Back-analysis of Excavation-Induced Ground Settlement in Hong Kong Using Mohr-Coulomb and Hardening Soil Models

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ABSTRACT

Deep basement excavations are routine in Hong Kong's dense urban environment, where excavation and lateral support (ELS) performance is often governed by serviceability criteria related to retaining wall deflection and ground settlement. Numerical assessment of excavation-induced ground movements in local practice commonly adopts the linearly elastic–perfectly plastic Mohr–Coulomb (MC) model in PLAXIS 2D as a baseline. However, the MC model employs a single elastic stiffness for both loading and unloading, which may be inconsistent with excavation stress paths and can lead to unrealistic deformation components, such as excessive unloading-induced heave. To bridge theory and practice in the spirit of the HKIE Geotechnical Division Annual Seminar theme, this paper presents a comparative back-analysis of a deep basement excavation in Hong Kong with comprehensive monitoring records, using both the MC model and the Hardening Soil (HS) model. The HS model is adopted to represent stress-dependent, non-linear soil stiffness and to distinguish stiffness during primary loading from unloading/reloading. Model set-up, parameter selection strategy and staged construction simulation are described. Predicted ground settlement profiles are compared with field measurements to illustrate the implications of constitutive model choice on deformation assessment. The paper concludes with practical observations for Hong Kong practitioners on when advanced constitutive modelling may be warranted for movement-sensitive ELS designs, and how monitoring-based back-analysis can be used to calibrate soil stiffness and improve confidence in deformation prediction.

Keywords: Deep excavation; ELS; Ground settlement

1 INTRODUCTION

1.1 Background

Deep excavations for basements, stations, and underground infrastructure are a defining feature of geotechnical engineering practice in Hong Kong. In dense urban settings, excavation-induced ground movements can affect adjacent buildings, utilities, and transport assets; therefore, ELS design is frequently controlled by serviceability performance rather than ultimate limit states. Field observations and case histories remain essential to build confidence in prediction methods and to refine design conservatisms, particularly where the ground profile is heterogeneous and the in-situ stress history is difficult to quantify (e.g., residual soils and weathered rock) (Chan 2003).

In Hong Kong, numerical modelling is widely used to estimate wall deflections and ground settlements for ELS works, with the linearly elastic–perfectly plastic Mohr–Coulomb (MC) constitutive model commonly adopted as the default baseline in finite element analysis. While MC is attractive due to its simplicity and limited parameter requirements, it represents pre failure stiffness using a single linear elastic modulus, which is applied identically in loading and unloading. For excavation problems dominated by unloading and stress-path dependency, this simplification may not reliably represent deformation behaviour, and unrealistic components (e.g., exaggerated heave during unloading) can be produced in certain cases (Chan et al. 2021).

More advanced constitutive models, such as the Hardening Soil (HS) model, provide an alternative framework to represent nonlinear stress–strain response and to distinguish stiffness under primary loading from unloading/reloading stiffness. Local and regional studies have reported that HS can improve the realism of

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predicted deformation profiles for excavations, particularly for ground surface settlement trough shape and for unloading dominated responses where MC may be less reliable (Chan et al. 2021).

1.2 Objectives

Against this backdrop, this paper presents a Hong Kong case study with comprehensive ground movement monitoring data and undertakes a comparative back analysis using PLAXIS 2D with both MC and HS models. The objectives are to:

(i) Compare the capability of MC and HS models in reproducing measured ground settlement associated with staged excavation;

(ii) Illustrate how constitutive model assumptions (especially stiffness under unloading) influence predicted ground movements; and

(iii) Provide practical observations that support calibration and model selection for movement sensitive ELS designs in Hong Kong, consistent with the Annual Seminar emphasis on "bridging theory and practice".

2 CONSITUTIVE SOIL MODELS

2.1 Mohr-Coulomb Model (MC)

The linearly elastic–perfectly plastic Mohr–Coulomb model is widely used in routine design of deep excavations, including Hong Kong applications. It is parameter efficient (e.g., strength and a single elastic stiffness), and its failure criterion remains practical for stability assessment when appropriate strength parameters are adopted.

For deformation prediction, the key limitation is that MC represents soil stiffness prior to yielding using a single linear elastic modulus and Poisson's ratio, without capturing nonlinear stiffness degradation with strain. Importantly for excavation, MC applies the same elastic stiffness in both loading and unloading, even though excavation-induced stress paths commonly involve significant unloading and reloading components. This can lead to unrealistic movement predictions in some scenarios, including overestimation of unloading heave and misrepresentation of settlement profiles behind retaining walls.

The MC model is a simple and widely used constitutive soil model in Hong Kong practice for predicting both stability and deformation of geotechnical structures, including deep excavations and slopes. It assumes linear elastic behaviour prior to yielding and perfectly plastic behaviour governed by the Mohr–Coulomb failure criterion, as illustrated in **Figure 1**. The model requires a limited number of input parameters, making it attractive for routine design applications.

However, soil stiffness in reality is stress-dependent, strain-dependent and stress-path dependent. In excavation problems involving significant unloading, the use of a single elastic stiffness for both loading and unloading in the MC model can lead to unrealistic deformation predictions, such as exaggerated unloading heave and misrepresentation of ground settlement profiles. While the MC model remains reliable for strength-controlled problems (e.g. overall stability), caution is required when it is used for deformation prediction in unloading-dominated problems such as deep excavations.

