

# Sensitivity study of the hardening soil model parameters based on idealized excavation

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**ABSTRACT:** The paper presents a study of the sensitivity of the hardening soil model (HSM) parameters to a change of values in an idealised excavation in normally consolidated soft clay soil. The HSM is a constitutive elasto-plastic-cap model which is presently implemented in the PLAXIS finite element program. By varying one parameter and keeping the other parameters constant, the influence of each parameter on the performance of the excavation can be studied. In this way, it can be proven whether the model parameters did perform exactly as they may theoretically be expected to perform.

## 1 Introduction

The influence of the hardening soil model parameters under drained conditions on the stress-strain and volume change behaviour has been discussed in the forgoing article in this proceeding (Gebreselassie & Kempfert, 2005) for a triaxial and one dimensional compression loading condition. The influence of these parameters on the performance of an idealised excavation are investigated in this paper. By varying one parameter and keeping the other parameters constant, the influence of each parameter on the performance of the excavation can be studied.

This study restrict itself to normally consolidated soft clays only, however, the result of the study may also apply to other type of soils. The outcome of the sensitivity study may help the user to have a clear picture of the influence of each parameter of a hardening soil model on the performance of an excavation. It helps to judge the confidence interval of the variation of the soil parameters

## 2 The idealised excavation problem

In order to perform the sensitivity study of the model parameters, an idealised excavation shown in Figure 1a has been chosen. The ground is assumed to be a deposit of a homogeneous lacustrine soft soil with the ground water table located at 1.5 m below the ground surface. The excavation 6.0 m deep is supported by a sheet pile wall of the type Hoech 134 which has a total length of 12.0 m and an embedment depth of 6 m. The wall is supported by two level of struts of the type IPB 360 St 37. A building load of 50 kN/m<sup>2</sup> at a distance of 3 m behind the wall and a traffic load of 10 kN/m<sup>2</sup> are assumed at the ground surface.

