Economical Design Approaches for Deep Excavation and Lateral Support (ELS) in Soil with Shallow Bedrock

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ABSTRACT

Deep excavation and lateral support (ELS) are integral components of urban development, particularly in the construction of multi-storey basements. The conventional ELS design employs the Mohr-Coulomb (M-C) Failure Criterion for both soil and rock parameters, even when bedrock is encountered at shallow depth. This may lead to over-conservative design by underestimating the rock mass strength and stiffness in the overall rock-structure interaction analysis. This paper explores an alternative design approach by suggesting the application of the Hoek-Brown (H-B) Failure Criterion for ELS design in the presence of shallow bedrock. While the M-C parameters are well-established for soil, the conservative nature of M-C Failure Criterion for rock parameters derived from rock joints or discontinuities defect shear strength properties can lead to avoidable costs. Additionally, the paper addresses common scenarios involving the design of temporary piled walls socketed into bedrock for deep excavation, which involve the design of groundwater seepage in rock. The paper suggests a more nuanced approach, advocating for an assessment of the likelihood of possible rock joint failures to derive sensible and cost-effective solutions. By reconsidering conventional design criteria and incorporating alternative geotechnical principles while highlighting the importance of rock testing and field observation, the proposed methodologies contribute to more efficient and economically viable deep excavation practices in rock.

1 INTRODUCTION

The most economical method for excavation in rock is open pit excavation with steep gradients, generally 60° to 80°. In such cases, rock support measures such as rock dowels / bolts, rock anchors or rock face dentitions would be required if unfavourable rock discontinuities are observed after geological rock face mapping by experienced geologists or geotechnical engineers. However, open rock pit excavation is not practical for deep basement excavation in the urban areas of Hong Kong. This is because the basements typically occupy the entire site to maximize the usable area leaving no space for steep rock cutting batters. Excavation and lateral support (ELS) for deep basements in urban Hong Kong commonly consists of temporary piled walls with layers of steel strutting. Even though the excavations are carried out in bedrock, the piled wall would still be required to be socketed with a certain embedment into the rock below the excavation levels, and some heavy strutting layers would be propped against the rock face. There seems to be little difference between excavation in "soil" and "rock", although it is widely accepted that "rock" should have significantly better engineering properties and may even be self-supporting safely.

In other words, the current design practices for ELS works in rock are generally over-conservative. This paper reviews the current design practices and tries to identify the processes leading to conservative outcomes. By understanding the properties, behaviours and potential failure mechanisms of rock, economical design approaches for deep ELS system with shallow bedrock can be applied to offer potential opportunities for cost-effective and safe design.

2 OVERVIEW ON MODELLING APPROACH FOR DEEP EXCAVATION DESIGN

In Hong Kong, limit state design is commonly adopted in the design of ELS works, which must satisfy the fundamental requirements of stability and serviceability (GEO Pub. 1/2023, 2023). To meet both limit state design requirements, especially in complex site setting, it is preferred to utilize numerical modelling to consider soil-

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structure interaction and excavation sequence. Numerical methods, including the boundary element method introduced in FREW and the finite element method adopted in PLAXIS 2D, are widely used in the design of ELS works. In both numerical methods, the fundamental Mohr-Coulomb (M-C) Failure Criterion is commonly adopted in local practice to determine the soil shear strength parameters of the principal material types encountered at depth. This criterion is well known as a linear function of shear strength parameters, including cohesion intercept (c') and angle of shearing resistance (ϕ'). The c'-ϕ' values of the soil strata are determined from consolidated undrained (CU) triaxial tests. The typical ranges of c'-ϕ' values for selected Hong Kong soils are given in Table 8 of Geoguide 1 (GEO, 2020).

In addition to shear strength parameters, the stress-strain behaviour, which can be expressed by the modulus of deformation of soil or Young's modulus, needs to be incorporated into the soil stiffness matrix for the calculation of ground movement. In prevailing practice, the Young's modulus is correlated with in-situ test results, such as standard penetration test (SPT) 'N' value (GEO Pub. 1/2023, 2023). By establishing the fundamental failure criterion and stress-strain relationship in the soil model, we can realistically model the ground conditions, describe soil-structure interaction interfaces in numerical analysis, and obtain subsequent estimations of geotechnical and structural results of the ELS system. These results include soil stress and ground movement from the geotechnical aspect, as well as deflection, shear force and bending moment of structural components such as piled walls and strut members.

The current design practices for ELS systems in rock assume that rock possess physical properties of "soil". In other words, the c'-ϕ' values for the rock are taken as the shear strength of the rock joints, which can be obtained from the results of direct shear box tests. If no direct shear box test is carried out, it may be considered to adopt the upper bound of the typical ranges of completely decomposed rock, such as:

- For granite bedrock: c'= 15kPa and ϕ' = 44°
- For volcanic bedrock: c'= 10kPa and ϕ' = 38°

For Young's modulus (E_s') of rock, it is assumed that SPT'N is greater than 200 blows, and thus the E value is empirically assumed to be twice the SPT'N value, i.e., greater than 400MPa. Typically, E_s' is proposed to be 400MPa for design purpose.

For the design groundwater pressure, hydrostatic pressure is adopted, with groundwater continuing to increase with depth from soil to rock. However, it should be noted that in the majority of rock excavation portions, grout curtain, lagging plates or shotcrete are not required between the piles. Therefore, the groundwater pressure should be relieved by seeping groundwater through persisting rock joints exposed between the piles. For the design of the required rock socket for the ELS piled wall, two failure modes should be considered. Firstly, the rock socket should be checked against the bearing failure in accordance with Figures 51 to 53 of Geoguide 1 (GEO, 2020) for different directions of bending moment and shear force at the top of rock socket. Secondly, the rock socket should be designed against planar discontinuity-controlled failure based on Figure 54 of Geoguide 1 (GEO, 2020). The variations of rock socket capacity are calculated with various dip angles of discontinuity and thus the calculated minimum ultimate lateral resistance should be greater than the required lateral resistance from numerical models, i.e., ULS bending moment and shear force from FREW or PLAXIS 2D. The latter checking mode often leads to much deeper rock sockets, which would require similar embedment as determined by kickout checking using K_p method.

