RS2

Excavation

Examples

Table of Contents

1. Undrained analysis for deep excavation in clayey soils using RS2

1.1. Introduction

When analyzing a geotechnical problem, different design parameters are utilized depending on drained or undrained conditions. A drained analysis is typically used for soils with large permeabilities, such as sands or gravel, or soils under long-term loading. On the other hand, undrained soil parameters need to be considered when simulating the short-term loading on a given clay model, as that examines the sample before it has been drained. Since undrained strength is not an intrinsic property of clay, that property changes within different depths of a model as the applied in-situ stresses change. In summary, the drained or undrained conditioned depends on soil types and/or rate of loading.

In RS2, undrained analyses can be modeled using three different approaches:

1. Using undrained strength and stiffness parameters (using Poisson' ratio close to 0.5 and back calculate the undrained young modulus (E) from the effective shear modulus (G)). Using this approach, no excess pore water pressure will be generated.
2. Using undrained strength parameters and effective stiffness parameters, the excess pore pressure can be generated using this approach, but it may not be accurate.
3. Using effective strength parameters and stiffness parameters, depending on the constitutive models that was utilized, accurate excess pore pressure can be generated.

In approaches 2 and 3, undrained behavior can be modelled by either activating "Undrained" in Consolidation analysis option or by setting Material behaviors to "Undrained" in RS2. Note that only one of the options should be selected.

A case study of an excavation in clays will be carried out in the following section to demonstrate undrained analysis using the three approaches.

1.2. The Taipei National Enterprise Center Excavation

The Taipei National Enterprise Center (TNEC) building is an 18-storey structure with five basement levels constructed in 1991 (Ou, Shiau, & Wang, 2000). The excavation for its site completed with the top-down construction method across seven stages, which took place over the span of approximately three hundred days.

In the first stage, which took place after installing the wall, the soil was excavated down to a depth of 2.8m, under the assumption that the ground surface elevation is zero. In the second stage, temporary H400 x 400 x 13 x 21 steel struts at the depth of 2.0m were installed, followed by the excavation of the soil to a depth of 4.9m. In the third stage, a slab was constructed at the depth of 3.5m, then the temporary struts from the previous stage were demolished, then the ground was further excavated to the depth of 8.6m. In the fourth stage, floor slabs were constructed at the depths of 0m (ground surface) and 7.1m, then the excavation was continued down to the depth of 11.8m. In the fifth stage, a floor slab was constructed at a 10.3m depth, followed by an excavation down to 15.2m. In the sixth stage, another slab was constructed at the depth 13.7m, and the excavation was further continued down to the depth of 17.3m, which was followed by installing temporary H400 × 400 × 13 × 21 steel struts. In the seventh stage, struts of the same dimensions and specifications were installed again, and the soil was excavated down to 19.7m. The purpose of the temporary struts was to reduce wall deformations throughout the excavation process prior to installing the concrete slabs.

Figure 1.1 shows a cross-section of the excavation site, highlighting the elevations and the concrete slabs constructed at different depths.

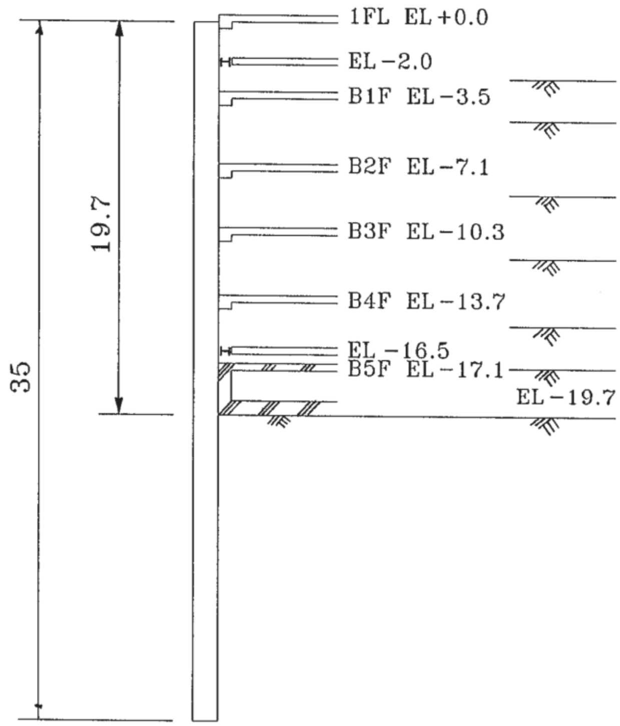

Figure 1.1: TNEC Excavation Project Cross-Section (Ou, Shiau, & Wang, 2000)

1.3. RS2 Analyses of the TNEC Excavation

1.3.1. Model Description

This case study covers the analyses of this excavation project modeled on RS2 with three general approaches, with four different models utilizing different soil parameters. The yielded results of those analyses include the horizontal displacement of the installed wall along its length, and the vertical settlement of the soil on the ground surface.

The purpose of the models was to present in detail different ways to model undrained analysis with RS2. Each approach models the undrained behaviours differently, while two different models were provided in the third approach to examine the benefit from using advance soil constitutive models. All four models have identical properties for the sand layers. It should be noted that the fourth model was more refined as it splits the clay layer between the depths of 11.8m and 33.0m at the depths of 17.3m, 25.0m, and 33.0m. The sand parameters across different depths for all the models are shown in Table 1 as below.

Table 1.1: Sand Parameters on RS2 Models

Depth	5.6m - 8.0m	33.0m - 35.0m	37.5m - 45.0m
Type	Loose Silty Fine Sand	Medium Dense Silty Fine Sand	Silty Fine Sand
Young's modulus (kPa)	68351	265473	300247
Poisson's ratio (v) (-)	0.3	0.2	0.3
Friction angle (ϕ) (º)	31	33	35
Cohesion (c) (kPa)	0.1	0.1	0.1
Unit Weight (kN/m3)	18.93	19.62	19.62
Failure Criterion	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb
Material Behaviour	Drained	Drained	Drained

The differences between all models are in the clay layers which will be discussed in detail in the following sections. It should be noted that the sand layers across the four models had their material behaviours set to drained due to their high permeability, and they also had their parameters unchanged.

Approach 1

In this approach, since both stiffness and strength parameters are undrained values. All clay layers were modelled using Mohr-Coulomb failure criterion with zero friction angle. Poisson's ratio was assigned a value of 0.495, and cohesion was assigned the value of undrained shear strength (*Su*). The shear strength and Young' modulus values were taken from Ou *et. al.*(2000) where *Eu* = 650 * *Su* for the clay layer from 0m to 5.6 m and *Eu* = 740 *Su* for other clays layers. The material behaviour for all clay layers were set to drained. The material properties are shown in Table 2. Figure 1.2 below shows the model prepared for this approach.

Table 1.2: Clay Parameters for the First Approach

Depth	0m - 5.6m	8.0m - 11.8m	11.8m - 33.0m	35.0m - 37.5m
Type	Soft Silty Clay 1	Soft Silty Clay 2	Soft Silty Clay 3	Medium Soft Clay
Young's modulus (Eu) (kPa)	28600	21075.2	28416	82251
Young's Modulus change with depth (kPa/m)	0	1835.2	1804.19	0
Poisson's ratio (v) (-)	0.495	0.495	0.495	0.495
Friction angle (ϕ) (º)	0	0	0	0
Cohesion (c) (kPa)	44	28.48	38.4	111.15
Cohesion change with depth (kPa/m)	0	2.48	2.226	0

Unit Weight (kN/m3)	18.25	18.15	18.15	19.13
Failure Criterion	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb
Material Behaviour	Drained	Drained	Drained	Drained

