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Abstract

The advance of computer technology has made the finite element method (FEM) more accessible than ever. Many engineers have tried FEM geotechnical software in handling their geotechnical projects. However, like a pilot with inadequate training, it would backfire if he were to fly a sophisticated jet fighter. Engineers with insufficient geotechnical background may gain access to the sophisticated FEM software without realizing the risk behind it. They make mistakes that may lead to the bad performance or even failure of the geotechnical structures. The author himself, along the years of learning and applying the geotechnical FEM software, has made many mistakes. This paper, with Plaxis application as example, tries to elaborate the common mistakes found in applying the FEM geotechnical software in handling excavation problems.

Keywords: Finite Element Method, Plaxis, Deep Excavation

1. Introduction

The application of Finite Element Method (FEM) is not new, it has been used in many engineering practice for over forty years. Throughout the seventies to mid-nineties, the method could only be applied by large universities that could afford to have the so called main frame computers. By the end of the 20th century, the advancement of computer technology had made personal and laptop computers able to run sophisticated FEM software, and hence, the method starts gaining its foot among engineers. To the author knowledge, specially built FEM software for geotechnical applications started to appear in the market in the early 1990s with the appearance of Feadam, Sage-Crisp, Plaxis, and others. As computer processors becomes faster and faster, many commercial geotechnical FEM software are becoming available, e.g.: Plaxis, Phase2, Geo5fem, Gfas, Sigma/w, Midas, Geofea, etc.

The author started becoming familiar with the geo-technical FEM software in the early 1990s. Started with Feadam in the 1990, Sage-Crisp in 1997, Plaxis from 1995, and lately also trying Phase2, Gfas, and Geo5fem. Along the years, in the process of learning, teaching and applying the FEM for geotechnical structures, through the mistakes of others and himself, he has gain some experiences that has made him wrote this paper to share the lessons learned. The write up given in this paper is based on Plaxis 2D software.

2. Modelling Excavation in Plaxis 2D

2.1 Plane Strain vs Axisymmetry Model

Though it is a relatively simple concept, many practicing engineers fail to understand the meaning of plane strain and axisymmetry. For example the shaded portion in Figure 1 will result in a long out of plane excavation if a plane strain model is adopted. On the other hand, it will result in a circular shape excavation if an axisymmetry model is adopted.

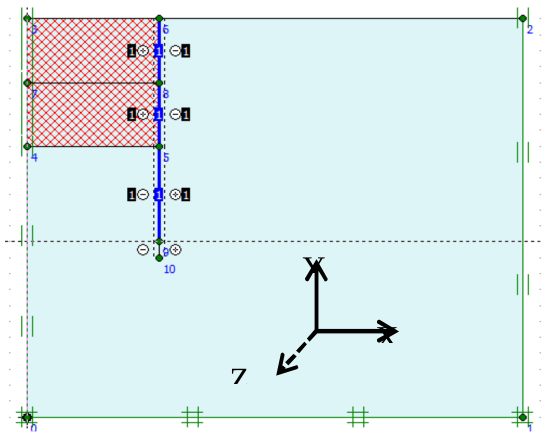


Figure 1. Plane Strain vs Axisymmetry Model

The plane strain model means the strains can only take place in the xy plane. Along the longitudinal axis (out of plane direction) the strain is assumed to be zero, $\varepsilon_z = 0$. Consequently, the length of the excavation must be significantly larger than the width of the excavation.

The axisymmetric model means the lateral, or more precisely, the radial strains of the model are equal in all direction, $\varepsilon_x = \varepsilon_z$. As the name implies the structures in the model is symmetrical along the vertical Y axis and the model is rotated at the Y axis, hence the model in Figure 1 results in a circular excavation. Note: in Plaxis the rotating axis is always at the left boundary.

Of course failure in choosing the right model of plane strain or axisymmetry will lead to incorrect output.

2.2 Interface Element

The interaction between the structural element and the soil is modeled by means of interface. The interface element is used to reduce the friction between the structural element and the soil. Introducing interface value, termed as R_{inter}, which has value between 0.01 and 1.0, does this. The lower bound value of 0.01 means there is practically no friction between the structural element and the soil. The upper bound value of 1.0 means the structural element and the soil is completely in contact, it means the soil and the structural component cannot slip one another. In this case, the contact is termed as rigid. Values in between mean the friction is reduced by the given number of R_{inter}, and the structural element and the soil mass can slip between one another.

The common mistake is to apply the interface element in modeling pressure grouted ground anchors as shown in Figure 2.

The free length of ground anchors is modeled by node to node anchor. As the name implies, in node to node anchor, the anchor is connected at both ends to nodes in the structural element, as if there is no contact along the anchor body to the surrounding soil. Therefore, there is no use to apply interface element along the body of the node to node anchor.

The bond length of ground anchors is modeled by geogrid (tensile) element. In practice the bond length is normally pressure grouted as such that the soil around the grouted body is completely in contact with the grouted body.

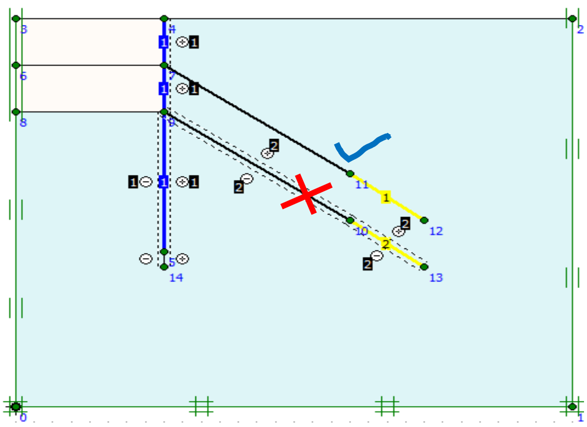


Figure 2. Mistake in Modeling Ground Anchor

Therefore, failure surface take place not between the grouted body and the soil, but within the soil seams that stick to the grouted body and the soil around it. It means the full frictional force of the soil can be developed; therefore, no interface element should be introduced. Applying interface element and giving R_{inter} less than 1 is a mistake.

