nevertheless, only limited studies have investigated how the spatial extent of these zones should be determined.

Meanwhile, fundamental concepts related to ground deformation and stress redistribution during excavation have been well established in classical soil mechanics and excavation studies [8–10].

A preliminary version of this study was presented in the Proceedings of GEOTECH HANOI 2023 [11], where the behavior of retaining walls and surrounding ground in soft cohesive soil was investigated using two-dimensional finite element analysis. Building on that work, the present study substantially extends the previous conference paper by introducing an optimization method for the zoning width of the soil deformation modulus using monitoring data obtained during the early excavation stages. In addition, the present paper includes expanded numerical analyses, validation of the proposed method, and more comprehensive discussions.

This study proposes a method for adjusting the zoning width of the soil deformation modulus using monitoring data obtained during the early excavation stages. Finite element analysis (FEM) with a Mohr–Coulomb constitutive model is employed, and the maximum wall displacement is used as the evaluation index. The zoning width is optimized by minimizing the difference between measured and simulated displacements, with the aim of improving prediction accuracy at the final excavation stage.

2. Background and Literature Review

The observational construction method is a framework that uses measurement data collected during construction to sequentially modify design values and has been widely applied in engineering practice. In the field of inverse analysis, various studies have sought to identify ground parameters from measured data using approaches such as Bayesian updating and Kalman filtering [12,13]. These approaches have been proven effective in improving consistency between numerical analyses and field measurements. Numerous studies have investigated the mechanisms of wall deformation and ground settlement induced by excavations through field measurements and numerical analyses [14–16]. Furthermore, several extensions of inverse analysis and observational construction frameworks have been proposed to improve the consistency between numerical predictions and field measurements in excavation problems [17–20].

Despite these advances, most existing studies have focused on identifying parameter values, while systematic investigations into the determination of zoning ranges have been limited. In practice, zoning ranges are often determined based on the designer's experience. If their validity is not adequately ensured, there is a risk that errors may expand as the excavation progresses, even if consistency is achieved at the early stages. Although recent studies have proposed dynamic inverse analysis frameworks that utilize real-time construction data for sequential updating, the optimization of parameter application ranges has not been explicitly addressed.

Therefore, this study has both academic and practical significance, as it presents a new framework that treats zoning ranges as design variables and optimizes them using measurement data. The limit state of the soil was evaluated using the Mohr–Coulomb constitutive model, and soil behavior was assumed to remain elastic until the failure criterion was reached. The Mohr–Coulomb model was adopted because it is widely used in practical excavation analyses and requires a limited number of input parameters. Although soil behavior during excavation is inherently nonlinear, the nonlinear response was approximated by reducing the elastic stiffness within the defined zoning regions.

3. Materials and Methods

3.1. Optimized Zoning Method for Soil Deformation Modulus in 2D FEM

In actual ground conditions, the deformation modulus during excavation varies depending on the excavation stage and location due to changes in confining pressure and shear strain levels within the ground. However, in FEM analyses used for practical design, it is difficult to assign deformation moduli to each individual mesh element. Therefore, this study introduced a zoning approach as a simplified method to represent the spatial distribution of ground stiffness, and the ground was divided into five areas. Specifically, Areas 1–3 represent regions below the excavation base where both confining pressure changes and shear strain effects were considered, Area 4 represents a region where stiffness reduction due to shear strain was considered, and Area 5 represents a region assumed to be unaffected by excavation.

The deformation modulus E in Areas 1 to 4 was set by multiplying the initial deformation modulus E_{ed} by the stiffness reduction rate α . The representative stiffness reduction rate α for each area can be set based on the change in confining pressure and shear strain at the representative position of each area. On the other hand, on the excavation side, the extent of the stiffness reduction zone is complex due to stress release and shear deformation induced by excavation, making it difficult to uniquely define the extent of the affected region. Furthermore, the width of the zone defined adjacent to the retaining wall on the excavation side influences the distribution of the apparent deformation modulus in the numerical model, which in turn affects the prediction of retaining wall displacement.

Therefore, in this study, the deformation modulus in each area (X) was fixed, while the zoning width X_i , representing the spatial extent of the stiffness reduction zone, was varied to investigate prediction accuracy. The evaluation criterion was defined as the absolute error between the measured and simulated maximum wall displacements, and the zoning width was optimized to minimize this error (Figure 1). In this study, the width of the ground behind the retaining wall (X_b) was fixed to the excavation depth, based on the assumption that significant shear strains occur within a zone extending at 45° from the excavation base. The excavation region width (X_e) was also fixed at half of the excavation width because a half-model was employed under symmetric boundary conditions. Consequently, the width of the zone adjacent to the retaining wall (X_i) was treated as the primary parameter in the parametric study. Specifically, monitoring data obtained during the second and third excavation stages were used to determine the optimal zoning width at each stage. The

optimized zoning configuration was then applied to subsequent analyses to evaluate its effectiveness in improving prediction accuracy at the final excavation stage.

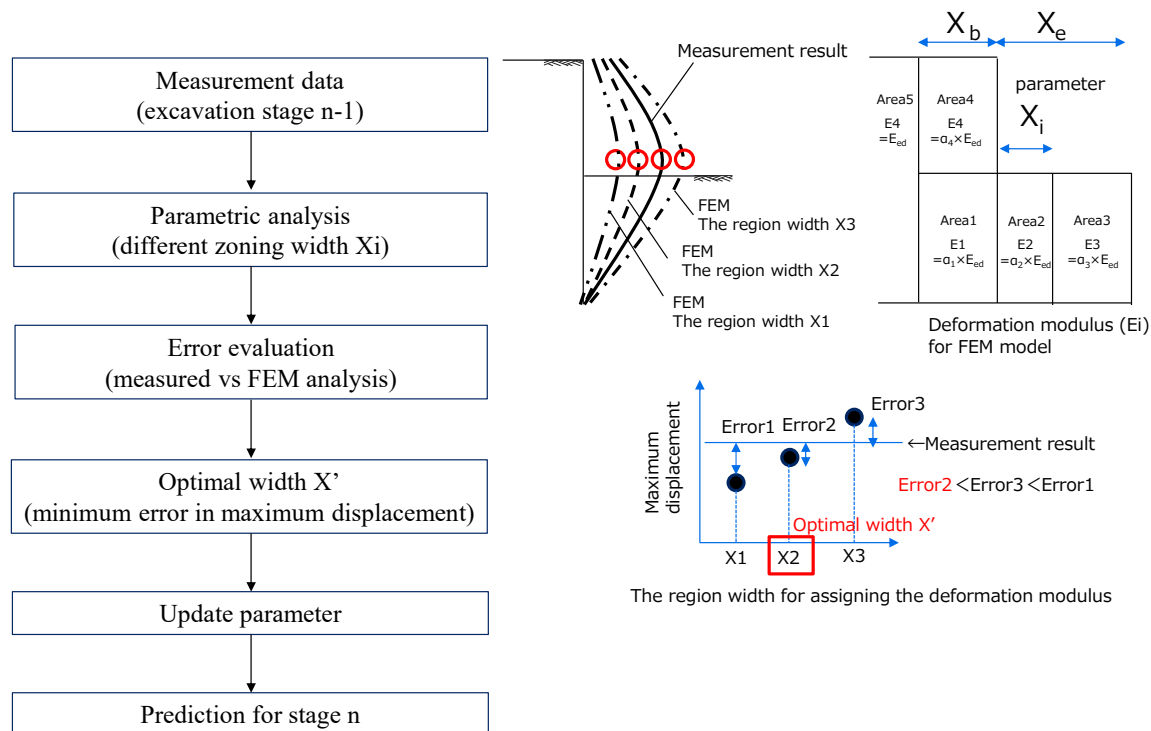


Figure 1. Optimization zoning method for soil deformation modulus.

Although geometric nonlinearity is also important, the present analysis was not considered a large deformation problem. Therefore, the analysis was treated as a small deformation problem using PLAXIS (Seequent, Bentley Systems).

3.2. Study Area and Project Overview

The case study was conducted at a deep excavation site with a plan dimension of 87 m $\times$ 56 m and an excavation depth of GL-12.1 m (GL: ground level). The site is located in proximity to a subway structure that is 14.3 m wide and extends to a basement level of GL-14.7 m. The horizontal distance between the subway structure and the retaining wall ranged from 5 to 10 m. To minimize the construction duration and mitigate its impact on the surrounding environment, an underground construction method known as the top-down approach was implemented.