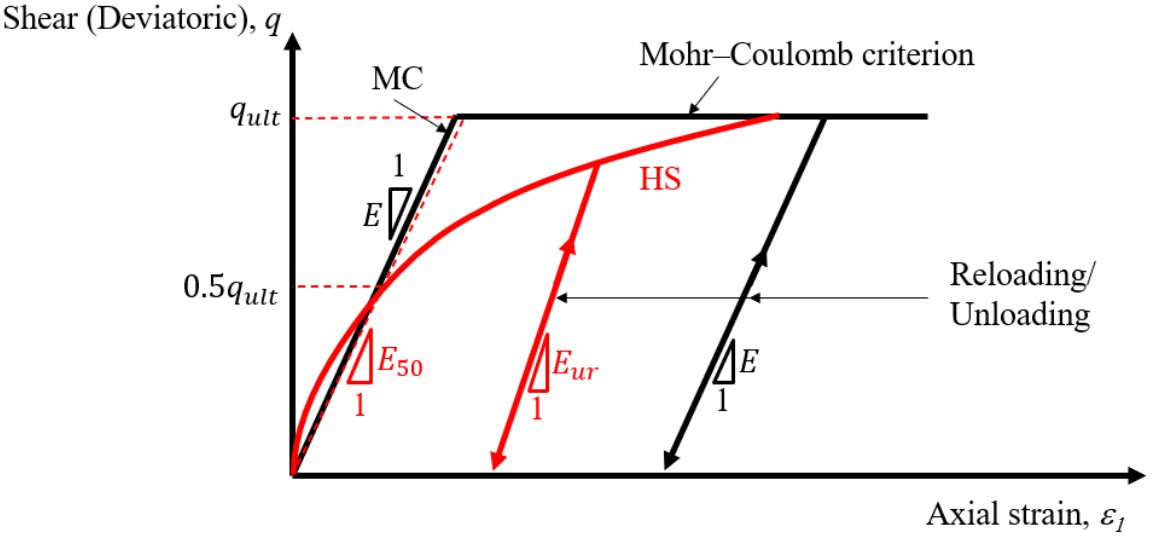


Figure 1 Stress-strain-strength behaviour of the MC and HS models in drained triaxial test

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2.2 Hardening Soil Model (HS)

The Hardening Soil (HS) model is an isotropic hardening constitutive model formulated within the framework of elasto-plasticity (Schanz et al., 1999). While the ultimate failure condition is still governed by the Mohr–Coulomb criterion, the HS model provides a more realistic stress-strain relationship by incorporating:

(i) Non-linear stiffness degradation;
(ii) Stress-dependent stiffness; and
(iii) Distinct stiffness parameters for primary loading, primary compression and unloading/reloading.

Three stiffness parameters are used: the secant stiffness in drained triaxial loading E_{50}^{ref}, the tangent stiffness in oedometer loading E_{oed}^{ref}, and the elastic unloading/reloading stiffness E_{ur}^{ref}. This distinction is particularly relevant for excavation problems, where unloading and stress relief dominate soil deformation behaviour. Stress-dependent stiffness in HS model can be represented by the following equations:

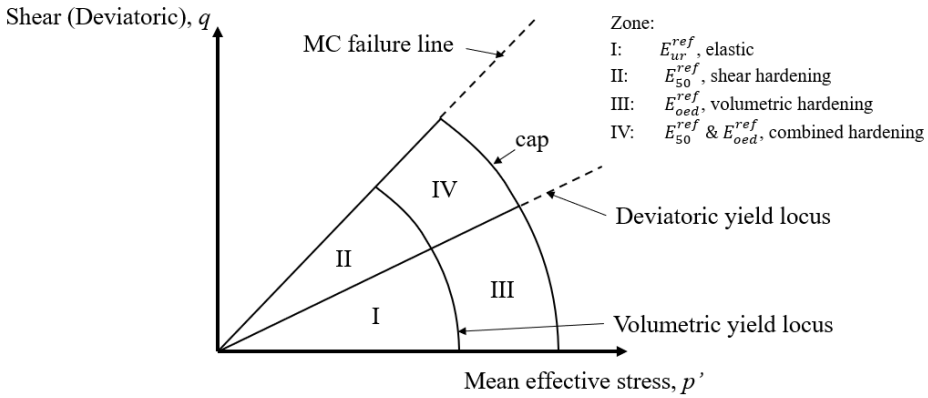
$$E_{50} = E_{50}^{ref}\left(\frac{c'\cos\varphi' + {\sigma_3}'\sin\varphi'}{c'\cos\varphi' + p_{ref}\sin\varphi'}\right)^m$$

$$E_{oed} = E_{oed}^{ref}\left(\frac{c'\cos\varphi' + {\sigma_1}'\sin\varphi'}{c'\cos\varphi' + p_{ref}\sin\varphi'}\right)^m$$

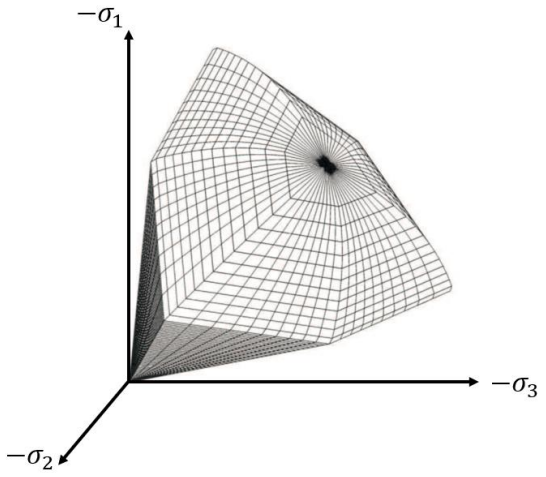
$$E_{ur} = E_{ur}^{ref}\left(\frac{c'\cos\varphi' + {\sigma_3}'\sin\varphi'}{c'\cos\varphi' + p_{ref}\sin\varphi'}\right)^m$$

where:

E_{50}^{ref}, E_{oed}^{ref}, E_{ur}^{ref} are reference stiffness at reference confining pressure p_{ref}; ${\sigma_1}'$, ${\sigma_3}'$ are major and minor effective principal stresses; m is the stress-dependent power.



