# Practical considerations for the appropriate use of finite element analysis in geotechnical design

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ABSTRACT: The current state of practice in Singapore for the design of geotechnical projects can be described with the routine usage of 2D and 3D finite element analyses, coupled with the well-known Mohr-Coulomb material model. The bi-linear Mohr-Coulomb model is simple to use as the parameters are easy to understand and can be easily obtained from conventional laboratory and in-situ tests. While the Mohr-Coulomb failure criterion is safe and appropriate for Ultimate Limit State design, the great deficiency of the Mohr-Coulomb model is that it is unable to predict correctly the strains and deformation of the ground. It is counterproductive to use a powerful tool such as the finite element method with simplistic constitutive models that cannot produce high levels of confidence and accuracy in the estimation of ground movements. With advanced constitutive soil models now widely available, more characteristics of soil behavior can be captured, namely the correct in-situ stresses in the initial state, nonlinear stress and strain dependent behavior, as well as distinguishing the various modes of plasticity in the soil. This paper will illustrate some of these considerations in what is to be considered as a good finite element analysis for design to control ground displacements in construction.

## 1 INTRODUCTION

With the rapid advances in computing technology and capabilities, as well as the wide availability of commercial finite element packages, the use of 2D and 3D finite element analysis has been progressively established worldwide as the standard tool for the design of geotechnical projects over the recent half-century. This phenomenon arose from the need for engineers to provide accurate predictions of soil deformation behavior for project concerns such as protection of sensitive structures adjacent to excavations, limiting ground subsidence due to tunneling or to ensure the stability of slopes. A full numerical analysis will be able to deliver accurate results by satisfying all the conditions of a theoretical solution while 1D limit equilibrium software or spreadsheet calculations fail to satisfy the compatibility, constitutive behavior and boundary condition requirements (Potts, 2003). A well calibrated finite element model that is constructed by an experienced geotechnical numerical analyst from rigorous data interpretation of project specific site investigations can greatly ease construction costs while still having a safe design by accurately predicting the performance of the project. However, the over-reliance on finite element methods (FEM) without good understanding of soil mechanics and numerical analyses may lead to design errors, with potential catastrophic consequences.

In the current state of practice in Singapore, the linear elastic perfectly plastic Mohr-Coulomb (MC) model is the most commonly used constitutive soil model due to its ease of usage. The parameters of the model are easy to understand, and can be easily determined from common laboratory and field tests. The frequent usage of the MC model has been further reinforced by the great wealth of project experiences accumulated from the use of the bilinear model, which leads to confidence in the design parameters assumed. While the MC failure criterion assumed in the MC model is safe and appropriate for use in the Ultimate Limit State (ULS) design, the MC model fails to correctly predict the strains in the soil and hence the ground deformation behavior predicted by the MC model can be grossly incorrect. This is because the MC model is firstly unable to correctly replicate the stress paths of real soils, secondly the model uses a single stiffness with neither stress nor strain dependency, and furthermore the model has no memory of stress history.

As cities grow to become more densely packed, the focus of geotechnical design begins to shift from the ULS to the Serviceability Limit State (SLS) in order to contain and minimize damage to neighboring properties or buried structures. Where the Mohr-Coulomb model is lacking, advanced constitutive models can capture the various characteristics of soil behavior required for a safe yet economical design. The Hardening Soil (HS) constitutive model for instance incorporates a nonlinear stress and strain dependent stiffness, has the ability to distinguish the different modes of plasticity in the soil such as volumetric hardening or shear hardening responses, and differentiates between the loading and unloading stiffness. Because of these capabilities, the HS model and similar advanced constitutive models are able to predict ground movement reasonably well for SLS design. Such models are widely available today, and the model parameters are not very difficult to determine from common laboratory and in-situ tests.

Apart from selecting constitutive models suitable to the nature of the project, there are many other considerations for a good numerical analysis. This paper will serve as a primer to appropriate numerical modelling by briefly discussing the proper use of constitutive models, correct initial conditions, appropriate soil identification and layering, drainage conditions of soil, modelling of groundwater, and safety analyses in FEM models.

## 2 INITIAL CONDITIONS OF FINITE ELEMENT ANALYSES

Before any computation in FEM model, the initial state of the soil must be correctly defined by the user. The initial state of the soil comprises of the initial stresses as well as the initial porewater pressures state in the soil. In the finite element code PLAXIS, the initial stresses are calculated based on the soil weight together with the loading history, and represents the equilibrium state of the ground before any calculation steps are performed. PLAXIS allows the user to define the initial stresses in the soil in the first calculation phase, or the initial phase of the calculation procedure. PLAXIS version 2016 onwards allows the user to choose between three calculation types: $K_0$ procedure, gravity loading and field stress.

The most commonly used $K_0$ procedure defines the initial stress condition based on the user's input of the coefficient of earth pressure at rest $K_0$, and over-consolidation parameters such as the Over-Consolidation Ratio (OCR) or the Pre-Overburden Pressure (POP). Equilibrium of the generated stresses are not guaranteed and should be checked by the user, unlike for the gravity loading calculation procedure.

The gravity loading procedure defines the initial stress condition by only considering weight loading, and by generating non-physical displacements which must be reset in the subsequent phase. While equilibrium of stresses is reached at the end of the phase, the gravity loading procedure does not consider the OCR or POP of the soil. There is no direct user control over the initial stress ratio, however the user may artificially adjust the weight of the soil through the $\sum M_{weight}$ multiplier in the initial phase to achieve the desired over-consolidation state. Consequently, only a uniform over-consolidation state of the soil throughout the model is possible.

The field stress procedure is an advanced calculation procedure that allows the user to directly specify a homogenous stress state that can account for rotated principal stresses in the initial conditions. These are used mainly in deep rock conditions which rotated principal stress trajectory is known from geologic investigations.

It is very important that the calculation of numerical models begins with a realistic initial effective stress field and pore pressure state, that is also in equilibrium with the soil weight. Initial stress state and consolidation history play an important role in the outcome of the numerical prediction.  Unfortunately, initial stresses that are generated by the MC model are meaningless and will not have any consequences on the results of the numerical model. This is because the MC model is a purely elastic model and is therefore unable to have any stress memory. That in itself is a serious numerical modelling issue. Advanced constitutive models, however are able to capture the correct in-situ stresses that can be created by the user.

The significance of using advanced constitutive models to describe the initial conditions of the soil can be illustrated with a simple soil test procedure within PLAXIS. The stress strain response of both the MC and HS constitutive models with the same basic soil parameters under the same drained triaxial stress conditions are plotted for comparison, as shown in Figure 1. It can be observed that within the elastic region of the MC model, there is no distinction between the virgin loading and unloading/reloading of the soil. This behavior is unlike observed soil behavior from laboratory or field tests. Constitutive models with the ability to model a cap yield surface will be able to distinguish an over-consolidation response from a virgin compression response, and will be able to define the initial state of the soil appropriately.