The retaining wall was constructed using a soil–cement mixture incorporating H-shaped steel beams, with due consideration given to water permeability. The toe depth of the soil–cement wall was set at GL-38.0 m, while the toe depth of the H-shaped steel beams was set at GL-32.5 m. The plan layout of the site is shown in Figure 2, and the cross-sectional views are shown in Figure 3.

The subsurface profile includes fill, alluvial sand, and alluvial silt extending to approximately GL-24 m. Below this depth, layers of diluvial silt and diluvial gravel are present (Figure 3). The primary soil layer excavated in this project is an alluvial cohesive (AC) layer, characterized by an N-value of approximately 1–3, a shear wave velocity (V_s) of approximately 140 m/s (measured by PS logging test), and a thickness exceeding 10 m. The alluvial cohesive layer is characterized by low stiffness and high compressibility, making it the dominant layer governing excavation-induced deformation. In contrast, the underlying diluvial deposits exhibit significantly higher stiffness and act as the primary

bearing stratum. The groundwater level was observed at GL-1.0 m, while the pressure head of the diluvial gravel layer was approximately GL-1.6 m.

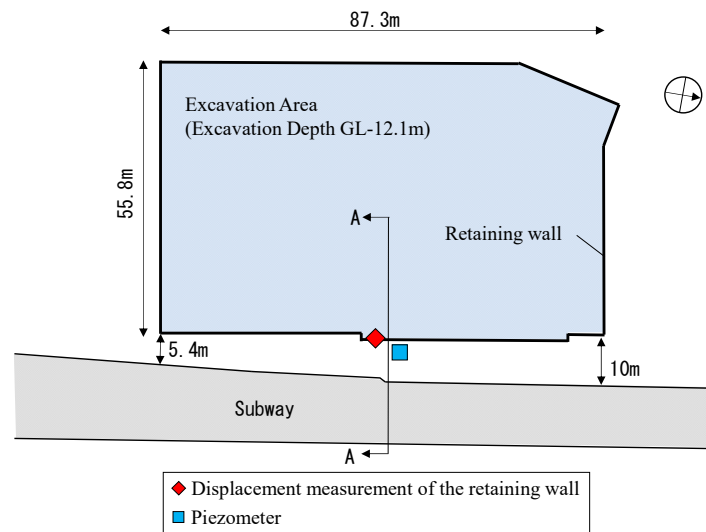


Figure 2. Plan view of construction and measurements.

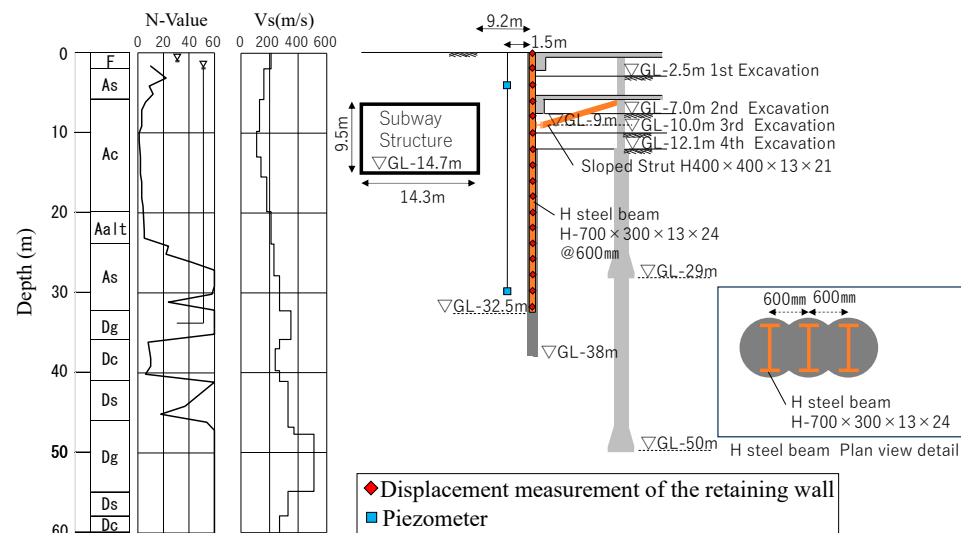


Figure 3. Soil profile and construction scheme.

3.3. Field Measurements

Figures 2 and 3 present comprehensive overviews of the measurement scheme employed in this study. The measurements included horizontal displacements of the retaining wall and variations in groundwater level. Specifically, the ground water level was measured at locations approximately 1.5 m from the wall.

The fixed inclinometers were used to measure the displacement of the retaining wall, and piezometers were used to measure the water pressure in the behind ground.

Figure 4 illustrates the measured horizontal displacement profiles of the retaining walls and the surrounding ground. The maximum displacement recorded for the retaining wall was 53 mm, occurring at a depth of approximately GL-12 m, which corresponds to the excavation level. Additionally, the groundwater level behind the retaining wall remained stable and was not affected by pumping activities within the excavation area.

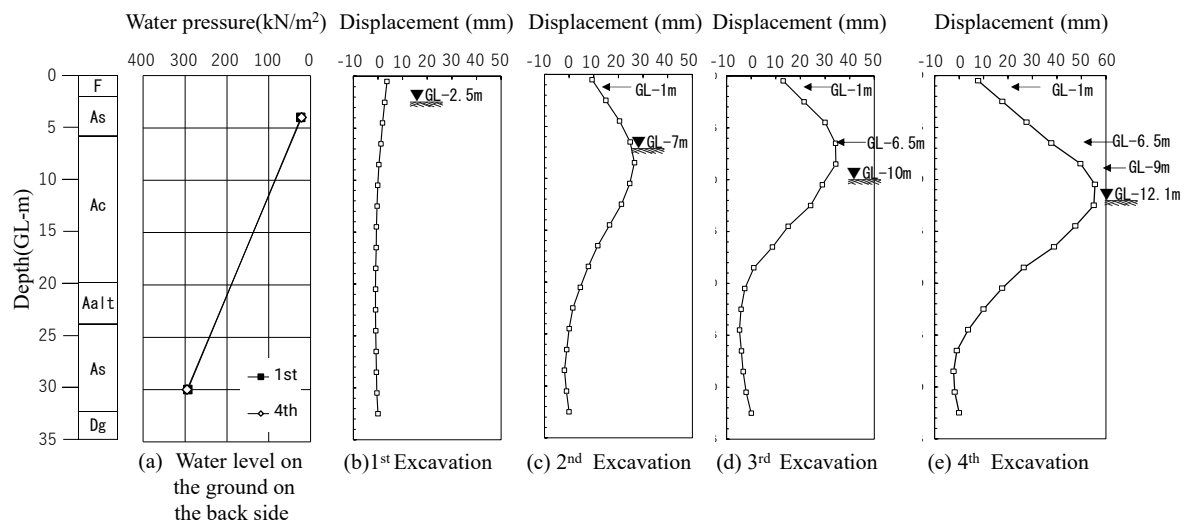


Figure 4. Horizontal displacement measurement results of the retaining wall.

3.4. 2D Finite Element Analysis

The behavior of the retaining wall and the surrounding ground was analyzed using the two-dimensional (2D) finite-element software PLAXIS 2D version 2025.1.3.5 (Seequent, Bentley Systems). Figure 5 shows the FEM analysis model employed in this study. The analyzed area extended 60 m horizontally from the retaining wall, which corresponds to approximately five times the excavation depth of 12.1 m. Vertically, the analysis covered a distance of 48 m from the excavation bottom, which is approximately 0.9 times the excavation width of 56 m.

