

Effect of D-Wall Penetration Depth on Hydraulic Stability in Deep Excavation for the Jakarta MRT Project Under Different Excavation Scenarios

Yuamar Imarrazan Basarah*

Department of Civil Engineering, Institut Teknologi Bandung
E-mail: yuamarbasarah@itb.ac.id

Mirna Dwi Lestari Salamah

Department of Civil Engineering, Institut Teknologi Bandung
E-mail: mirnadwi67@gmail.com

Masyhur Irsyam

Department of Civil Engineering, Institut Teknologi Bandung
E-mail: masyhur.irsyam@yahoo.co.id

Abstract

Rapid urban growth has increased the need for underground space, often involving deep excavations in dense urban areas. Hydraulic stability becomes a key concern due to soft ground, high groundwater pressure, and nearby structures, factors that, if neglected, can cause excessive ground movement or failure. This paper examines how diaphragm wall (D-Wall) penetration depth affects three main hydraulic failure modes: basal heave, piping, and uplift. The study is based on a 22.4 m deep MRT station excavation in North-Central Jakarta area, surrounded by adjacent existing buildings. Finite element analyses were performed with CPTu-based soil parameters to evaluate safety factors under various embedment depths, wall stiffnesses, and excavation scenarios, including construction duration. The results show that the hydraulic stability for all failure modes increased with penetration depth ratio (H_p/H_e). A 1.0 m of wall thickness was found sufficient to ensure stability for a three-level excavation. Both predicted maximum deflection of approximately 44 mm and ground settlement of 40 mm remained within the serviceability limits. An empirical correlation between penetration depth and hydraulic stability is proposed for use in preliminary design. A minimum H_p/H_e of 1.65 is recommended to achieve overall hydraulic stability of the excavation with a minimum factor of safety of 1.5.

Keywords: Deep excavation, diaphragm wall, land subsidence, wall deflection, stability, uplift.

Abstrak

Pertumbuhan perkotaan yang pesat meningkatkan kebutuhan pemanfaatan ruang bawah tanah, yang umumnya melibatkan galian dalam di kawasan perkotaan padat. Pada kondisi tersebut, stabilitas hidraulik menjadi isu krusial akibat keberadaan tanah lunak, tekanan air tanah yang tinggi, serta bangunan eksisting di sekitarnya. Apabila tidak diperhitungkan dengan baik, kondisi ini dapat menyebabkan pergerakan tanah berlebihan atau kegagalan konstruksi. Makalah ini mengkaji pengaruh kedalaman penetrasi dinding diafragma (D-Wall) terhadap tiga mekanisme utama kegagalan hidraulik, yaitu basal heave, piping, dan uplift. Studi ini didasarkan pada galian stasiun MRT sedalam 22,4 m di wilayah Jakarta Pusat, yang dikelilingi oleh bangunan eksisting. Analisis elemen hingga dilakukan dengan menggunakan parameter tanah berbasis CPTu untuk mengevaluasi faktor keamanan pada berbagai variasi kedalaman penanaman dinding, kekakuan dinding, serta skenario galian, termasuk pengaruh durasi konstruksi. Hasil analisis menunjukkan bahwa stabilitas hidraulik seluruh mekanisme kegagalan meningkat seiring bertambahnya rasio kedalaman penetrasi (H_p/H_e). Ketebalan dinding 1,0 m terbukti memadai untuk menjamin stabilitas pada galian dengan tiga tingkat, dengan lendutan maksimum sekitar 44 mm dan penurunan tanah sekitar 40 mm, yang masih berada dalam batas kinerja layan. Selain itu, diusulkan korelasi empiris antara kedalaman penetrasi dan stabilitas hidraulik untuk keperluan perancangan awal, dengan rekomendasi H_p/H_e minimum 1,65 untuk mencapai faktor keamanan minimum 1,5.

Kata-kata Kunci: Galian dalam, dinding diafragma, penurunan permukaan tanah, defleksi dinding, stabilitas, uplift.

1. Introduction

Rapid urban development in Jakarta has increased the need for underground space, requiring deep excavations in soft, saturated soils near existing structures. Hydraulic instability is a major concern, as it can trigger basal heave, uplift, or seepage failures. Diaphragm walls (D-Walls) are commonly used to provide both structural support and groundwater cutoff, with penetration depth being a key factor for basal stability.

Classical studies on deep excavations in soft clay have shown that stability problems such as basal heave and excessive deformation are primarily governed by undrained shear strength and stress redistribution during excavation (Bjerrum and Eide, 1956; Terzaghi and Peck, 1967). Empirical observations further indicate that increasing embedment depth beyond a critical limit does not always result in proportional improvements in stability, highlighting the need for rational design approaches that balance safety and constructability (O'Rourke, 1992; Das, 2011).

Several previous studies have addressed this issue. Hashash and Whittle (1996) demonstrated that finite element methods (FEM) provide more realistic predictions of wall moments and deflections than traditional approaches. O'Rourke (1992) emphasized that beyond a certain threshold, deeper embedment does not always yield proportional improvements in stability. Hsiung et al. (2018), through 3D modeling of a deep MRT excavation in Jakarta, found that flexural stiffness (EI) and embedment depth significantly influence wall deformation patterns, and that asymmetric soil profiles across the site can impact wall behavior. However, the effects of depth of excavation for different level of underground station were not considered in their study. Additionally, the duration of excavation construction was assumed to be always ontime, and hence, there is no construction delay was taken into account in the analysis that would increase the deformation of the wall.

In addition, due to limited space with adjacent buildings, maintaining the hydraulic stability during underground structure construction is critical as excessive deformation due to the excavation might affect the surrounding building. Previous numerical studies on deep excavations in Jakarta, particularly the three-dimensional analysis conducted by Hsiung et al. (2018), have demonstrated the critical influence of diaphragm wall stiffness and embedment depth on excavation-induced deformation. However, the assessment primarily focused on wall deflection behavior and did not explicitly address hydraulic stability or construction time effects, which are known to govern failure mechanisms in deep excavations constructed in soft ground. Therefore, this study investigates the influence of excavation configurations and construction duration on hydraulic failure mechanisms through a case study of a deep

underground station for MRT in Jakarta. An empirical design chart is also proposed to aid preliminary design under similar geotechnical conditions.