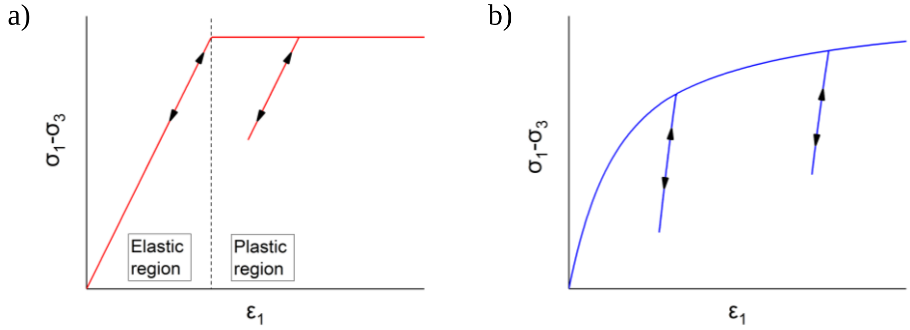


Figure 1. Simulation of a drained triaxial test with a) Mohr-Coulomb model and b) Hardening Soil model.

### *2.1 Initial conditions of sloping ground*

Obtaining the correct initial stresses in sloping ground has always proved to be a challenge in numerical modelling. While gravity loading is recommended by PLAXIS to be used for non-horizontal layers, the $K_0$ procedure can also be used with several additional procedures in place. As previously mentioned, the $K_0$ procedure is able to produce the desired over-consolidation state of soil by user input of OCR or POP, but is unable to satisfy equilibrium conditions. The principal stress rotations are incorrect if only the $K_0$ procedure is performed. Indicated in Figure 2, the principal stress directions produced by the $K_0$ procedure are only in horizontal or vertical directions, signifying that equilibrium is not achieved. This shortcoming can be resolved by the creation of a high accuracy nil step computation to achieve equilibrium before the start of the analysis. In this calculation step, the default value of 0.01% of the tolerated error is reduced by 10 to 100 times higher accuracy. The achieved state of equilibrium as a result of the higher accuracy nil step is presented in Figure 3, comparable to a result by the gravity loading procedure. However, contrary to the calculation outcome of the gravity loading procedure, the over-consolidation state is correctly captured by this procedure, with appropriate values of $K_0$. The summarized procedure is listed in Table 1.

Table 1. Recommended numerical procedure for initial stress calculation in sloping ground.

| Calculation phase | Calculation steps |
|---|---|
| Initial phase | Standard $K_0$ procedure calculation. |
| Phase 1 | Higher accuracy calculation step with adjusted error tolerance. |
| Phase 2 | Reset displacements to zero, continue with staged construction. |
| Phase 3 onwards | Continue with staged construction. |

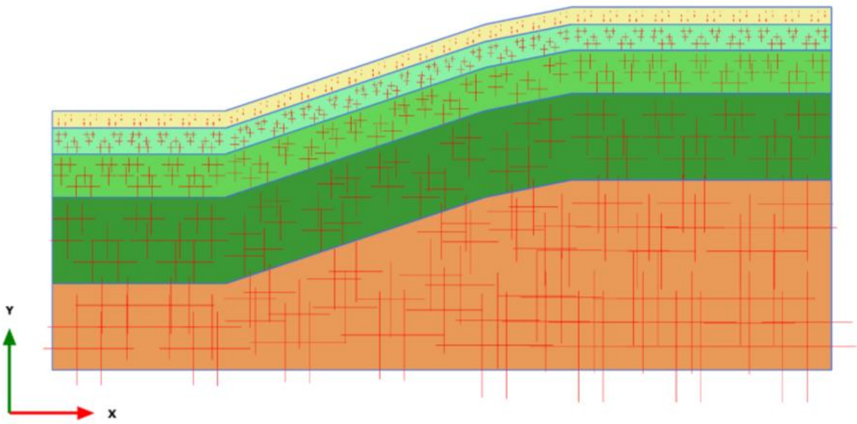


Figure 2. Principal stress directions of the initial stress state from the $K_0$ procedure.

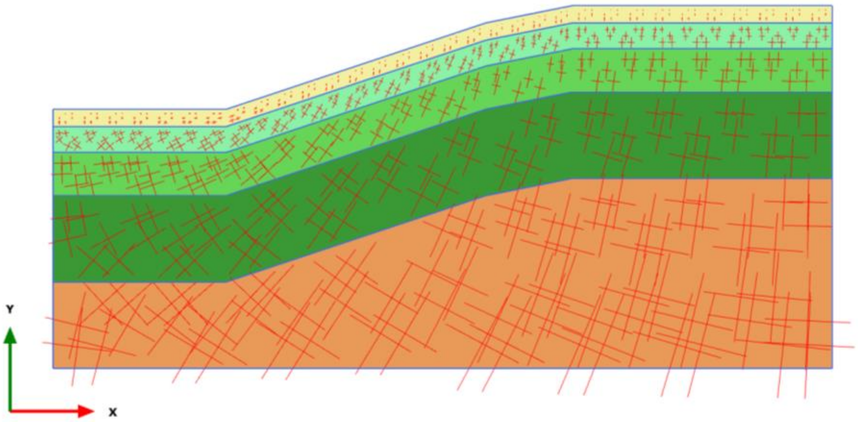


Figure 3. Principal stress directions of the initial stress state after the nil step for $K_0$ procedure, or gravity loading.

## 3 SOIL LAYERING

In general, it is expected that soils appear in laminated layers. This is because sedimentary soils are deposited in layers, and the weathering profile of residual soils is a function of depth. For a natural sloping ground there should be no horizontal layering but parallel layers. Incidentally, if the N-value from the Standard Penetration Test (SPT) are plotted with depth instead of ground level, the N-values are naturally banded. This intuitive concept can also be verified from typical soil layering observed in many past projects, as illustrated by Figure 4. Such layering of soils should be reflected in finite element models.

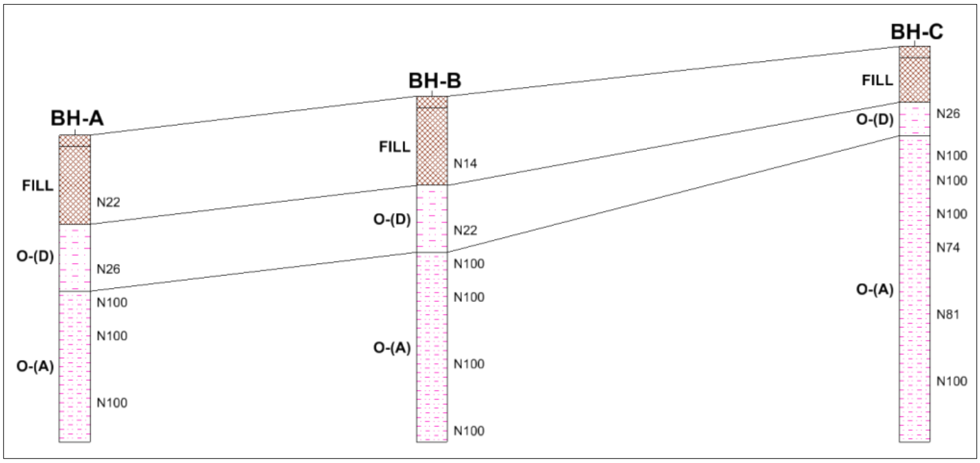


Figure 4. Typical soil layering for a natural sloping ground.