2.3 Material Models

There are many options to simulate soil behavior, e.g.: Mohr-Coulomb (MC) model, Soft Soil Model, Hardening Soil Model, Soft Soil Creep, Hardening Soil with Small Strain, Modified Cam-clay, etc. Every model has each own pro and cons. Two of the soil models that are often adopted for modeling deep excavation problems, shall be discussed below,

2.3.1Mohr-Coulomb (Mc) Model

Being the simplest and the one that engineers were being trained with in their undergraduate study, Mohr-Coulomb is widely adopted by practicing engineers, often without realizing the limitation. Mohr-Coulomb modeled the non-linear behavior of the soil into two bilinear lines, as presented in Figure 3.

Inherent in this Mohr-Coulomb bilinear elastoplastic approached, the soil stiffness, taken as E_{50}, is constant throughout the elastic zone, until the stress state reaches the plastic (failure) zone. In reality, the soil behaves non linearly which means the soil stiffness is never constant, instead it changes with the stress level within the soil mass. Therefore, at stress level less than 50% of the ultimate strength, the MC model will over-predict the ground movement, whereas at stress level higher than 50% (means factor of safety less than 2) it can dangerously under predict the ground movement.

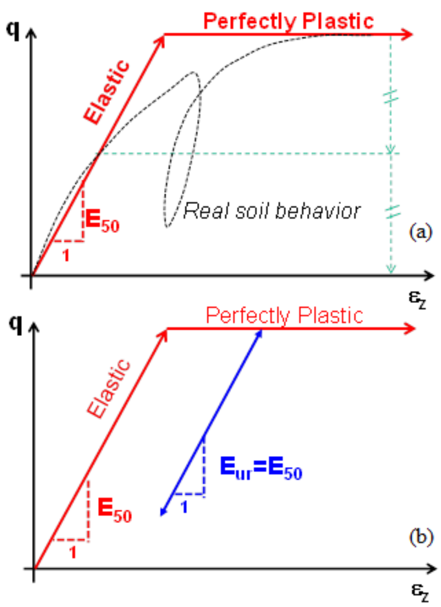


Figure 3. Mohr Coulomb Model

Another serious drawback in the MC model is: it assumed the soil unloading-reloading stiffness modulus, E_{ur}, equal the soil loading stiffness, E_{50}, i.e. $E_{ur}=E_{50}$ as presented in Figure 3b. In reality, under unloading-reloading condition soils generally have much stiffer modulus compared to under loading condition (see Figure 4). The unloading-reloading stiffness can be easily higher by a factor of 2 to 5 as compared to the loading stiffness i.e. $E_{ur} \approx 2\sim5\ E_{50}$. This means that when applied to evaluate excavation problems, the MC model will generally over predict the soil heave in an unrealistic manner. Due to this reason, in excavation problem, when MC model is adopted, it is suggested to input the soil stiffness in E_{ur} value rather than E_{50}.

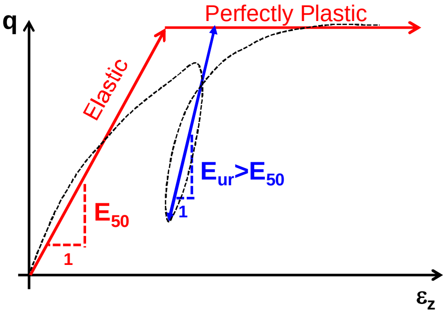


Figure 4. Real Soil have Higher Unloading-Reloading Modulus

Apart from the drawback on the assumption of the stiffness modulus, which leads to the inaccuracy of predicted soil movement, MC model also have its limitation in analyzing undrained problem. This shall be discussed in the next section.

2.3.2Hardening Model

The real soil stress strain behavior shows that when loaded the soil behaves non-linearly. As the load goes higher the stiffness modulus of the soil becomes lower and lower (see Figure 5).

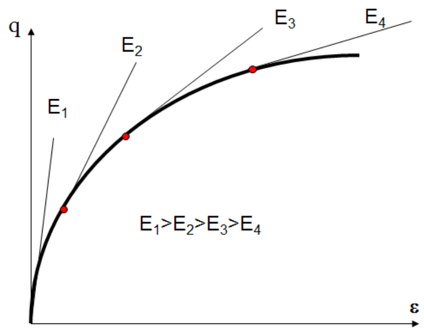


Figure 5. Non-linear Stress Strain Curve and The non Constant Soil Stiffness

This non-linear stress strain behavior can be approximated by hyperbolic model developed by Duncan & Chang, 1970. In Plaxis, this hyperbolic model is known as Hardening Soil model (HS model), and often applied in evaluating soft soil or hard ground condition. The formulation of the model is shown in Figure 6 below,

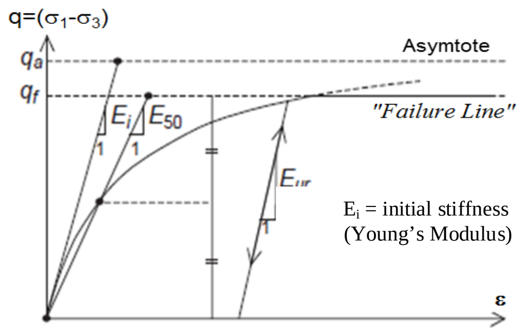


Figure 6. Formulation of Hardening Soil Model

For

$q < q_f \rightarrow -\varepsilon = \frac{1}{E_i}\frac{q}{1-q/q_a}$ (1)

$E_i = \frac{2E_{50}}{2-R_f}$ (2)

$R_f = \frac{q_f}{q_a} = 0.9$ (3)

The failure stress is determined by:

$q_f = (c'cot\varphi' - \sigma_3')\frac{2\sin\varphi'}{1-\sin\varphi'}$ (4)

The dependency of the soil stiffness to the stress level is calculated by:

$E_{50} = E_{50}^{ref}\left(\frac{c'\cos\varphi' - \sigma_3'\sin\varphi'}{c'\cos\varphi' + p^{ref}\sin\varphi'}\right)^m$ (5)