2. Project Background and Site Condition

The study focuses on a three-level underground MRT station in North-Central Jakarta (**Figure 1**), constructed using the top-down method. The excavation reached a final depth of 22.4 m, with a 1.2 m-thick diaphragm wall embedded approximately 32 m below the ground. Reinforced concrete slabs were installed sequentially as both structural struts and permanent station floors, supported by steel king posts on 1 m bored piles foundation extending to 40 m depth. The site is in a densely developed area, bounded by a 7-story building which is 4.5 m to the east and a 2-story building which is 4 m to the west. These nearby structures were represented in the model using equivalent uniform surcharge loads to capture realistic boundary effects.

Figure 2. illustrates the cross-section of the underground MRT station.



Figure 1. Area of study

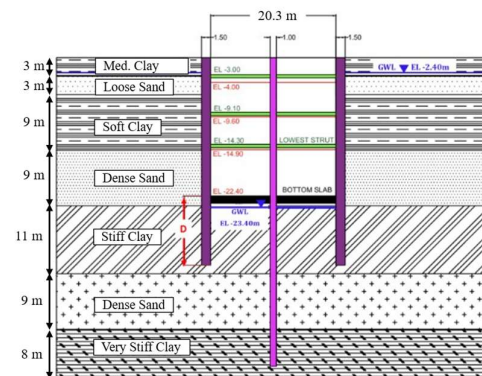


Figure 2. Cross-sectional view of the excavation

Geotechnical investigations included 19 boreholes with SPT data and 7 CPTu tests, complemented by vane shear, pressuremeter, and laboratory tests. The subsurface profile interpreted from SPT logs shows soft clay with low blow counts (NSPT = 0–5) in the upper 14 m, underlain by stiffer clay and dense sand layers (NSPT = 20–40) between 15–40 m, as shown in **Figure 3**. These results indicate a predominance of soft, compressible clay overlying denser strata that provide better bearing resistance. CPTu data were used

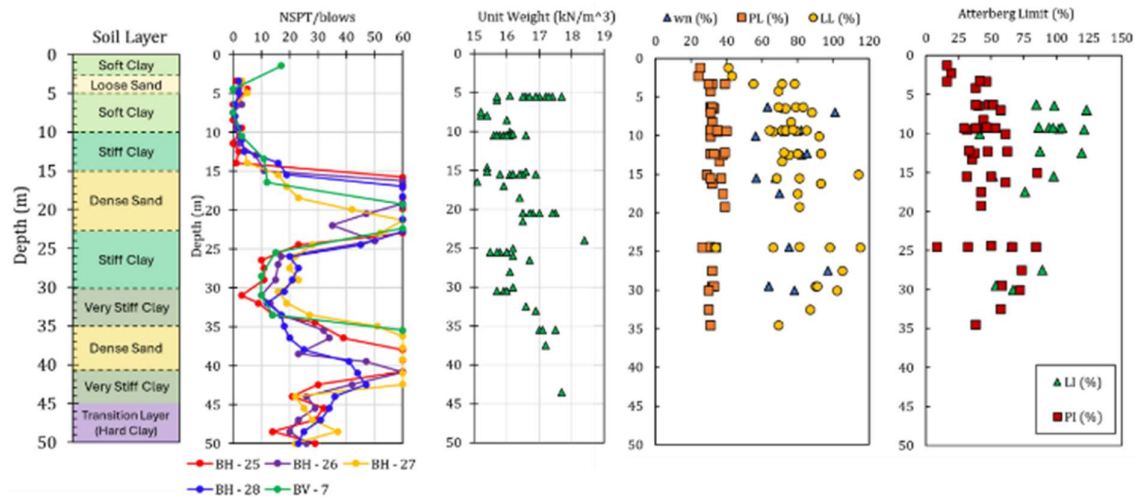


Figure 3. Soil stratigraphy, SPT blow count, and index properties at the MRT underground station site.

to characterize the soft clay layer in the upper 10 meters. Ten CPTu boreholes were evaluated, selected based on their location near boreholes with vane shear or triaxial test data. This allowed for cross-checking the consistency of results and supported the estimation of undrained shear strength parameters that reflect site conditions.

To obtain representative undrained shear strength (S_u) values for this soft soil, cone penetration tests with pore pressure measurement (CPTu) were used as the primary data source. CPTu testing offers continuous profiling and is considered more reliable for capturing the behavior of soft cohesive soils compared to other field tests such as SPT. Interpretation of S_u from CPTu data was carried out by calibrating the cone factor (N_{kt}) using results from nearby vane shear tests (VST) and consolidated undrained (CU) triaxial tests. The interpretation of undrained shear strength from CPTu data using a calibrated cone factor (N_{kt}) was carried out according to Mayne and Peuchen (2018). The N_{kt} of 14 was used in this study.

3. Research Methodology

After collecting and interpreting the results from soil investigation, the study was continued by determining the preliminary penetration depth of the diaphragm wall for two different levels of station (two- and three-story station). Using these D-wall lengths, the stability of the excavation was evaluated through three stability aspects against: 1) basal heave, 2) upheaval, and 3) piping. Once the stability requirements were satisfied, the study was followed by 2D numerical modeling using Plaxis 2D to perform parametric study to evaluate the effects of excavation configurations and durations on the stability of the excavation system and the deformation of the diaphragm wall. A chart was proposed to provide a preliminary guideline to determine the length of the D-wall for the case with similar conditions evaluated in this study.

4. Preliminary Design of D-wall Penetration Depth

In this study, the preliminary determination of diaphragm wall (D-Wall) embedment depth was based on a moment equilibrium approach to ensure the wall could resist lateral loads and maintain stability. This method balances the driving moment from active earth pressure with the resisting moment below the excavation base. In this analysis, three pressure components were considered: active earth pressure above the excavation base, passive pressure below the base, and water pressure (both hydrostatic and uplift) acting at the bottom of the wall.