The geotechnical engineer ought to provide for sufficient site investigation boreholes to adequately characterize his project site. With enough site investigation, lenses of soil that can interfere with design will be detected. Furthermore, with an adequate number of laboratory and field tests, the correct soil types can be identified and reasonable geotechnical design parameters can be determined for an accurate numerical prediction of soil behavior in geotechnical projects.

### *3.1 Soil parameters*

Soil parameters for use in advanced constitutive models can more often than not be obtained from common laboratory and field tests. However, the engineer should always first identify and provide test specifications suited to the nature of the project. As one of the most common uses of FEM in Singapore is mainly for deep excavation projects, obtaining soil parameters for excavations will be discussed.

The dominant behavior of soils in excavations are unloading and reloading, hence it is good practice to specify tests to have unloading and reloading cycles. The standard triaxial test and one-dimensional consolidation oedometer test conducted with unloading and reloading cycles will allow for a clear distinction between the behavior of the shear hardening and cap hardening surfaces.

It is often recommended to conduct isotropically consolidated drained triaxial (CID) tests over the isotropically consolidated undrained triaxial (CIU) tests. The CID test directly measures the effective stress of the soil during shearing, as opposed to the CIU test first measuring the total stress of the soil and then deriving the effective stress by measuring the excess pore pressure generated. Moreover, the pore pressure transducer is commonly installed at the base of the triaxial sample where it is unable to obtain an accurate representation of the average excess pore pressure in the sample, which can lead to erroneous test result interpretation. The CID test is excellent for numerical calibration of advanced constitutive models due to the constant effective confining stress, allowing for a straightforward calibration of stress paths against a numerical model.

Advanced constitutive model numerically calibrated by laboratory test data can be verified by simulation of field tests. An appropriate in-situ test to calibrate the numerical model would be the Pressuremeter test (PMT). The PMT considers all environmental factors, in-situ stresses, stress history, drainage boundary conditions, and is conducted with minimal soil disturbance in stable borehole conditions. The calibration process can be carried out by the application of the cavity expansion theory in finite element models. Figure 5 shows the results of a simulated PMT from PLAXIS plotted against results from the actual PMT. A successfully calibrated model against both laboratory and in-situ tests gives confidence that the numerical model has a comparable behavior to the real soil. This practice should be the aim of all site investigation testing programs.

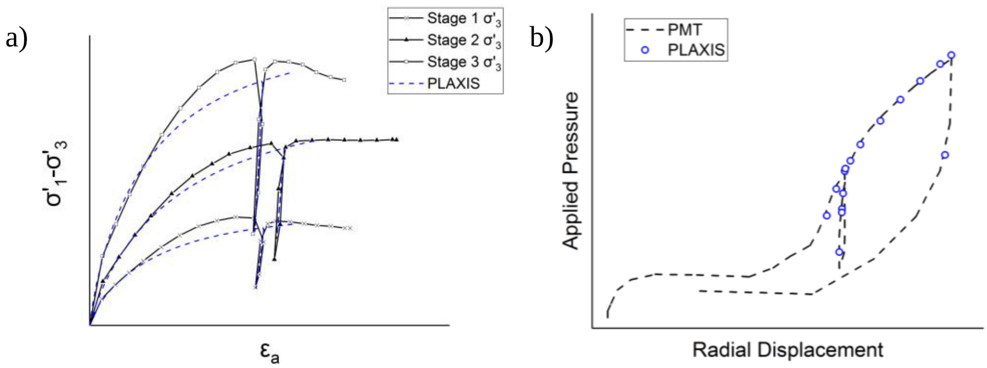


Figure 5. Numerical simulations verified against real project data for a) Isotropically consolidated drained Triaxial test and b) Pressuremeter test.

Excavations exhibit a fairly predictable behavior, and the value of constitutive models can be classified by their ability to capture the variation of soil stiffness within the excavation. A noteworthy advanced constitutive model suitable for the modelling of excavations is the HS model. There are some shortcomings of the HS model such as the inability to model strain softening or the effects of creep. Even so, the model has an appropriate behavior within the low strain levels expected, and is unlikely to cause an unrealistic prediction of the excavation behavior.

*3.2 The Hardening Soil constitutive model*

The HS model developed by Schanz et al. (1999) was formulated from the hyperbolic relationship between the vertical strain and the deviatoric stress in primary triaxial loading. The basic features of the standard elastoplastic HS model include having a nonlinear stress dependent stiffness, separate stiffness for primary loading and unloading/reloading stiffness in shear and compression, and memory of pre-consolidation stresses. Additionally, the model has the ability to distinguish between the different modes of plasticity in the soil, such as volumetric hardening, shear hardening or even a combination of the two soil responses. Further development of the HS model led to the Hardening Soil model with small-strain stiffness (HSS) by Benz (2007), which includes strain dependency in the calculation of soil stiffness.

The key parameters used for constitutive model are strength parameters c', ϕ' & ψ and stiffness parameters $E_{50}^{ref}$, $E_{oed}^{ref}$, $E_{ur}^{ref}$ and stress dependency power m. As the standard HS model utilizes the MC failure criterion, the traditional approach of Mohr envelopes from triaxial tests may be used to obtain the effective strength parameters of the soil samples. The stiffness parameter $E_{50}$ is the confining stress dependent stiffness secant modulus for primary loading, while the stiffness parameter $E_{oed}$ is the vertical stress dependent tangent stiffness for primary loading. While the stiffness in primary loading are plastic stiffness parameters, the unloading and reloading stiffness parameter $E_{ur}$ follows Hooke's theory of elasticity.

The relationship between $E_{50}$ and $E_{50}^{ref}$ is described by Equation 1, where the reference stress $p^{ref}$ determines the reference stiffness modulus, and the amount of stress dependency is given by the power m. The $p_{ref}$ can be taken as the default value of 100 kPa or calibrated against the in-situ effective confining stress, either value will give the right soil behavior provided the calibration against the laboratory and field tests were done correctly. Equation 1 can be linearized to obtain Equation 2. With three different confining stresses conducted in a triaxial test, three sets of stiffness can be obtained, thus a straight line can be plotted with Equation 2. The stress dependency power may be directly obtained from the gradient, and the reference stiffness modulus interpreted from the intercept. Similar steps may be performed for the unloading/reloading stiffness $E_{ur}^{ref}$.

$$E_{50} = E_{50}^{ref}\left(\frac{c'\cos\phi' + \sigma_3 \sin\phi'}{c'\cos\phi' + p^{ref}\sin\phi'}\right)^m \tag{1}$$

$$\ln\left(E_{50}\right) = \ln\left(E_{50}^{ref}\right) + m\ln\left(\frac{c'\cos\phi' + \sigma_3 \sin\phi'}{c'\cos\phi' + p^{ref}\sin\phi'}\right) \tag{2}$$

While test results from triaxial tests can be used to characterize the shear hardening response of soils, test results from oedometer tests can be used to characterize the volumetric hardening response of the soil. Equation 3 derived from the relationship between the vertical strain and vertical pressure of the oedometer test may be used to determine the reference oedometer stiffness $E_{oed}^{ref}$. The pre-consolidation stress $p_c$' may be obtained from the oedometer test with several approaches such as Casagrande's empirical method, the bi-logarithmic method by Onitsuka et al. (1995) or the incremental work method by Becker et al. (1987).

$$E_{oed}^{ref} = \frac{2.3(1+e_0)\,p^{ref}}{C_c} \tag{3}$$

A more modular form of the HS model entitled as the Generalized Hardening Soil model (GHS) has been developed to allow the user to specifically select or deselect functions of the constitutive model. Apart from the availability of options, the GHS model further adds to the practical capability of this constitutive model.