$E_{ur} = E_{ur}^{ref}\left(\frac{c'\cos\varphi' - \sigma_3'\sin\varphi'}{c'\cos\varphi' + p^{ref}\sin\varphi'}\right)^m$ (6)

Where:

- E_{50}^{ref} is reference soil modulus at reference confining pressure, p_{ref}, of 100kPa
- σ_3' = confining pressure
- the power m is generally equal to 0.5 for sand, and 1.0 for clay and silt.
- c' and ϕ' are effective strength parameters
- E_{ur} is the unloading-reloading modulus.
- E_{ur}^{ref} is reference unloading-reloading modulus at reference confining pressure, p_{ref}, of 100kPa

Apart from the loading stiffness, E_{50}, and the unloading-reloading modulus, E_{ur}, the hardening soil model also taken into account the oedometer modulus, E_{oed}, as presented in the following equation:

$E_{oed} = E_{oed}^{ref}\left(\frac{c'\cos\varphi' - \frac{\sigma_3'}{K_O^{nc}}\sin\varphi'}{c'\cos\varphi' + p^{ref}\sin\varphi'}\right)^m$ (7)

Where:

- E_{oed} = oedometer modulus, i.e. the soil stiffness with no lateral strain (obtained from oedometer test result)
- $K_o^{nc} = 1 - \sin \phi'$; coefficient of earth pressure at rest

2.3.3 Material Behaviour in Excavation Problem

Figure 7 shows a typical excavation problem with the stress paths experienced by soil mass below the excavation level and behind the retaining wall. It is clearly demonstrated that the soils at point B (below the excavation level) undergo unloading case at all construction stages, while point A (behind the retaining wall) goes through several changes, at stage 1 it undergoes unloading, at stage 2 (prestressing) it undergoes reloading, and at stage 3 again it undergoes unloading.

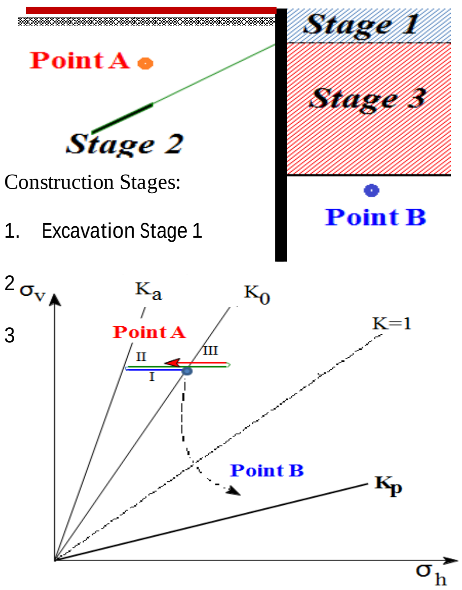


Figure 7. Stress Path under a Typical Excavation Problem

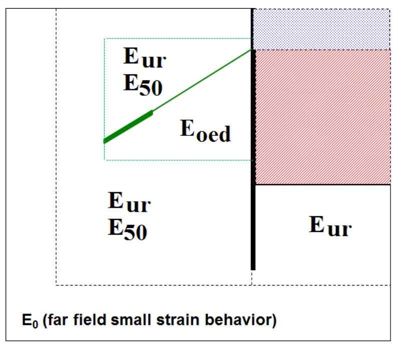


Figure 8. Expected Material Behavior in Excavation Problem (Brinkgreeve, R.B.J., Shen, R.F., 2011)

The stress paths clearly illustrated the need to use different soil stiffness in evaluating excavation problems. The expected material behavior at various zones in a typical excavation problem is depicted in Figure 8 (Brinkgreeve and Shen, 2011).

Since Mohr Coulomb model uses only a single E value, it fails to cater for the complex material behavior at various zones. It gives unrealistic deformations, overestimates bottom heave, and sometimes predicted unrealistic heave of soil behind the wall. Soils below excavation behaves with E_{ur}, even soils behind wall behaves between E_{ur} and E_{50}. Use of E_{50} is too conservative.

The soil stiffness for isotropic loading, shearing and unloading-reloading can be automatically catered for in the model Hardening Soil model. Therefore, it predicts more realistic wall deformations, bottom heave, and settlement trough behind wall.

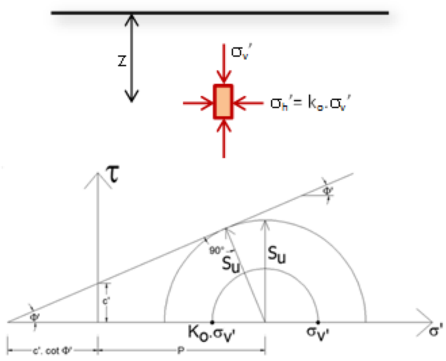
An improved version of HS model, i.e. the Small Strain Hardening Soil Model (HS small model), can take care of far field small strain behavior. Therefore, it gives even better and more realistic settlement trough behind the retaining wall (narrower and deeper).

2.4 Undrained Parameters

The undergraduate study of soil mechanics told us that analyzing undrained behavior of clay, has to be done with total strength parameters, S_u or c_u, ϕ=0, and undrained/total stiffness parameters, E_u and undrained Poisson's ratio, ν_u=0.5. However, in many FEM codes, the undrained analysis is often calculated by effective stress approach. The reasoning behind is a mathematical relationship between the undrained and drained shear strength parameters as presented in Equation (8) shown in Figure 9.

In Plaxis, there are three combination of input in modeling the undrained shear strength, as presented in Table 1.