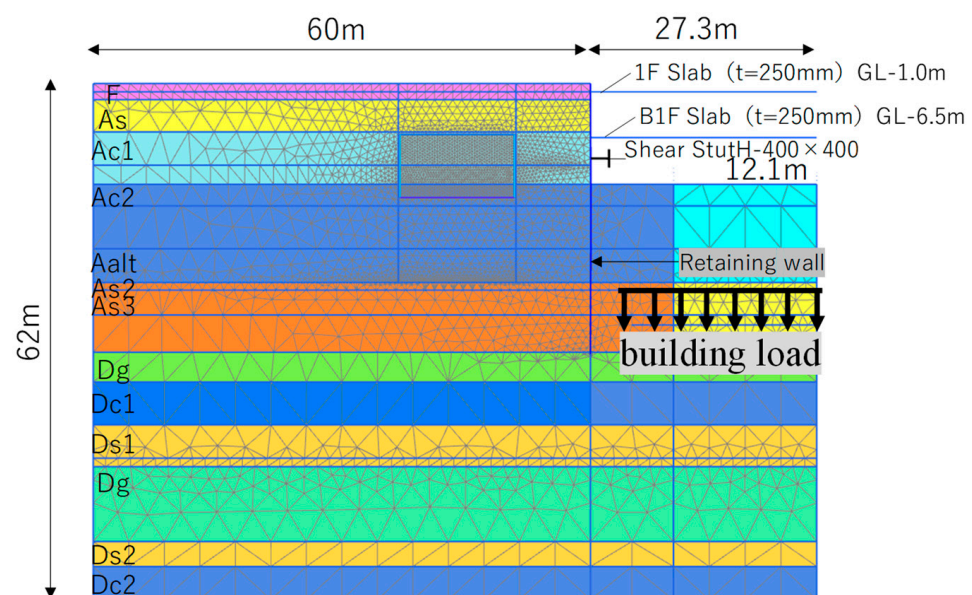


Figure 5. Finite element model.

The applicability of 2D FEM and the constitutive soil models employed in this study has been verified in numerous excavation analyses reported in the literature.

The boundary conditions were defined such that vertical rollers were applied to the lateral boundaries and fixed constraints were imposed at the bottom boundary. This 2D analysis involved a step-by-step approach to simulate the behavior of the retaining wall and surrounding ground during each excavation stage. Soil behavior was modeled using an elastic–perfectly plastic constitutive model for all soil layers. In this model, soil responds elastically up to the yield stress τ , beyond which plastic deformation occurs while the

stress remains constant. The yield stress was determined using the Mohr–Coulomb failure criterion. The Mohr–Coulomb model was selected because it requires a relatively small number of input parameters and is widely used in practical geotechnical engineering. In addition, it is implemented as a standard constitutive model in many commercial finite element programs, not only PLAXIS, making the proposed zoning approach readily applicable to various numerical analysis platforms. Although more advanced constitutive models, such as the Hardening Soil (HS) and Hardening Soil with Small-Strain Stiffness (HSS) models, may further improve prediction accuracy by considering stress-dependent stiffness and unloading–reloading behaviour, they require a larger number of material parameters. Therefore, the application of the proposed zoning approach to these advanced constitutive models will be investigated in future studies. The retaining wall and floor slabs, which act as supports, were modeled using linear-elastic plate elements. In addition, a spring element was used to represent the shore strut installed at GL-9 m.

Table 1 summarizes the soil parameters used in this analysis. All soil layers were modeled under drained conditions. Poisson’s ratio was set to 0.3 for sandy soils and 0.4 for cohesive soils. Regarding the Poisson ratio, the change in the process of strain change is not taken into account in the design of the retaining wall, so the Poisson ratio is treated as constant in this study. The cohesion (c) and internal friction angle (φ) of the AC layer were determined based on consolidated undrained triaxial compression test with pore pressure measurement results. The coefficient of earth pressure at rest K_0 was determined using Jaky’s formula, expressed as $K_0 = 1 - \sin \varphi$ [21]. Table 2 summarizes the specifications of the retaining wall and struts.

Table 1. Soil types and their geotechnical parameters.

Soil Layer	Layer Bottom Level (m)	Unit Weight (kN/m ³)	Poisson’s Ratio	c (kN/m ²)	φ (°)	E_{ed} (MPa)	K_0
F	2.0	18.5	0.4	10.0	29	233	0.52
As1 *	5.8	18.5	0.3	0	31	126	0.48
Ac1 **	12.1	17.0	0.4	8.3	31	95	0.48
Ac2 **	19.9	17.0	0.4	7.2	29	95	0.52
Aalt **	23.9	17.0	0.4	9.9	32	95	0.47
As2	24.8	19.0	0.3	0	36	278	0.41
As3	32.3	19.0	0.3	0	42	300	0.33
Dg	35.9	20.5	0.3	0	45	666	0.29
Dc1	41.0	17.0	0.4	87.0	0	328	1.0
Ds1	46.0	18.3	0.3	0	45	437	0.29
Dg	55.0	20.0	0.3	0	45	1027	0.29
Ds2	58.0	19.5	0.3	0	45	563	0.29
Dc2	62.0	16.5	0.4	200.0	0	344	1.0

*. The values cohesion (c) and internal friction angle (φ) of were determined based on CD triaxial compression test result. **: The values cohesion (c) and internal friction angle (φ) of were determined based on CU triaxial compression test with pore pressure measurement result.

The detailed assignment of the deformation modulus is described in Section 3.4. For the hydraulic boundary conditions, the groundwater level was maintained at GL-1.0 m. During excavation, the groundwater level in the excavation area was set to GL-3.0 m in the first stage, GL-9.0 m in the second stage, and GL-13.0 m in the third and fourth stages. In addition, the building load was applied as an average contact pressure at a depth of GL-29

m in the excavation area. The applied building load was updated at each construction stage to represent the increase in structural dead load associated with the progress of the top-down construction method. Table 3 summarizes the construction steps and applied loads used in the analysis.

Table 2. Specifications of the retaining wall and struts.

	Young's Modulus (kN/m ²)	The Second Moment of Area (m ⁴ /m)	Cross-Sectional Area (m ² /m)
Retaining wall (H-beam)	2.1×10^8	1.97×10^{-3}	0.0385
Strut	1F RC Slab	2.1×10^7	1.3×10^{-3}
	B1F RC Slab	2.1×10^7	1.3×10^{-3}
	Strut (H-beam)	2.1×10^8	0.006

Table 3. Analysis steps and load.

Step	Status	Groundwater Level Behind the Wall	Groundwater Level in Excavation Area	Building Load (kN/m ²)
Step1	Initial stress analysis	GL-1 m	GL-1 m	-
Step2	Installation of retaining wall and nearby structures		GL-1 m	-
Step3	1st Excavation (GL-2.5 m)		GL-3 m	-
Step4	1F Slab construction		GL-3 m	-
Step5	2nd Excavation (GL-7.0 m)		GL-9 m	15
Step6	B1F Slab construction		GL-9 m	15
Step7	3rd Excavation (GL-10.0 m)		GL-13 m	47.4
Step8	Installation of shore strut		GL-13 m	47.4
Step9	4th Excavation (GL-10.0 m)		GL-13 m	100.2

Different deformation modulus values were assigned to specific zones within the ground. Figure 6 shows the zoning scheme used in the analysis. The zoning configuration consisted of the following areas: The widths of Areas 4 and 1 were set to 12 m, which is approximately equivalent to the excavation depth. This value was adopted based on the assumption that stiffness reduction caused by shear strain is likely to occur within a zone of this extent around the excavation. A preliminary sensitivity analysis was conducted by changing the widths of Areas 1 and 4 from 6 m to 12 m. The calculated maximum wall displacement differed by less than 1 mm, indicating that the influence of these widths on the predicted wall behavior was negligible within the investigated range. Therefore, the widths of Areas 1 and 4 were fixed at 12 m to reduce the number of optimization parameters, and only the width of Area 2 was treated as the optimization parameter.

- Area 1: Behind the retaining wall and deeper than the excavation;
- Area 2: Near the retaining wall on the excavation side;
- Area 3: Center of excavation area;
- Area 4: Shallower than excavation depth;
- Area 5: An area 12 m or more away from the retaining wall to the behind side, which corresponds to the excavation depth.

Notably, the parametric study focused on optimizing the width X of Area 2, which was considered to have a dominant influence on wall deformation. The zoning width X was varied from 1 to 12 m, and representative values of 1, 2, 4, 6, 8, 10, and 12 m were

examined. The range of zoning width X (1–12 m) was determined based on preliminary parametric analyses conducted prior to the main analysis.