The GHS model allows the user to specify the configurations of stress and strain dependency, modes of plasticity and failure criterion. The strain dependency option allows the user to utilize the strain dependency capabilities of the HSS model. Thereafter the failure criterion can be selected to be either the MC criterion or the Matsuoka-Nakai criterion. After which four different plastic yield functions can be selected for the plasticity mode, as shown in Table 2. It is generally recommended to enable both the shear hardening and cap hardening, as that is the realistic behavior of most soils. Lastly, there are two sets of options for stress dependency in the GHS model. The first of which will define how often the stiffness of the soil is updated in the calculation, as shown in Table 3, and the second will define how the stiffness of the soil is calculated, as shown in Table 4.

Table 2. Modes of plasticity within the Generalized Hardening Soil model.

| | Plastic yield function |
|---|---|
| Option 1 | Linear elastic and perfectly plastic |
| Option 2 | Shear hardening |
| Option 3 | Cap hardening |
| Option 4 | Both shear hardening and cap hardening |

Table 3. Frequency of updated stiffness within the Generalized Hardening Soil model.

| | Frequency of update for stress dependent stiffness |
|---|---|
| Option 0 | Constant $E_{ur}$ throughout the calculation |
| Option 1 | Updates $E_{ur}$ for every calculation phase |
| Option 2 | Updates $E_{ur}$ for every calculation step |

Table 4. Formulation of stress dependency within the Generalized Hardening Soil model.

| | Formulation of stress dependency |
|---|---|
| Option 0: Stress dependency is based on $\sigma_3$ and strength parameters | $E_{ur} = E_{ur}^{ref}\left(\frac{\sigma_3 + c\cdot\cot\phi}{\sigma^{ref} + \cot\phi}\right)^m$ |
| Option 1: Stress dependency is based on $\sigma_3$ and pre-consolidation stress | $E_{ur} = E_{ur}^{ref}\left(\frac{(\sigma_3 + p_c)/2}{p^{ref}}\right)^m$ |
| Option 2: Stress dependency is based on mean effective stress and pre-consolidation stress | $E_{ur} = E_{ur}^{ref}\left(\frac{(p' + p_c)/2}{p^{ref}}\right)^m$ |

Of practical value is the ability of the GHS model to include stiffness dependency on the pre-consolidation stress and the mean effective stress in the updating of stiffness during FEM computations. This can be illustrated by the comparison of material models in common excavation performance. Upon excavation of a ground, the soil in reality may have weakened due to the decrease in confining stress. However, the stiffness of a ground modelled by MC model does not change, because the model does not have a stress dependent stiffness. This is an unsafe and optimistic numerical outcome. With the standard HS model the soil stiffness is pegged only to the minor principle stress $\sigma_3$, and thus the soil becomes too weak, leading to a pessimistic numerical outcome. The GHS model with option 2 of stress dependency formulation achieves a balance, which is a realistic expectation of how the soil behaves during an excavation. In 3D situations, the true stresses that control the behavior is a combination of the mean effective stress parameter p’ and the pre-consolidation stress $p_c$’. This is likely to be the best option to model realistic soil stiffness response in excavation unloading of soil stresses.

Another example with a sloping ground can reiterate the merits of the GHS model. The minor principal stress in a sloping ground is a somewhat rotated stress, which is not ideal for stress dependency. Figure 6 shows the stiffness $E_{ur}$ of the ground before and after excavation for the hardening soil model. It can be seen that there is a significant drop of stiffness after the excavation, which is unlikely to be true in reality. Figure 7 shows the shear modulus G of the soil in the excavation modelled with the GHS soil model. The shear modulus of the soil can be relatable to the young’s modulus of the soil with the Poisson’s ratio. Thus, it can be seen that the stiffness of the soil does not decrease as much as the HS model, which is a more realistic expectation.

The mean effective stress is independent of how the stress rotates and should be used in FEM calculations. Soil in reality would not be needlessly dependent on the rotation of stresses. The other parameter $p_c$’ takes into account the soil loading history. It is expected that soil behaves relatively stiffer when the loading exerted on it is less than what it had experienced in the past, due to factors such as natural cementation. However, should the soil be loaded beyond its pre-consolidation pressure, the structural formation of the soil breaks down leading to irrecoverable volume change. With these considerations, the GHS constitutive model makes the least assumptions in the modelling of ground movement.

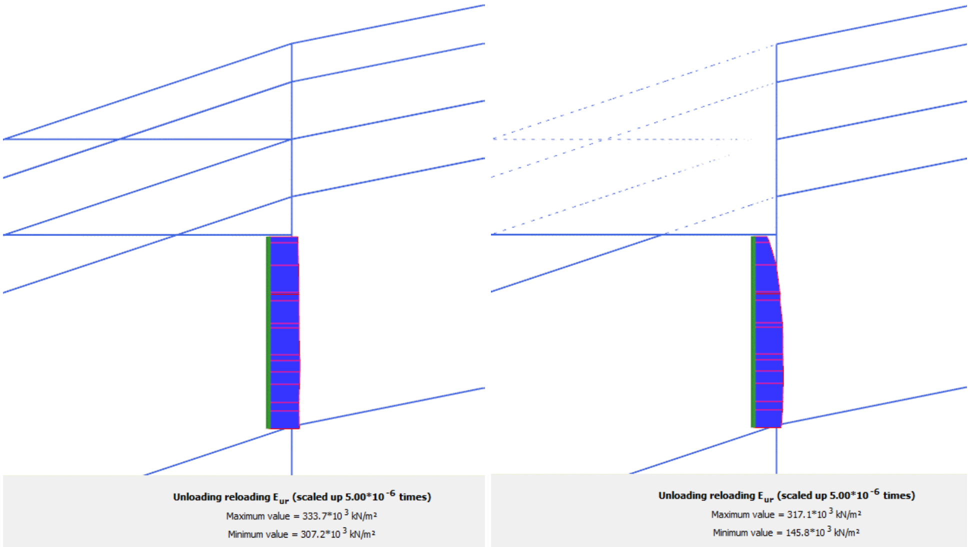


Figure 6. Unload/reload stiffness before and after excavation, for Hardening Soil model (Minimum value of $E_{ur}$ reduced from 307.2E+03 to 145.8E+03 kN/m$^2$).