Plaxis automatically adds stiffness of water when undrained material type is chosen, therefore, if total stiffness parameters are adopted as taught in the conventional soil mechanics, then the undrained stiffness becomes very much higher than it should be. In turn, it will lead to inaccurate predicted deformation.



$$C_u = Su = Sin\,\varnothing'.\left(c'.\cot\varnothing' + \frac{\sigma_{v'} + k_0.\sigma_{v'}}{2}\right)$$

Figure 9. Effective Stress Formulation of Undrained Strength

Table 1. Modeling Undrained Analysis

UNDRAINED A Analyzed in term of effective stress Material type: undrained Effective strength parameters c´, φ´, ψ´ Effective stiffness parameters E_{50}´, ν´
UNDRAINED B Analyzed in term of effective stress Material type: undrained Total strength parameters c= c_uφ=0, ψ=0 Effective stiffness parameters E_{50}´, ν´
UNDRAINED C Analyzed in term of total stress Material type: non-porous / drained Total strength parameters c= c_uφ=0, ψ=0 Undrained stiffness parameters E_u,ν_u = 0.495

Since soil behaviour is always governed by effective stresses, undrained A is a preferable method in modeling undrained behaviour of clay. It can predict the excess pore water pressure in a relatively accurate manner, and increases of shear strength during consolidation can be calculated. However, care must be taken if Mohr Coulomb soil model is adopted as undrained A may over predicts the undrained shear strength (see Figure. 10).

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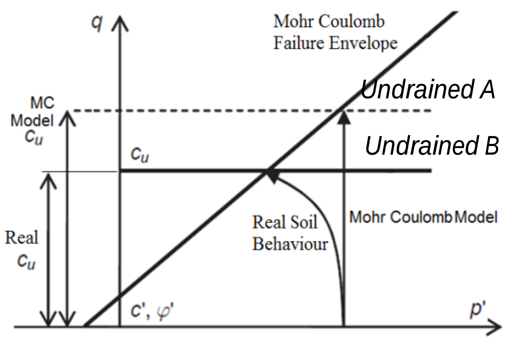


Figure 10. Mohr Coulomb Model Over Predict C_u

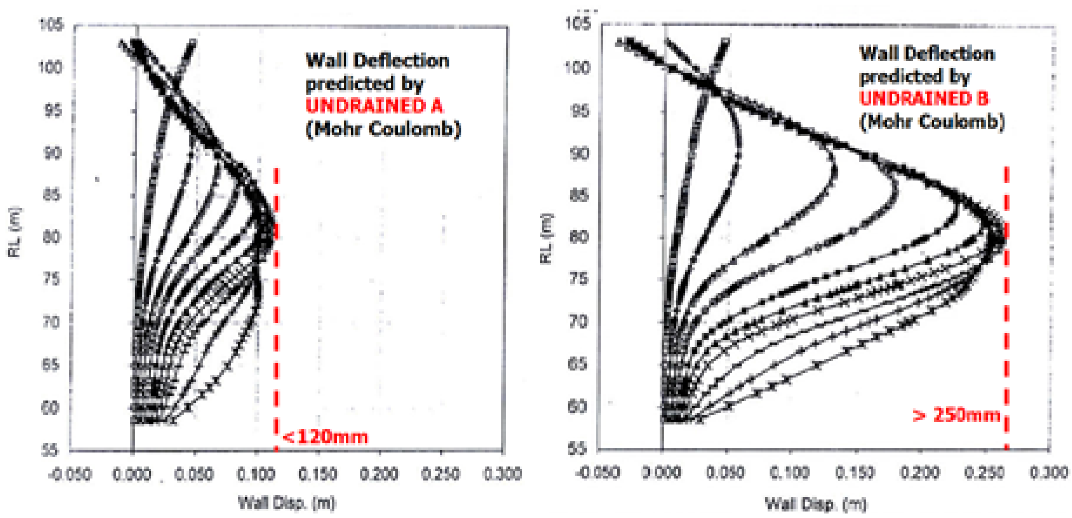


Figure 11. Wall Deflection Predicted by Undrained A and B (Richard Magus, et al 2005)

Singapore Nicoll highway deep excavation failure on April 21, 2004 gives very valuable lessons in modeling the undrained behavior of soft clays. The investigation report revealed the importance of analyzing both Undrained A and Undrained B methods (Magus et al, 2005) as presented in Figures 11 and 12 which show the comparison of the wall movement and the wall bending moment obtained from Undrained A and B. In this Nicoll highway case, the undrained B showed more critical results. The lesson learned is: while it is generally true that drained condition govern the safety of deep excavation retaining wall, when facing excavation in very soft clay, it is very important to check the undrained behavior (in both undrained A and B) as well.

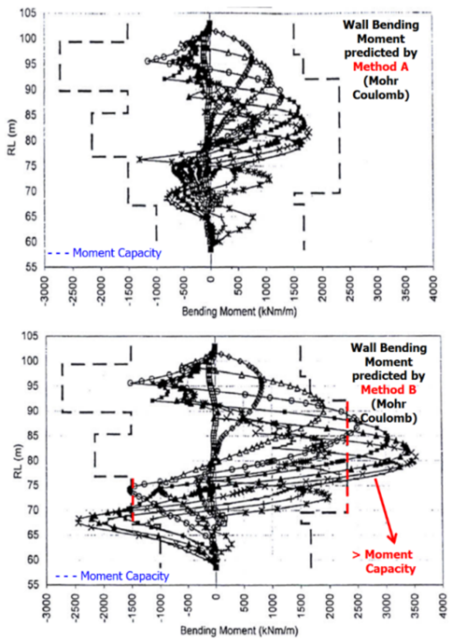


Fig. 12. Wall Bending Moment Predicted by Undrained A and B (Richard Magus, et al 2005)

3. Process Calculation

3.1 Initial Condition

Initially, when creating the finite element model, although the soil parameters has been assigned and the finite element mesh has been created, the soil body self-weight, i.e. the initial stresses, has not been counted for. A special procedure is necessary to generate or to calculate the initial stresses within the soil body. As the name implied, initially only the original soil body is in existence, therefore, all the structural elements and geometry changes, e.g.: backfilling, excavation, all structural elements, and groundwater changes (e.g. dewatering, sudden water drawdown) must not be activated.