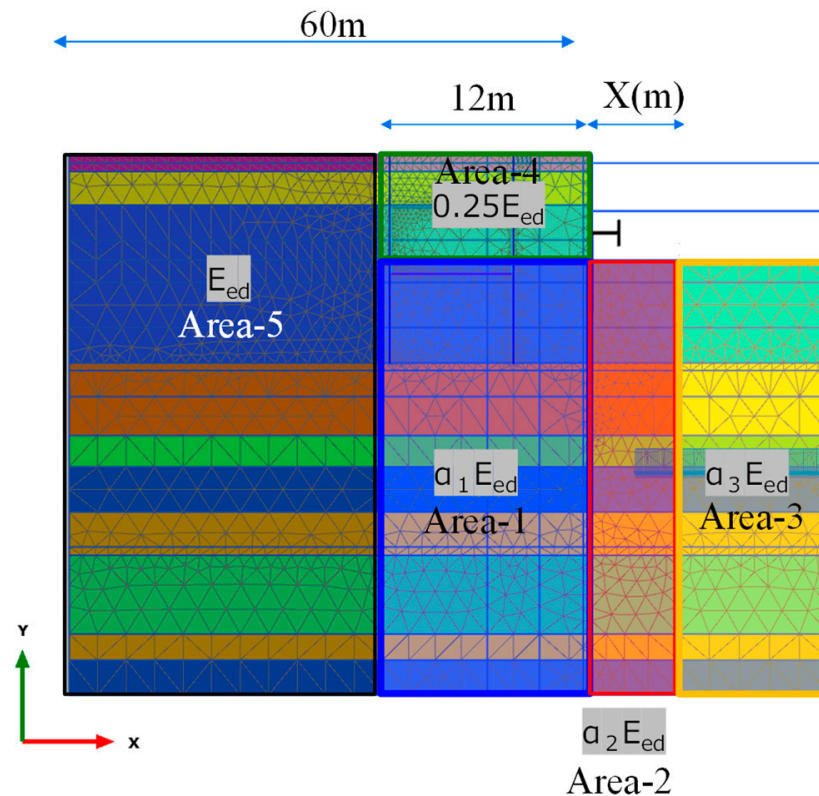


Figure 6. Zoning of the deformation modulus.

The deformation modulus for Areas 1 to 3 was determined by considering the initial deformation modulus (E_{ed}) and the stiffness reduction rate (α) induced by excavation. The value of E_{ed} was calculated from the shear wave velocity V_s obtained from PS logging using Equation (1). Since it is a deformation modulus due to excavation and unloading, it is thought that the deformation modulus does not change even if the stress state of the ground is overconsolidated state or normally consolidated state. The stiffness reduction rate (α) was calculated using Steinbrenner's elastic solution [22] by evaluating the changes in confining pressure and shear strain during excavation (for example, unloading: 214 kN/m^2 ; building load: 100 kN/m^2). α is calculated by multiplying the stiffness reduction rate α_{mR} due to changes in confining pressure by the stiffness reduction rate α_s due to strain. α_{mR} is calculated using Equation (3), and α_s is calculated using Equation (4). Figure 7a illustrates the calculation line for the stiffness reduction rate, while Figure 7b shows the resulting stiffness reduction rate α . For each excavation stage, the calculations were performed at a representative point located 2 m below the excavation bottom, where the influence of excavation-induced stress release was considered to be significant. The stiffness reduction rate α was calculated for each excavation stage, and the average value obtained from all excavation stages was assigned to each zoning region as an input parameter before performing the FEM analysis. The deformation modulus of Area 4, located shallower than the excavation depth behind the retaining wall, was set to one-quarter of the initial deformation modulus (E_{ed}) obtained from shear wave velocity measurements. It has been reported that in the shear strain range of 10^{-4} to 10^{-3} , the shear stiffness of the ground decreases to approximately 20–40% of the initial shear stiffness. Based on this, it is thought that a similar strain level will occur in the ground behind the earth retaining wall due to excavation, so the reduction rate for Area 4 was set to 0.25 in this analysis. The

deformation modulus of Area 5 was assigned as the small-strain deformation modulus E_{ed} . Previous studies have extensively investigated strain-dependent stiffness characteristics and small-strain soil behavior, providing a basis for evaluating stiffness reduction during excavation [23–25]. Figure 8 shows the deformation modulus assigned to each zoning area at the forth excavation stage.

$$E_{ed} = 2\rho V_s^2(1 + \nu) \quad (1)$$

where

E_{ed} : Elastic modulus derived from PS logging;

ρ : Density of soil;

V_s : Shear wave velocity;

ν : Poisson's ratio.

$$\alpha = \alpha_{mR} \times \alpha_s \quad (2)$$

$$\alpha_{mR} = \sqrt{\frac{1 + \sqrt{\sigma'_{z0}/\sigma'_z}}{2 \times (\sigma'_{z0}/\sigma'_z)}} \quad (3)$$

$$\alpha_s = \gamma_{mR} / \gamma_R \quad (4)$$

where

α : The stiffness reduction rate;

α_{mR} : The stiffness reduction rate due to changes in confining pressure;

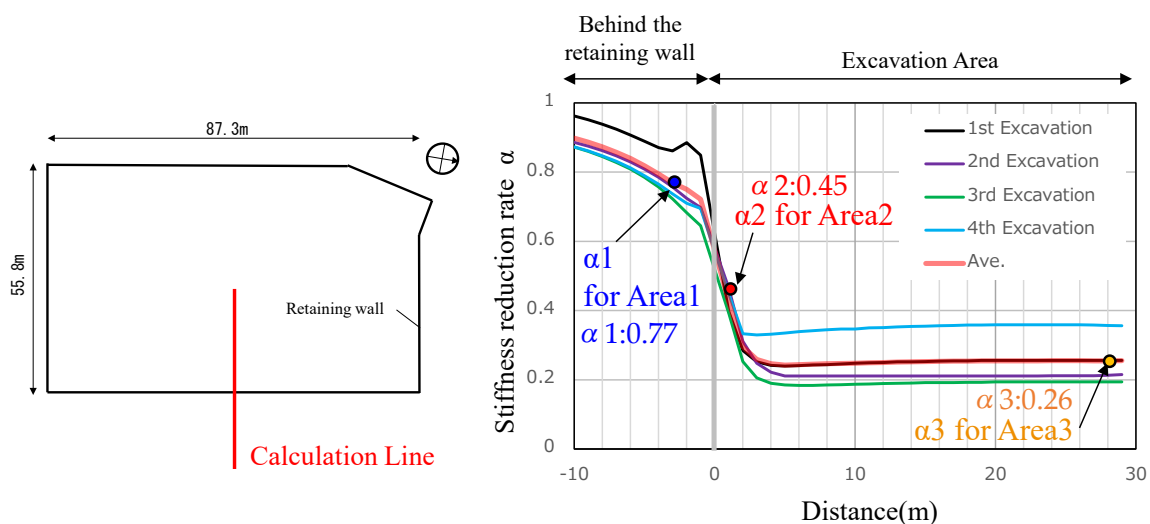
α_s : The stiffness reduction rate due to strain;

σ'_{z0} : Initial effective stress;

σ'_z : Effective stress after excavation;

γ_{mR} : Shear strain in the ground considering stiffness changes associated with variations in confining pressure;

γ_R : Shear strain considering stiffness reduction due to strain.



(a) Calculation line for calculating the stiffness reduction rate

(b) Stiffness reduction rate α

Figure 7. Stiffness reduction rate.