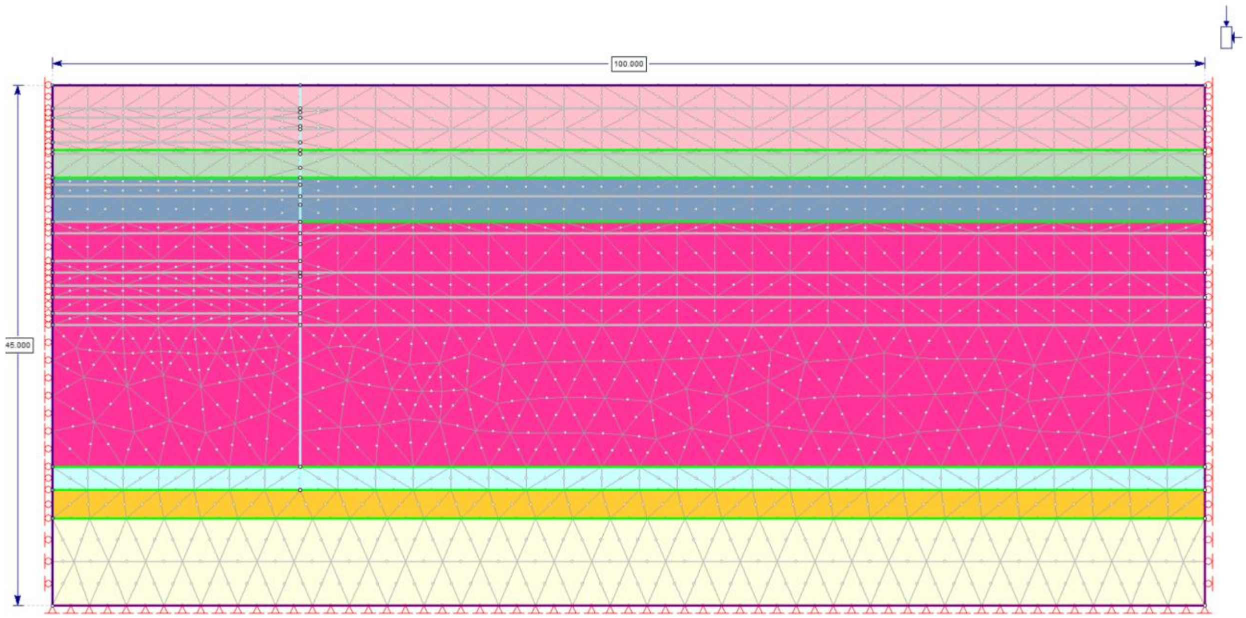

Figure 1.2: Model Geometry – Approach 1

Approach 2

In this approach, since the effective stiffness parameters will be utilized together with undrained shear strength, clays layer was modelled using Mohr Coulomb failure criterion with Poisson's ratio value of 0.3 and zero friction angles. To model undrained behaviors and generate excess pore pressure, the material behaviour of clay layers was set to undrained. Table 3 below shows the clay parameters of this approach, and the model prepared for this approach can be seen in Figure 1.3:

Table 1.3: Clay Parameters for the Second Approach

Depth	0m - 5.6m	8.0m - 11.8m	11.8m - 33.0m	35.0m - 37.5m
Type	Soft Silty Clay 1	Soft Silty Clay 2	Soft Silty Clay 3	Medium Soft Clay
Young's modulus (E) (kPa)	24869.6	18326.3 (+1595/m)	24709.6 (+1568.86 /m)	71522
Poisson's ratio (v) (-)	0.3	0.3	0.3	0.3
Friction angle (ϕ) (º)	0	0	0	0

Cohesion (c) (kPa)	44	28.48	38.4 (+2.226/m)	111.15
Unit Weight (kN/m3)	18.25	18.15	18.15	19.13
Failure Criterion	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb
Material Behaviour	Undrained	Undrained	Undrained	Undrained

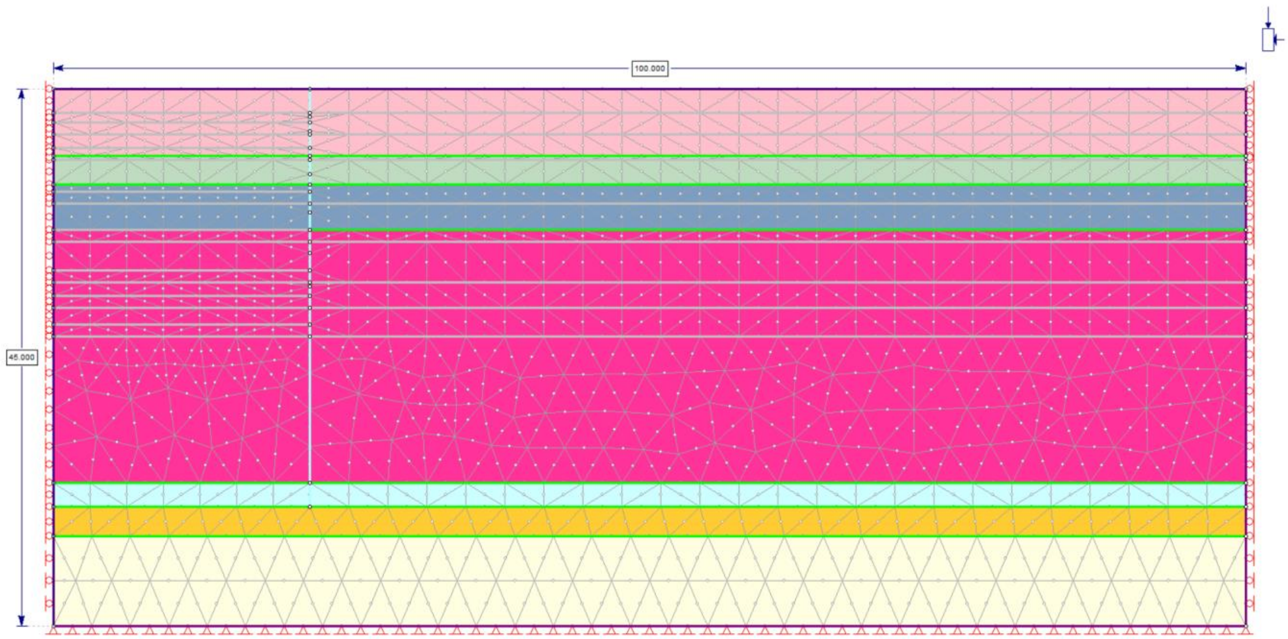

Figure 1.3: Model Geometry – Approach 2

Approach 3

In this approach, effective parameters were used for both strength and stiffness parameters. Two different models were created using two different constitutive models for clay layers. The first model used Mohr Coulomb model and the second one utilized Hardening Soil model. Undrained behaviours were modelled by setting material behaviour to undrained. Details of the two models are given below.

<u>Mohr Coulomb model</u>

The clay layers in the model used in this approach had a Poisson's ratio of 0.3, friction angles around 30°, and cohesion was not considered. The failure mode that the model was designed with was the Mohr-Coulomb failure criterion. The soil parameters of this approach were obtained from the journal article "Building response and ground movements induced by a deep excavation" (Ou, Liao, & Cheng, 2000). The clay layers' parameters can be seen in Table 4, along with a representation of this model in Figure 1.4 below:

Table 1.4: Clay Parameters for the Third Approach

Depth	0m - 5.6m	8.0m - 11.8m	11.8m - 33.0m	35.0m - 37.5m
Type	Soft Silty Clay UD	Soft Silty Clay	Soft Silty Clay Unloading	Medium Soft Clay UD
Young's modulus (E) (kPa)	24869.6	18326.3	24709.6 (+1568.86/m)	71522
Poisson's ratio (v) (-)	0.3	0.3	0.3	0.3
Friction angle (ϕ) (º)	33.9	29	29	0
Cohesion (c) (kPa)	0	0	0	31.6
Unit Weight (kN/m3)	18.25	18.15	18.15	19.13
Failure Criterion	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb
Material Behaviour	Undrained	Undrained	Undrained	Undrained