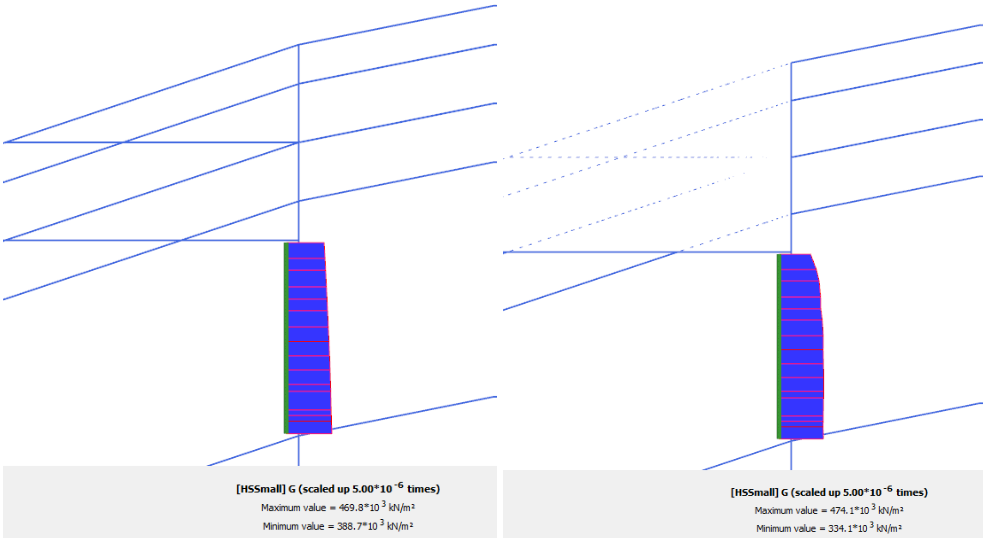


Figure 7. Shear modulus before and after excavation, for Generalized Hardening Soil model (Minimum value of G reduced from 388.7E+03 to 334.1E+03 kN/m$^2$).

## 4 DRAINAGE CONDITIONS

With the initial conditions and soil layering defined, it is now possible to proceed to the staged construction calculation phases. Most geotechnical problems can be solved with either/or a combination of the two extremes of soil drainage conditions, namely the drained or undrained state. However, the drainage condition of soil in reality is time dependent. PLAXIS allows for three main types of calculation to determine porewater pressures and they are namely the plastic (drained or undrained), consolidation and fully coupled flow deformation analysis. The commonly used plastic calculation type performs an elastic-plastic deformation analysis without taking into account the pore pressure changes due to time or the pore pressure changes due to soil deformation. In the consolidation calculation, the development and dissipation of excess pore pressures with respect to time can be tracked. The fully coupled flow-deformation analysis is most suitable for projects where the simultaneous development of deformation and pore pressures due to time-dependent changes of the hydraulic boundary conditions is important for an accurate prediction of ground and pore pressure behavior.

The main factors that affect the drainage condition of soil are the rate of loading, hydraulic boundary conditions and the soil permeability and stiffness. The common misunderstanding that drainage conditions of the soil is defined by only permeability must be corrected. Coincidentally, soft soils happen to have low values of permeability. A combination of permeability and stiffness should be used to determine the drainage conditions of soils, with the use of the coefficient of consolidation parameter $c_v$, as described by Equation 4. In current state of practice, permeability values of 1E-08 m/s or less are often defined to be undrained. However, it is commonly reported that instrumentation results show little excess pore water pressures are generated in stiff residual or sedimentary soils with low permeabilities. Likewise, it is also possible for a sandy soil with low stiffness to be a draining material. For a construction period of 6 months to a year, soils with $c_v$ values of less than 10 $m^2$/year can be said to be approximately undrained, and soils with $c_v$ values of more than 50 $m^2$/year as approximately drained. The soil is partially draining in-between $c_v$ values of 10 $m^2$/year and 30 $m^2$/year. Table 5 indicates common values of $c_v$ with the corresponding drainage conditions.

$$c_v = \frac{k \times E_{oed}}{\gamma_w} \tag{4}$$

A further illustration of soil drainage conditions can be considered, as suggested by Vermeer & Meier (1998). Equation 5 describes the use of a dimensionless time factor T with the drainage length D to characterize the drainage conditions of soil with respect to the construction time.

$$T = \frac{c_v \times t}{D^2} \tag{5}$$

Table 5. Coefficient of consolidation, $c_v$ and corresponding approximate drainage conditions for construction periods of 6 months to a year.

| Soil types | k (m/s) | $E_{oed}$ (kPa) | $C_v$ ($m^2$/yr) | Approximate drainage condition |
|---|---|---|---|---|
| Sand | 1E-05 | 1E+04 | 315360 | Drained |
| | 1E-06 | 1E+04 | 31536 | Drained |
| | 1E-07 | 1E+04 | 3154 | Drained |
| | 1E-08 | 1E+04 | 315 | Drained |
| | 1E-09 | 1E+04 | 31.5 | Partially draining |
| | 1E-10 | 1E+04 | 3.2 | Undrained |
| Soft Clay | 1E-10 | 5E+03 | 1.6 | Undrained |

When the value of T is less than 0.1, the degree of consolidation is less than 10%, leading to approximately undrained conditions. Subsequently when T is more than 0.4, the degree of consolidation is more than 70%, leading to drained conditions. Implications of this association regarding soil permeability, soil stiffness and drainage path can be illustrated with Figure 8, an example of a common excavation. Given the same characteristic drainage path, defined as the depth from the toe of the wall to the formation level, in an excavation, the drainage length is long when the excavation just begins, and decreases as the excavation proceeds. Implying that the soil starts as at an undrained state and progressively moves to the drained state. Whereas the soil on the unexcavated side of the wall tends to remain in the undrained condition, due to there being no significant change to the drainage path.

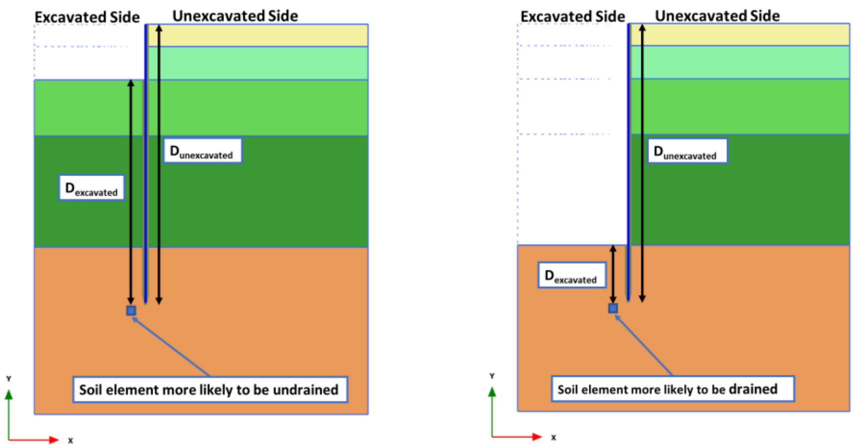


Figure 8. Drainage conditions of a typical excavation.