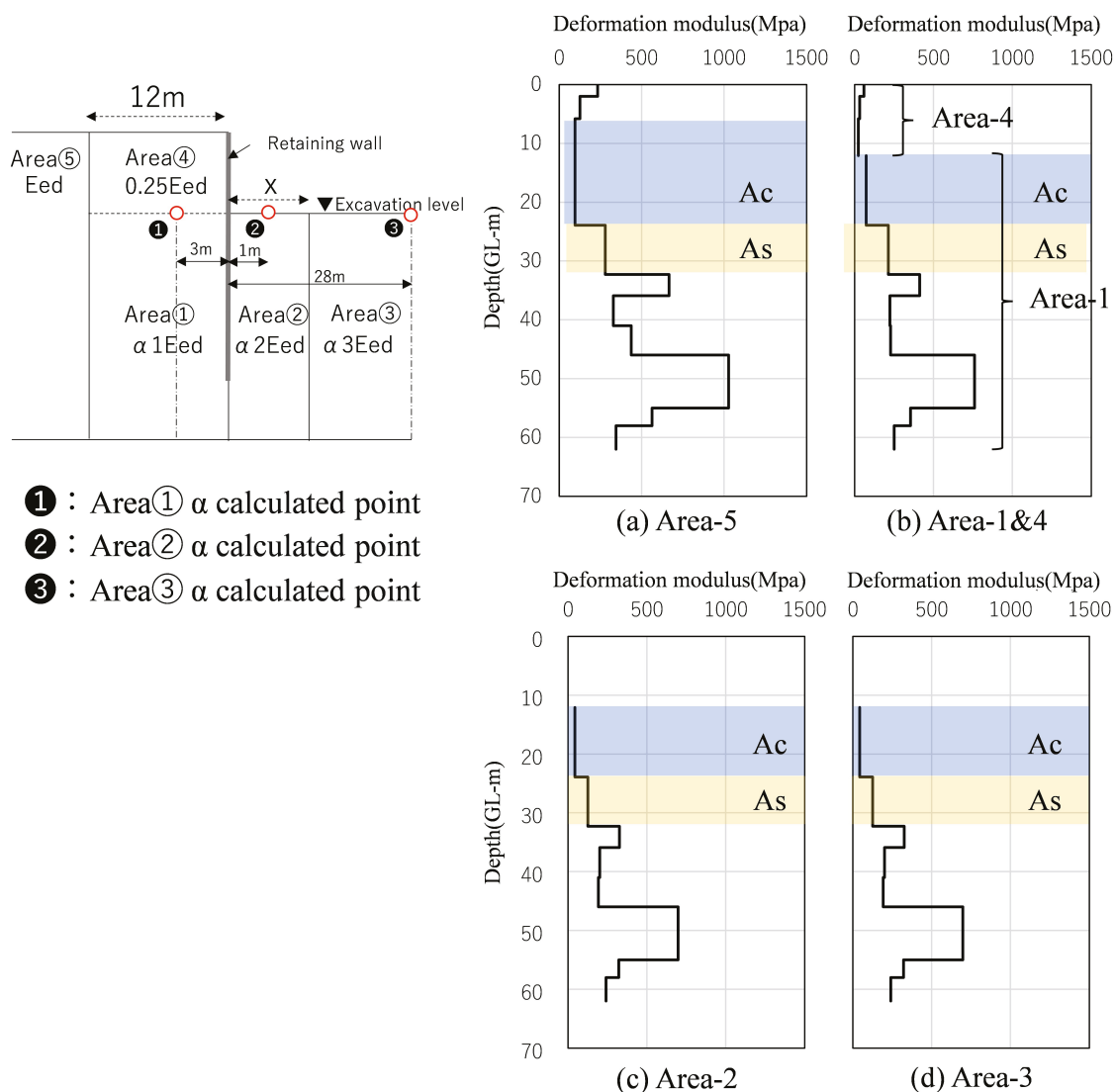


Figure 8. Deformation modulus of each area at the fourth excavation stage.

4. Results and Discussion

4.1. Retaining Wall Displacement

Figure 9 shows the horizontal and vertical displacement contours for a model with $X = 1$ m, and Figure 10 shows the horizontal and vertical displacement contours for a model with $X = 12$ m.

Figure 11 compares the measured horizontal displacement of the retaining wall with the results obtained from the FEM-based parametric study. During the first excavation stage, the maximum measured displacement occurred at the top of the wall, whereas the FEM analysis indicated the maximum displacement of approximately GL-10 m. From the second excavation stage onward, both the measured and simulated maximum displacements occurred near the excavation level, indicating that the FEM analysis reasonably reproduced the overall displacement distribution. Figure 12 shows the relationship between maximum displacement of retaining wall and excavation depth. Among the analysis cases, the model with an Area 2 width of $X = 12$ m exhibited smaller wall displacements than the model with $X = 1$ m. This trend can be attributed to the expansion of the zone with a higher deformation modulus, which increases the apparent stiffness of the excavation area. In this study, the maximum wall displacement was adopted as the primary evaluation index because it is one of the most important performance indicators in excavation design.

and construction management. Although the displacement distribution, including the depth of the maximum displacement and wall-top displacement, is also important for evaluating retaining wall behavior, the present study focused on the maximum wall displacement to clarify the influence of the zoning width on prediction accuracy. Furthermore, since this study is based on a single case history, the applicability of the proposed method should be verified using additional excavation projects. A more comprehensive evaluation incorporating multiple performance indices and case histories will be conducted in future studies.

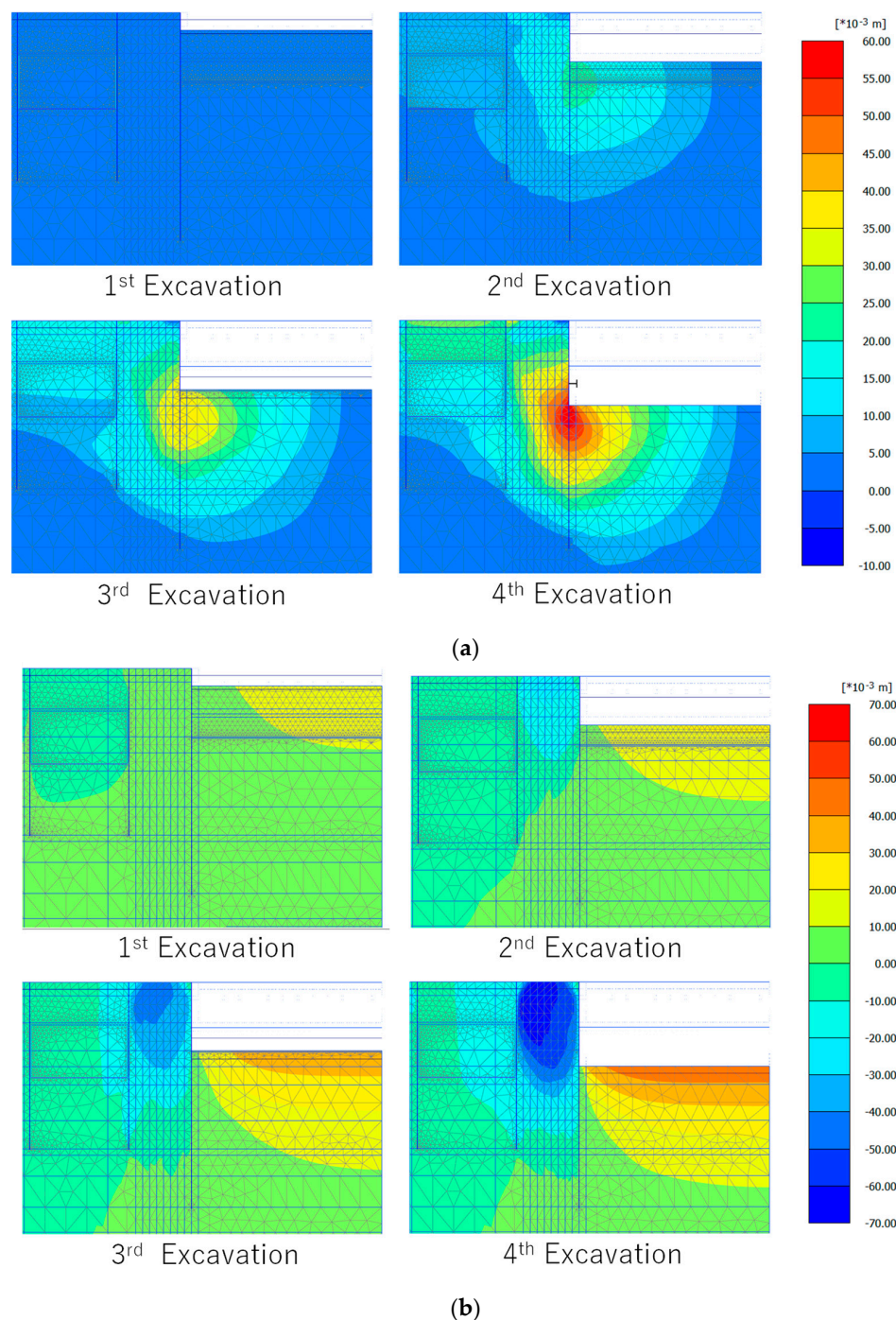


Figure 9. Contour distributions of horizontal and vertical displacement $X = 1$ m model. (a) Horizontal displacement. (b) Vertical displacement.