The embedment depth was initially estimated using the dimensionless factor method proposed by Ou (2019), which applies moment equilibrium between active and passive zones. The calculated depth was then increased by 20–40% to account for uncertainties and ensure design safety.

Error! Reference source not found. illustrates the force distributions acting on the diaphragm wall for moment equilibrium analysis. The diagram decomposes the active and passive earth pressures along with water pressure and surcharge loads, applied to both sides of the wall. Each pressure component is labeled and quantified based on stratified soil layers, surcharge intensity, and water level location. The calculation references the lowest strut as the pivot point for determining the driving and resisting moments, assuming the rotation occurred at this level. Based on the moment equilibrium analysis, the diaphragm wall embedment length was found to be 28 meters for the two-level model and 32 meters for the three-level model. The resulting embedment length serves as the baseline for further evaluation in hydraulic stability checks, including basal heave, uplift, and piping, which are elaborated in subsequent sections.

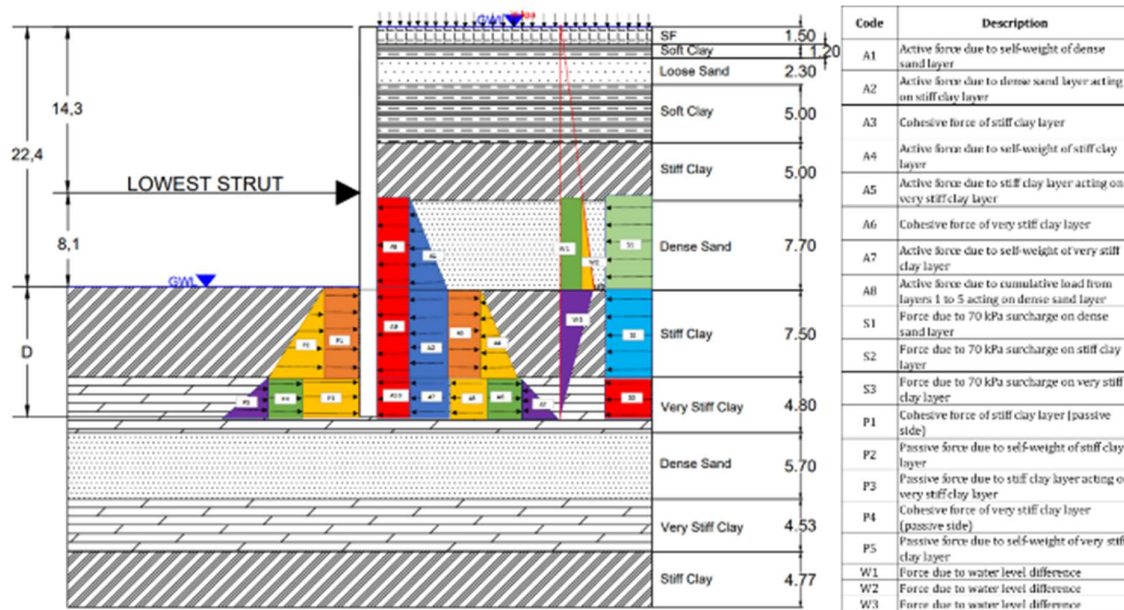


Figure 4. Distribution of active, passive, and water pressures on the diaphragm wall

4.1. Basal heave

Basal heave failure refers to the upward movement of soil at the bottom of an excavation due to the insufficient bearing capacity of the underlying soil to resist the soil weight behind the excavation and external loads such as surcharge. This phenomenon is particularly critical in deep excavations involving clayey soils, where the undrained shear strength is a key parameter governing stability. In this study, two analytical approaches are used to assess the risk of basal heave: the Terzaghi bearing capacity method and the slip circle method.

The Terzaghi method provides a simplified and conservative estimation of the factor of safety (FS) against basal heave by comparing resisting and driving forces using the soil's undrained shear strength. However, it does not allow direct evaluation of the required wall embedment depth, as the calculations are only based on excavation depth and width. For the Terzaghi method, the results indicate that for both excavation depths, the factor of safety exceeds the allowable threshold (1.25–1.5), indicating a stable condition with respect to basal heave. The summary of the key parameters and results is presented in **Table 1.**

To address the limitations of the Terzaghi method—which cannot directly evaluate the required wall embedment depth due to its reliance solely on excavation geometry—the slip circle method is adopted as a complementary approach. This method enables a more refined assessment of basal stability by considering rotational failure mechanisms that originate below the base of excavation. In this approach, the failure surface is assumed to rotate about a point located near the lowest strut level, simulating the most critical potential slip surface. An increase of

approximately 21% from the preliminary D-wall embedment length is required to enhance the safety factor related to hydraulic stability.

Table 1. Summary of basal heave calculation

Parameter	2-Story Model	3-Story Model
Excavation Depth, H_e (m)	18.52	22.40
Excavation Width, B (m)	20.30	20.30
$B/\sqrt{2}$ (m)	14.35	14.35
Distance to Stiff Soil, D (m)	22.18	18.30
Condition	$D < B/\sqrt{2}$	$D < B/\sqrt{2}$
Undrained Shear Strength, Su_1 (kPa)	39	40
Undrained Shear Strength, Su_2 (kPa)	104	107
Saturated Unit Weight, γ (kN/m ³)	17	17
Surcharge, q_s (kPa)	50	50
Factor of Safety (FS)	1.75	1.58
Stability Status	OK	OK

4.2. Blow-in/upheaval

Upheaval or blow-in failure occurs when upward water pressure from an underlying permeable layer exceeds the self-weight of the overlying impermeable soil layer at the base of an excavation. This failure mechanism is closely associated with seepage forces and upward hydraulic gradients. Under such conditions, excessive pore water pressure may lead to hydraulic rupture and upward soil movement, particularly in deep excavations where vertical hydraulic gradients are significant (Harza, 1935). According to SNI

8460:2017, the base of the excavation is considered stable against blow-in failure if $F_{up} \geq 1.25$. To further investigate the effects of interaction between the retaining wall and the soil mass, two calculation scenarios were considered:

1. Without wall-soil friction — a conservative assumption where no shear resistance is mobilized along the d-wall interface.
2. With wall-soil friction — frictional resistance is mobilized along the wall from the base of the excavation to the toe, contributing to the resisting forces against uplift (**Figure 5**).