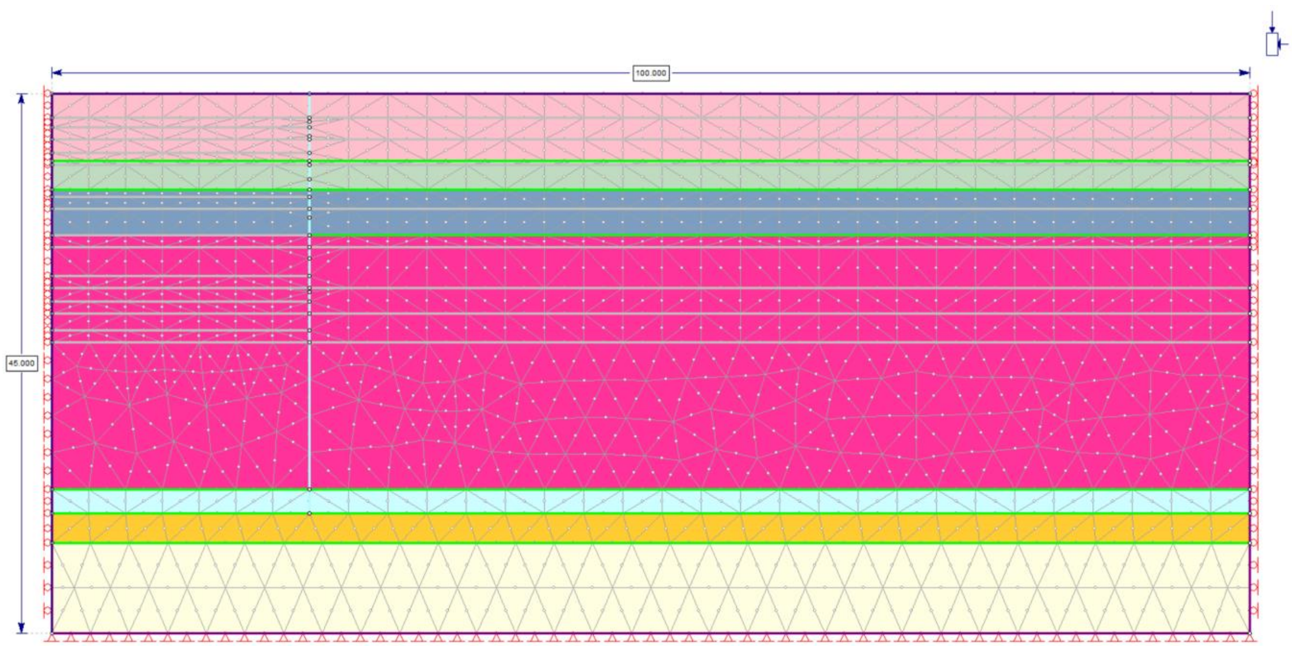

Figure 1.4: Model Geometry – Approach 3

Hardening soil model

In this model, the clay layer starting at 11.8m was split at the depths of 17.3m, 25.0m, and 33.0m, resulting in four different layers within that depth range with each having slightly different parameters. All the clay layers utilized the Hardening Soil failure criterion, unlike the Mohr-Coulomb one that was implemented in the three other models. The parameters of this model were obtained from the article "Evaluation of Soil Constitutive Models for Analysis of Deep Excavation under Undrained Condition journal article" (Lim, Ou, & Hsieh, 2010). Soil parameters and model are shown in Table 5 and Figure 5 respectively.

Table 1.5: Clay Parameters for the Refined Model of the Third Approach

Depth		0m - 5.6m	8.0m - 11.8m	11.8m - 17.3m	17.3m - 25.0m	25.0m - 33.0m	35.0m - 37.5m
Type		Soft Silty Clay UD	Soft Silty Clay		Soft Silty Clay Unloading	Medium Soft Clay UD	
Young's modulus (kPa)	E	-	-	-	-	-	71522.6 (+1911.13 /m)
	E_{50}	4574	9375.01	15952	19220.3	20750	21227
	E_{oed}	3202	6562	11167	13454	20750	6562
	E_{ur}	13723	28125	74858.4	57660	14525	28125
Poisson's ratio (v) (-)		0.2	0.2	0.2	0.2	0.2	0.2
Friction angle (ϕ) (º)		30	30	30	30	30	30
Cohesion (c) (kPa)		10	0	0	0	0	10
Dilitancy angle (ψ) (º)		5	5	5	5	5	5
Unit Weight (kN/m3)		18.25	18.15	18.15	18.15	18.15	19.13
Porosity (-)		0.5	0.5	0.5	0.5	0.5	0.5
Initial Pore Water Pressure (kPa)		1	1	1	1	1	1
Failure Criterion		Hardening Soil	Hardening Soil	Hardening Soil	Hardening Soil	Hardening Soil	Hardening Soil
Material Behaviour		Drained	Drained	Drained	Drained	Drained	Drained

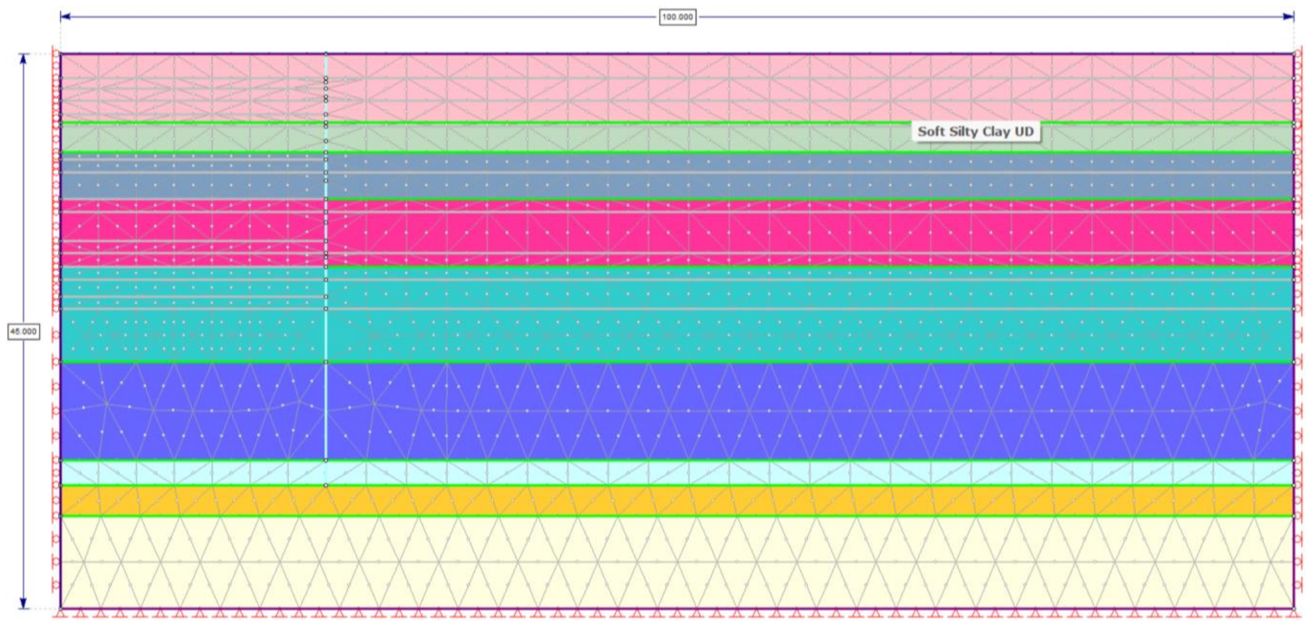

Figure 1.5: Model Geometry – Approach 3 Refined

1.3.2. Results

The observed and computed wall deflection are compared in Figure 1.6 to Figure 1.12. The computed wall deflection at the latter stages from stage 4 compared well with observed values. However, for the stage 1, 2 and 3 none of the approach can capture the wall deflection correctly. That may be contributed by the fact that the pore pressure has been dissipated partly during the construction (Ou and Lai, 1994) and its effects are more noticeable as the wall displacement are small. The results for the wall installation stage along with stages 1 to 7 can be seen in Figure 1.6 to Figure 1.12 in their respective order below:

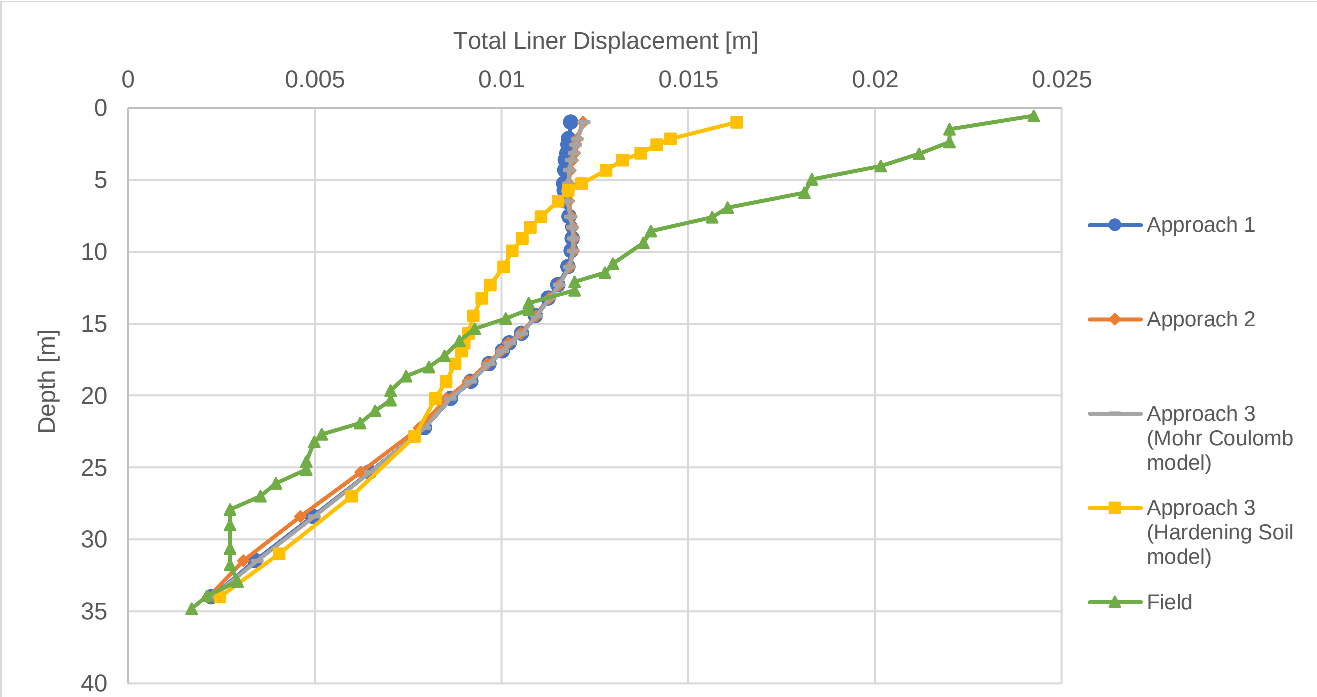

Figure 1.6: Total Liner Displacement – Stage 1: Excavate 2.8m

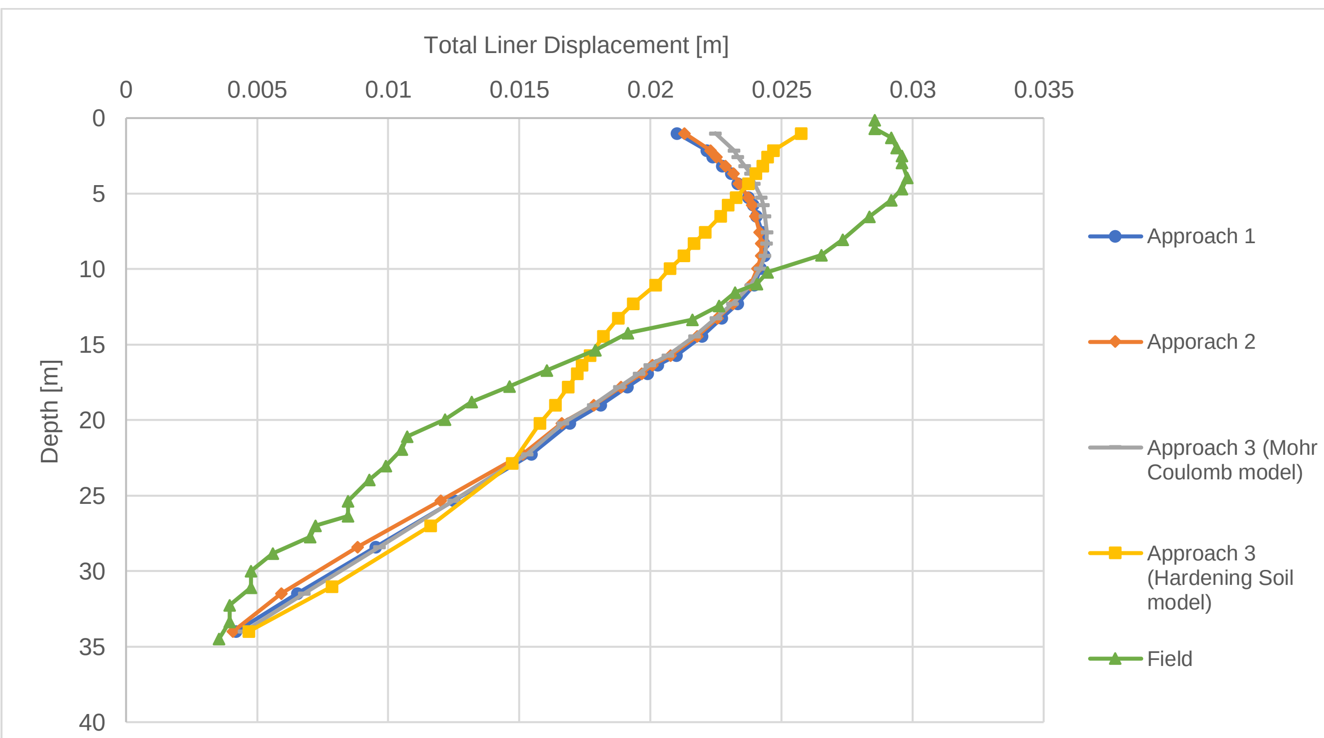

Figure 1.7: Total Liner Displacement – Stage 2: Excavate 4.9m

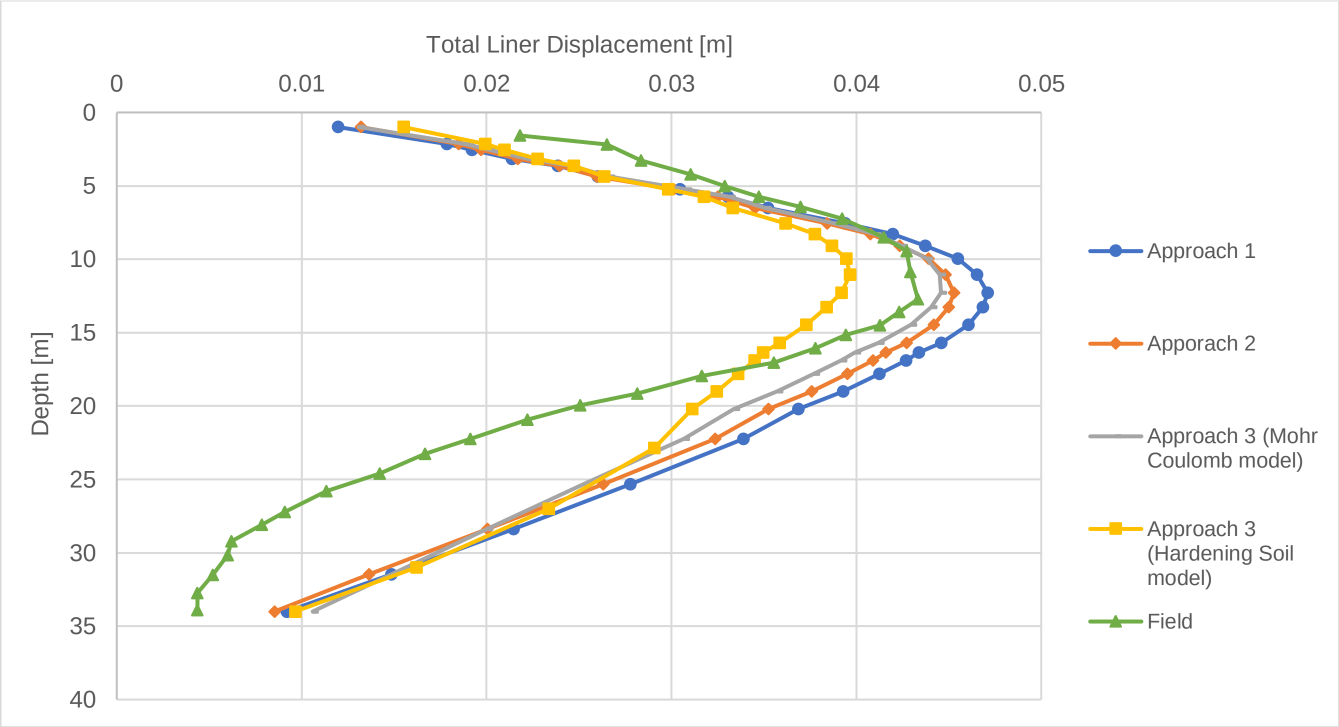

Figure 1.8: Total Liner Displacement – Stage 3: Excavate 8.6m

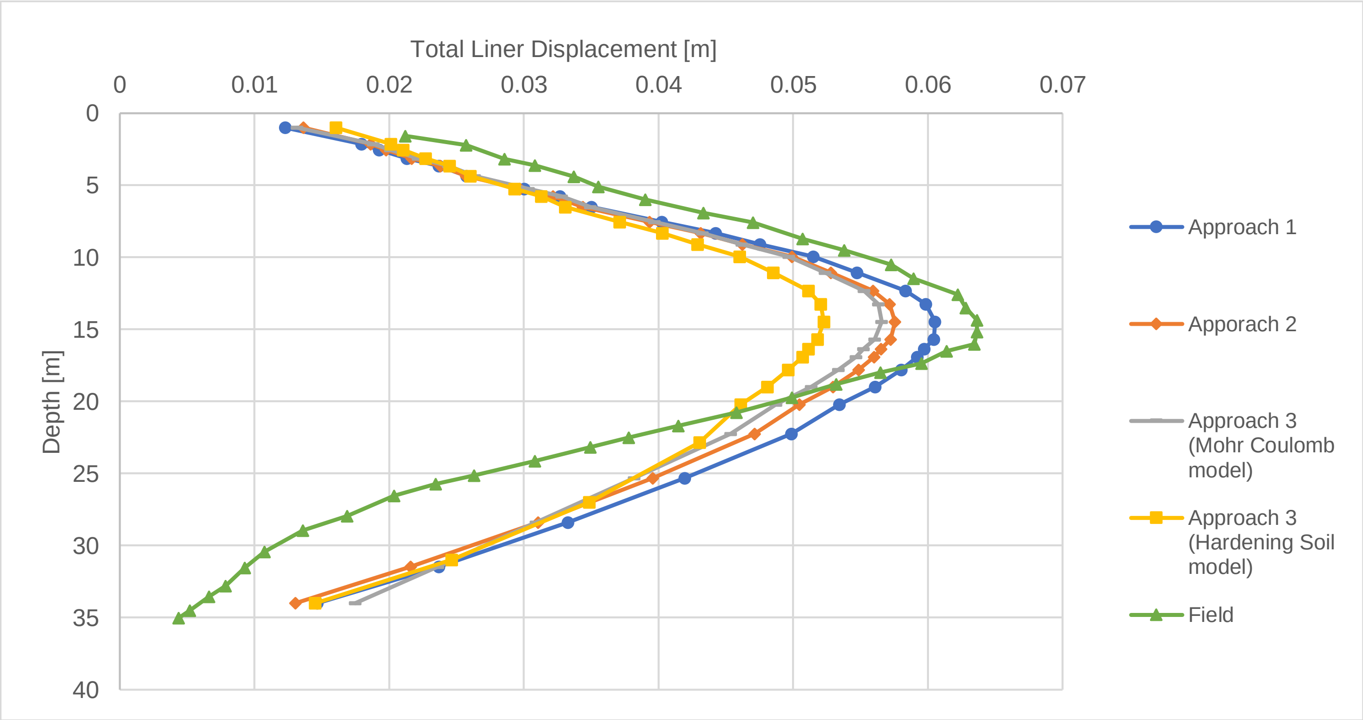

Figure 1.9: Total Liner Displacement – Stage 4: Excavate 11.8m

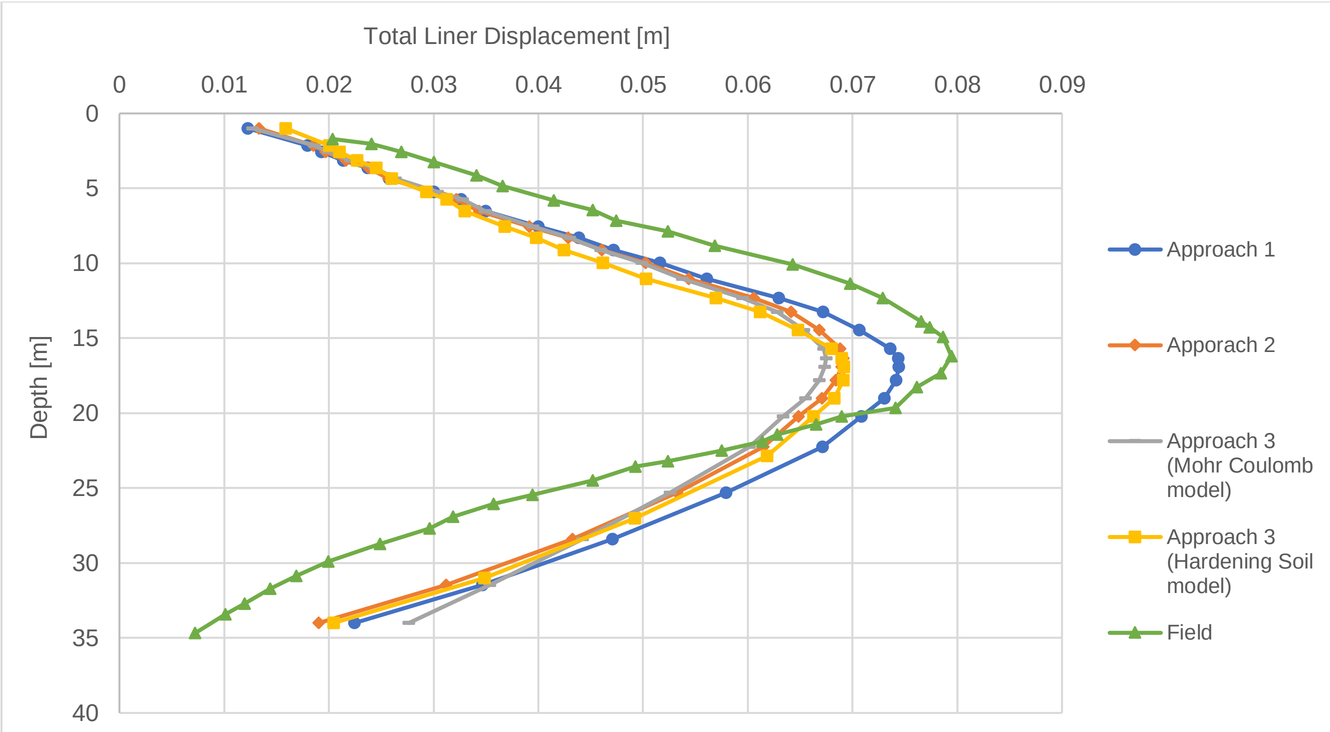