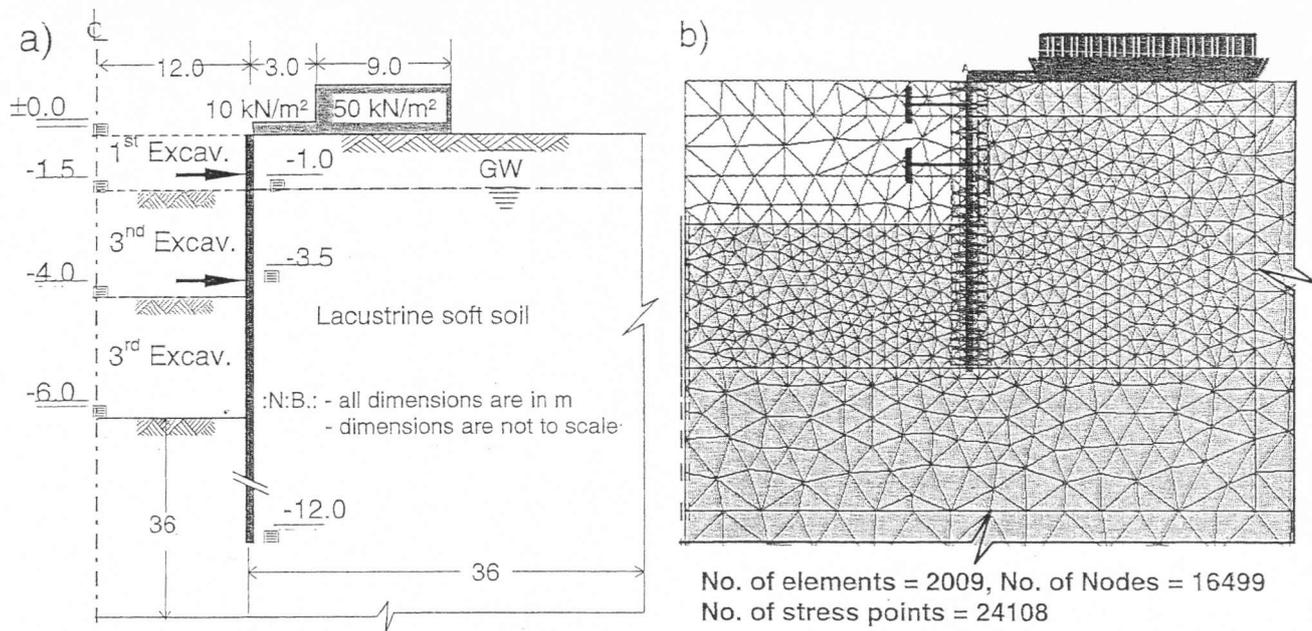


Figure 1. a) Idealized excavation, b) the finite element model and its mesh

### 3 The constitutive soil model

The constitutive model to which the soil parameters are being calibrated in this paper is an elasto-plastic-cap soil model known as the hardening soil model (HSM). The HSM is implemented in the finite element code for soils and rocks "PLAXIS" (Brinkgreve, 2002). It is originally developed based on the so called the Duncan-Chang hyperbolic model. It, however, supersedes the hyperbolic model, because it uses the plasticity theory instead of the elasticity theory, it includes the dilatancy soil behaviour and it introduces the yield cap. The HSM also considers the stress dependant stiffness of the soil according to the power law. For detail Information on the constitutive model and the program PLAXIS refer to Brinkgreve (2002) (see also Gebreselassie & Kempfert, 2005).

As mentioned above, the HSM is used to simulate the soil behaviour, whereas a Mohr-Coulomb Model (MCM) is used for the interface elements. A drained type of analysis is chosen, because it is believed that this condition is most unfavourable condition for excavations in soft deposits. The reference soil parameters required for the HSM are given in Table 1 and the soil parameters for the interface elements according to the MCM are given in Table 2. The stiffness of the soil is adopted as it is for the interface element, whereas the shear strength parameter are reduced by a factor of 1/3. The wall and the struts are assumed to behave linear elastic with the following material properties:

Wall:  $EA = 3.591 \times 10^6$  kN/m,  $EI = 5.355 \times 10^4$  kN-m<sup>2</sup>/m,  $w = 1.34$  kN-m/m,  $\nu = 0.30$

Strut:  $EA = 3.801 \times 10^6$  kN,  $L_{\text{spacing}} = 2.0$  m

Table 1. Reference soil parameters for the HSM

$\gamma_{\text{sat}}$	$\varphi'$	$c'$	$E_{30}^{\text{ref}}$	$E_{\text{oad}}^{\text{ref}}$	$E_{\text{ur}}^{\text{ref}}$	$p^{\text{ref}}$	$m$	$R_f$	$K_0^{\text{nc}}$	$\nu_{\text{ur}}$
[kN/m <sup>3</sup> ]	[°]	[kN/m <sup>2</sup> ]	[kN/m <sup>2</sup> ]	[kN/m <sup>2</sup> ]	[kN/m <sup>2</sup> ]	[kN/m <sup>2</sup> ]	[-]	[-]	[-]	[-]
19.5	25.3	13.2	3253	2948	19170	100	0.63	0.83	0.573	0.20

Table 2. Reference soil parameters for the interface elements according to the MCM

$\gamma_{sat}$ [kN/m <sup>3</sup> ]	$\delta = \frac{1}{3} \cdot \varphi'$ [°]	$c'$ [kN/m <sup>2</sup> ]	$E_{ref}$ [kN/m <sup>2</sup> ]	$E_{increment}$ [kN/m <sup>2</sup> ]	$\gamma_{ref}$ [kN/m <sup>2</sup> ]	$c_{ref}$ [kN/m <sup>2</sup> ]	$\nu_{ur}$ [-]
19.5	8.43	4.4	3253	0	0	0	0.35

#### 4 The finite element model and the calculation stages

An important part of the finite element model is shown in Figure 1b. The model is extended to a depth of 42 m where a fixed boundary is imposed. At a distance of 36 m behind the wall and at the symmetry axis, a zero horizontal displacement is imposed. The size of the model as a whole is 48 m wide and 42 m high. Triangular elements with 15 nodes are used in generating the mesh. This element provides a fourth order interpolation for displacements and it involves twelve numerical integration stress points (Gauss points). The model consists of 2009 elements, 16499 nodes and 24108 stress points.