(a)



(b)

Figure 2 (a) Double yield loci of the HS model, and (b) the HS model in principal stress space

HS parameters can be derived from laboratory tests (e.g., drained triaxial and oedometer tests), but uncertainties related to sample quality and representativeness are well recognised, especially in heterogeneous Hong Kong ground. Consequently, field tests and back analysis of well documented case histories are frequently used to calibrate stiffness parameters for practical design. In Hong Kong engineering design practice, Standard Penetration Test (SPT) is commonly adopted to determine the stiffness of soils. Empirical correlations between SPT-N values and large strain soil stiffness for geotechnical engineering problem with MC model are well established, the empirical correlations are reviewed and suggested in Chan 2001, and are summarized in **Table 1**. Some local back-analysis studies from Hong Kong have been also carried out to establish the empirical correlation to soil stiffnesses for HS model. Chan et al. 2021 summarises the suggested correlations and relationships between soil stiffnesses as summarized in **Table 2**.

Table 1 Empirical correlations between SPT-N values and Young's modulus for the MC model

Soil	Modulus of Elasticity, E (MPa)
Fill	1.5 N
Alluvium	1.5 N
CDG/ HDG	2.0 N

*N means uncorrected SPT-N values.

Table 2 Suggested Relationships between HS Stiffness Parameters

HS Model Input Parameters	Relationship	Reference
E_{50}^{ref}	1.2 N (MPa), for p^{ref} = 100 kPa	Hsiung & Dao (2014)
E_{oed}^{ref}	$1.0E_{50}^{\text{ref}}$, for sand	Schanz, Vermeer and Bonnier (1999)
	$0.5E_{50}^{\text{ref}}$, for soft clays	
	0.7 to $1.0E_{50}^{\text{ref}}$ for soils in Hong Kong	Lu et al. (2018), Lee et al. (2008), and Lam (2018)
E_{ur}^{ref}	3 to $6E_{50}^{\text{ref}}$, for loose sand	Schanz, Vermeer and Bonnier (1999)
	2 to $4E_{50}^{\text{ref}}$, for dense sand	
	3 to $5E_{50}^{\text{ref}}$, for clays	
	$3E_{50}^{\text{ref}}$	Khoiri & Ou (2013)
	2 to $3E_{50}^{\text{ref}}$, for soils in Hong Kong	Lu et al. (2018), Lee et al. (2008), and Lam (2018)
m	0.5 for sands and gravels 0.75 for silts 1.0 for clays	Ohde (1951 – 1955); Janbu (1963), Janbu (1970)

3 CASE STUDY

3.1 Background

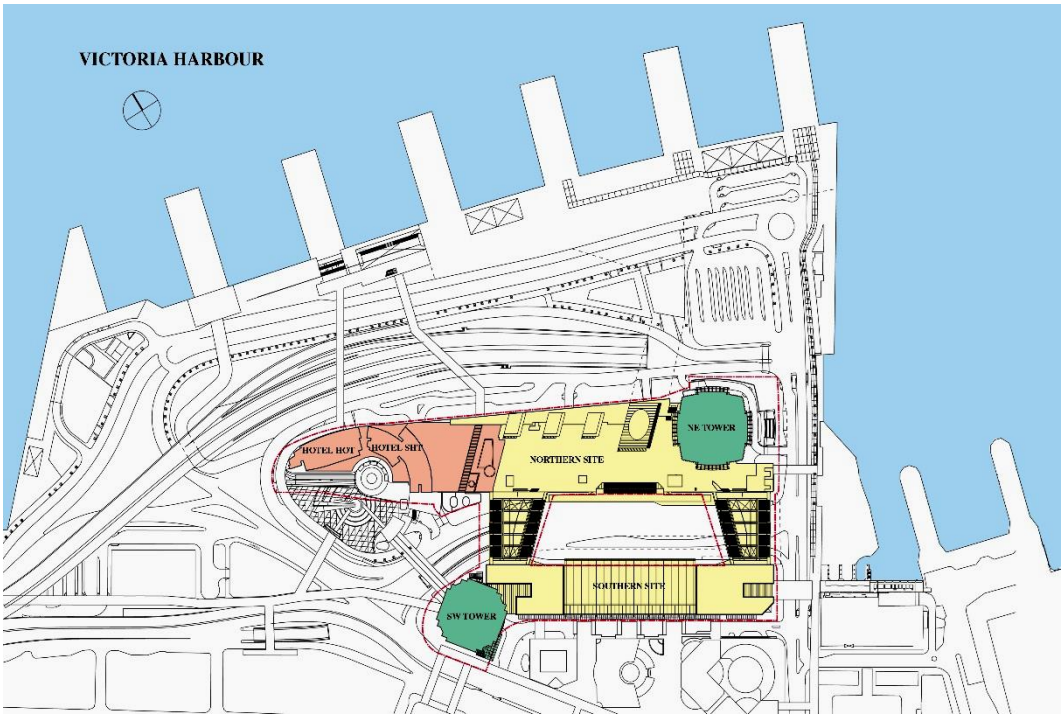
The Mass Transit Railway Corporation (MTRC) proposed to construct an airport railway from Hong Kong to Check Lap Kok. Hong Kong Station was going to form the terminus for the Airport Express Line (AEL) to the new airport at Check Lap Kok and also the Lantau Line (LAL). The Station was constructed in three phases as shown in **Figure** 3. Phase III basement excavation works are selected as the case study for back-analysis.