The calculated uplift safety factors (F_{up}) showed decreasing values with deeper excavation, especially in the 3-story model where F_{up} drops below the minimum requirement of 1.25. These results, however, do not account for wall-soil friction, making them overly conservative. In practice, friction along the embedded diaphragm wall may increase uplift resistance. **Table 2.** shows the calculation of the uplift safety factor (F_{up}) that incorporates wall-soil interface friction, which can nearly double the safety factor. Although the F_{up} method without friction is useful for preliminary assessments, ignoring friction may lead to unrealistic stability evaluations and overly conservative designs.

Table 2. Summary of upheaval stability calculation

Parameter	3-Story Model	2-Story Model
D-Wall Thickness (m)	1.2	1.2
D-Wall Length (m)	32	27
Assumed Wall Width (m)	5.0	5.0
Cross-Sectional Area (m ²)	23	20.4
Structure Weight (kN)	2765	2448
Uplift Force (kN)	1750	1550
Wall-Soil Friction Force, Q_s (kN)	7390	14855
SF Against Uplift (with Friction)	5.8	11.2

4.3. Piping

Piping failure typically occurs in excavations involving permeable sandy layers, where water seepage into the excavation can initiate internal erosion or sand boiling. In deep excavations, the retaining wall also functions as a cut-off wall, intended to interrupt, or lengthen the seepage path and reduce the hydraulic gradient at the base of the excavation. Therefore, sufficient wall embedment depth is required to effectively limit upward seepage and mitigate piping risk.

However, for the case under consideration, the excavation base consists of stiff clay layers, which are less susceptible to piping-type failures due to their low permeability and high cohesion. Thus, no piping failure analysis was performed in this study. The focus

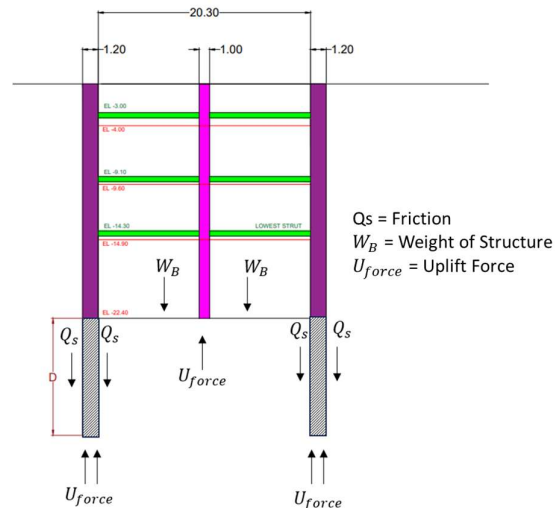


Figure 5. Upheaval with Friction Consideration

was then directed instead toward hydraulic uplift and basal heave mechanisms that are more relevant under such ground conditions.

5. Numerical Modelling Approach

5.1. Parametric study design

A series of parametric study was conducted to assess the effects of wall penetration depth, stiffness, and construction duration on hydraulic stability and deformation. Twelve numerical models were developed for two excavation depths (18.5 m and 22.4 m) to represent two- and three-levels of underground station. Additional variations included wall thicknesses of 0.8, 1.2, and 1.5 m to evaluate stiffness effects, and construction durations of 28, 180, and 1825 days to capture time-dependent pore pressure responses **Table 3.** summarize the configuration of these parametric analyses, including wall embedment levels, excavation stages, and parameter combinations used in the numerical models.

5.2. Finite element simulation

Based on the embedment depth estimations obtained from the previous stability analyses, this section presents the simulation setup used to evaluate excavation performance under various conditions. A summary of the calculated wall penetration depths that were used as input parameters in the L1, L2, and L3 parametric study scenarios is presented in **Table 4.** A Fully Coupled Flow-Deformation (FCFD) analysis was used to simulate time-dependent behavior, capturing both stress-strain response and pore pressure variation. This type of analysis has been increasingly adopted in deep excavation studies to capture the interaction between stress redistribution and pore pressure dissipation (Luna, 2019). The analysis required input of soil permeability and excavation timing to reflect consolidation and seepage effects. Excavated layers were modeled under dewatering

Table 3. Summary of Parametric Study Scenarios

Excavation Case	Wall Thickness (Stiffness)	Excavation Time Scenarios	Total Models
2 Levels (He = 18.5 m) L1, L2, L3	1.2 m (base case)	28 days	3
2 Levels (He = 18.5 m) L (Design Length)	1.2 m	180 days, 1825 days	2
3 Levels (He = 22.4 m) L1, L2, L3	1.2 m (base case)	28 days	3
3 Levels (He = 22.4 m) L (Design Length)	1.2 m	180 days, 1825 days	2
3 Levels (He = 22.4 m) L (Design Length)	0.8 m & 1.5 m	28 days	2
Total			12

Table 4. D-Wall embedment lengths for parametric study scenarios

Parametric Model	2-Level Model (m)	3-Level Model (m)
L1	25	27
L2	29	32
L3	31	35

conditions with the phreatic line set 1 meter below the base and assumed drained to represent groundwater control.

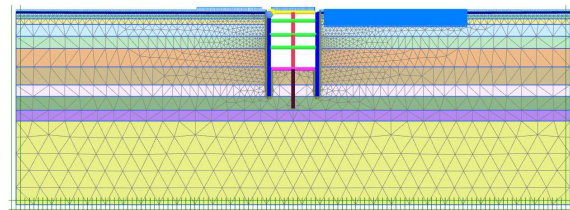
In the absence of exact construction data, stage durations were adapted from Hsiung (2018). **Table 5.** summarizes the excavation stages, time intervals, and model components activated in each phase. Adjacent buildings were simulated using uniformly distributed loads applied on both sides of the excavation. On the east side, a 7-story structure was modeled with a 70 kN/m² load applied at -6.00 m, assuming deep foundation support with an equivalent raft depth of 2/3 the total embedment (9 meters). On the west side, a 2-story structure was represented by a 20 kN/m² load applied at ground level. **Figure 6.** displays the finite element model geometry and the location of applied building loads.