3 MATERIAL MODELLING OF ROCK MASS

In consideration of the shear strength parameters of rock, it can be classified as discontinuity shear strength or rock mass shear strength based on scale effects and geological conditions, as represented in Figure 1. The relationship between shear stress and effective normal stress varies for rock with different characteristics of discontinuities and rock strength. As presented in Figure 1, there are shear strength properties for "Discontinuities" and "Rock Masses". The shear strength properties for "Discontinuities" refer to the properties of individual joints / joint sets in the rock mass, which are related to the joint's properties and can be determined from direct shear box tests of rock joint specimens. The shear strength properties for "Rock Masses" refer to the properties of the overall rock structure, which take into account of intact rock strength obtained from the Unconfined Compression Strength (UCS) Test, and Geotechnical Strength Index (GSI), concentrating on the description of two rock characteristics: rock structure and block surface conditions.

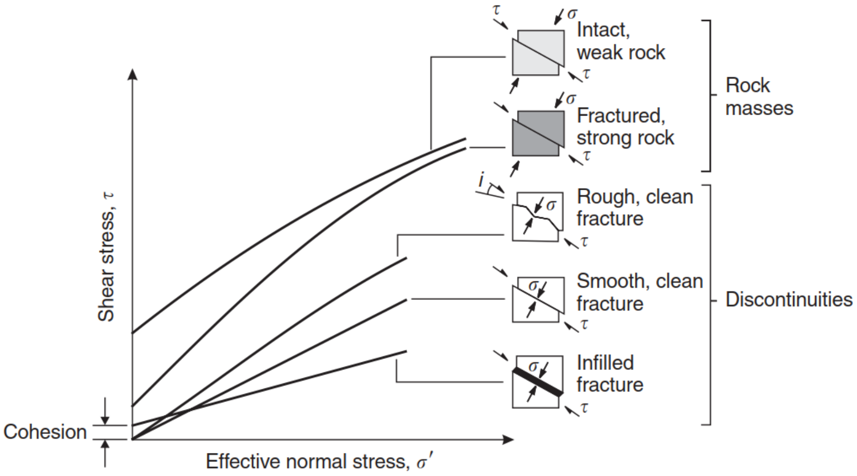


Figure 1: Shear Strength Parameters of Rock (Wyllie and Mah, 2004)

Depending on the different checking scenarios, designers should determine which set of rock shear strength parameters should be adopted for the design. For example, when designing of rock plane or wedge failure at localized areas, the shear strength properties of "Discontinuities" should be used. While working on numerical analysis, the rock strata should be modelled with the shear strength properties of "Rock Masses".

The Hoek-Brown (H-B) Failure Criterion is a widely used rock mass strength criterion that provides a method for estimating the strength of rock masses and predicting their behaviour under various loading conditions. Developed by Evert Hoek and E. T. Brown in the 1980s, this criterion has become a fundamental tool in geotechnical engineering for assessing the stability of rock slopes, tunnels, and mining works globally. The original H-B Criterion has a major limitation in handling some geotechnical problems, especially for slope stability and underground excavation design, since it is defined in the terms of principal stresses rather than shear and normal stress, as shown in the following equation:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m\frac{\sigma_3'}{\sigma_{ci}} + s\right)^{0.5} \qquad (1)$$

where:

- $\sigma'_1$and σ'_3 are the major and minor effective principal stresses at failure;
- σ'_{ci} is the uniaxial compressive strength of the intact rock material;
- m and s are material constants, where s = 1 for intact rock.

In contemplation of the limitation of H-B Criterion, Hoek endeavoured to provide a comprehensive presentation of the H-B Criterion within the framework of M-C Failure Criterion, and the generalized H-B Criterion was developed. In the generalized H-B Criterion, the shape of the principal stress plot or the Mohr envelope could be adjusted by replacing the square root term in Eq. (1) with a variable coefficient, resulting in the following equation:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b\frac{\sigma_3'}{\sigma_{ci}} + s\right)^{a} \qquad (2)$$

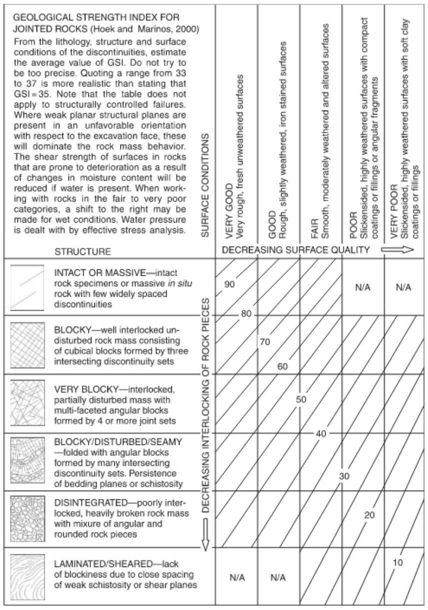
On the other hand, the introduction of the material constant (m_i), Geological Strength Index (GSI) and the degree of Disturbance (D) to which the rock mass subjected by stress change due to any activities such as blast damage and stress relaxation completes the expression of Generalized H-B criterion by quantifying the value of material constants and the variable coefficient (i.e., m_b, s and a) and given by the following relationships:

$$m_b = m_i \, e^{\frac{GSI-100}{28-14D}} \qquad (3)$$

$$s = e^{\frac{GSI-100}{9-3D}} \qquad (4)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{GSI}{15}} - e^{-\frac{20}{3}}\right) \qquad (5)$$

The values of GSI, m_i and D are generally empirical and subject to the rock particle interlocking condition, types and texture of intact rock, and the disturbance to the rock mass itself due to the nature of geotechnical works involved. The Disturbance Degree Factor (D) for normal ELS rock excavation by mechanical method is 0.7 in according to Hoek, Carranza-Torres & Corkum, 2002. The suggested values for the GSI, m_i and D are shown in the tables of Figure 2. Apart from the GSI chart, several literatures suggest different methodologies for determining the value of GSI based on well-established systems such as the rock mass rating system (RMR) and Q-system (Ágnes, Kovács, Somodi & Vasarhelyi, 2016) and (Hussian et al., 2020).