The case study relates to a deep basement excavation forming part of a major underground development in a dense urban area of Hong Kong. The project involved construction of a large plan-area basement supported by diaphragm walls and temporary strutting, with excavation extending to more than 25 m below ground level. Owing to the proximity of sensitive structures and transport infrastructure, excavation and lateral support (ELS) design was governed primarily by serviceability requirements related to ground settlement and wall deformation.

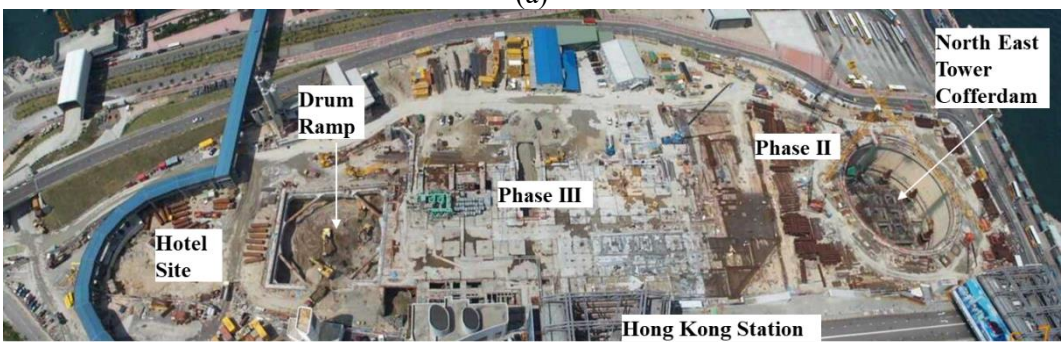
Phase III excavation was instrumented with a comprehensive monitoring system, including diaphragm wall inclinometers, ground settlement markers and groundwater observation points, providing high-quality field data for comparison with numerical predictions. The monitoring system is depicted in **Figure 4**.

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(a)



(b)

Figure 3 (a) Overall site layout plan, and (b) Phasing of construction works

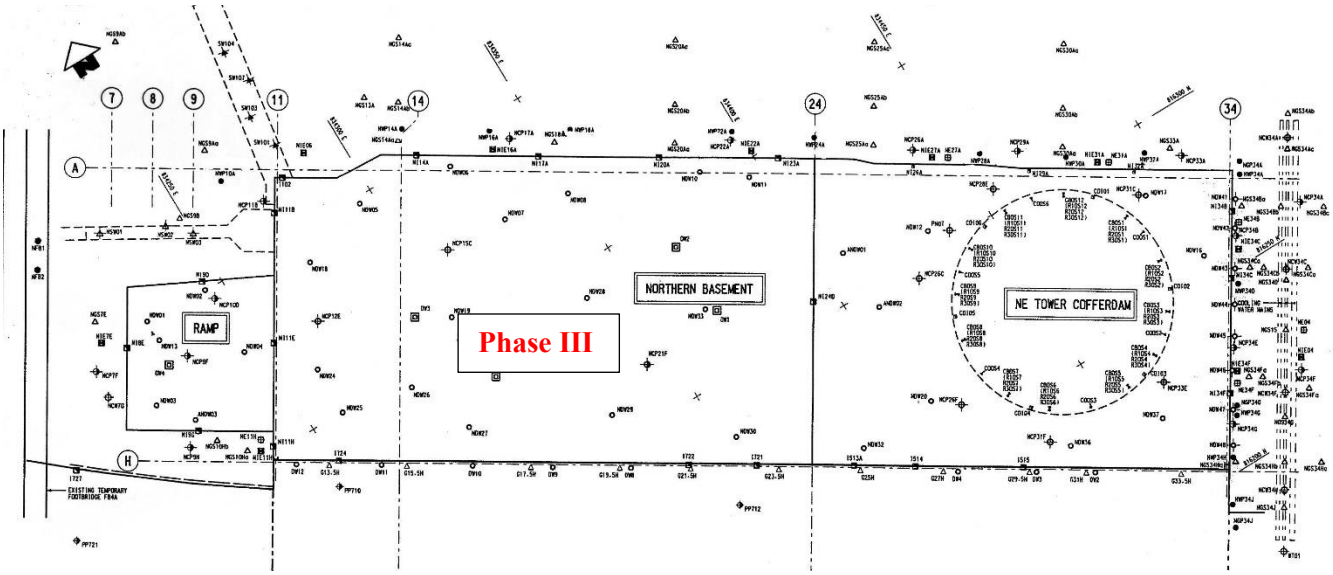


Figure 4 Monitoring and Instrumentation Plan

3.2 Ground Conditions

The site is underlain by typical Hong Kong geological materials comprising fill, marine deposits and alluvium overlying completely decomposed granite (CDG), with bedrock encountered at depth. Based on site investigation records along Gridline A as shown in **Figure 5**, the generalized ground profile consists of:

(i) Fill at ground surface, variable in thickness;
(ii) Marine deposits and/or alluvium, comprising soft to firm silty clay and sandy clay;
(iii) Completely decomposed granite (CDG), transitioning gradually to moderately decomposed granite (MDG) with depth;
(iv) Granite bedrock at greater depth.