Figure 1.10: Total Liner Displacement – Stage 5: Excavate 15.2m

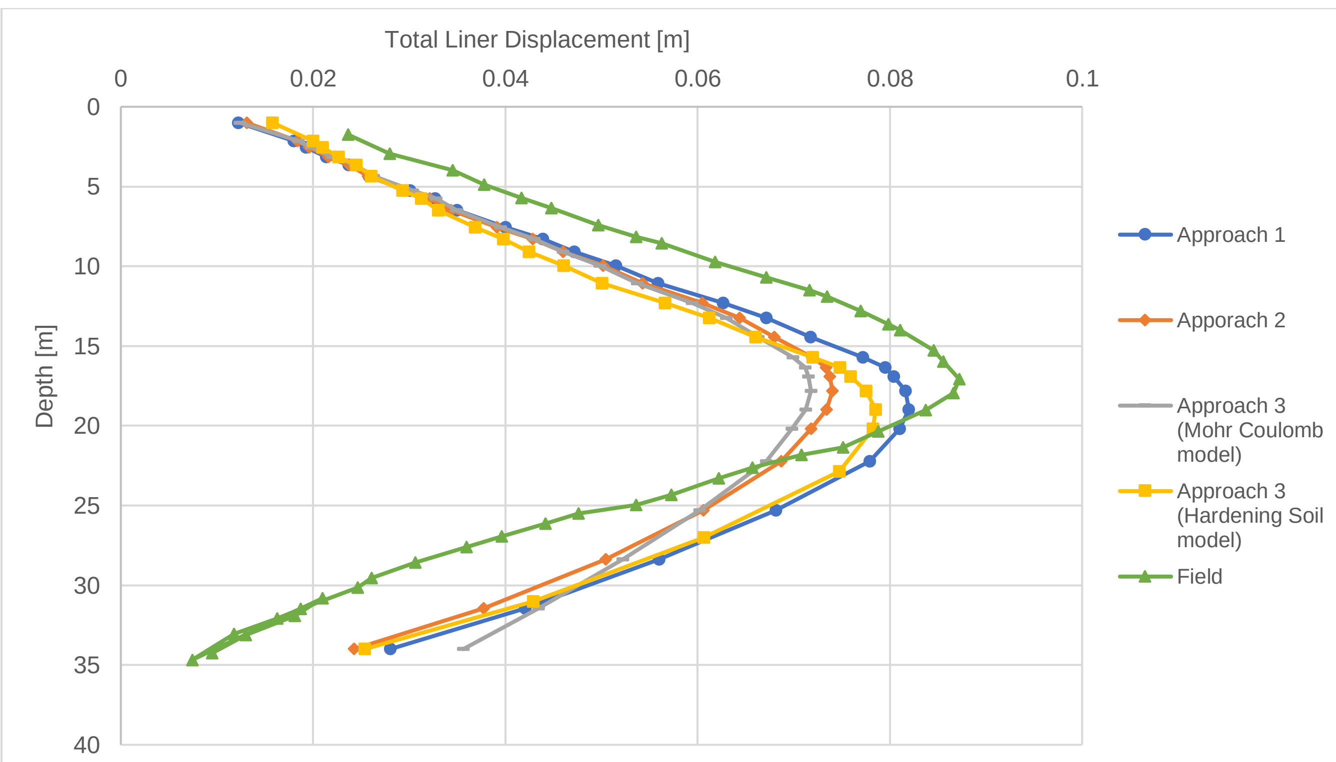

Figure 1.11: Total Liner Displacement – Stage 6: Excavate 17.3m

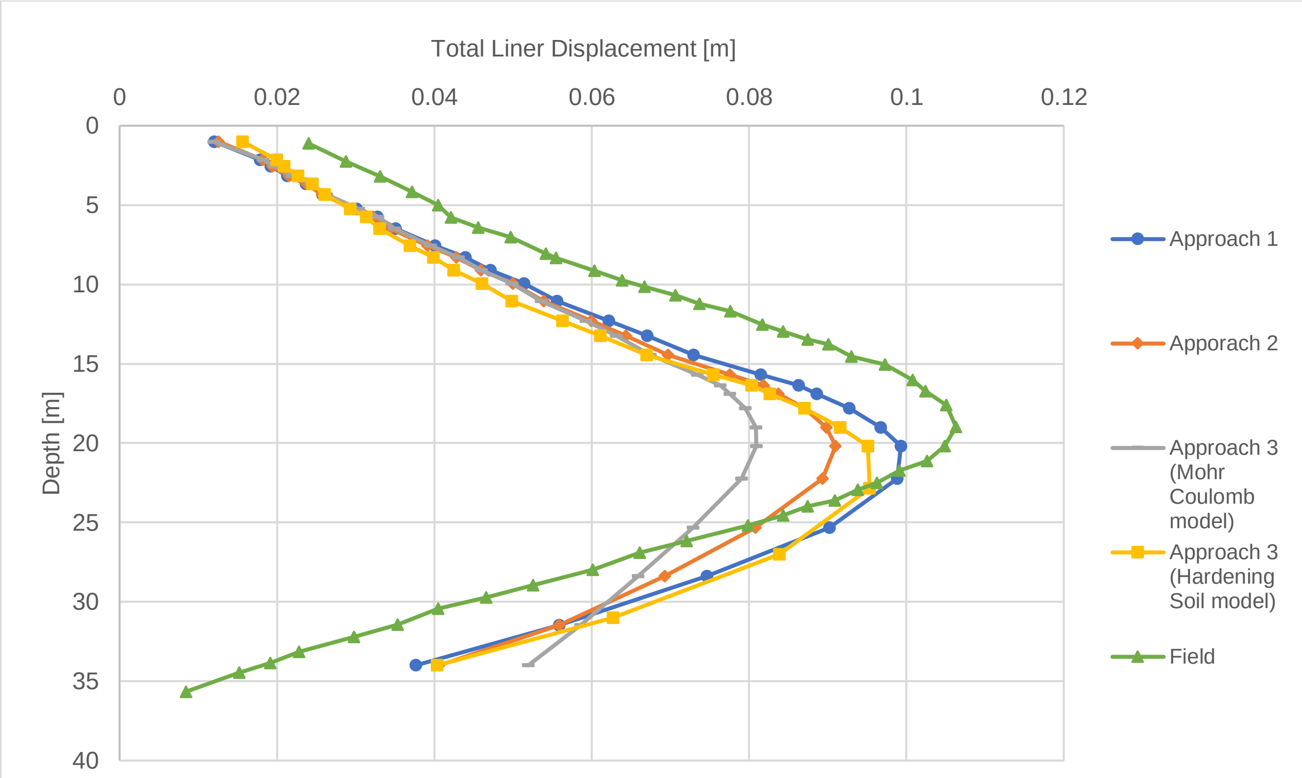

Figure 1.12: Total Liner Displacement – Stage 7: Excavate 19.7m

In the calculated ground settlements, there were more variations across the stages; hence, no specific approach was found to be very viable. Stage 2 was closely represented by approach 2. Stages 3 and 4 were not really closely represnted by any of the approaches, though it should be noted that approach 3 with Hardening soil model's results were somewhat close to the field measurements. As for stage 5, either approaches 2 or 4 could be followed as they were both similar to the field results. Both stages 6 and 7 were best resprented by approach 1, but approach 2 could also be used for stage 7. The results for the wall installation stage to stage 7 can be seen in Figure 1.13 to Figure 1.19 below:

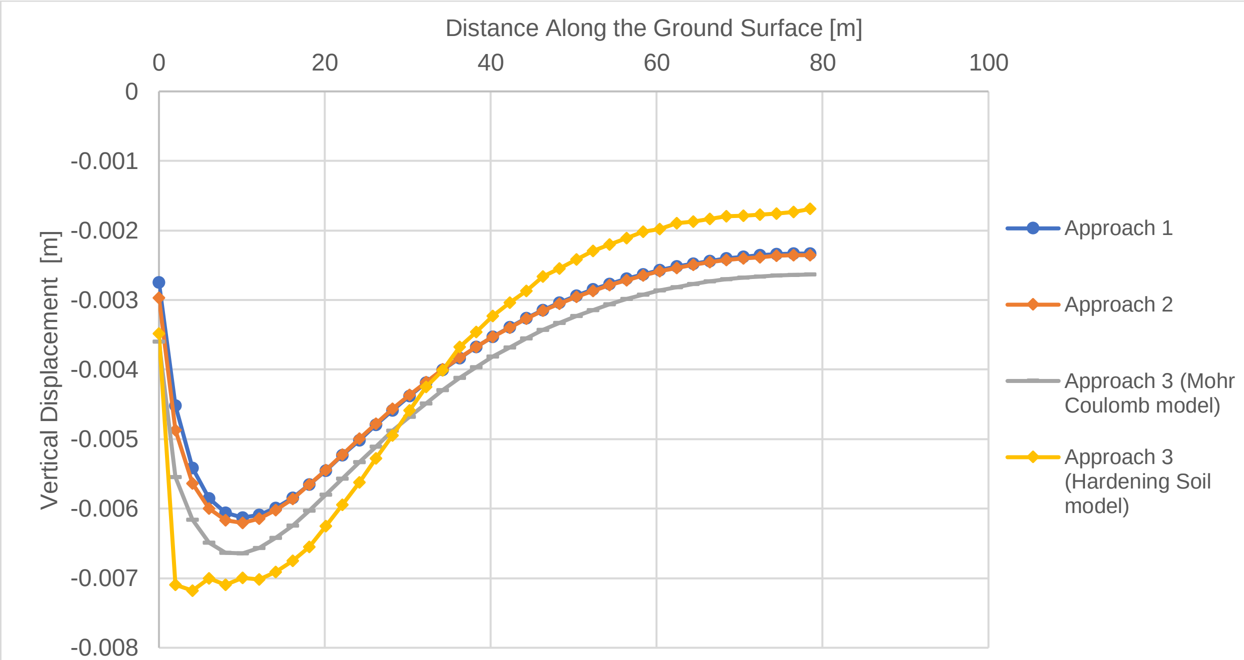

Figure 1.13: Distance Along the Ground Surface – Stage 1: Excavate 2.8m

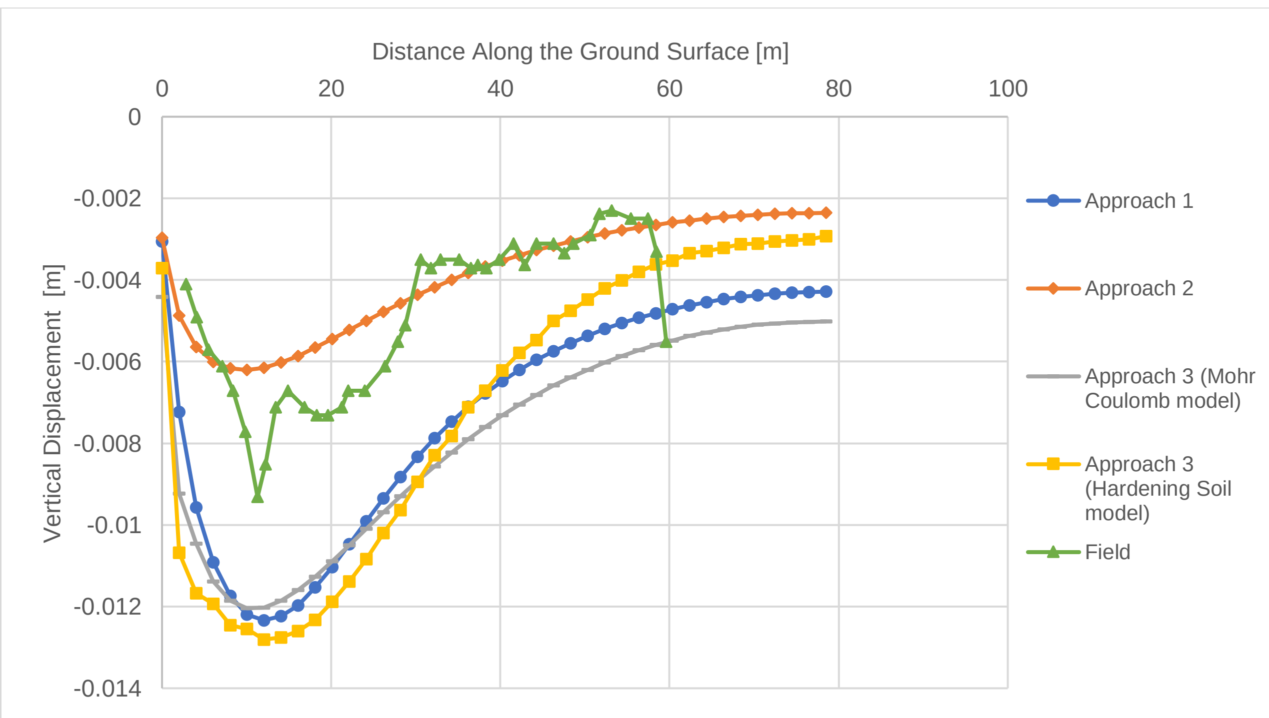

Figure 1.14: Distance Along the Ground Surface – Stage 2: Excavate 4.9m

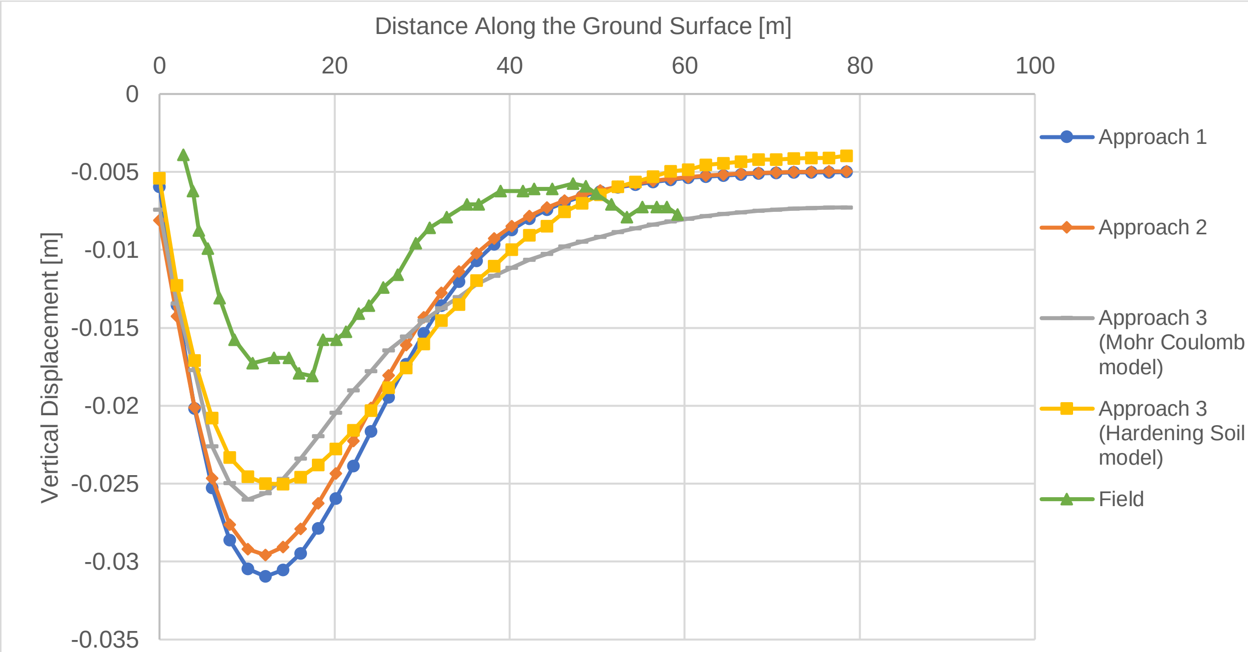

Figure 1.15: Distance Along the Ground Surface – Stage 3: Excavate 8.6m

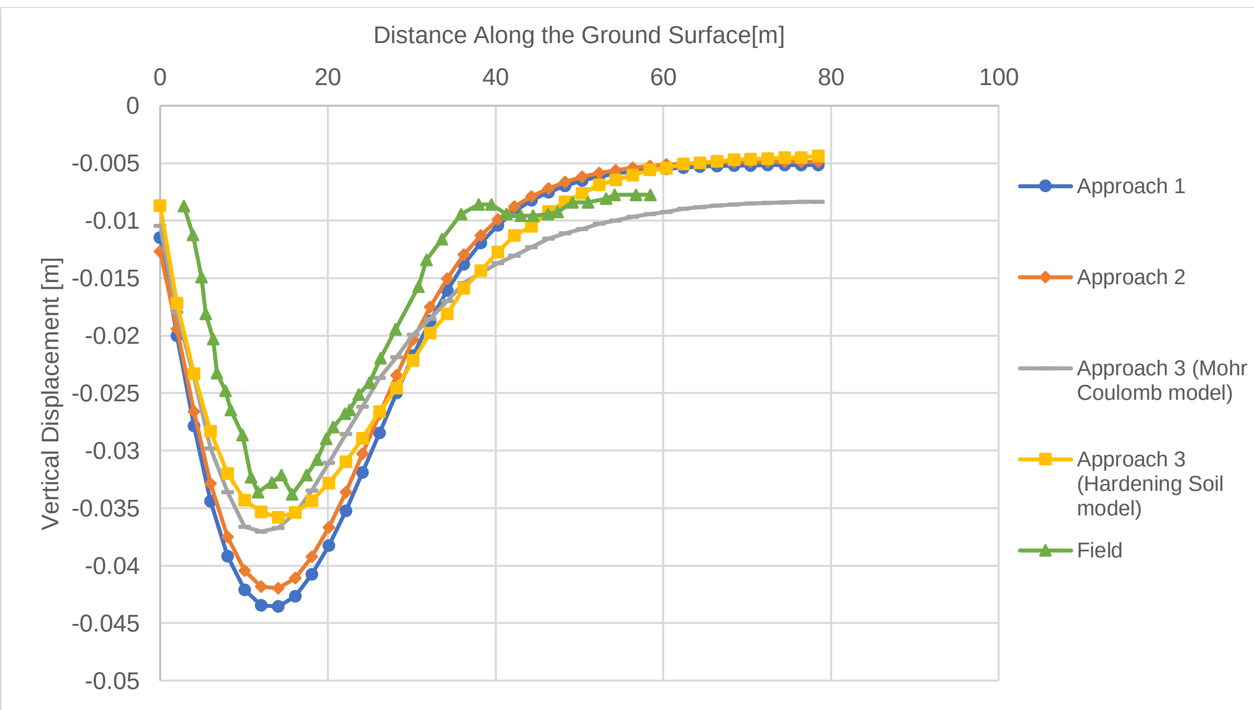

Figure 1.16: Distance Along the Ground Surface – Stage 4: Excavate 11.8m

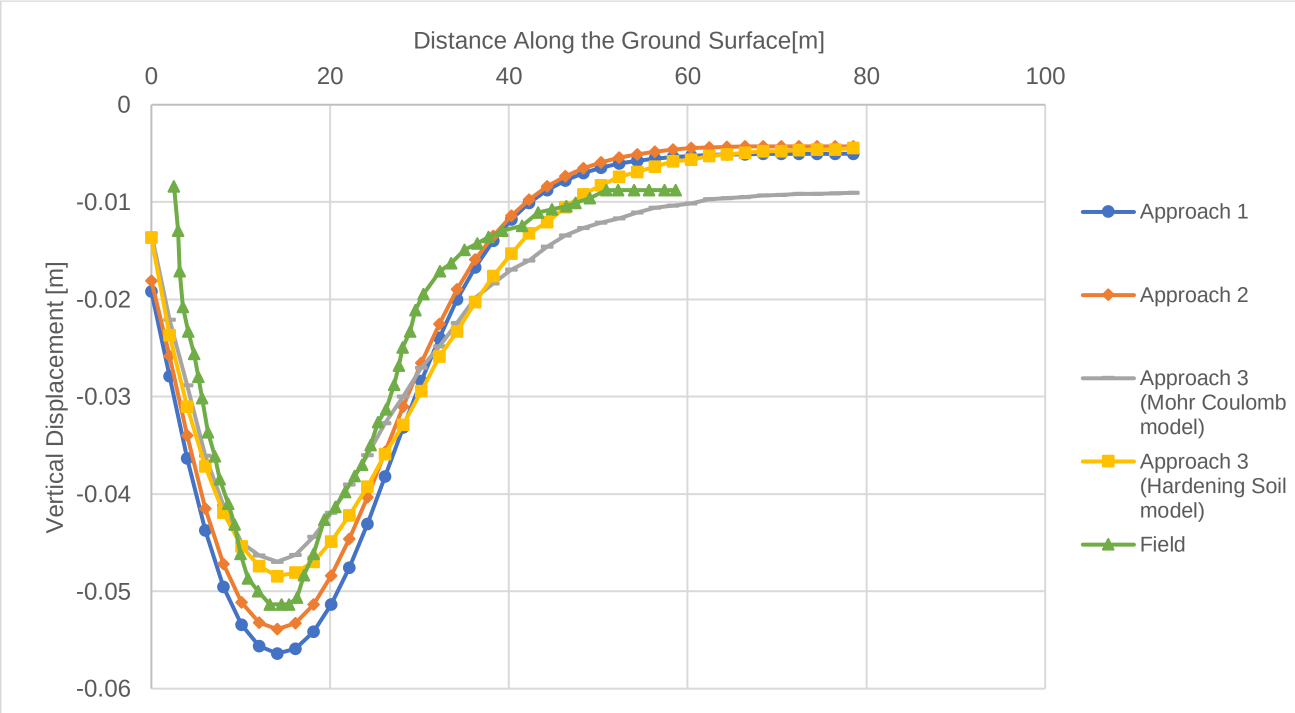

Figure 1.17: Distance Along the Ground Surface – Stage 5: Excavate 15.2m

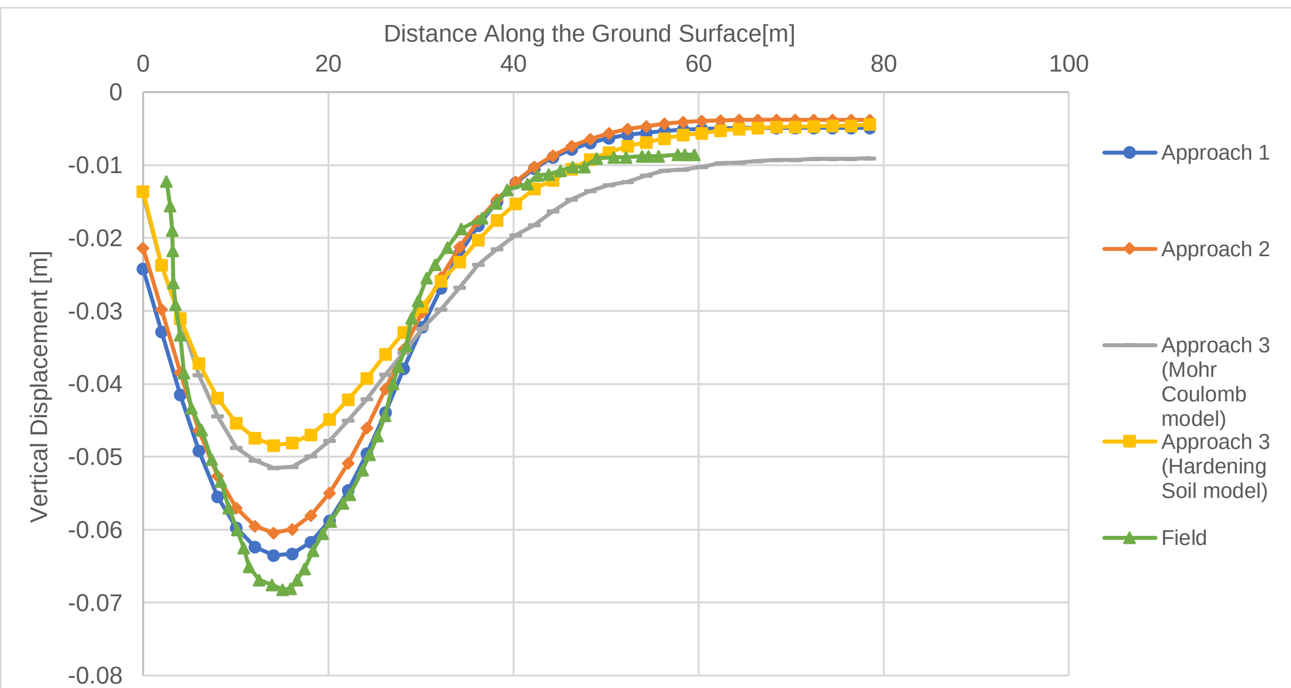

Figure 1.18: Distance Along the Ground Surface – Stage 6: Excavate 17.3m

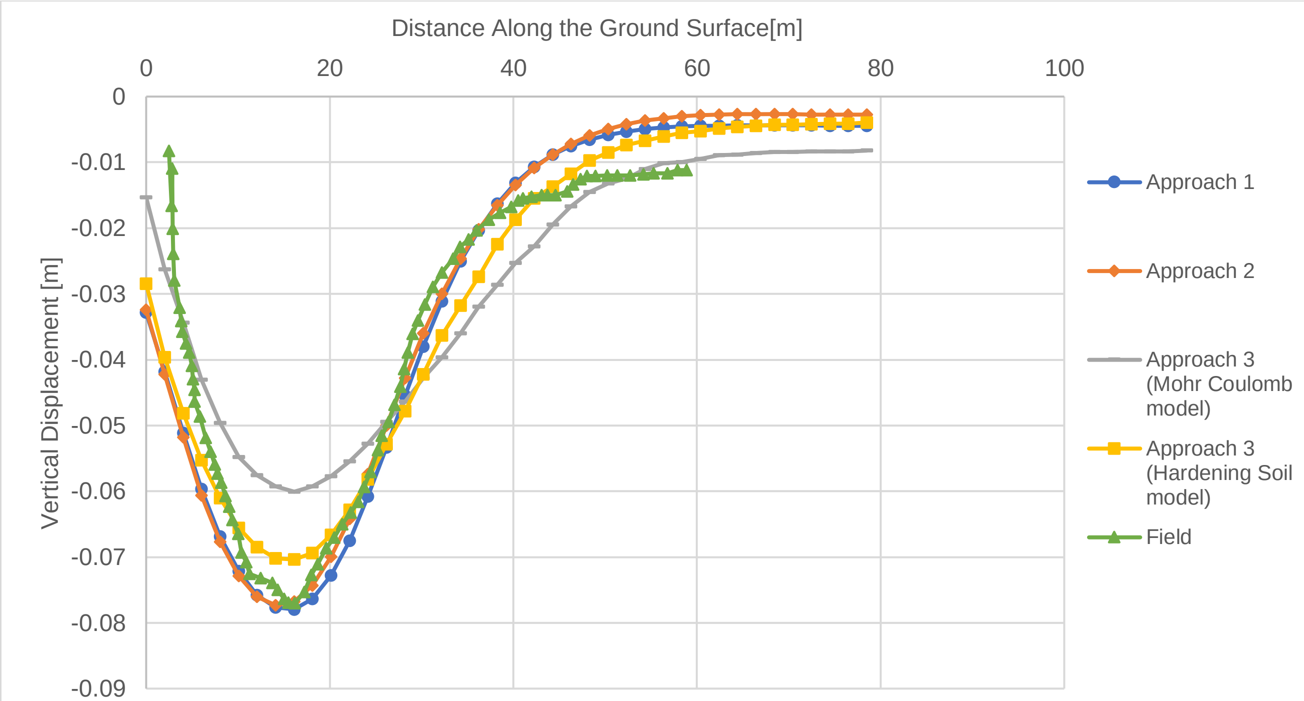

Figure 1.19: Distance Along the Ground Surface – Stage 7: Excavate 19.7m

1.4. Conclusions

An undrained analysis of an excavation in layered sandy and clayey soil using RS2 was presented. Three approaches were considered in the analysis:

1. Using undrained strength and stiffness parameters
2. Using undrained strength parameters and effective stiffness parameters
3. Using effective strength parameters and stiffness parameters

From the results, it can be shown that all the approaches can be used to predict wall deflection and soil settlement during the excavation. Given the long construction period, analysis that consider pore-water dissipation may give better prediction than using the undrained analysis.

1.5. References

Lim, A., Ou, C.-Y., & Hsieh, P.-G. (2010, April). Evaluation of Soil Constitutive Models for Analysis of Deep Excavation under Undrained Condition. *Journal of GeoEngineering, 5*, 9-20. doi:10.6310/jog.2010.5(1).2

Ou, C.-Y., Liao, J.-T., & Cheng, W.-L. (2000, January). Building response and ground movements induced by a deep excavation. *Geotechnique, 50*, 209-220. doi:10.1680/geot.2000.50.3.209

Ou, C.-Y., Shiau, B.-Y., & Wang, I.-W. (2000). Three-dimensional deformation behavior of the Taipei National Enterprise Center (TNEC) excavation case history. *Canadian Geotechnical Journal, 37*(2), 438-448. doi:10.1139/t00-018