GEOLOGICAL STRENGTH INDEX FOR JOINTED ROCKS (Hoek and Marinos, 2000)

From the lithology, structure and surface conditions of the discontinuities, estimate the average value of GSI. Do not try to be too precise. Quoting a range from 33 to 37 is more realistic than stating that GSI = 35. Note that the table does not apply to structurally controlled failures. Where weak planar structural planes are present in an unfavorable orientation with respect to the excavation face, these will dominate the rock mass behavior. The shear strength of surfaces in rocks that are prone to deterioration as a result of changes in moisture content will be reduced if water is present. When working with rocks in the fair to very poor categories, a shift to the right may be made for wet conditions. Water pressure is dealt with by effective stress analysis.

Rock type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerates (21±3) Breccias (19±5)	Sandstones 17±4	Siltstones 7±2 Greywackes (18±3)	Claystones 4±2 Shales (6±2) Marls (7±2)
	Non-Clastic	Carbonates	Crystalline Limestone (12±3)	Sparitic Limestones (10±2)	Micritic Limestones (9±2)	Dolomites (9±3)
		Evaporites		Gypsum 8±2	Anhydrite 12±2	
		Organic				Chalk 7±2
METAMORPHIC	Non foliated		Marble 9±3	Hornfels (19±4) Metasandstone (19±3)	Quartzites 20±3	
	Slightly foliated		Migmatite (29±3)	Amphibolites 26±6	Gneiss 28±5	
	Foliated*			Schists 12±3	Phyllites (7±3)	Slates 7±4
IGNEOUS	Plutonic	Light	Granite 32±3 Granodiorite (29±3)	Diorite 25±5		
		Dark	Gabbro 27±3 Norite 20±5	Dolerite (16±5)		
	Hypabyssal		Porphyries (20±5)		Diabase (15±5)	Peridotite (25±5)
	Volcanic	Lava		Rhyolite (25±5) Andesite 25±5	Dacite (25±3) Basalt (25±5)	Obsidian (19±3)
		Pyroclastic	Agglomerate (19±3)	Breccia (19±5)	Tuff (13±5)	

* These values are for intact rock specimens tested normal to bedding or foliation. The value of m_i will be significantly different if failure occurs along a weakness plane.

Appearance of rock mass	Description of rock mass	Suggested value of *D*
	Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel.	*D* = 0
	Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass. Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed.	*D* = 0 *D* = 0.5 No invert
	Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass.	*D* = 0.8
	Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance.	*D* = 0.7 Good blasting *D* = 1.0 Poor blasting
	Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal. In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less.	*D* = 1.0 Production blasting *D* = 0.7 Mechanical excavation

Figure 2: GSI chart for jointed rock (Hoek & Marinos, 2000) (Left), Table of values of constant m_i for intact rock by rock group (Wyllie & Mah, 2004) (Middle), Guiding table for estimating disturbance factor D (Hoek, Carranza-Torres & Corkum, 2002) (Right)

After introducing the Generalized H-B Failure Criterion, the next step is to derive the equivalent friction angle and cohesion of rock mass and stress range. By fitting an average linear relationship with the curve generated by solving Eq. (2) for a range of minor principal stress values defined by $\sigma_t < \sigma_3 < \sigma'_{3max}$, as shown in Figure 3, the value of friction angle can be expressed in the H-B Failure Criterion, and the relationship is presented in the following equations and figure (Hoek, Carranza-Torres & Corkum, 2002):

Substitute the values from Eq (3) to Eq (5) to Eq (6) then Eq (7).

$$\sigma'_{cm} = \frac{\sigma'_{ci}\left(m_b+4s-a(m_b-8s)\right)\left(\frac{m_b}{4}+s\right)^{a-1}}{2(1+a)(2+a)} \qquad (6)$$

$$\sigma_{3n} = \sigma'_{3max}/\sigma_{ci} = \sigma'_{cm} \times 0.72\left(\frac{\sigma'_{cm}}{\gamma H}\right)^{-0.91}\Big/\sigma_{ci} \qquad (7)$$

Then, substitute the values from Eq (3) to Eq (5) and Eq (7) to Eq (8) and Eq (9).

$$\varphi' = \sin^{-1}\left[\frac{6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}}{2(1+a)(2+a)+6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}}\right] \qquad (8)$$

$$c' = \frac{\sigma_{ci}\left[(1+2a)s+(1-a)m_b\sigma'_{3n}\right]\left(s+m_b\sigma'_{3n}\right)^{a-1}}{(1+a)(2+a)\sqrt{1+\frac{6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}}{(1+a)(2+a)}}} \qquad (9)$$

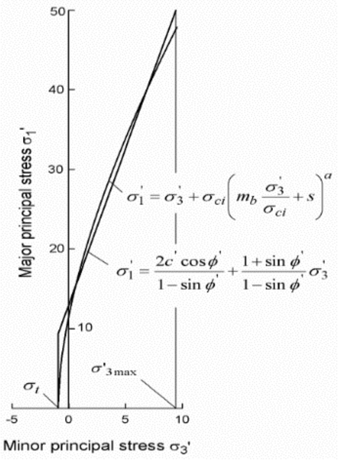


Figure 3: Relationship between major and minor principal stresses for H-B and equivalent M-C criteria

Apart from the rock mass shear strength parameters, the rock mass modulus of deformation, E_m in (GPa) can also be derived from σ'_{ci}, D and GSI. This is given by the following equations when σ'_{ci} is smaller or equal to 100MPa and larger than 100MPa respectively:

$$E_m = \left(1-\frac{D}{2}\right)\sqrt{\frac{\sigma'_{ci}}{100}}\cdot 10^{\frac{GSI-10}{40}} \quad \text{where } \sigma'_{ci} \leq 100\text{MPa} \qquad (10)$$

$$E_m = \left(1-\frac{D}{2}\right)\cdot 10^{\frac{GSI-10}{40}} \quad \text{where } \sigma'_{ci} > 100\text{MPa} \qquad (11)$$

Alternatively, the rock mass deformation parameter can be determined using either the equations in Section 5.9 of Geoguide 1 (GEO, 2020) or Table 10 (Section 1 for Granite Rocks) and Table 7 (Section 2 for Volcanic Rocks) of GEO Report No. 8 (GEO, 1995).

4 DESIGN GROUNDWATER PRESSURE IN ROCK EXCAVATION

Hong Kong is a wet city with heavy rainstorms in the summer season. The groundwater table is generally high, especially along the shorelines and low-lying areas. Therefore, the design groundwater level is critical for the ELS design. It is essential for geotechnical engineers to confirm the groundwater regime and subsequent effect on groundwater level due to dewatering during the deep excavation. In common practice, ELS designers would consider rock strata as low permeability materials. Hence, water cut-off barrier such as grout curtains would be embedded a nominal 1m to 2m into the rockhead to cater to rock fracture zones at the soil-rock interface. Then, the groundwater pressure for ELS kickout checking and numerical analysis is assumed as hydrostatic pressure from the design groundwater level to the toe of the piled wall.