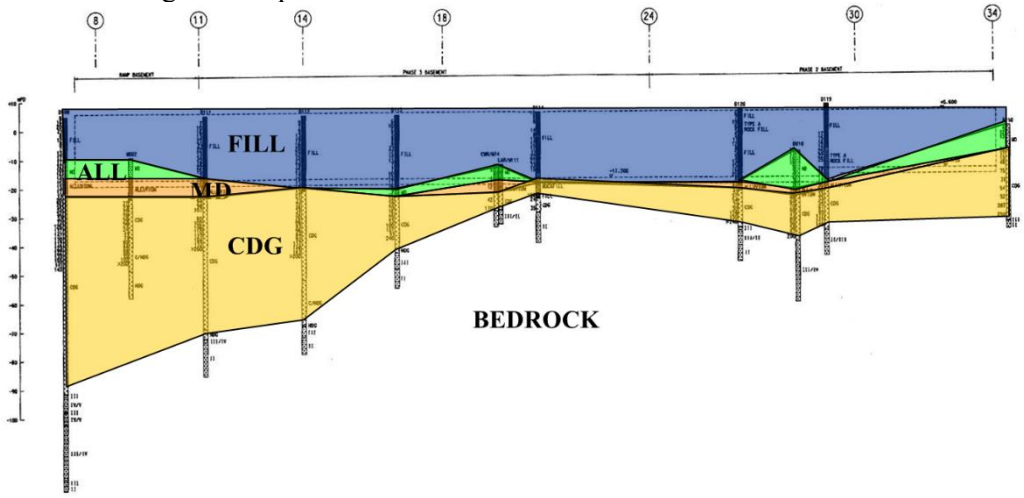


Figure 5 Geological Section along Gridline *A*

Groundwater levels were observed to be close to ground level under long-term conditions, with temporary drawdown implemented during excavation as part of the dewatering strategy. The excavation therefore involved significant unloading of both effective stress and total stress within the soil mass. Such stress paths are known to be sensitive to soil stiffness non-linearity and unloading–reloading behaviour, making this case particularly suitable for comparing the Mohr–Coulomb (MC) and Hardening Soil (HS) models.

3.3 Construction Sequences

The excavation was carried out in a staged manner with the installation of temporary supports at intermediate levels. A simplified summary of the construction sequence adopted for numerical modelling is as follows:

(i) Installation of diaphragm walls extending into the underlying decomposed granite, forming the permanent retaining system.
(ii) Initial excavation to an upper level, followed by installation of the first level of strutting.
(iii) Subsequent staged excavation and installation of additional struts at progressively lower levels.
(iv) Continued excavation to final formation level, with dewatering implemented to control groundwater inflow and pore water pressure beneath the excavation base.
(v) Completion of temporary works prior to construction of permanent basement slabs.
(vi) Backfill the unexcavated side to a higher ground level.

The staged excavation resulted in repeated cycles of unloading and partial reloading of the retained soil mass. The timing and elevation of excavation stages and strut installations were well documented and were explicitly reproduced in the numerical analyses to enable meaningful comparison with measured performance. **Figure 6** shows the design ground profiles and structural elements of the plane-strain PLAXIS models. Note that the fill layer at the excavation side was compacted before excavation commenced, hence its friction angle Φ' was set larger than uncompacted fills, as compared in **Table 3**.

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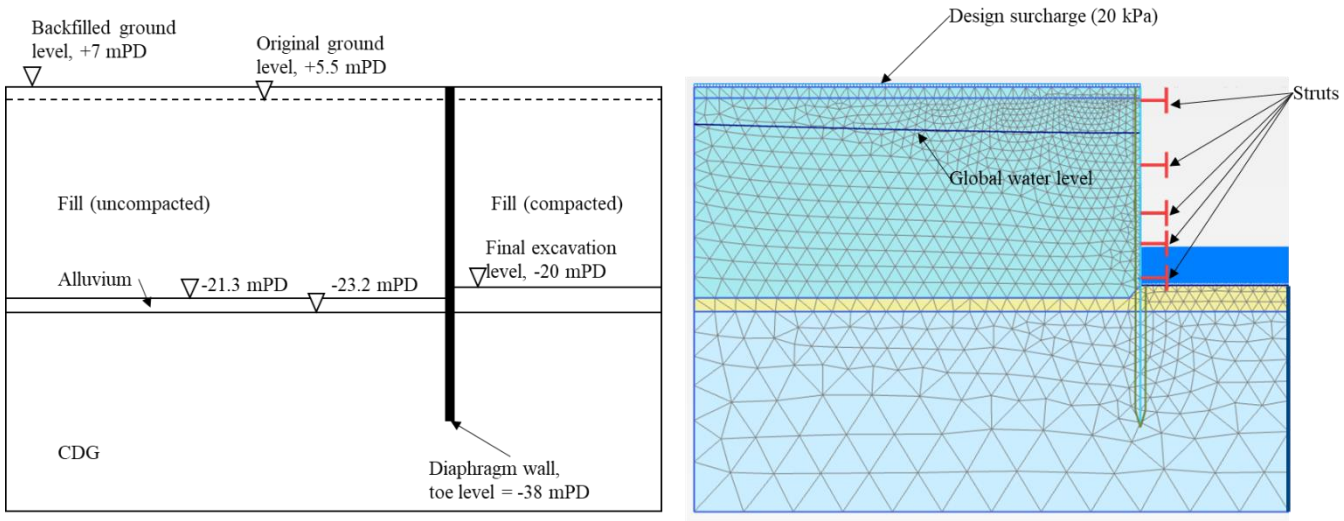