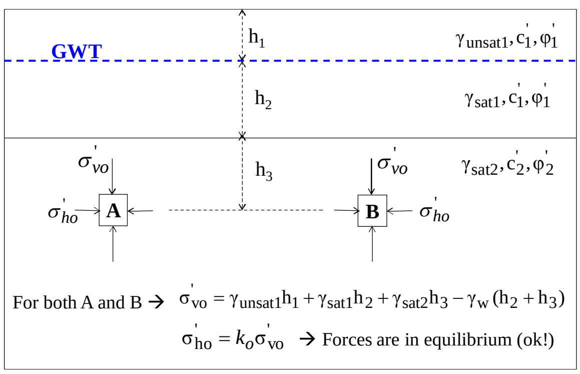


Fig. 13. K_o Procedure for Horizontal Geometry

Engineers, very often, again without understanding the proper theoretical knowledge, directly go through the so called k_o procedure, to generate the initial water pressure and the initial effective stresses of the ground. The k_o procedure, calculates the stresses within the soil body by the following simple equations:

$$\sigma'_{ho} = k_o \sigma_{vo}' \qquad (9)$$

Where σ_{ho}' is the horizontal earth pressure at rest, k_o is the coefficient of earth pressure at rest, σ_{vo}' is the effective vertical overburden pressure. This procedure is correct only and only when all the geometry of the ground surface, the ground layers, and the ground water table are horizontal (Figure 13).

Where the ground surface, the subsoil layer, or the ground water level is not horizontal, as shown in Figure 14, the k_o procedure will lead to the existence of unbalance forces or non-equilibrium of initial forces within the soil body, which are obviously not correct. In such cases, to maintain equilibrium, there should be shear stresses developed within the soil body. Therefore, the k_o procedure should not be used, instead a *gravity loading procedure*, where the shear stresses are calculated should be chosen.

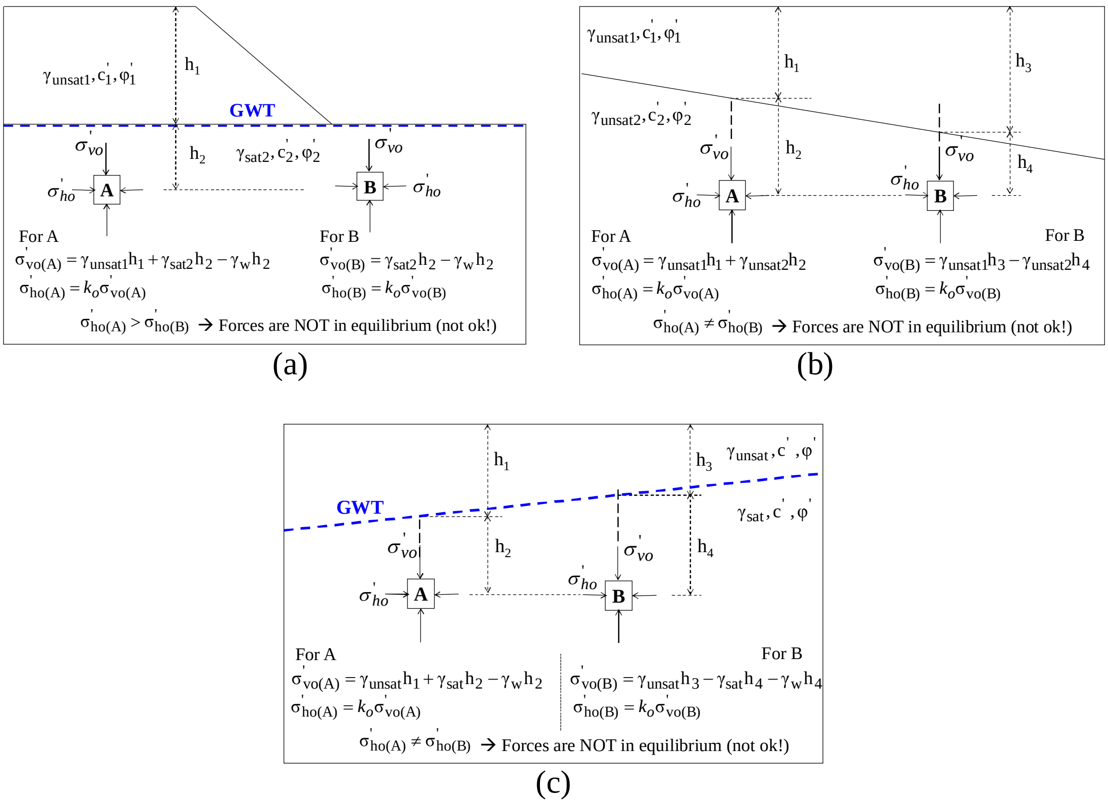


Figure 14. Cases where K_o Procedure is Inaccurate

The option of gravity loading and k_o procedure in the initial phase is only available in Plaxis 2D version 2011 and above. For Plaxis 2D version 9 and below, the gravity loading stage needs to be done by skipping the k_o procedure. This is done by setting ΣMweight=0 in the k_o procedure i.e. in the initial stage. This way no initial stresses within the soil body is developed. The initial stresses of the soil body is then calculated in the calculation module of the program by selecting the first phase as plastic 'Calculation type', and if any of the soil layer is modeled as undrained, the 'Ignore undrained behavior' option in the 'Parameter' tab has to be selected (this is due to the fact that initially, when no external load and no geometry changes is made, the soil is in a drained condition). In the 'Loading input' section, the 'Total multiplier' option is selected, and in the 'Multiplier' tab, key in ΣMweight=1. Then the next actual construction stages are modeled.

3.2 Ground Water Pressures

Gouw (2012) highlighted the importance of determination the ground water level and ground water pressures acting during deep excavation process. The effect of ground water seepage toward the excavated area is often overlooked or improperly modeled by inexperience design engineers and contractors and in many instances it can lead to the instability of the earth retaining structures.