5.3. Soil parameters input

The Hardening Soil (HS) model was used to represent the nonlinear, stress-dependent behavior of the subsoil, providing more accurate deformation predictions than simpler models especially in excavation case. Undrained B conditions were applied for short-term excavation stages, simulating rapid construction where pore pressure remains constant and undrained strength controls behavior. For long-term or delayed scenarios, drained conditions were used to reflect consolidation and pore pressure dissipation. Soil parameters such as stiffness, strength, and permeability were derived from data in Chapter 2. The full set of HS model inputs for each soil layer is listed in **Table 6.**

Table 5. Construction stage and analysis setup

Stage Construction	Time Interval (days)	Description
Initial Phase [InitialPhase]	-	Initial Condition
DW & KP Installation [Phase_1]	101	Activate D-wall, king post, king post pile, & surcharge
Initial Excavation [Phase_2]	18	Reset Displacement, Deactivate soil for initial excavation (-1.00 m)
Deck [Phase_3]	20	Activate Deck & free-connection
EX-1 [Phase_4]	30	Deactivate soil for 1st excavation (-4.00 m), GWL at -5.00 m
Top Slab [Phase_5]	14	Activate Top Slab
EX-2 [Phase_6]	30	Deactivate soil for 2nd excavation (-9.60 m), GWL at -10.6 m
Middle Slab 1 [Phase_7]	20	Activate Middle Slab 1
EX-3 [Phase_8]	22	Deactivate soil for 3rd excavation (-14.90 m), GWL at -15.90 m
Middle Slab 2 [Phase_9]	22	Activate Middle Slab 2
EX-4 [Phase_10]	28	Deactivate soil for 4th excavation (-22.40 m), GWL at -23.40 m
EX-5 [Phase_11]	28	Deactivate soil for 5th excavation (-23.65 m), GWL at -24.65 m
Bottom Slab [Phase_12]	21	Activate Bottom Slab
SF for Each Excavation Stage	-	Safety factor check for Phase_13–17

**Figure 6. Finite Element Model Geometry**

5.4. Structural elements and interface

The excavation system components—including the diaphragm wall, king posts, and strut slabs—were modeled as plate elements with assigned thickness and stiffness based on the design specifications. King posts, used to support the deck slabs during excavation, were modeled as embedded beam rows spaced at 4.5 meters to simulate realistic interaction with the surrounding soil. A free connection was assumed between the deck slabs and diaphragm wall, meaning no moment or force transfer at the interface, consistent with the construction detail where slabs rest on king posts independently. Table 7. summarizes the structural properties and input parameters used in the model.

Table 5. Soil parameter input

Layer	Depth (m)	Soil Type	γ (kN/m ³)	S_u (kPa)	E_{50} (kPa)	E_{oed} (kPa)	E_{ur} (kPa)	k (cm/s)
1	0–1.5	Fill	18	40	16000	16000	48000	1E-07
2	1.5–2.7	Soft Clay (MH)	16	33	7370	7370	22110	1E-07
3	2.7–5	Loose Sand (SM)	18	-	8000	8000	24000	1E-06
4	5–10	Soft Clay (MH)	16	33	7370	7370	22110	1E-07
5	10–15	Stiff Clay (MH)	17	48	32000	32000	96000	1E-08
6	15–22.7	Dense Sand (SM)	19	-	128000	128000	384000	1E-06
7	22.7–30.2	Stiff Clay (MH)	17	90	60000	60000	180000	1E-08
8	30.2–35	Very Stiff Clay (MH)	17	120	80000	80000	240000	1E-08
9	35–40.7	Dense Sand (SM)	18	-	123200	123200	369600	1E-06
10	40.7–45.23	Very Stiff Clay (MH)	17	186	97600	97600	292800	1E-08
11	45.23–50	Very Stiff Clay (CL)	17	150	88000	88000	264000	1E-08

Table 6. Structural properties

Element	Material Model	EA (kN/m)	EI (kPa/m)
King Post	Plate/Elastic	6520000	196000
D-wall	Plate/Elastic	31388000	3642054
Top Slab	Plate/Elastic	25700000	2141667
Middle Slab 1	Plate/Elastic	12850000	267708
Middle Slab 2	Plate/Elastic	15420000	462600
Deck	Plate/Elastic	7370000	98200
King Post Pile	Embedded beam row/Elastic	29066700	2616260

6. Result and Discussion

6.1. Hydraulic stability assessment

In the 3-level model, FEM consistently yielded the highest SF (>1.4), while the upheaval method showed the lowest, especially for $H_p/H_e < 1.5$, indicating high failure risk. Stability against boiling and heave showed similar increasing trends. Similar to 3-level model, the 2-story model showed improved SF across all failure modes with slightly greater SF, implying that shallower excavations in general have better stability. Based on these results, at $H_p/H_e = 1.65$, all failure modes meet the standard required factor of safety criteria for excavation stability of $SF \geq 1.5$.

6.2. Wall deflection and ground surface settlement

The assessment of diaphragm wall performance based on deformation profiles rather than a single maximum displacement value has been recommended in previous studies, as it provides better insight into excavation behavior and failure mechanisms (Hsiung and Hwang, 2009; Massarsch, 1979). Numerical simulation results indicated that wall deflection across all design variations remains within the allowable deformation specified in the project of 0.5% of excavation depth ($\delta_w < 0.5H_e$). This criterion was adopted considering nearby structures in Zone 1, located approximately $x = 4$ to 4.5 meters from the excavation, with x/H ratios

below 1.0. The maximum wall deflection for 3-story model was about 40 mm which occurred at the depth of the final base excavation. Deflection was assessed on both eastern and western walls throughout each excavation stage. **Table 8.** summarizes the maximum deflections compared to allowable limits.