Figure 6 Illustration for Ground Profiles and Design Models

Table 3 Design Soil Densities and Strength Parameters

Soils	Fill	Compacted fill	Alluvium	CDG
Saturated unit weight γ_{sat}' [kN/m^3]	19	19	19	19.5
Friction angle ϕ' [deg]	32	38	32	37
Cohesion c' [kPa]	0	0	0	0

3.4 Instrumentation, Monitoring and Measured Performance

A comprehensive instrumentation and monitoring scheme was implemented for Phase III excavation works to track excavation-induced ground and structural movements. The monitoring system included:

(i) Diaphragm wall inclinometers, measuring lateral displacement profiles of the retaining wall at various construction stages;

(ii) Ground settlement markers, installed at multiple offsets from the diaphragm wall to record development of surface settlement troughs;

(iii) Piezometers and standpipes, monitoring groundwater levels and assessing the effectiveness of dewatering measures.

Monitoring data indicate that wall deflection and ground settlement developed progressively with excavation depth, with the maximum movements occurring during deeper excavation stages when unloading effects dominated. The measured settlement profiles show a well-defined trough extending beyond the toe of the diaphragm wall, providing a valuable basis for assessing the ability of different constitutive models to reproduce both the magnitude and shape of ground settlement.

The completeness and consistency of monitoring records for Phase III excavation works, combined with clearly defined construction stages and boundary conditions, make this section well suited for numerical back-analysis and for evaluating the implications of constitutive soil model assumptions on deformation prediction. The measured wall deflection and the measured ground settlement profiles are used for back-analysis and will be presented in the following paragraphs.

3.5 PLAXIS Models for Prediction and Back-analysis

Three plane-strain PLAXIS models were built following the sequences introduced in Section 3.3. The first one adopted MC model, and the stiffnesses of soils were designed as in **Table 1**. This was to reproduce the prediction models in typical design submissions for acquiring approval in local industry. The second one also adopted MC model, however the stiffnesses of soils were adjusted with E = 3.0 N (in MPa) to match actual monitored deformations (the back-analysis model). The third one adopted HS model for back-analysis purposes. The densities and strengths of each soil layer are shown in **Table 3**, and the stiffness profiles are listed in **Table 4**.

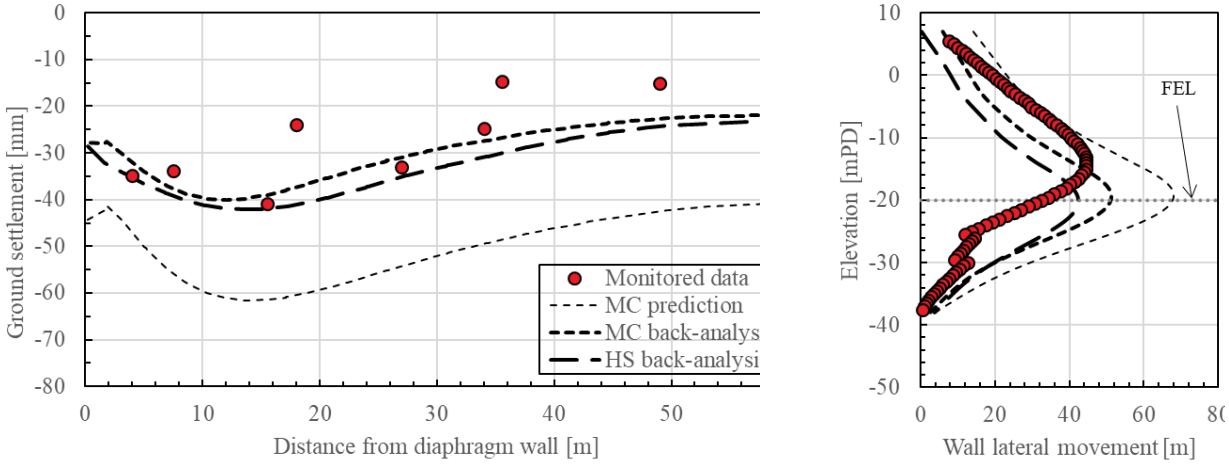
In the HS model, p^{ref} for fill and compacted fill was designed as the at-rest effective confining stress σ_3', at 0 mPD; p^{ref} for alluvium and CDG layers was designed as σ_3' at the top level of each layer. Correspondingly, E_{50}^{ref} of each soil layer was the reference secant modulus at the respective reference pressure. E_{oed}^{ref} was assumed to be equal to E_{50}^{ref} and E_{ur}^{ref} was assumed to be 3 times of E_{50}^{ref}, which are default PLAXIS settings.