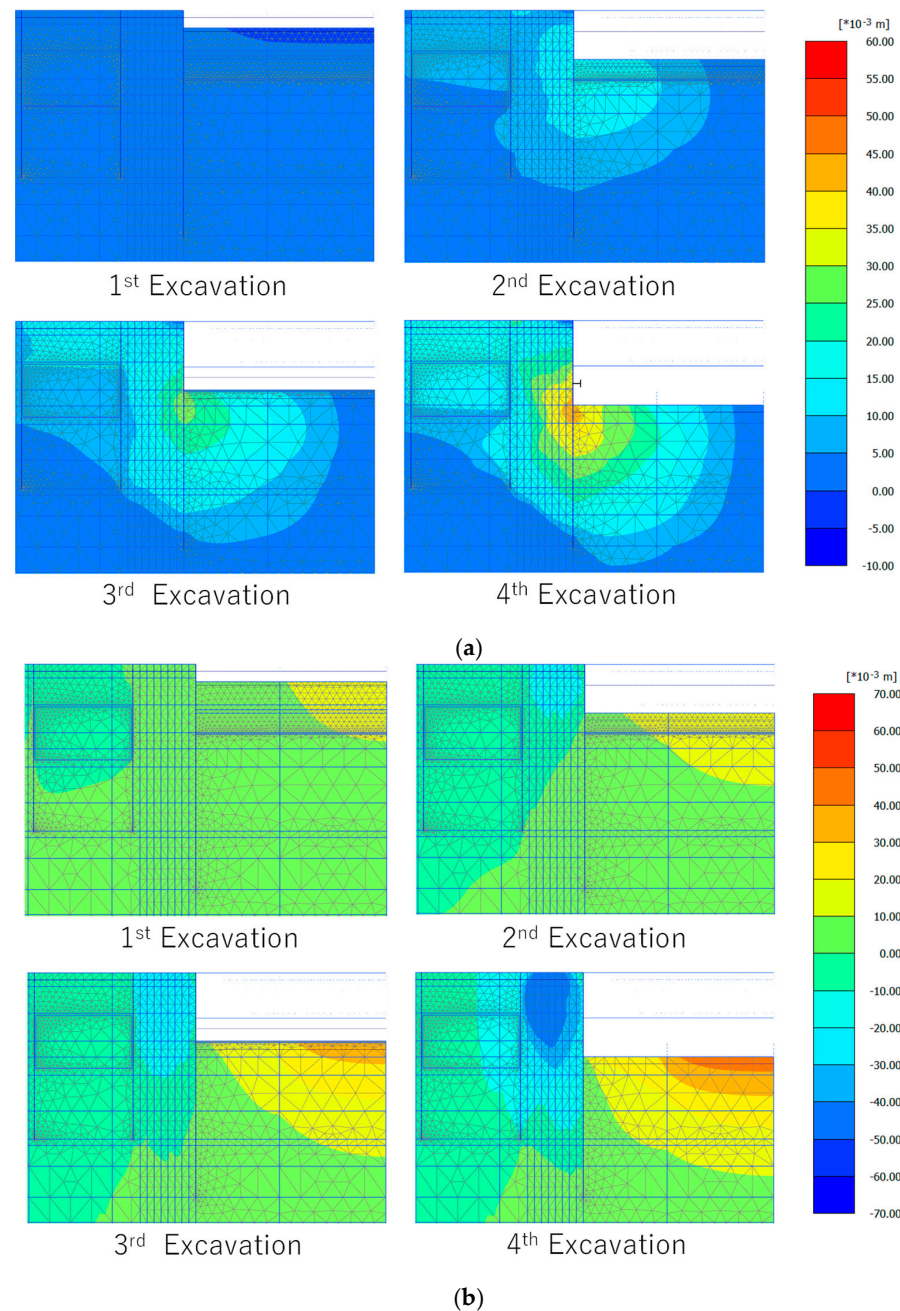


Figure 10. Contour distributions of horizontal and vertical displacement $X = 12$ m model. (a) Horizontal displacement. (b) Vertical displacement.

4.2. Difference in Maximum Displacement of the Retaining Wall

To determine the optimal zoning width, the absolute errors between the measured and simulated maximum wall displacements were evaluated for the first time through three excavation stages (Figure 13). During the first stage, the displacement error remained below 1 mm for all cases, indicating negligible sensitivity to the zoning width. In the second stage, the error decreased as the zoning width decreased, reaching nearly zero at $X = 2$ m. Conversely, during the third stage, the error decreased with increasing zoning width and reached a minimum at $X = 8$ m, after which it increased again. Based on these results, the optimal zoning widths were identified as 2 m for the second excavation stage and 8 m for the third stage, corresponding to the conditions that minimized the displacement error. Compared with the central excavation area, the zone with relatively higher ground stiffness,

defined as X, expanded from approximately 2 m in the second stage to approximately 8 m in the third stage.

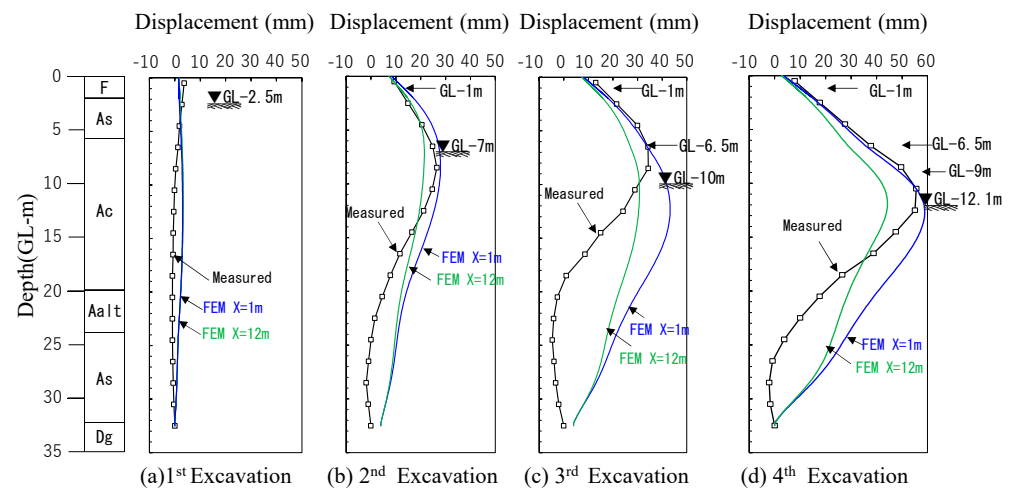


Figure 11. Simulated and measured displacements of the retaining wall.

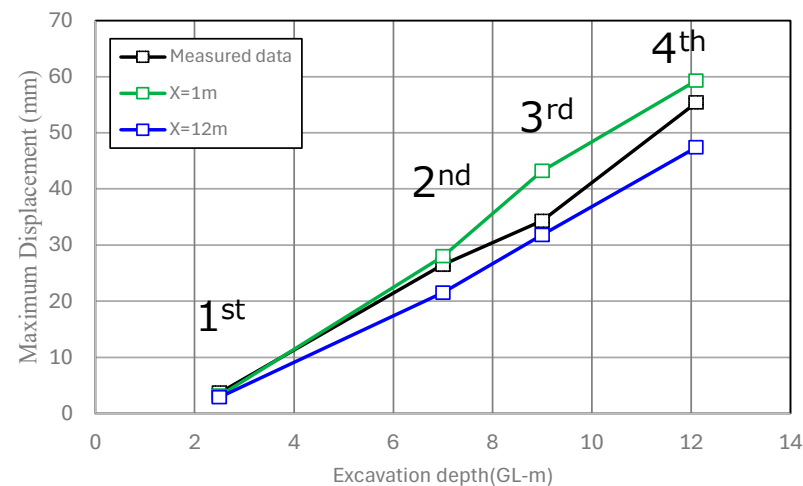


Figure 12. Relationship between maximum displacement of retaining wall and excavation depth.