However, when there is no impermeable water cut-off throughout the excavated rock face, such as pre-bored sheetpile, interlocking pipe pile or steel / shotcrete lagging, the groundwater would be able to flow through the rock fissures / rock joints, leading to a deviation in the assumed groundwater pressure distribution. Such seepage through rock mass would relieve the groundwater pressure and reduce the loading to the ELS system. With sufficient ground investigation (G.I.) data, such as in-situ mass permeability parameters for both soil and rock strata, it is suggested to conduct a numerical seepage analysis under steady-state seepage conditions. The groundwater pressure acting at the back the ELS piled wall can be obtained for ultimate limit state (ULS) design and while the changes of groundwater table inducing ground settlement can also be obtained for serviceability limit state (SLS) design.

It is common for ELS designers to assign in-situ soil permeability tests using either the falling head or constant head method. However, rock permeability parameters must also be ascertained to ensure a realistic groundwater pressure distribution under seepage for robust ULS and SLS design of the ELS system. Rock permeability tests like the Lugeon test or Packer test are generally recommended for obtaining the in-situ averaged hydraulic permeability value of rock mass for ELS design.

In the following example, the design groundwater pressure distribution acting on the ELS piled wall was determined by conducting a numerical seepage analysis with the aid of SEEP/W. In this hypothetical seepage analysis, it was assumed that the deep excavation would be 24m below the existing ground with the level of +4mPD and grout curtain, by means of TAM and / or rock fissure grouting, from existing ground level to 5m below rockhead level, was provided as water cut-off barrier. The design groundwater table, mass permeability parameters and geological profile adopted are shown in Figure 4.

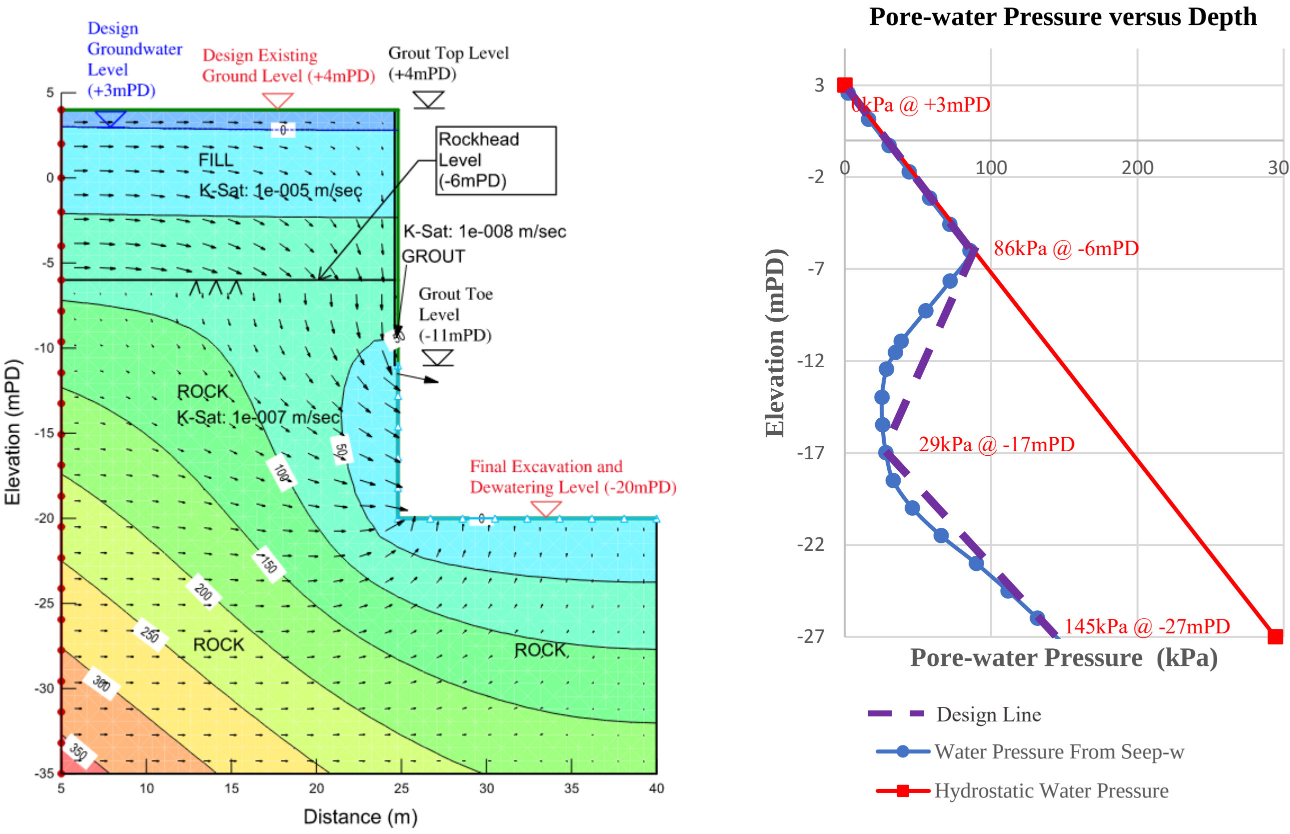


Figure 4: Numerical seepage analysis (Left) and groundwater pressure distribution on the active side (Right)