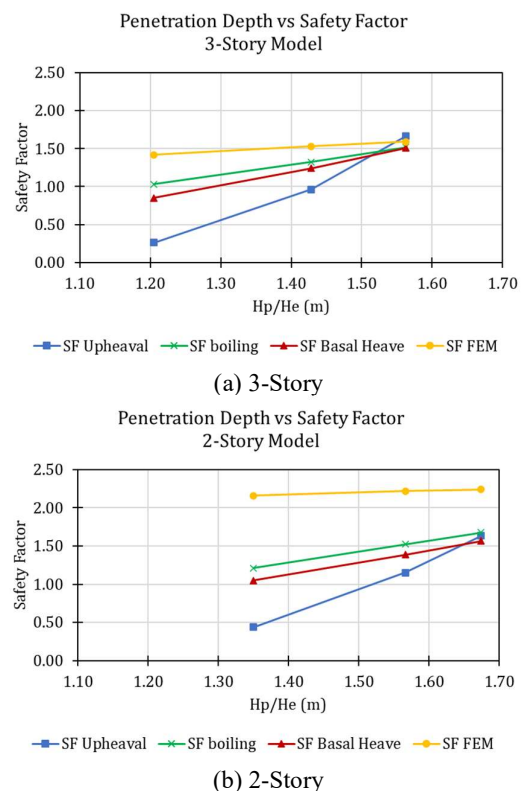


Figure 7. Penetration depth against safety factors for different failure modes, a) 3-story; b) 2-story models

Table 7. Maximum allowable deformation for each excavation stage

Excavation Stage	3-Story Model	2-Story Model
Excavation 1	20.0 mm	22.0 mm
Excavation 2	48.0 mm	50.0 mm

Table 8. Maximum allowable deformation for each excavation stage

Excavation 3	74.5 mm	92.6 mm
Excavation 4	112.0 mm	98.8 mm
Excavation 5	118.3 mm	–

Ground surface settlement, although reduced through appropriate design measures, remains inevitable due to lateral wall movement. As the wall deflects, small gaps form between the wall and soil, allowing gradual downward movement of the overlying soil mass. While complete elimination is not possible, ground surface settlements must remain within tolerable limits to avoid impacting nearby buildings.

Figure 8. presents the combined results of wall deformation and ground settlement of each stage for the 3-story excavation models. The profiles illustrate the progressive deformation behavior during staged excavation and confirm that lateral displacement and settlement remain within acceptable serviceability limits. The maximum wall deflection occurred around the bottom of the excavation at the final excavation stage, showing the slab acted as a strut for the upper wall.

The effects of wall penetration depth represented by H_p/H_e ratio on wall deflection and ground settlement for 3-story excavation model are shown in **Figure 9.** A noticeable difference can be observed in the deformation behavior of shorter wall embedments ($H_p/H_e=1.2$), where the wall base exhibits a “kicking in” pattern toward the excavation. This deformation pattern indicates insufficient embedment depth to resist both lateral earth pressure and uplift forces. As a result, the wall deflects inward, compromising overall stability. In contrast, longer wall embedments ($H_p/H_e=1.4$ and $H_p/H_e=1.6$) demonstrate more restrained deformation profiles, providing better resistance against hydraulic failure mechanisms.

In the selected design scenario, the 3-story excavation model exhibited a maximum diaphragm wall deflection of 44 mm and a ground surface settlement of 39 mm. Meanwhile, the 2-story model showed slightly lower values, with a maximum wall deflection of 39 mm and settlement of 34 mm. These values were still below the allowable wall deflection of 118.3 mm and 98.8 mm for 3- and 2-story excavation models, respectively.

The ground surface settlement profiles behind the diaphragm wall indicate that the maximum predicted settlement remains within the range of approximately 10–50 mm for all evaluated wall embedment ratios ($H_p/H_e = 1.2$ – 1.6) for each stage of excavation. According to the damage classification criteria defined in SNI 8460:2017, this magnitude of settlement corresponds to Risk Category II, which represents a low level of risk where minor, non-structural damage may occur, while no structural damage is expected **Table 10.** This implies that the excavation system performance satisfies acceptable serviceability requirements for adjacent structures in dense urban environments.

Table 9. Typical damage risk classification based on maximum building tilt and settlement (SNI 8460:2017, adapted from CIRIA PR30, 1996)

Risk Category	Maximum Building Tilt	Maximum Building Settlement (mm)
I	< 1/500	< 10
II	1/500 to 1/200	10 to 50
III	1/200 to 1/50	50 to 75
IV	> 1/50	> 75

Table 10. Typical damage risk description based on maximum building tilt and settlement (SNI 8460:2017, adapted from CIRIA PR30, 1996)

Risk Category	Risk Description
I	Negligible: superficial damage is unlikely to occur.
II	Slight: superficial damage may occur, but unlikely to have structural significance
III	Moderate: superficial damage accompanied by potential structural damage to buildings; possible damage to relatively rigid pipelines.
IV	High: significant structural damage to buildings and rigid pipelines or other buried utilities is expected.

6.3. Construction time effect

To investigate the influence of construction time on excavation stability, numerical simulations were conducted for the final excavation stage of a selected design scenario: a three-level excavation supported by a 35-meter-deep diaphragm wall. The assessment focuses on the time-dependent response of the excavation system, including changes in safety factors, wall displacement, and excess pore water pressure. Three construction time scenarios were analyzed:

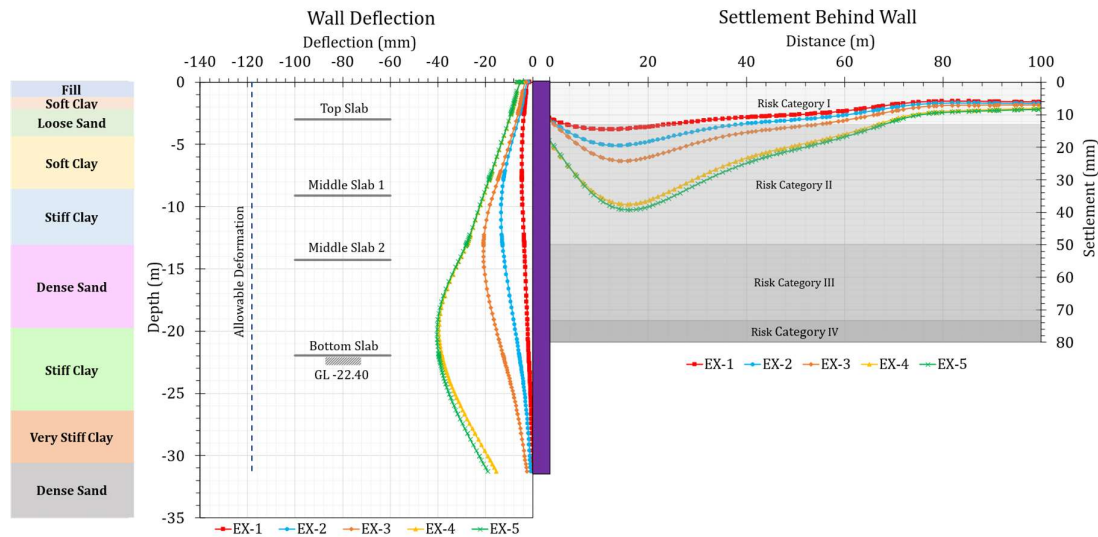


Figure 8. Wall displacement and ground surface settlement – 3-story excavation model

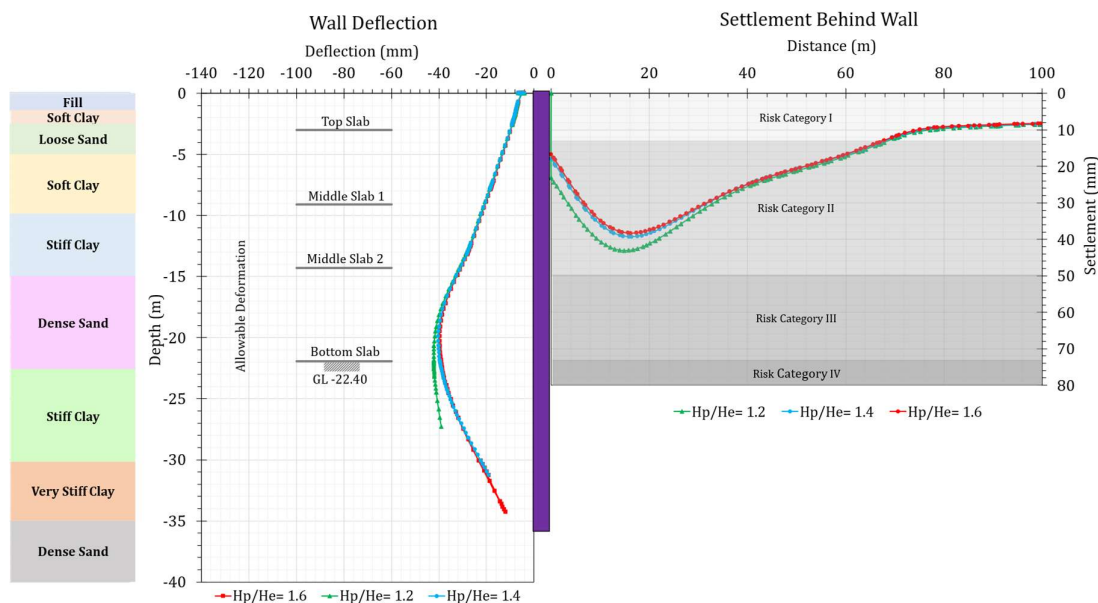


Figure 9. Diaphragm wall deflection and ground surface settlement for varying embedment depths

1. **Ideal construction timeline:** 28 days (short-term condition)
2. **Realistic construction:** 180 days (approximately 6 months)
3. **Delayed construction:** 1825 days (5 years – long-term condition)

The results are summarized in **Table 11.**, highlighting the variation in hydraulic response and deformation patterns over time.

Excavations in general are most vulnerable under long-term conditions, as excess pore pressure tends to accumulate and reduce effective stress, thereby lowering shear strength and overall stability (T.B. Edil, 1982). This is consistent with the modeling results, which show a decreasing factor of safety as

Table 11. Effect of time on safety factor, uplift, and pore water pressure (PWP) at the final stage of excavation

Time (day)	Safety Factor	Uplift (m)	Excess PWP (kPa)	PWP (kPa)
28	1.59	0.16	27.8	77.8
180	1.36	0.40	29.2	76.4
1825 (5 years)	1.35	2.70	29.2	76.4

construction time increases. In the short-term model, an unloading mechanism prevails, generating negative excess pore pressure at the excavation base, which temporarily enhances effective stress and stabilizes the system. Over time, however, excess pore pressure gradually dissipates and stabilizes

toward an asymptotic value, reflecting drained conditions as shown in **Table 11**.

Failure patterns observed in the simulations, suggest a dominant uplift mechanism, particularly influenced by upward seepage pressures at the excavation base. As time progresses, this uplift force becomes more pronounced, resulting in increased deformation and potential instability. **Figure 10**, compares the active pore water pressure within the excavation to the hydrostatic pressure. The results indicate that the active pore pressure inside the excavation increased about the twice of the hydrostatic pressure. This additional pressure highlights the influence of water flow accumulation beneath the excavation base due to the seepage during the construction. The presence of seepage-induced pore pressure reduced the effective stress, thus decreasing soil shear strength and potentially triggering hydraulic failure if not adequately controlled.

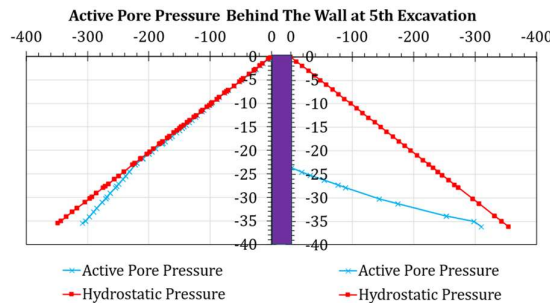


Figure 10. Active pore pressure around D-Wall

6.4. Wall stiffness effect

The effect of wall stiffness on excavation stability was examined by varying diaphragm wall thickness and adjusting axial (EA) and flexural (EI) stiffness accordingly. Table 12 lists the input parameters of the diaphragm wall for thicknesses of 0.8 m, 1.2 m, and 1.5 m. Figure 11 shows that increasing wall thickness improves safety factor (SF) for both $H_p/H_e = 1.2$ and 1.6 , due to greater structural stiffness, which reduced deformation and improved resistance to basal failure. At a given thickness, the safety factor is higher for $H_p/H_e = 1.6$ than that for $H_p/H_e = 1.2$, indicating that wall stiffness and embedment depth simultaneously improving the excavation stability. These results also aligned with the study by Do (2016). Optimal performance of the diaphragm wall configuration is achieved when both parameters are sufficiently high.