To properly model the ground water seepage on a deep excavation problem, engineers must first understand whether water can pass through the retaining wall or not and whether the retaining is installed as a water cut-off system. For an excavation with impervious retaining walls, e.g. diaphragm walls or secants piles, where the toe of the walls is located in a relatively permeable soil layer, then the walls will not act as a water cut-off system. This means, during dewatering and excavation process, water can seep from outside the walls into the excavation area through the permeable soil layer below the walls' toe as shown in Figure 15. This ground water seepage creates seepage force which increases the effective overburden pressure in the active side of the walls, and on the other hand, reduces the effective stress in the passive side of the walls. This means the seepage force increases the lateral pressure to the walls and decreases the passive pressure. A large seepage force may significantly reduce the effective overburden pressure and subsequently may lead to piping and boiling.

Figure 16 shows the Plaxis modelling procedures for the bottom seepage of an impermeable wall. In the water analysis mode of the corresponding stage of calculation phase, the first step is to activate the interface along the wall. The activation of the interface in the soil mode and the water mode are independent each other, active interface in soil mode does not mean it must be active in water mode, and vice versa.

In the soil mode the active interface means to reduce the contact friction and allow slippage within soil and wall, in water mode active interface means the wall is impermeable and inactive interface means water can pass through the wall.

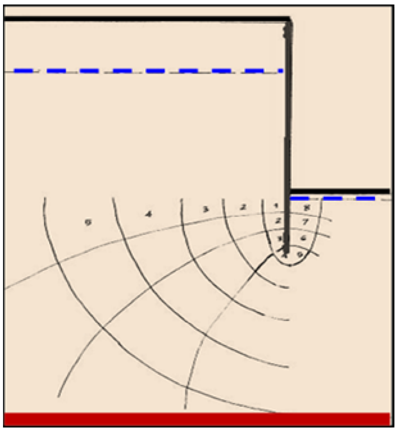


Fig. 15. Impermeable Wall and Bottom Seepage

The second step is to draw the general water line (phreatic level) in the so called z method. In the example given in Figure 16, at the level of the existing ground water, draw the horizontal water level from the left hand side boundary to any point between the left boundary to the wall, but not on the point at the wall itself. Then, continue draw the phreatic water line in a slanted way to a point at the wall on the level of the planned water level on the excavation side, followed by drawing the water line horizontally to the right boundary (at the planned water level). Plaxis will automatically calculate the water level drawdown on the unexcavated side of the wall.

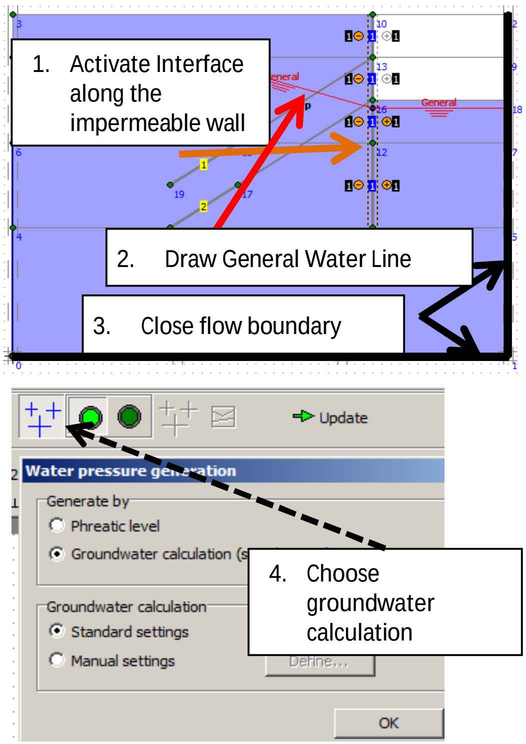


Figure 16. Modelling Impermeable Wall and Bottom Seepage

The third step is to restrict the ground water flow along the boundary of the system considered. The option is carried out by choosing the closed flow boundary in Plaxis v8 and v9 (the thick black vertical line icon) or by closed boundary icon in Plaxis v2011 and above (the thick slanted black vertical line icon). In this example, the left hand side boundary should not be closed, because it is where the water comes from. The bottom boundary has to be closed as the water will not flow out to the bottom. The right hand boundary, as it is the line of symmetry (the model only draw half of the excavation), there will be no flow crossing the boundary, therefore, it should be closed. The last step is to choose the groundwater calculation in Plaxis v8 and v9, or to choose the steady state groundwater flow in Plaxis v2011 and above. The example of the calculation result is presented in Figure 17. One note to be added here is the correct input of the subsoil permeability is important.

If the retaining wall is of permeable type such as: soldier piles where there is a gap in between the piles where water can seep through (Figure 18). Then, in water mode, the interface along the wall should be deactivated as shown in Figure 19. In such a case, example result of the analysis is presented in Figure 20. Comparing Figure 17 and Figure 20, it can be seen that the leaked wall causes deeper water drawdown on the unexcavated side. One note that should be highlighted here is Plaxis cannot predict whether there is any erosion of soil particles behind the wall or not.

If an impermeable retaining wall is embedded into an impermeable layer and/or the excavation time is relatively short compared to the speed of the water to permeate from the soil below the wall, there will be unbalance water pressure within the active and passive sides as shown in Figure 21. Here the wall also functions as water cut off system; hence the modeling follows the steps presented in Figure 22.