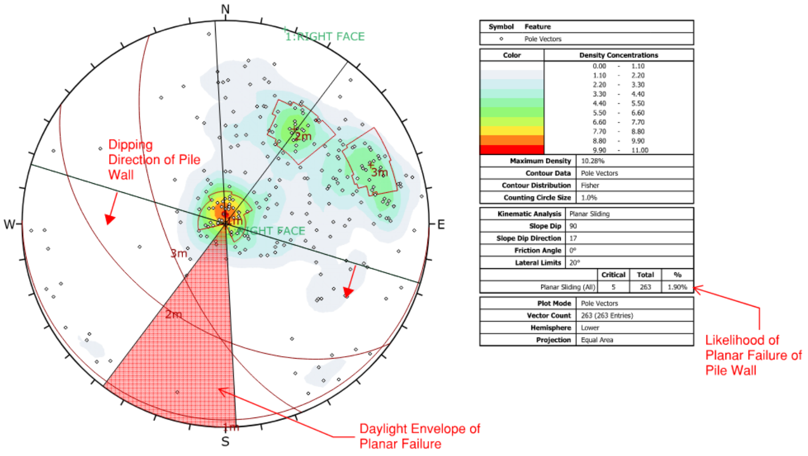
The results of seepage analysis revealed that the groundwater pressure behind the piled wall on the active side was generally hydrostatic in fill layer from the design groundwater table to the rockhead. Below the rockhead level, the groundwater pressure decreased and was much lower than the hydrostatic pressure profile due to the pressure relief through groundwater seepage. The linear approximation of the design groundwater pressure profile as presented in the purple dotted line in Figure 4 could be applied for stability checking for ULS design such as toe stability checking and wall structural capacity checking as well as for SLS design of wall deflection and induced ground settlement.

Based on the findings from the above example, there is a significant reduction in the design groundwater pressure in the rock strata if the soil and rock permeability parameters are obtained from the G.I. field tests and then simulated by seepage analysis.

5 ROCK SOCKET DESIGN

For the design of an ELS piled wall system, the overall stability checking against overturning failure is a basic procedure to determine the required pile toe embedment, i.e., rock socket design. As discussed in Section 2 above, there are two possible failure modes for the design of a rock socket which are bearing failure of intact rock and planar discontinuity-controlled failure of rock mass. In the absence of sufficient rock joints data, all rock discontinuity combinations should be checked to obtain the lowest ultimate lateral resistance of the proposed rock socket length. However, assuming that the rock mass has the most adverse joint in the stability checking may lead to over-conservative and unrealistic rock socketed required for the piled wall. It is suggested that the rock joint data can be measured using borehole acoustic televiewer (ATV) surveys. If there is no potential rock joint planar failure sets against rock socket kickout, the rock socket would then be controlled by the bearing failure mode, resulting in a much shorter length. Even if there are potential rock joint sets causing planar failure of rock socket kickout, the dip angles and dip directions can be identified for checking against the planar discontinuity-controlled failure by applying the equations in Figure 54 of Geoguide 1 (GEO, 2020). In this case, the rock socket length would still be generally shorter by confining the dip angles. Additionally, such potential adverse rock joint set only affects one side of the excavation face and the remaining excavation faces can still undergo the bearing failure mode checking only. Thus, the rock socket design would be more economical and specific based on the site-specified G.I. information. This approach significantly saves time and costs by avoiding the installation of unreasonably deep pile wall into bedrock.

In the following sample of stereo plot analysis, the approach for finding the adversely oriented joint set(s) is demonstrated. Firstly, relevant rock joint data, including dip angle and dip direction at levels within the influence zone of planar failure (i.e., from the final excavation level to the design toe level of the piled wall), are selected. With the aid of computer-aided stereographic projection software (e.g., DIPS developed by Rocscience) a variety of poles representing the plane orientation of each rock joint are plotted in the stereonet as shown in Figure 5. Additionally, the daylight envelope for planar failure against rock socket kickout is plotted with the known dip direction of the piled wall.



Dipping Direction of Pile Wall

Daylight Envelope of Planar Failure

Likelihood of Planar Failure of Pile Wall

Symbol	Feature
○	Pole Vectors

Color	Density Concentrations
⬜	0.00 - 1.10
⬜	1.10 - 2.20
🟦	2.20 - 3.30
🟦	3.30 - 4.40
🟩	4.40 - 5.50
🟩	5.50 - 6.60
🟩	6.60 - 7.70
🟨	7.70 - 8.80
🟧	8.80 - 9.90
🟥	9.90 - 11.00

Maximum Density	10.28%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Planar Sliding
Slope Dip	90
Slope Dip Direction	17
Friction Angle	0°
Lateral Limits	20°

	Critical	Total	%
Planar Sliding (All)	5	263	1.90%

Plot Mode	Pole Vectors
Vector Count	263 (263 Entries)
Hemisphere	Lower
Projection	Equal Area

Figure 5: Kinematic analysis of rock discontinuities as rock socket of pile wall

When a high concentration of poles falls within the daylight envelope for planar failure, it indicates that there is a high likelihood of those rock joint sets, with their range of dip angles in their corresponding dip direction, experiencing planar failure under external load. Conversely, if the rock joint sets lay outside the planar failure zone, it is concluded that no planar discontinuity failure will occur under lateral load for the retaining wall. Therefore, we are confident in designing the rock socket checking against the bearing failure of intact rock.

6 CASE STUDIES

6.1 Case Study 1 – ELS design with shallow rockhead under M-C and H-B Failure Criterions

The site was located at the toe of the hillside with existing ground level varying from +5.0mPD to +8.7mPD. The site-specific G.I. results indicated that the site was underlaid with fill followed by moderately/slightly decomposed granite (M/SDG). The engineering rockhead level varied from +2.0mPD to -5.0mPD, while the final excavation level was -23.75mPD, resulting in a 32.5m deep excavation, including approximately 25.8m of deep rock excavation. The groundwater table of the site varied from +1.3mPD at its lowest and +3.0mPD at its highest.

In the initial design, the analyses of ELS for ULS and SLS design were based on a constitutive soil model which relates linear stress to strain relationship controlled by the M-C Failure Criterion of rock joints. The design geotechnical parameters were assumed as follows:

- Fill: c' = 0kPa, ϕ' = 30° and Es =110MPa
- M/SDG: c' = 10kPa, ϕ' = 40° and Es = 250MPa
- Design groundwater pressure adopted from Seep-W results

To satisfy the ULS and SLS design requirements, the ELS system required a 323mm diameter pipe pile wall at 500mm centre-to-centre spacing with 8 layers of steel strut propping and heavy preloading on the bottom 6 layers of shoring support, with at least 4 layers to be installed below the bedrock.

Design optimization was carried out to achieve cost saving design by adopting M-C fit values derived from the H-B Failure Criterion for rock mass. Based on the rock core obtained from the boreholes and laboratory results of unconfined compressive strength (UCS) tests, the adopted values for the inputs are listed out in the Table 1 and with the steps of calculation to obtain the rock strength and stiffness parameters are shown in Figure 6.