Table 12. Structural properties of d-wall sections for thickness parametric study

Thickness (m)	I (m ⁴)	EI (kNm ² /m)	EA (kN/m)
0.8	0.043	1134933	21280000
1.2	0.137	3642054	31388000
1.5	0.281	7481250	39900000

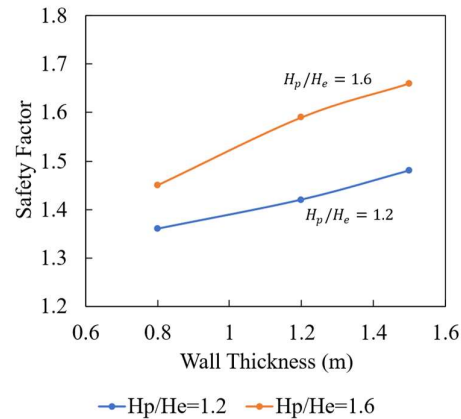


Figure 11. Relationship wall thickness & safety factor for 3-story excavation model

7. Conclusion

Based on the results of the parametric analysis and numerical simulations, several key conclusions can be drawn regarding the stability of deep excavations for underground station supported by diaphragm walls:

1. Increasing the penetration depth ratio (H_p/H_e) substantially improved the safety factor (SF) across all hydraulic stability failure modes. Shallower excavations showing greater stability. For preliminary design, a minimum of H_p/H_e of 1.65 was recommended to ensure $SF \geq 1.5$.
2. Wall stiffness (EI and EA) also affects excavation stability. Thicker walls yield higher SF, especially when combined with deeper embedment, highlighting the synergistic role of stiffness and depth in resisting basal failure. The thickness of 1.0 m was sufficient to achieve the requirement stability for 3-story excavation level with the excavation height of 22.4 m.
3. The retaining wall deflection behavior observed in the simulations complies with Indonesian serviceability limits ($\delta_w < 0.5\%H_e$). This confirms that the excavation design was not only stable but also reducing a potential damage to adjacent buildings located as close as 4 meters from the excavation. Deflection maximum of 44 mm remains within tolerable limits throughout all construction stages.
4. The maximum ground settlement of 40 mm predicted by the simulations falls into Risk Category II according to SNI 8460:2017, which corresponds to a low level of risk where only minor, non-structural damage to adjacent buildings may occur, and no structural significance.

5. Construction delays (up to 5 years) reduce the safety factor by ~15% due to increased pore pressures induced by the seepage and lower effective stress. Shorter construction periods benefit from temporary stress release, enhancing stability. Timely excavation and groundwater control are therefore crucial. Proper scheduling helps maintain overall stability of the excavation and performance.

These conclusions provide insight into the behavior of underground station excavation under different diaphragm wall configurations. Nevertheless, further validation using actual field measurements is required to confirm the applicability of the findings for practical implementation.

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References

- Badan Standardisasi Nasional (BSN). (2017). SNI 8460:2017 – Persyaratan Perencanaan Geoteknik. Jakarta, Indonesia.
- Bjerrum, L., & Eide, O. (1956). Stability of strutted excavations in clay. *Géotechnique*, 6, 32–47.
- CIRIA. 1996. Prediction and effects of ground movements caused by tunnelling in soft ground beneath urban areas. Construction Industry Research and Information Association, Project Report 30.
- Das, B. M. (2011). *Principles of Geotechnical Engineering* (8th ed.). Stamford, CT: Cengage Learning.
- Do, T. N. (2016). A study of stability of deep excavations in clay with consideration of a full elastoplastic support system. PhD Dissertation, National Taiwan University of Science and Technology, Taipei, Taiwan.
- Edil, T. B. (1982). Seepage, slopes, and embankments. CEE-530 Class Notes, University of Wisconsin–Madison, USA.
- Harza, L. F. (1935). Uplift and seepage under dams in sand. *Transactions of the American Society of Civil Engineers (ASCE)*.
- Hashash, Y. M. A., & Whittle, A. J. (1996). Ground movement prediction for deep excavations in soft clay. *Journal of Geotechnical Engineering*, 122(6), 474–484.
- Hsiung, B. C. B., & Hwang, R. N. (2009). Evaluating performance of diaphragm walls by wall deflection path. *SEAGS Special Issue on Excavation and Tunneling in Geotechnical Engineering*, 81–90.
- Hsiung, B. C. B., Yang, K. H., Aila, W., & Ge, L. (2018). Evaluation of the wall deflections of a deep excavation in Central Jakarta using three-dimensional modeling. *Tunnelling and Underground Space Technology*, 72, 84–96. <https://doi.org/10.1016/j.tust.2017.11.013>
- Luna, C. A. B. (2019). Parametric analysis in deep urban excavations using fully coupled flow–deformation analysis. Master's Thesis / Technical Report.
- Massarsch, K. R. (1979). Lateral earth pressure in normally consolidated clay. In *Proceedings of the 7th European Conference on Soil Mechanics and Foundation Engineering*, Vol. 2, 245–250, Brighton, UK.
- Mayne, P. W., & Peuchen, J. (2018). Evaluation of CPTU Nkt cone factor for undrained strength of clays. *Research Report / Technical Publication*.
- O'Rourke, T. D. (1992). Base stability and ground movement prediction for excavations in soft clay. In *Proceedings of the ICE Conference on Retaining Structures*, Thomas Telford, London, 657–687.
- Terzaghi, K., & Peck, R. B. (1967). *Soil Mechanics in Engineering Practice* (2nd ed.). New York: John Wiley & Sons.