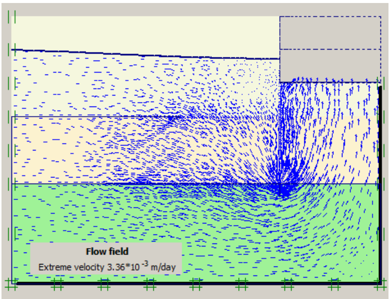


Figure 17. Predicted Drawdown due to Bottom Seepage

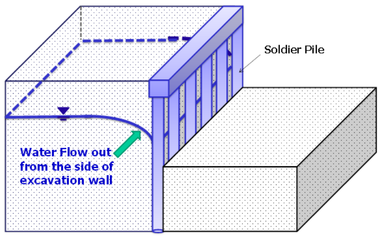


Figure 18. Modeling Permeable (leaked) Wall

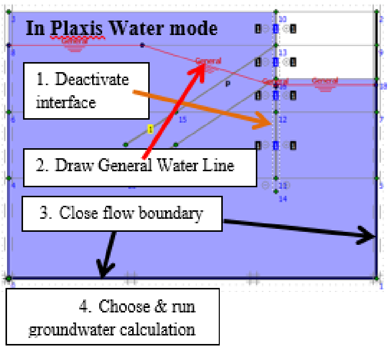
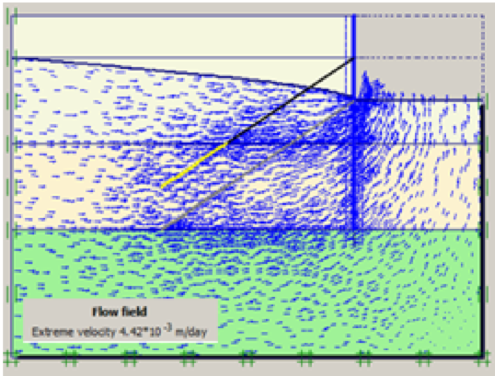


Figure 19. Modeling Permeable (leaked) Wall



Figre 20. Groundwater Flow through Permeable Wall

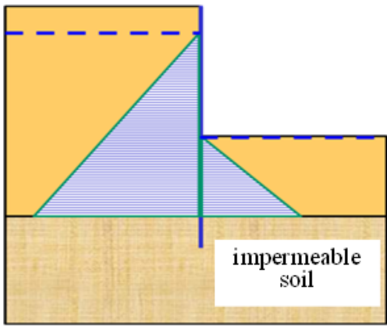


Figure 21. Unbalance Water Pressure

In this case of water cut off wall system, there will be no ground water flow, therefore, the activation or deactivation of interface element along the soil wall system in the water mode is not important. As there is no groundwater flow, the generation of the water pressure should be done by choosing the phreatic level option and not groundwater flow option. After the calculation is done, the resulting groundwater pressure at both sides of the wall can be obtained by clicking the interface along the wall (Figure 23).

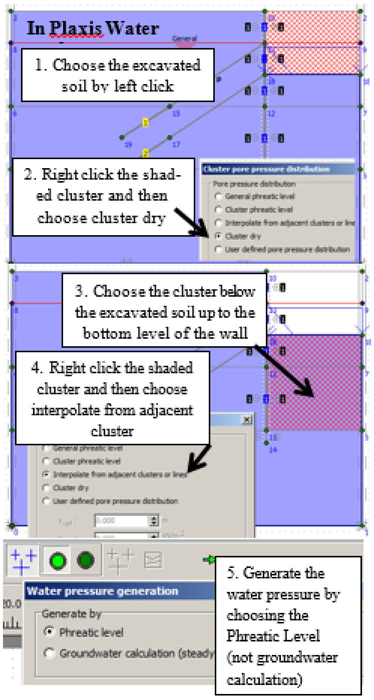


Figure 22. Steps in Modeling Unbalance Water Pressure

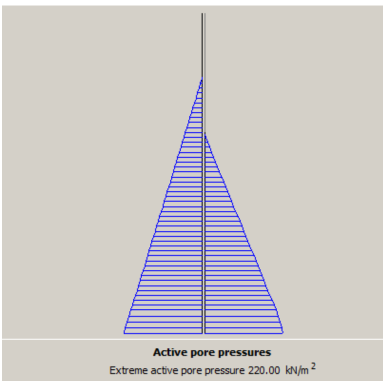


Figure 23. Unbalance Water Pressure in Water Cut-off System

To get a smooth distribution of the water pressure on the excavated side, it is suggested to draw a horizontal geometry line right at the toe of the wall during the input stage. It is very often that the interpolate from adjacent cluster step is forgotten. In such a case, the resulting water pressure will not start from zero at the bottom of the excavation, and of course it is not correct (see Figure 24).

3.3 Soil Parameter

Like in any other geotechnical analysis, a reasonably accurate input of soil parameters is very important, otherwise, the calculation shall be as good as the adage says: “Garbage in garbage out.” Some notes on the effect of soil parameters on the performance of the retaining wall are elaborated here.

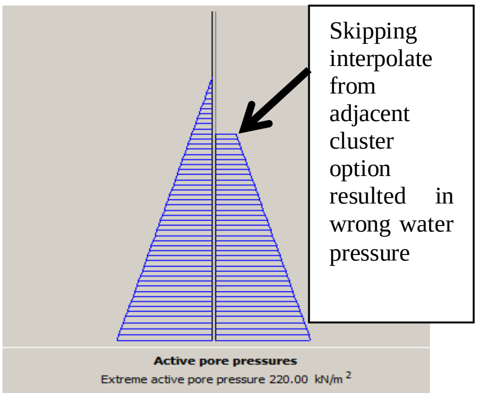


Figure 24. Incorrect Unbalance Water Pressure Calculation

3.3.1 Cohesion Parameter, c’ or cu

The basic formula for calculation of lateral earth pressures acting on a retaining wall requires the input of the soil cohesion, as presented in the equations below:

$$P_a = k_a\sigma_v' - 2c'\sqrt{k_a} \tag{10}$$

$$P_p = k_p\sigma_v' - 2c'\sqrt{k_p} \tag{11}$$

where:

P_a = Active Earth Pressure,
P_p = Passive Earth Pressure,
k_a = Active Earth Pressure Coefficient,
k_p = Passive Earth Pressure Coefficient,
σ_v’ = Effective Overburden Pressure, and
c’ = Drained Cohesion.

The c’ value in the above formula reduces the active earth pressure, on the other hand it increases the passive earth pressure, hence, over estimating c’ will lead to unsafe condition. One has to understand that in soft normally consolidated soils, c’≈0, and even if the triaxial test results show the existence of c’, which is normally the case of unsaturated samples, it is suggested to take c’ = 0.

In total stress analysis where the undrained cohesion c_u prevailed, the k_a and k_p values for soft clay is equal to one, because the undrained internal angle of friction, ϕ_u = 0. Note that soil investigation reports often show $\phi_u > 0$, this happened because the samples tested were not in a fully saturated condition; the water content somehow had reduced during the preparation or the keeping of the soil samples, whereas in situ soft soils are generally in a fully saturated condition.