It is generally a good practice to design for both drained and undrained cases for the worst structural loadings as it is very difficult to predict consolidation due to the variability of soil and the complexity of each project. It should be noted that certain soils are acknowledged to be undrained throughout the entire construction period.

Singapore marine clay and peaty clay with low values of permeability and very low stiffness are two such materials. Within the construction period of a few years, the clayey material generates high excess porewater pressure during the excavation, and the high excess porewater pressures remain in the material throughout the project construction period, causing the soil to behave practically as an undrained material. The other residual or sedimentary soils may have different drainage conditions throughout the project duration. That is why there is a need to identify the materials that have the possibility of a more undrained response if the excavation is fast enough, to have an economical and safe design.

The advantages of using advanced pore pressure calculation procedures can be significant from a cost perspective. Consider a case when contiguous bored pile walls are proposed to be used to retain a certain height for a fully drained excavation. In granular cohesive soils, it can be contended that the material behaves like a partially draining material due to low permeability values, and in that case a soldier pile wall might actually work. If a coupled consolidation analysis is performed, the loading on the earth retaining structure will be considerably less than for a fully drained condition. In reality, such soils are always in a state between drained and undrained conditions. Should the site conditions be quantified correctly for use in FEM, a more cost-efficient design can be adopted.

The value of performing a coupled consolidation analysis is on a case by case basis, with greater considerations on the correct modelling of the hydraulic boundary conditions, permeability of the ground, suitability of the earth retaining stability structures with appropriate embedment depths, and a realistic time frame for the completion of the project. The engineer must appropriately select the calculation type suitable for his project, and in turn examine the results from the FEM analysis, specifically the current operative soil strength at each stage of construction to ensure safe design.

### *4.1 Undrained soil strength*

The change in soil strength at every stage of the excavation should always be checked by the engineer. Realistically, the initial strength of the soil is at its peak and will decrease as the excavation proceeds because of the gradual decrease in effective stresses due to unloading. The changes in porewater pressure in the excavation will likewise affect the strengths of the soil. Consequently, identification of correct drainage conditions can lead to a more efficient design. With today's FEM capabilities, it is possible to check the variation of undrained strength with respect to each calculation phase or with time, depending on the calculation mode selected. It is advised to verify the shear strength $\tau_{mob}$ and $\tau_{max}$ of soils at each stage. An example of an FEM output of $\tau_{mob}$ is shown in Figure 9.

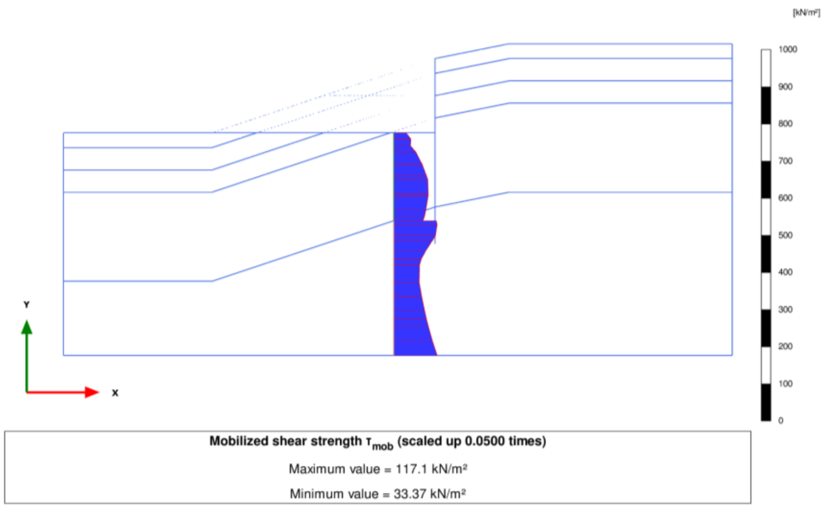


Figure 9. Output of shear strength with the Generalized Hardening soil model in PLAXIS.

Volume change in a real soil may be due to volumetric compression or due to shear distortion. As the MC model is unable to simulate volume change due to shear, the stress path of the MC model must follow a constant mean effective stress path. Hence, implications of using the MC model for undrained soil strength can be dangerous. Figure 10 shows the effective stress path of the MC model in undrained simulation. Method A of undrained modelling uses effective stress strength parameters for effective stress analysis while Method B of undrained modelling uses total stress strength parameters in effective stress analysis. While method B can limit the undrained strength to appropriate values, the stress paths predicted by the model is still incorrect. It is suggested to use advanced constitutive models with the ability to capture the complete effects of volumetric compression and shear distortion on effective stresses, in other words a model with double hardening capabilities such as the HS model as shown in Figure 10.

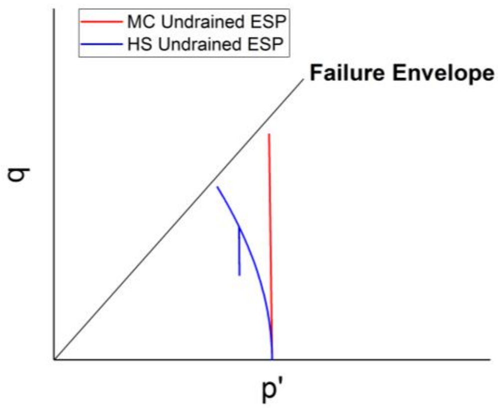


Figure 10. Effective stress paths of the Mohr-Coulomb and Hardening Soil model.

## 5 GROUNDWATER MODELLING

In the modelling of groundwater in FEM, it should be first identified whether the groundwater system is an unconfined or confined flow. Consequently, a decision should be made on whether to use the phreatic control or steady state groundwater flow for the calculation of pore pressures. The phreatic control method is based on the input of a global water level where the calculated porewater pressures are hydrostatic. While for the steady state groundwater flow method the calculation of pore pressures is dependent on the hydraulic boundary conditions input by the user, with seepage of soils enabled. The steady state groundwater flow condition is ideal for simulation of groundwater behavior in the long term condition.

The condition that two distinct levels of groundwater exist at opposing sides of a retaining wall happens when toe of the wall is embedded in relatively impermeable material, and there is water recharge. Such conditions of groundwater are described by Figure 11. Phreatic control can be used to model the pore pressures of this groundwater condition where there is no significant water drawdown. However, the phreatic control method should be used together with the interpolation of pore pressures at the base of the excavation as shown in Figure 12.