Table 1: Summary table of adopted input parameters for H-B Failure Criterion

Input Parameters	Descriptions	Adopted Values
GSI	• Based on GI records, the encountered rock mass was intact with minor fracture • The rock cores were generally fully recovered (i.e. TCR=100% and RQD=100%) • The fracture index was low (i.e. generally less than 1.0) • From Figure 2 (Left), range of GSI was considered from 60 to 90 for intact to blocky rock with good to very good surface conditions	55
σ'_{ci}	• From laboratory testing results, average intact uniaxial compressive strength (UCS) = 100MPa	75MPa
m_i	• From Figure 2 (Middle), the material constant was range from 32 +/-3 for granite rock	32
D	• From Figure 2 (Right), the disturbance factor was 0.7 for mechanical excavation of open pit	0.7
H	• Average excavation depth	30m
γ	• Typical unit weight for granite rock	26kN/m^3

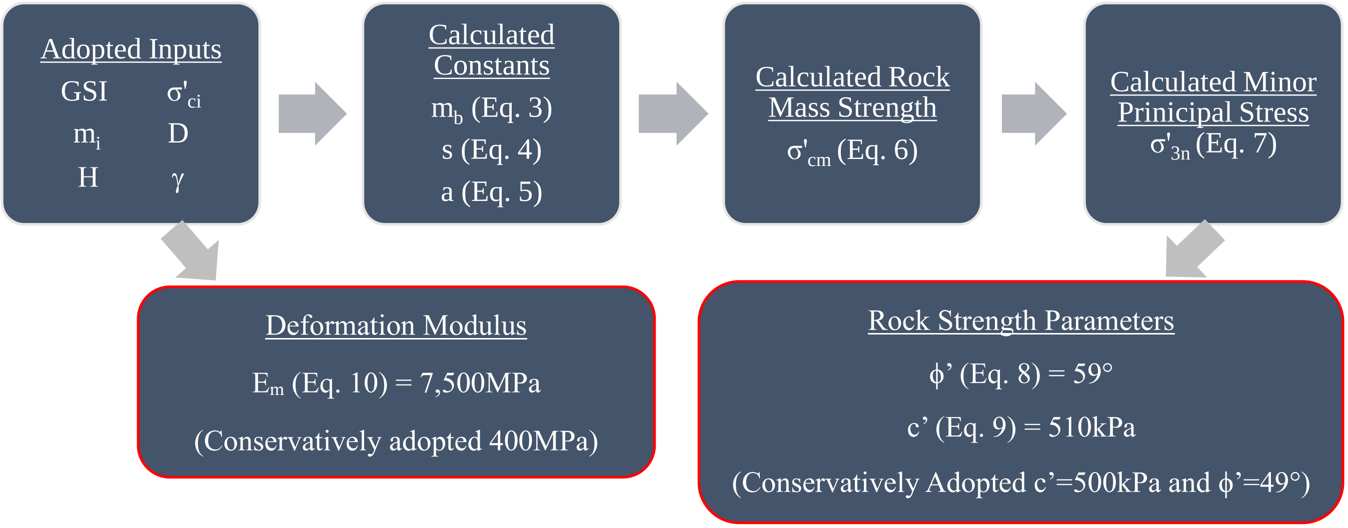


Figure 6: Flow chart for calculation of equivalent M-C parmeters based on H-B failure criterion

Even though conservative design parameters for M/SDG were adopted, significant improvements were achieved in the design, resulting in significant time and cost savings. The number of shoring layers could be reduced to 6 layers and all preloading works were no longer necessary while obtaining a satisfying wall movement. By the time of cost saving design, the pipe pile installation had been completed and so no optimization on the proposed piled wall. The comparison of the initial design and cost saving design is presented in Figure 7.