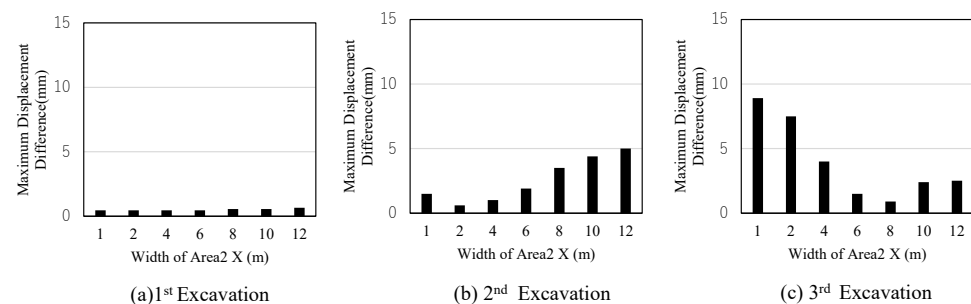


Figure 13. The maximum displacement error of the retaining wall between the measured and numerical values.

This difference is attributed to changes in the distance between the excavation bottom and the lowest strut, which was approximately 6 m in the second stage and approximately 3.5 m in the third stage.

The shorter span between the supports in the third stage enhanced the confinement effect, suppressing the development of lateral strains near the retaining wall and expanding the high-stiffness region. For the fourth excavation stage, although the building load

increased from 47 kN/m^2 to 100 kN/m^2 , an additional analysis showed that the calculated maximum wall displacement increased only from 47.7 mm to 48.6 mm , corresponding to a difference of 0.9 mm (approximately 1.9%). This indicates that the influence of the increased building load was limited. Therefore, because the distance between the excavation bottom and the lowest strut remained similar to that in the third excavation stage (approximately 3 m), the same region width ($X = 8 \text{ m}$) was adopted for the fourth excavation stage. The corresponding results are presented in the following section.

4.3. Verification of Optimization Effects

To verify the effectiveness of the proposed optimization, a comparison was conducted between the optimized zoning width ($X = 8 \text{ m}$) and a wider zoning case ($X = 12 \text{ m}$). The value of $X = 12 \text{ m}$ was selected based on surface wave exploration results conducted near the excavation bottom, which indicated a noticeable change in wave velocity at approximately 10 m from the retaining wall. Accordingly, a width of 12 m was considered representative of the zone in which ground stiffness was inferred to differ near the excavation base. Figure 14 shows the depth distribution of retaining wall displacements obtained after zoning optimization. The optimized zoning width improved the prediction accuracy of the maximum wall displacement at the final excavation stage. The error in the maximum wall displacement was reduced from approximately 8 mm for $X = 12 \text{ m}$ to approximately 3 mm for $X = 8 \text{ m}$, quantitatively demonstrating the effectiveness of the proposed zoning approach. The remaining discrepancy in the displacement profile is considered to be partly attributable to the assumption of constant stiffness within each zoning region in the Mohr–Coulomb constitutive model. Nevertheless, the proposed zoning approach effectively improved the prediction of the maximum wall displacement while maintaining a simple and practical analysis framework.

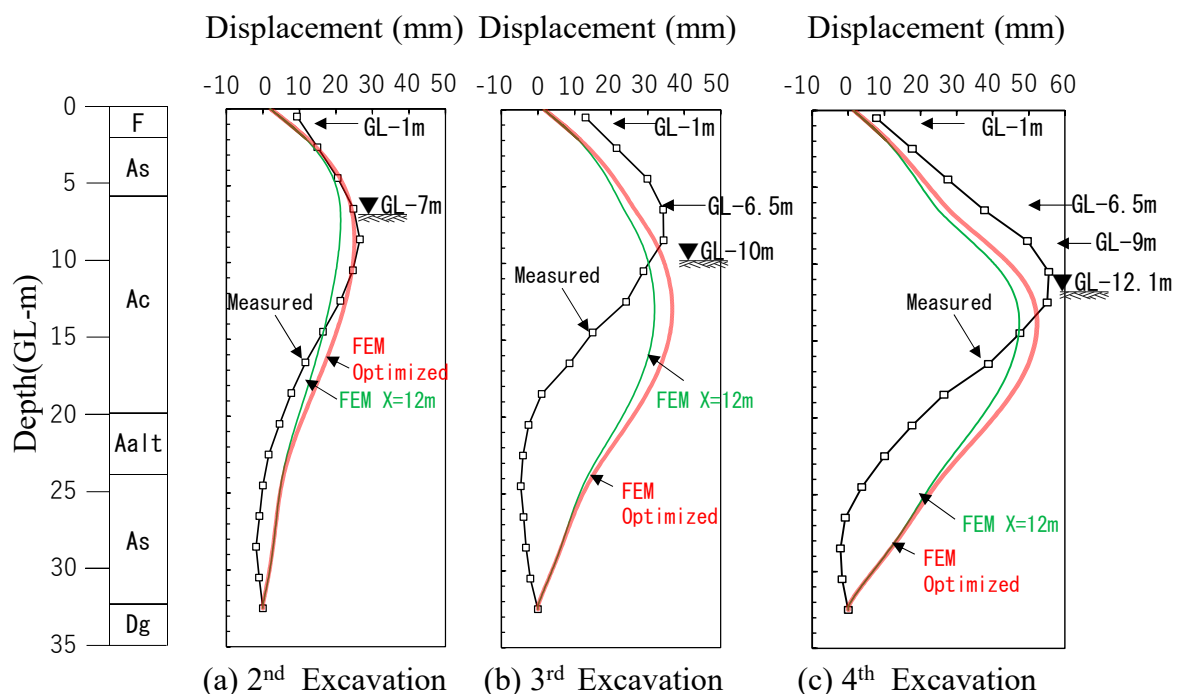


Figure 14. Comparison of wall displacements for two various optimized zoning widths.

5. Conclusions

This study aims to improve the prediction accuracy of retaining wall displacements during excavation by integrating finite element analysis with early-stage monitoring data, with particular emphasis on optimizing the zoning width used to assign the deformation

modulus. The results demonstrated that defining the optimal zoning width as the condition that minimizes the difference between measured and simulated maximum wall displacements significantly enhances predictive accuracy in subsequent excavation stages. In this study, the maximum wall displacement was adopted as the primary evaluation index because it is one of the most important performance indicators in excavation design and construction management. Although displacement distributions are also important for evaluating retaining wall behavior, they were not included in the optimization process and should be considered in future studies. Future research will extend this framework by optimizing both the zoning width and the deformation modulus, incorporating machine learning techniques, and applying the proposed zoning approach to advanced constitutive models, such as the Hardening Soil (HS) and Hardening Soil with Small-Strain Stiffness (HSS) models. In addition, future studies will address the reproducibility of displacement distributions along the wall depth by introducing more comprehensive evaluation indices. These developments are expected to contribute to the development of a real-time system for assessing excavation performance.

The proposed method was validated using a single excavation case. Therefore, further verification using multiple excavation projects under different geological and construction conditions is required to confirm the general applicability and robustness of the proposed approach. In addition, integrating comprehensive analytical frameworks, case histories of deep excavations, and three-dimensional effects together with advanced constitutive models will strengthen the generality of the proposed approach [26–30].

Author Contributions: Conceptualization, H.K. and T.H.; Methodology, H.K.; Validation, K.K. and T.H.; Formal analysis, H.K. and K.K.; Investigation, H.K.; Resources, H.K.; Data curation, H.K.; Writing—original draft, H.K.; Writing—review & editing, K.K. and T.H.; Visualization, H.K.; Supervision, T.H.; Project administration, T.H. All authors have read and agreed to the published version of the manuscript.

Funding: This research received no external funding.

Data Availability Statement: The original contributions presented in this study are included in the article. Further inquiries can be directed to the corresponding author.

Conflicts of Interest: Author Hiroto Kumagai was employed by Takenaka Corporation. The remaining authors declare that the research was conducted in the absence of any commercial or financial relationships that could be construed as a potential conflict of interest.