For the drained analysis of the idealised excavation problem, the HSM parameters in Table 1 are adopted as a reference parameters for the soil body. In order to study the sensitivity of the soil parameters, their values are varied above and below the reference values. The contact between the wall and the soil body is simulated by mean of interface elements whose material properties are given in Table 2.

The following construction stages has been followed in the computation:

- Stage 0: generation of the initial stresses ( $K_0$  - method)
- Stage 1: application of the surcharge and traffic loads
- Stage 2: installation of the wall
- Stage 3: first excavation
- Stage 4: installation of the 1st strut and 2nd excavation
- Stage 5: installation of the 2nd strut and 3rd excavation

#### 5 Analysis of the computation results

##### 5.1 The effect of the variation of the Poisson's ratio $\nu_{ur}$

As it can be seen from Figure 2, the parameter  $\nu_{ur}$  seems to be a pure deformation parameter. In other words,  $\nu_{ur}$  may affect the deformation of the wall and soil movements but not the earth pressure and bending moment of the wall. A change in  $\nu_{ur}$  from its reference value of 0.2 to a smaller

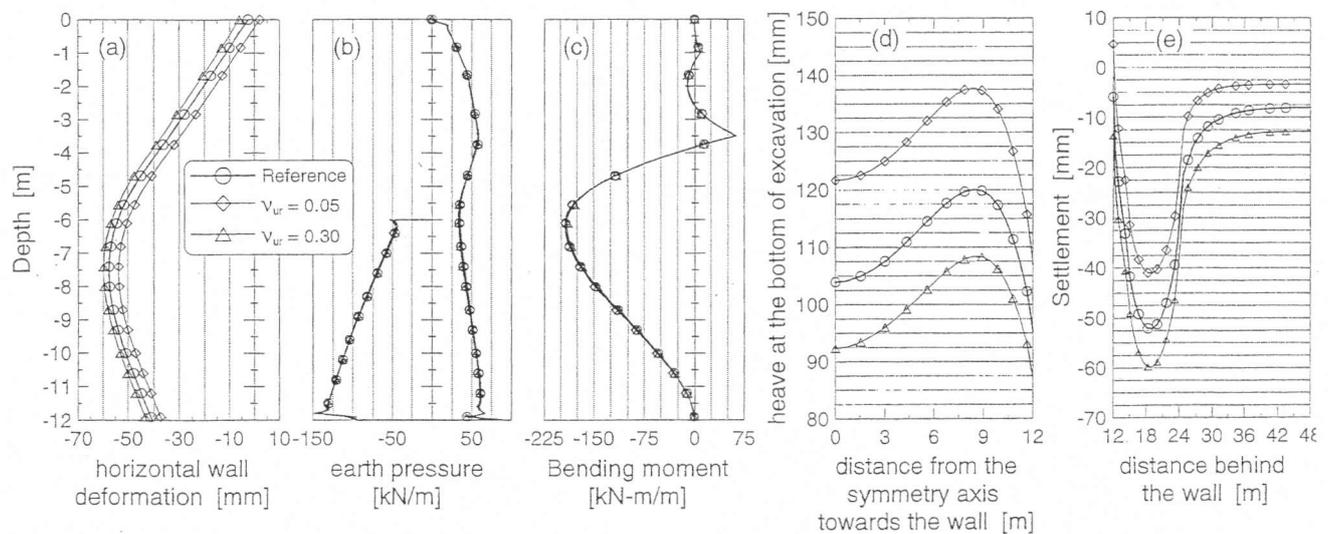


Figure 2. The effect of the variation of  $\nu_{ur}$  on a) deformation of the wall, b) earth pressure, c) bending moment of the wall, d) heave of the bottom of excavation, and e) settlement at the surface

value of 0.05 and a larger value of 0.3 has resulted in a uniform change of the wall deflection by about -7 and 4% respectively. Similarly, a change of the heave by about 15 and -10%, and a change of the surface settlement by about -21 and 15% respectively are calculated.