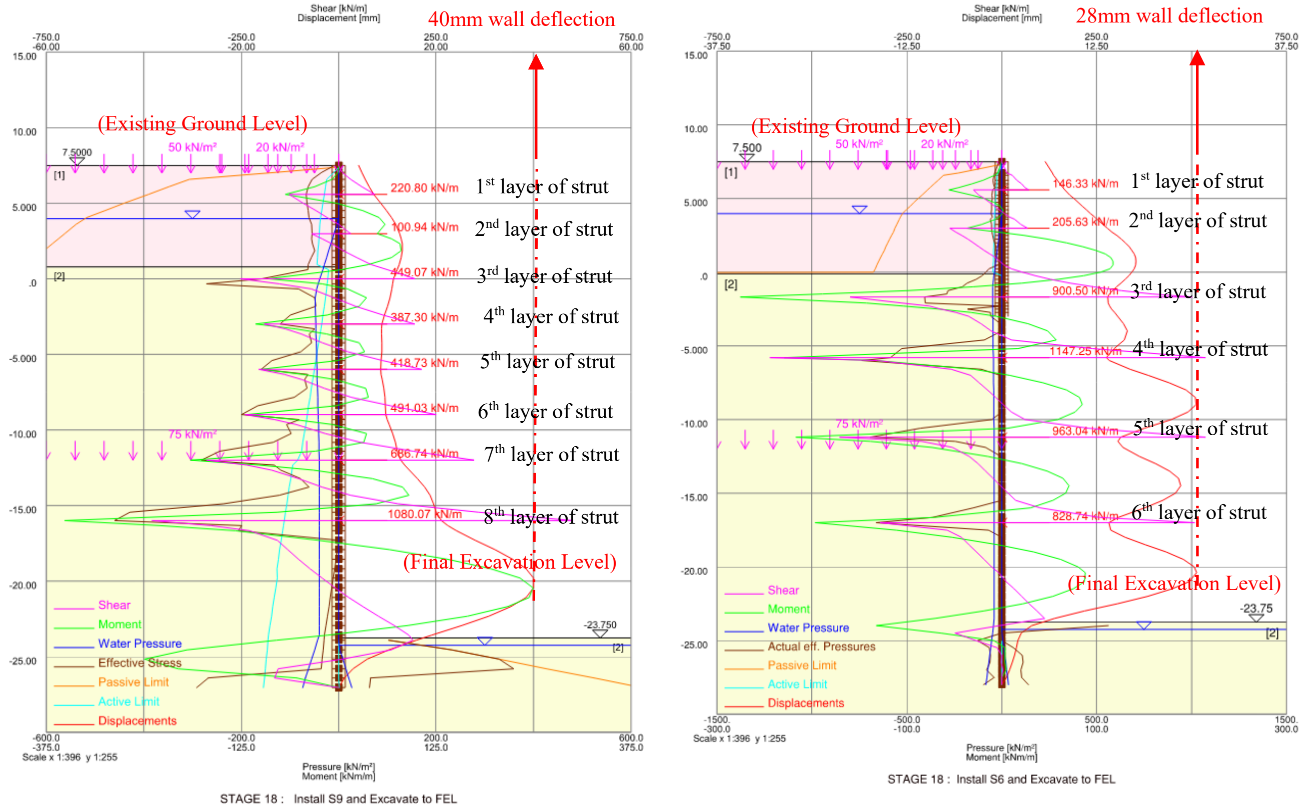


Figure 7: Oasys FREW model of initial design adopted M-C Criterion (Left) and cost saving design adopted H-B Criterion (Right)

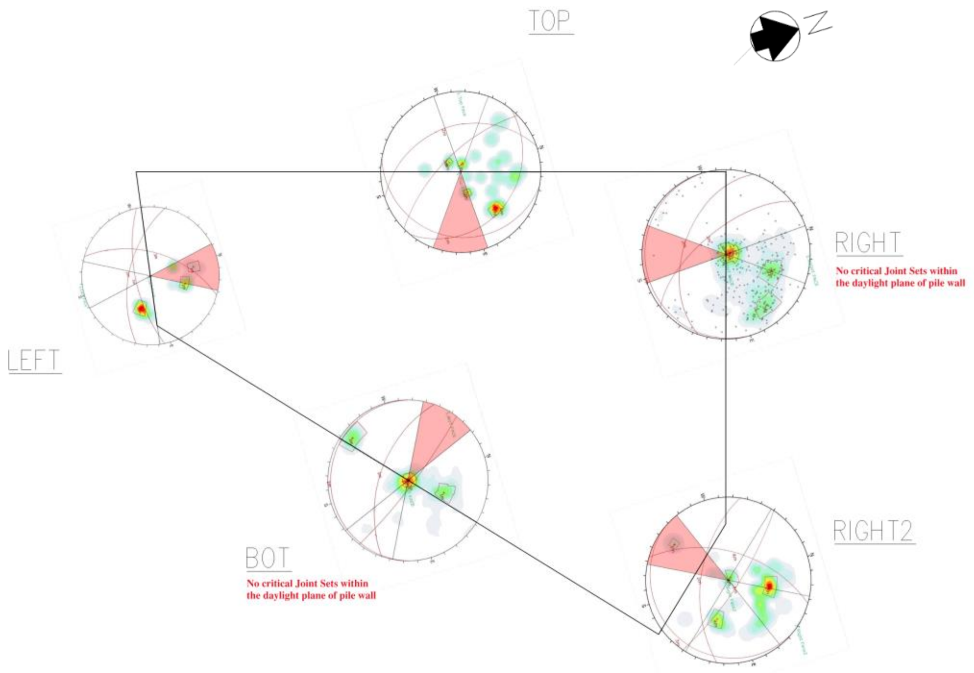
6.2 Case Study 2 – Rock socket design with review of rock joint conditions

The site was located in an urban area at Tsim Sha Tsui, Hong Kong with an existing ground level of approximately +4.5mPD. According to the site-specific G.I. information, the site was underlain by a vertical sequence of fill or alluvium, completely decomposed granite (CDG), highly decomposed granite (HDG) and underlain by moderately/slightly decomposed granite (M/SDG). The engineering rockhead level varied from -18mPD to -25mPD. The proposed final excavation level varied from -22 to -25mPD, with a maximum depth of excavation of 29.5m. A multi-strutted interlock pipe piled wall embedded in rock was adopted as the temporary ELS wall for deep excavation to construct the underground permanent basement structure.

A PLAXIS 2D model using the global safety factor method was employed to model the construction sequences of proposed excavation and lateral support works. The bending moment and shear force of the wall could be extracted from the model for the design of the rock socket. With reference to Figures 51 to 54 of Geoguide 1 (GEO, 2020), the estimated embedment length of piled wall into bedrock was about 5m in general, while the checking of planar discontinuity-controlled failure dominated the overall design of rock socket length.

A comprehensive design review was carried out to minimize the construction cost and time for ELS works. One of the solutions was to reduce length of the rock socket by conducting a detailed study on the discontinuities condition of the rock mass to provide lateral resistance to the pile. By compiling a considerably large sample size of dip angle and direction of rock joints from the acoustic televiewer (ATV) surveys, more confidence could be gained in the rock joint conditions around the site. In general, it is recommended including ATV in 25% of total proposed G.I. boreholes which are assigned evenly along the excavation face near the rock socketed piled wall.

According to Figure 8, the joint sets with high concentration were sorted out into generally five directions of the site. Among these, only joint sets at the top, left and bottom-right of the site fell within the zones of planar failure of the rock socket kickout. The concerned joint sets (highlighted in red in the table below) with specific dip angles and directions were checked against failure due to planar discontinuities. For all three critical joint sets, the dip angles were relatively large, resulting in a high ultimate lateral resistance supporting the piled wall under lateral load. As a result, the design rock socket length was halved to 2.5m, which was controlled by the bearing failure of intact rock.



Face	Right	
Joint Set No.	Dip Angle	Dip Direction
1	4	181
2	48	216
3	66	248
Face	**Right 2**	
Joint Set No.	Dip Angle	Dip Direction
1	76	50
2	53	205
3	5	147
4	52	303
Face	**Bot**	
Joint Set No.	Dip Angle	Dip Direction
1	51	216
2	2	197
3	82	54
Face	**Left**	
Joint Set No.	Dip Angle	Dip Direction
1	65	185
2	55	209
3	52	303
Face	**Top**	
Joint Set No.	Dip Angle	Dip Direction
1	12	117
2	30	268
3	64	242
4	21	56

Figure 8: Plan showing rock joints at each excavated face of piled wall (Left) and summary table of joint sets (Right)

6.1 Case Study 3 – Deep excavation in Sedimentary Rock

It is uncommon to encounter deep Sedimentary Rock excavation in Hong Kong. The site for this case study was located in Central Business District (CBD) of Sydney which involved deep shaft excavation in Sandstone and adjacent to high-rise buildings. The rock mass shear strength parameters were developed by H-B Failure Criterion as listed in Table 2. The PLAXIS 2D analysis inputs and outputs are presented in Figure 9.