3.3.2 Coefficient of Earth Pressure at Rest, k_o

The value of coefficient of earth pressure at rest, $k_{o,}$ has the effect on the predicted bending moment acting along a retaining wall. Higher k_o value means higher horizontal stresses, which in turn leads to higher bending moment.

3.3.3 Interface Value, R_{inter}

Numerical exercises show that the lower the interface value, R_{inter}, the larger the bending moment. Therefore, it is important to estimate a reasonably "right" value for this interface or friction reduction factors, R_{inter}. Table 2 shows the suggested reduction values.

Table 2. Suggested Reduction Factors, Rinter (Brinkgreeve and Shen, 2011)

Interaction sand/steel	= Rinter ≈ 0.6 – 0.7
Interaction clay/steel	= Rinter ≈ 0.5
Interaction sand/concrete	= Rinter ≈ 1.0 – 0.8
Interaction clay/concrete	= Rinter ≈ 1.0 – 0.7
Interaction soil/geogrid (grouted body) (interface is not necessary)	= Rinter ≈ 1.0

3.4 Other Factors

3.4.1 Elastic vs Elastoplastic Plate Element

In excavation problem the plate element is used to model the sheet pile walls, soldier piles, secant piles, diaphragm walls, and basement walls. There are two options in assigning the material properties of the plate element. The first option is to assume the material behaves elastically; another option is to assume elastoplastic material behavior. If the material is assumed to be elastic, then only the input of flexural stiffness, EI, and the axial stiffness EA, are required (note: E = Young modulus of the wall material, I = moment of inertia per m run of wall, and A = cross sectional area per m wall). When elastoplastic material model is adopted, two additional parameters are required, i.e., Ultimate axial force and ultimate moment of the retaining wall system is required. Figure 25 shows the comparison of the two options. It can be seen that elastic option may violate the limit of the structural strength. Therefore, whenever possible it is better to input the material behavior in elastoplastic model.

3.4.2 Weight of Plate Element

The weight of plate element is not the full weight of the structures, but it is the weight of the structures minus out the weight of the soil removed. Brinkgreeve & Shen, 2011, suggested the input as presented in Figures 26 for the case of wall in the soil and for the case of excavated soil on one site of the wall.

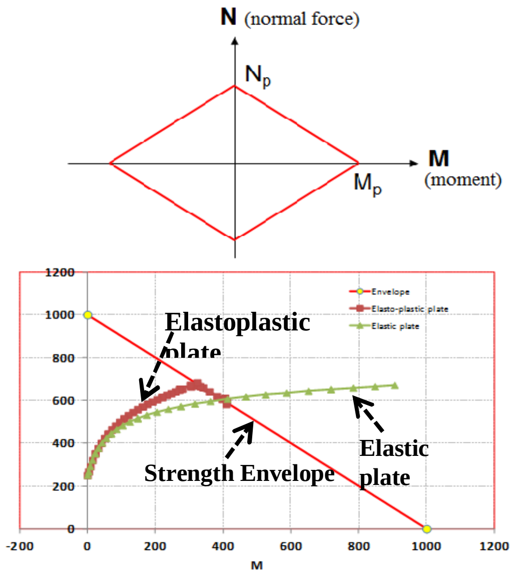


Figure 25. Elastic Plate vs Elastoplastic Plate (after Brinkgreeve & Shen, 2011)

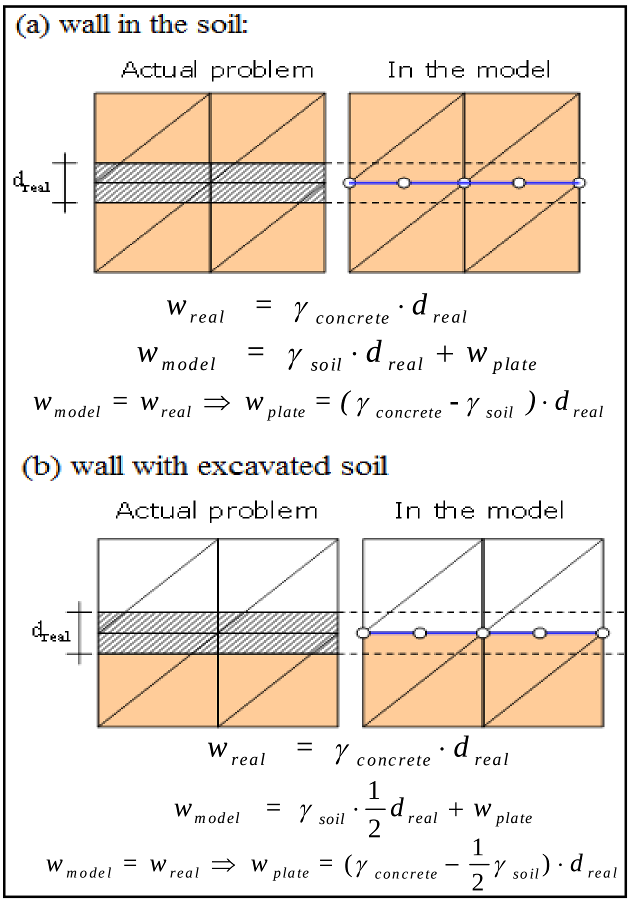


Figure26. Wall within the Soil Mass

4. Conclusion

The advance of computer technology and the availability of the FEM geotechnical software provide engineers with sophisticated tools for analyzing geotechnical problems on hand. However, like a pilot with inadequate training, it would backfire if he were to fly a sophisticated jet fighter. The author himself, along the years of learning and applying the geotechnical FEM software, has made many mistakes. Some of the mistakes often encountered and the correct way of modeling deep excavation are elaborated in this paper. Finally, the success of analyzing deep excavation with FEM greatly depends on good understanding of soil mechanics, the soil behavior and its relevant parameters, the structural properties, and also on the background of the software on hand. In short: Treat the soil as a lady and the comfort is yours! Treat the soil as step children, the revenge is waiting for you!

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