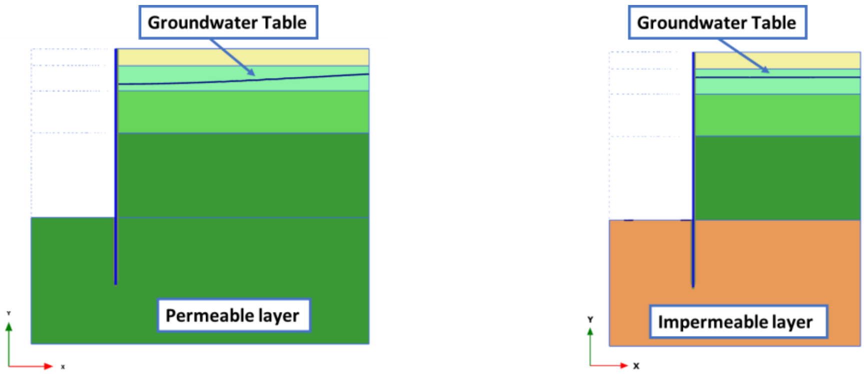


Figure 11. Possible groundwater conditions.

If a material has high enough values of permeability, steady state seepage can be achieved within a few months. The steady state groundwater flow calculation within PLAXIS is not time dependent, and consequently water drawdown due to seepage in this type of analyses is a function of geometry and relative permeability of soil layers. It should be noted that the influence of relative permeability rather than absolute permeability controls the behavior of the seepage problem. Figure 13 illustrates the porewater pressure generated by the steady state groundwater flow calculation. In the modelling of groundwater conditions, it is important to understand that soils are rarely homogenous in reality and the permeability of sub-layers must be recognized. Thin lenses of clay can prevent the groundwater table from lowering due to seepage, and likewise lenses of sand that is partially exposed to the atmosphere may draw water away rapidly.

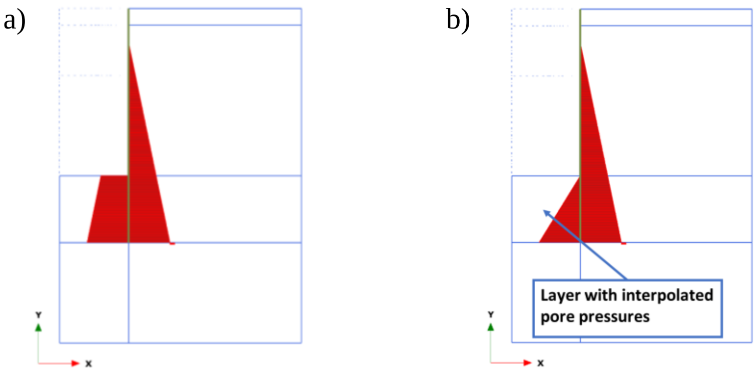


Figure 12. Porewater pressure generation by phreatic control a) without interpolation b) with interpolation.

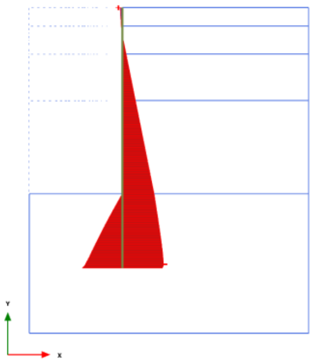


Figure 13. Porewater pressure generation by steady state groundwater flow.

The modelling of the groundwater table in FEM involves engineering judgement. The hypothesis of how the groundwater behaves can be verified with FEM by executing a transient flow analysis with realistic permeability and construction time of the excavation. Reviewing the results of the transient flow analysis will reveal how the water table changes with time, and a decision can be made on how to place the groundwater table in the numerical model. Figure 14 shows a typical example of an unconfined flow where transient flow calculations can be used to determine the approximate phreatic level.

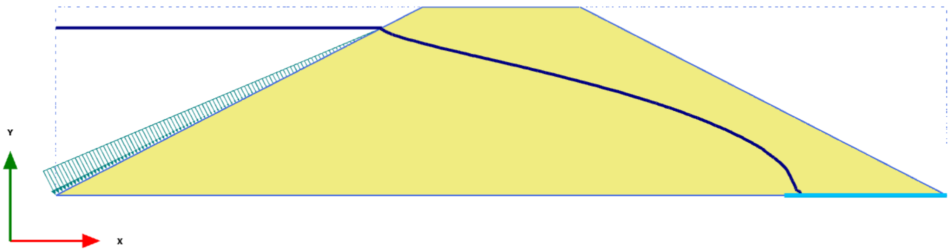


Figure 14. Unconfined flow within an earth dam.

For practical engineering design, the highest water table due to heavy rainfall could be used for ULS design. But for SLS design, a realistic water table should be determined with proper site instrumentation and be captured within the numerical analysis.

## 6 SAFETY ANALYSIS

More commonly known as the c'-phi' reduction method, safety analysis performed in PLAXIS calculates the global factor of safety by reducing strength parameters incrementally. This calculation procedure is useful to find the most critical failure mechanism. There are certain considerations in FEM for a reliable safety analysis, and they will be discussed in this section.

PLAXIS uses a multiplier named the strength reduction factor ∑Msf to reduce the strength of soil until failure occurs. There must be sufficient load steps such that the strength reduction factor ∑Msf converges so that it is certain that failure had been reached. Figure 15 shows an appropriate convergence of the ∑Msf factor.

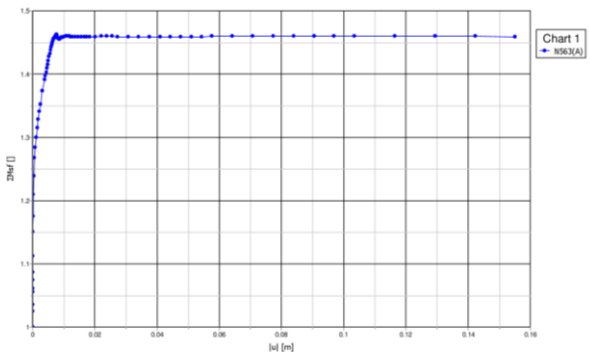


Figure 15. Appropriate convergence of the strength reduction factor.

The presence of undrained materials in the FEM calculations is another concern in safety analyses. A decision should be made whether to consider undrained behavior in the safety analysis. The influence of undrained behavior in a safety analysis may lead to unreasonable results as the continuous reduction of the friction angle of the soil influences the behavior of the stress paths causing it to reach the yield locus repeatedly, generating unreasonable amounts of excess pore water pressure. Appropriate inclusion of the behavior of undrained materials is dependent on site conditions.

Apart from reducing the strength of the soil clusters, structures must also be selected for strength reduction. A purely elastic analysis for structures is unsafe as the rotation of earth retaining structures will carry on indefinitely during a safety analysis, resulting in the yield zone for the soil clusters to enlarge unrealistically, as illustrated by Figure 16. A workaround is to include an elastoplastic material type for the structures in the FEM and subject them to strength reduction calculations to obtain the true factor of safety. Plasticity is taken into account by specifying the maximum bending moment $M_p$ and maximum axial force $N_p$. With the presence of both bending moment and axial force, the actual bending moment or axial force which cause plasticity are lower than the respective values of $M_p$ and $N_p$, because of interaction factors. With an elastoplastic behavior defined for structural elements, it is possible for them to develop plastic hinges, illustrated in Figure 17. Figure 18 displays the incremental shear strain output from PLAXIS, which shows the failure mechanism of safety analysis with the inclusion of elastoplastic structures. An appropriate safety analysis with an elastoplastic wall will reduce the yield zone and a true factor of safety may be obtained, as shown in Figure 19.