Table 4 Summary of MC and HS Stiffness Parameters

Soils			Fill	Compacted fill	Alluvium	CDG
Model parameters	MC model (prediction), E [kPa]		10,830 + 839z, z ≤ 5 mPD	24,600 + 3417z, z ≤ 5 mPD	28,500 + 1,500z, z ≤ -19 mPD	18,000 + 14,235z, z ≤ -23 mPD
	MC model (back-analysis), E [kPa]		21,660 + 1,679z, z ≤ 5 mPD	49,200 + 6,834z, z ≤ 5 mPD	57,000 + 3,000z, z ≤ -19 mPD	27,000 + 21,350z, z ≤ -23 mPD
	HS model (back-analysis)	p^{ref} [kPa]	23.3	19.0	103.7	102.1
		E_{50}^{ref} [kPa]	25,000	80,000	60,000	100,000
		E_{oed}^{ref} [kPa]	25,000	80,000	60,000	100,000
		E_{ur}^{ref} [kPa]	75,000	240,000	180,000	300,000
		m	0.6	0.6	1.0	0.6

Figure 7 compares the modelled ground settlement and wall lateral movement against the in-situ monitored data, with both MC and HS models. The MC prediction model gave over-estimated settlement and wall movement compared to the monitored data, primarily due to the conservative assumptions in soil stiffness. This is the routine result in the local design experience, to avoid unforeseen large in-situ ground settlement. In contrast, the MC back-analysis model and the HS back-analysis model can well capture the maximum monitored ground settlement, around 41 mm at 10 - 15 m away from the wall. At further distance, all modelled settlement profiles were larger than the monitored data, probably because a strip surcharge of 20 kPa was applied in the models.

The lateral movement of walls was monitored by inclinometers at certain locations and **Figure 7**(b) presents the average monitored movements. The maximum wall movement estimated from the MC back-analysis model and HS back-analysis model was comparable to the in-situ monitored data, around 45 mm, and the elevation at which maximum movement was observed was slightly below the final excavation level (FEL). However, the monitored data showed that maximum wall movement occurred at a higher level.



(a) (b)

Figure 7 Predicted and Back-analysed Profiles of (a) Ground Settlement and (b) Lateral Movement of Walls, by MC and HS Models

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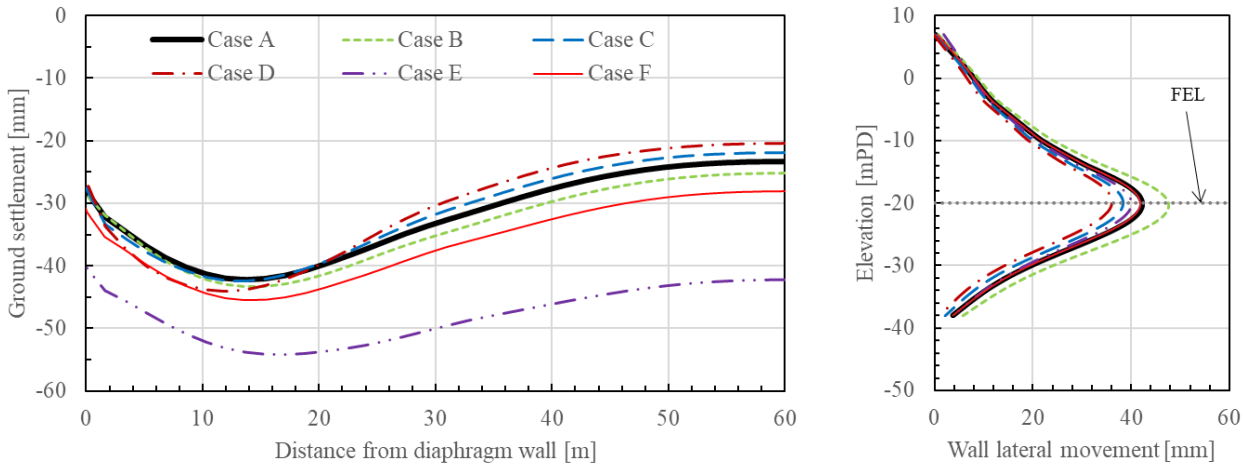
3.6 Sensitivity Study on HS Parameters

To explore the influence of the HS model stiffness parameters on the modelling responses during excavation works, a sensitivity study was also performed. The ratios of E_{oed}^{ref} / E_{50}^{ref} and E_{ur}^{ref} / E_{50}^{ref} were varied, as shown in **Table 5**, from 0.5 to 1 and from 2 to 10, respectively. These ratios are deemed to cover typical ranges adopted in previous studies, as shown in **Table 2**. Other soil and structural properties and model parameters remain the same as above.

The profile of ground settlement and wall lateral movement of each case is plotted in **Figure 8**. Case A to D investigate the influence of E_{ur}^{ref}. Maximum ground settlement is not sensitive to E_{ur}^{ref}, which varies in a narrow range between 42.1 mm to 44.1 mm. On the other hand, there is a clear tendency that maximum wall movement decreases with increasing E_{ur}^{ref}. Case A, E and F investigate the influence of E_{oed}^{ref}. With increase in E_{oed}^{ref}, ground settlement reduces significantly, while wall lateral movement increases slightly. Base heave at final excavation level at varying E_{ur}^{ref} / E_{50}^{ref} ratios is also investigated. Maximum heave reduces by about 50% when E_{ur}^{ref} / E_{50}^{ref} is increased from 2 to 10.