Abbreviations

The following abbreviations are used in this manuscript:

FEM	Finite Element Method
GL	Ground Level
F	Fill
As	Alluvial Sand
Ac	Alluvial Clay
Aalt	Alternating Alluvial Deposits
Dg	Diluvial Gravel
Dc	Diluvial Clay
Ds	Diluvial Sand
2D	Two-dimensional
PS logging	P–S (Primary–Secondary) Wave Logging

References

1. Peck, R.B. Advantages and limitations of the observational method in applied soil mechanics. *Géotechnique* **1969**, *19*, 171–187. [\[CrossRef\]](#)
2. Ou, C.Y.; Hsieh, P.G.; Chiou, D.C. Characteristics of ground surface settlement during excavation. *Can. Geotech. J.* **1993**, *30*, 758–767. [\[CrossRef\]](#)
3. Zhang, J.; Qiao, G.; Feng, T.; Zhao, Y.; Zhang, C. Dynamic Back Analysis of Soil Deformation during the Construction of Deep Cantilever Foundation Pits. *Sci. Rep.* **2022**, *12*, 13112. [\[CrossRef\]](#) [\[PubMed\]](#)
4. Liu, W.; Tong, L.; Li, H.; Wang, Z.; Sun, Y.; Gu, W. Multi-Parameter Intelligent Inverse Analysis of a Deep Excavation Considering Path-Dependent Behavior of Soils. *Comput. Geotech.* **2024**, *174*, 106597. [\[CrossRef\]](#)
5. Ng, C.W.W.; Zheng, G.; Ni, J.; Zhou, C. Use of Unsaturated Small-Strain Soil Stiffness to the Design of Wall Deflection and Ground Movement Adjacent to Deep Excavation. *Comput. Geotech.* **2020**, *119*, 103375. [\[CrossRef\]](#)
6. Xu, W.; Lin, L.; Zhu, N.; Zhao, Y.; Yang, H.; Mei, Y.; Zhou, D. Three-Dimensional Numerical Analyses of a Monitored Deep Excavation Pit: A Case Study in the Guangzhou Metro. *Buildings* **2025**, *15*, 4018. [\[CrossRef\]](#)
7. Arslan, E.; Akmaz, E.; Çakır, U.F.; Öztürk, Ö.; Pir, H.; Acartürk, S.; Akça, N.Ç.; Karakuş, Y.; Sert, S. Finite Element Modeling with Sensitivity and Parameter Variation Analysis of a Deep Excavation: From a Case Study. *Buildings* **2025**, *15*, 658. [\[CrossRef\]](#)
8. Clough, G.W.; O'Rourke, T.D. Construction induced movements of in situ walls. In Proceedings of the ASCE Specialty Conference on Design and Performance of Earth Retaining Structures, Ithaca, NY, USA, 12–21 June 1990; pp. 439–470.
9. Hashash, Y.M.A.; Whittle, A.J.; Guo, Y.; Chowdhury, S.; Hung, C.J. Evaluation of deep excavation-induced ground movements using three-dimensional numerical analyses. *J. Geotech. Geoenviron. Eng. ASCE* **2010**, *136*, 1107–1121. [\[CrossRef\]](#)
10. Finno, R.J.; Roboski, J.F.; Voss, F.T. Subsurface conditions and performance of a deep excavation. *J. Geotech. Geoenviron. Eng. ASCE* **2007**, *133*, 1503–1513. [\[CrossRef\]](#)
11. Kumagai, H.; Kono, T.; Nishiie, S. Evaluation of Retaining Wall and Surrounding Ground Behavior in Soft Cohesive Soil by FEM Analysis. In *Proceedings of the 5th International Conference on Geotechnics for Sustainable Infrastructure Development (GEOTEC 2023). Lecture Notes in Civil Engineering*; Springer Nature: Singapore, 2024; Volume 395, pp. 761–774. [\[CrossRef\]](#)
12. Gioda, G.; Sakurai, S. Back analysis procedures for the interpretation of field measurements in geomechanics. *Int. J. Numer. Anal. Methods Geomech.* **1987**, *11*, 555–583. [\[CrossRef\]](#)
13. Sakurai, S.; Takeuchi, K. Back analysis of measured displacements of tunnels. *Rock Mech. Rock Eng.* **1983**, *16*, 173–180. [\[CrossRef\]](#)
14. Kung, G.T.C.; Juang, C.H.; Hsiao, E.C.L. Simplified model for wall deflection and ground surface settlement caused by braced excavation. *J. Geotech. Geoenviron. Eng. ASCE* **2007**, *133*, 731–747. [\[CrossRef\]](#)
15. Hsieh, P.G.; Ou, C.Y. Shape of ground surface settlement profiles caused by excavation. *Can. Geotech. J.* **1998**, *35*, 1004–1017. [\[CrossRef\]](#)
16. Long, M. Database for retaining wall and ground movements due to deep excavations. *J. Geotech. Geoenviron. Eng.* **2001**, *127*, 203–224. [\[CrossRef\]](#)
17. Zhang, R.; Goh, A.T.C.; Li, Y.; Liu, H.; Wang, L.; Chen, Z.; Zhang, W. A simple estimation model for basal heave stability of braced excavations in anisotropic clay. *Acta Geotech.* **2022**, *17*, 5789–5800. [\[CrossRef\]](#)
18. Kim, T.; Jung, Y.-H. Optimizing Material Parameters to Best Capture Deformation Responses in Supported Bottom-Up Excavation: Field Monitoring and Inverse Analysis. *KSCE J. Civ. Eng.* **2022**, *26*, 3384–3401. [\[CrossRef\]](#)
19. Kogure, K.; Soga, K.; Klar, A. Inverse analysis for braced excavation using field observations and finite element method. *Soils Found.* **2012**, *52*, 449–466. [\[CrossRef\]](#)
20. Hashash, Y.M.A.; Marulanda, C.; Ghaboussi, J.; Jung, S. Systematic update of excavation performance using field observations. *J. Geotech. Geoenviron. Eng. ASCE* **2004**, *130*, 392–403.
21. Winterkorn, H.F. *Foundation Engineering Handbook*; Van Nostrand Reinhold: New York, NY, USA, 1975; p. 403.
22. Terzaghi, K. *Theoretical Soil Mechanics*; John Wiley & Sons: New York, NY, USA, 1943. [\[CrossRef\]](#)
23. Jardine, R.J.; Potts, D.M.; Fourie, A.B.; Burland, J.B. Studies of the influence of non-linear stress–strain characteristics in soil–structure interaction. *Géotechnique* **1986**, *36*, 377–396. [\[CrossRef\]](#)
24. Atkinson, J.H.; Sallfors, G. Experimental determination of soil properties at small strains. In Proceedings of the 10th European Conference on Soil Mechanics and Foundation Engineering, Florence, Italy, 26–30 May 1991; pp. 915–956.
25. Viggiani, G.; Atkinson, J.H. Stiffness of fine-grained soil at very small strains. *Géotechnique* **1995**, *45*, 249–265. [\[CrossRef\]](#)
26. Ou, C.Y. *Deep Excavation: Theory and Practice*; Taylor & Francis: London, UK, 2006.
27. Finno, R.J.; Roboski, J.F. Three-dimensional effects in supported excavations. *J. Geotech. Geoenviron. Eng. ASCE* **2005**, *131*, 1529–1537.
28. Hashash, Y.M.A.; Hook, J.J.; Schmidt, B.; Yao, J.I.C. Seismic design and analysis of underground structures. *Tunn. Undergr. Space Technol.* **2001**, *16*, 247–293. [\[CrossRef\]](#)

29. Tabarroki, M.; Ching, J.; Yuan, S.-H.; Phoon, K.-K.; Teng, F. Data-Driven Hierarchical Bayesian Model for Predicting Wall Deflections in Deep Excavations in Clay. *Comput. Geotech.* **2024**, *168*, 106135. [[CrossRef](#)]
30. Phoon, K.-K.; Cao, Z.; Ji, J.; Leung, Y.F.; Najjar, S.; Shuku, T.; Tang, C.; Yin, Z.-Y.; Ikumasa, Y.; Ching, J. Geotechnical Uncertainty, Modeling, and Decision Making. *Soils Found.* **2022**, *62*, 101189. [[CrossRef](#)]

Disclaimer/Publisher's Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.