Table 2: Shear strength rock mass parameters for Sandstone

Material	**Unit Weight, γ (kN/m^3)**	**Cohesion, c' (kPa)**	**Friction Angle, ϕ'(deg)**	**Young's Modulus, E (kPa)**	**Poisson's Ratio**	**Principle Stress (MPa)**
Sandstone (Class III)	24	300	36	1,000	0.3	Major: $\sigma_H = 2\sigma_v$ Minor: $\sigma_H = 1.2\sigma_v$
Sandstone (Class II)	24	500	36	2,000	0.2	Major: $\sigma_H = 2\sigma_v + 2.5$ Minor: $\sigma_H = 1.4\sigma_v + 1.75$
Sandstone (Class I)	24	700	36	3,000	0.2	

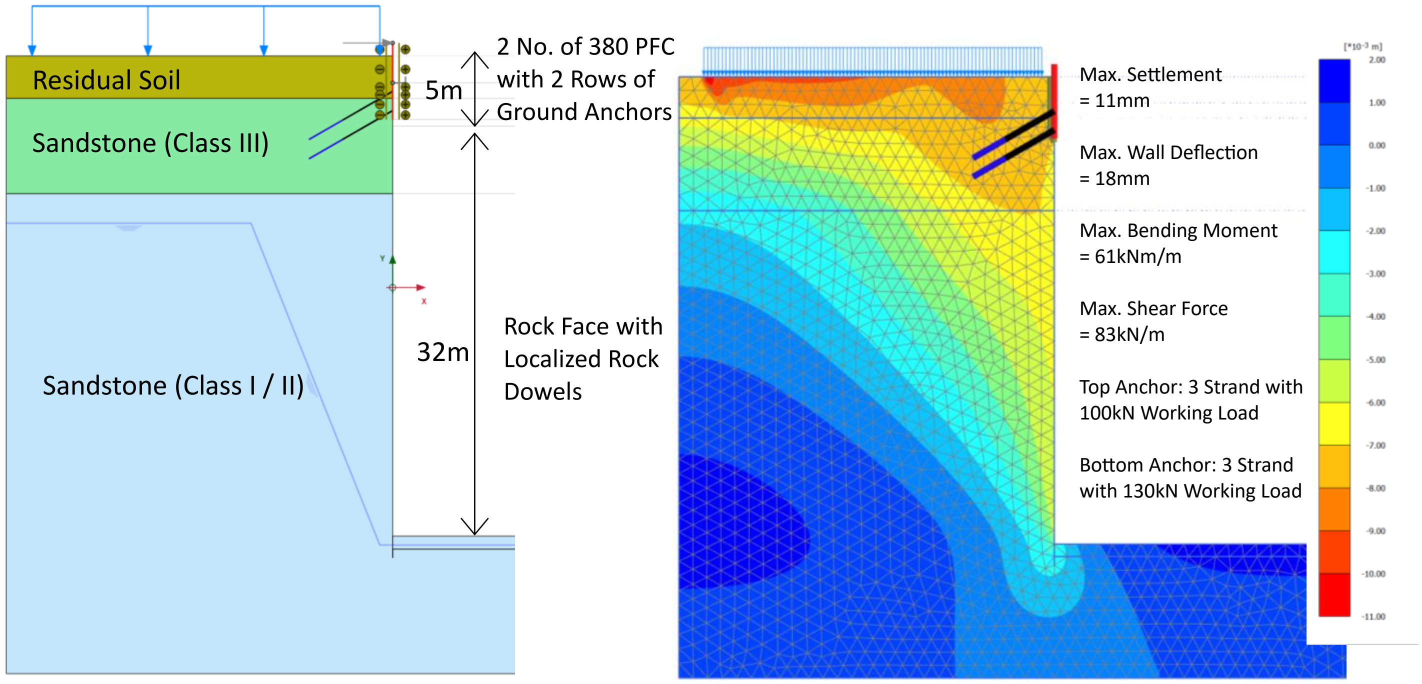


Figure 9: PLAXIS 2D Input of Deep Excavation in Sandstone (Left) and PLAXIS 2D Outputs (Right)

7 CONCLUSION AND RECOMMENDATION

The purpose of this paper is to explore more economical design approaches in the ELS design with deep excavation and shallow bedrock. Three main ideas are discussed in the paper:

1. Determination of fundamental material strength and stiffness parameters of rock mass based on H-B Failure Criterion;
2. Design groundwater pressure in rock excavation by considering the free seepage from excavated rock surface; and
3. Rock socket design with the rock joint survey information

It is emphasized that the implementation of suitable material modelling for rock is crucial in the overall ELS design in terms of safety and project finance. Providing unreasonable stiff and heavy shoring members with large preloading based on over-conservative assumptions of low rock strength parameters can be costly and time-consuming and sometimes dangerous. Moreover, it is revealed that the design of groundwater pressure and rock socket can be optimized by understanding the rock joint conditions in terms of their quantities, dip angles and directions. However, the design approach heavily relies on field tests and examinations, as well as laboratory tests on the rock samples. It is recommended that sufficient quantities and types of rock tests and surveys shall be assigned for obtaining realistic and proper strength and stiffness parameters, including but not limited to the following:

1. Lugeon test or Packer test in rock mass;
2. Acoustic or optical televiewer survey;
3. Direct shear box test of rock joint samples; and
4. Unconfined compressive strength test and point load test for intact rock samples.

Apart from the field and laboratory testing for the rock mass in the design stage, it is essential to carry out geological mapping, including rock joints, weathering / fracture zones, adverse features and groundwater seepage from rock joints, of the excavated rock faces during the construction stage. All mapping information should be continuously reviewed by the designers to verify and confirm the design assumptions. If required, some remedial measures, such as local rock dowels / bolts, additional strutting, and raking drains, should be applied. It is strongly believed that when designers progressively pay more attention to the importance of testing and observation of rock mass and discontinuity in the planning stage of G.I. works, it would greatly benefit the design and construction of ELS works when shallow rockhead is encountered. The construction time and project cost could be significantly reduced, allowing a more efficiently use of resources without compromising the robustness and safety of the ELS system.

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