Table 5 Sensitivity Study on the Stiffness Ratios of HS Models

Scenarios	A (Base case)	B	C	D	E	F
E_{oed}^{ref} / E_{50}^{ref}	1	1	1	1	0.5	0.8
E_{ur}^{ref} / E_{50}^{ref}	3	2	5	10	3	3



(a) (b)

Figure 8 Ground Settlements and Wall Deflections at Varying E_{oed}^{ref} / E_{50}^{ref} and E_{ur}^{ref} / E_{50}^{ref} Ratios

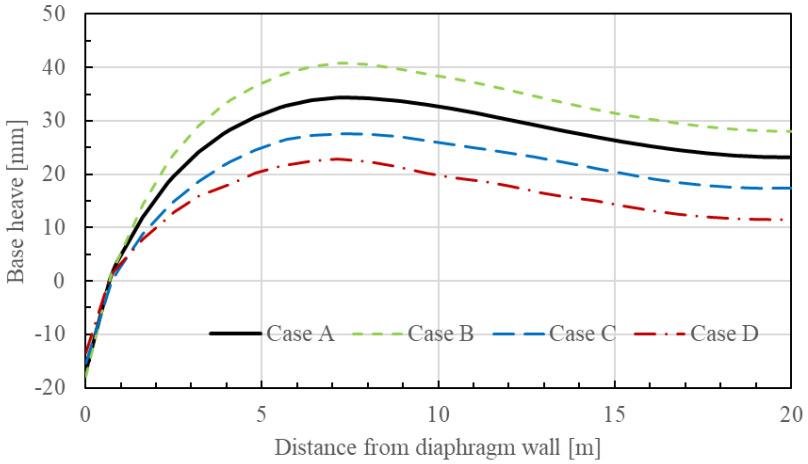


Figure 9 Variation of Base Heave with E_{ur}^{ref} / E_{50}^{ref} Ratios

4 CONCLUSIONS

This paper presents a comparative back-analysis of a deep basement excavation in Hong Kong using the Mohr–Coulomb (MC) and Hardening Soil (HS) constitutive models, with reference to comprehensive field monitoring data. Based on the case study and sensitivity analyses, the following conclusions are drawn:

The MC model, when used with stiffness parameters derived from conventional empirical correlations for design prediction, produced conservative estimates of ground settlement and wall deflection. While such conservatism is commonly adopted in Hong Kong practice to manage serviceability risk, the MC model's use of a single elastic stiffness for both loading and unloading can lead to unrealistic deformation components in unloading-dominated excavation problems, including exaggerated settlement and base heave.

Back-analysis using an adjusted MC stiffness profile and the HS model both achieved good agreement with the monitored maximum ground settlement and wall lateral movement. However, the HS model provided a more realistic representation of soil behaviour by explicitly accounting for stress-dependent stiffness and distinguishing between primary loading and unloading/reloading stiffness. This was reflected in improved reproduction of the settlement trough shape and more consistent deformation mechanisms under staged excavation.

Sensitivity studies indicate that, for the HS model, the unloading/reloading stiffness ratio (E_{ur}^{ref} / E_{50}^{ref}) has a limited influence on the magnitude of maximum settlement but affects settlement at greater distances from the wall, wall deflection profiles, and excavation-induced base heave. In contrast, the oedometer stiffness ratio (E_{oed}^{ref} / E_{50}^{ref}) exerts a more significant influence on ground settlement magnitude, highlighting the importance of appropriate stiffness selection when modelling volume change and stress redistribution during excavation. It is suggested that sufficient triaxial compression tests and consolidation tests are carried out to determine the E_{50}^{ref}, E_{ur}^{ref} and E_{oed}^{ref} accurately, for a site-specific investigation. In absence of such test data, default settings with E_{ur}^{ref} / E_{50}^{ref} = 3 and E_{oed}^{ref} / E_{50}^{ref} = 1 may be adopted.

From a practical perspective, the study suggests that advanced constitutive modelling, such as the HS model, is particularly justified for movement-sensitive ELS designs where unloading effects are significant, monitoring data are available for calibration, and deformation performance rather than ultimate stability governs the design. For routine applications, the MC model remains suitable for strength-controlled assessments, provided its limitations in deformation prediction are recognised.

The case study also demonstrates the value of monitoring-based back-analysis in reducing uncertainty associated with soil stiffness selection. By calibrating numerical models against high-quality field data, designers can improve confidence in deformation prediction, refine constitutive model parameters, and support rational decision-making in ELS design. This approach aligns with the observational method and reinforces the importance of integrating theory, numerical modelling, and field performance in Hong Kong geotechnical practice.

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5 LIMITATIONS AND SCOPE OF APPLICATION

The findings and observations presented in this paper are based on a project-specific case study involving a deep basement excavation in Hong Kong, supported by comprehensive instrumentation and monitoring data. While the back-analysis provides useful insights into the influence of constitutive soil model selection on predicted ground movements, the results should be interpreted within the context of the ground conditions, excavation geometry, construction sequence, and monitoring quality of the study site.

The numerical analyses were carried out using two-dimensional plane-strain models, which represent an idealisation of the actual three-dimensional excavation and support system. Localised three-dimensional effects, construction imperfections, and spatial variability in ground conditions may not be fully captured. In addition, the soil stiffness parameters adopted in the analyses were calibrated primarily against measured wall deflections and ground settlements for the selected section; different parameter combinations may be required for other sites with different geological profiles, stress histories, or groundwater regimes.

The empirical correlations and parameter relationships discussed in this paper reflect prevailing Hong Kong practice and published local case histories. Their applicability to projects outside Hong Kong, or to ground conditions significantly different from those considered (e.g. thick soft clay deposits, heavily structured soils, or highly anisotropic ground), should therefore be assessed with caution. Designers are advised to carry out project-specific calibration, sensitivity studies, and independent checks when applying advanced constitutive models in excavation and lateral support design.

Notwithstanding these limitations, the study demonstrates the value of monitoring-based back-analysis in improving the understanding of soil stiffness mobilisation and in supporting informed selection of constitutive models for movement-sensitive excavation problems. The approach adopted in this paper is intended to complement, rather than replace, sound engineering judgement, established empirical experience, and observational methods in geotechnical design.

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