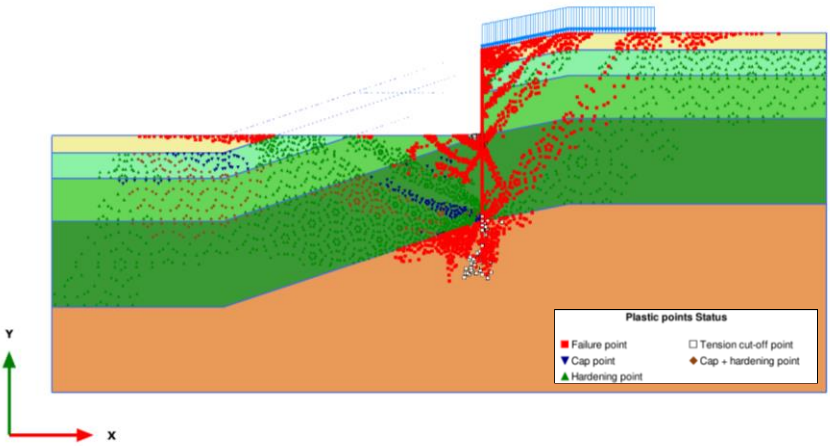


Figure 16. Distribution of plastic points during a safety analysis with an elastic wall.

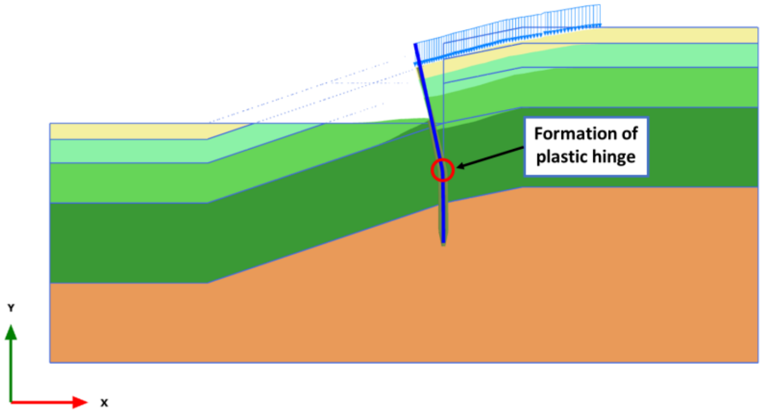


Figure 17. Formation of plastic hinge (Deformations scaled up 40 times).

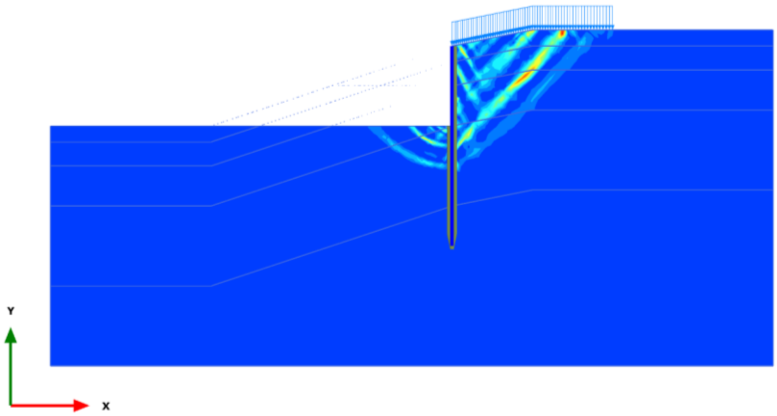


Figure 18. Incremental shear strains showing the failure mechanism of safety analysis with the inclusion of elastoplastic structures.

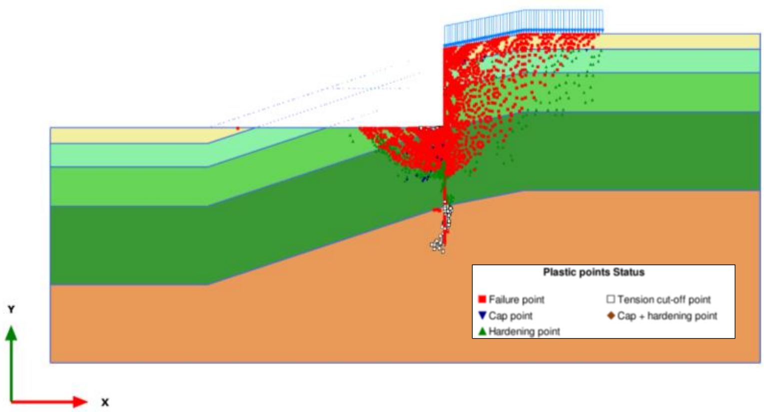


Figure 19. Distribution of plastic points during a safety analysis with an elastoplastic wall.

# 7 CONCLUDING REMARKS

Proper procedures for FEM analyses should be in place for an accurate numerical prediction of soil behavior. The importance of initial conditions of FEM projects were discussed, and it is shown that the elastic Mohr-Coulomb model is unsuitable for replicating the initial stress state of soils. There are various methods of establishing the initial stress state for FEM meshes and with certain additional steps they all work reasonably well for various cases.

Layering of soils are almost always parallel to the ground due to the nature of soil deposits and weathering profiles being a function of depth. For excavations, it is crucial that site investigation laboratory and field tests be conducted with loading and unloading sequences for suitable representation of soil unloading behavior. The use of the hardening soil constitutive model with its double hardening capabilities and stress and strain dependent stiffness is very suitable for the modelling of excavations for the reasons discussed in the paper. The inclusion of mean effective stress and pre-consolidation pressure for stiffness calculations in the generalized hardening soil model further enhances the capability of the finite element modelling tool.

A combination of soil permeability and stiffness should be used to approximate the drainage conditions of soils. Strength of soils are dependent on time and a fully coupled flow-deformation analysis should be used to verify the strength changes with time. Great care must be taken when using the Mohr-coulomb model for undrained soils. Even with artificial strength reductions, the stress path produced by the model is still incorrect. It is recommended to use constitutive models with more correct effective stress paths to model soils in partially draining conditions when needed.

Groundwater behavior is complex in reality due to the presence of sub-layers. It is important to characterize the soil by its relative permeability instead of absolute permeability for an accurate prediction of groundwater behavior in steady state conditions. Hydrostatic phreatic specification of ground water levels must be used with proper understanding of the realistic seepage conditions that apply in reality.

Safety analysis must include the strength reduction of structures to avoid unrealistic enlarged yield zones of soil. The inclusion of undrained soil behavior in safety analyses is debatable, and judgement must be used to ensure no disproportionate amounts of excess porewater pressure is generated in safety analysis with undrained behavior activated.

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