## 5.2 The effect of the variation of the coefficient of the earth pressure at rest $K_0^{nc}$

The HSM treats  $K_0^{nc}$  and  $K_0$  separately. Whereas  $K_0^{nc}$  is a model parameter which is closely related to the stiffness parameters  $E_{50}$ ,  $E_{ur}$ ,  $E_{oed}$  and  $\nu_{ur}$ ,  $K_0$  is purely used to define the initial state of the stresses. For normally consolidated soft soils, however, these values are more or less the same. The value of  $K_0^{nc}$  as a model parameter can not be varied indefinitely. For example, for the given reference parameters, the minimum and maximum possible values of  $K_0^{nc}$  are 0.437 and 0.71 respectively. Here  $K_0$  is assumed to vary with  $K_0^{nc}$ . As it can be seen from Figure 3, the parameter  $K_0^{nc}$  would affect the deformation of the wall, the soil movements, the earth pressure and bending moment of the wall, although the magnitude of its influence is moderate as compare to the triaxial case of loading (Gebreselassie & Kempfert, 2005). Varying the value of  $K_0^{nc}$  from the reference value of 0.573 to those extreme values has resulted in a change of the maximum wall deformation of about -21 and 3% respectively. Similarly, a change of the heave by about 2 and -9%, a change of the surface settlement by about -5 and 7%, a change of the earth pressure by about -21 and 8%, and a change of the bending moment by about -18 and 2% respectively are calculated. From the above percentage difference presentation and the Figure 3, it appears that varying the  $K_0^{nc}$  value towards the lowest limit is more sensitive than varying its value towards the upper limit, although the difference between the reference value and the extreme values is almost the same.

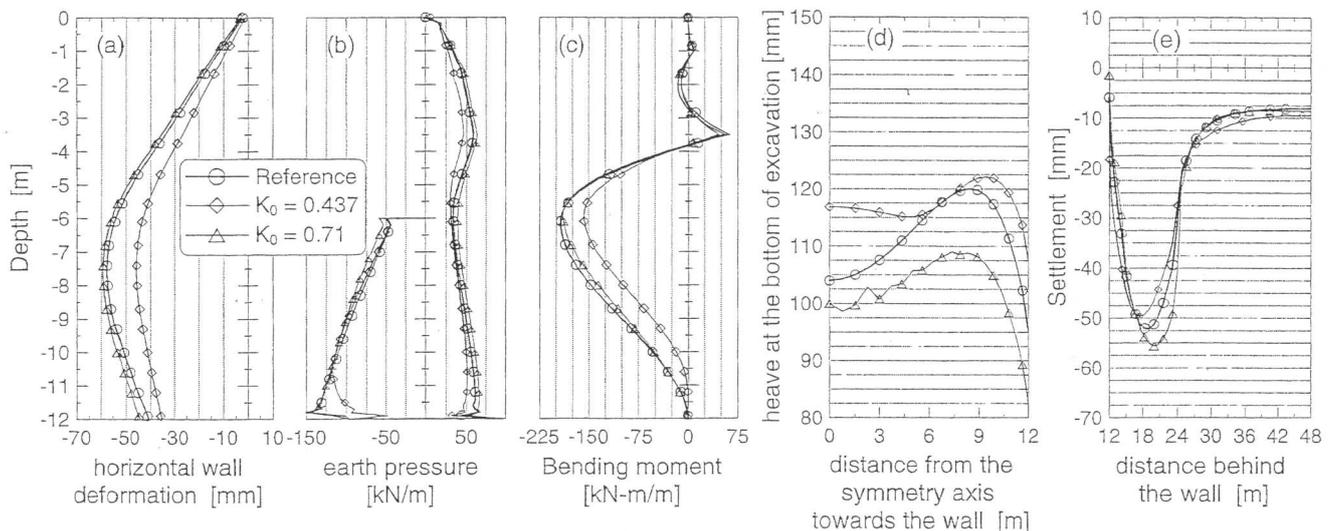


Figure 3. The effect of the variation of  $K_0^{nc}$  on a) deformation of the wall, b) earth pressure, c) bending moment of the wall, d) heave of the bottom of excavation, and e) settlement at the surface

## 5.3 The effect of the variation of the failure factor $R_f$

In triaxial and oedometer loading conditions, it has been proved that the failure factor  $R_f$  plays an important role in enhancing or retarding the failure of the soil body (Gebreselassie & Kempfert, 2005). Its influence on the idealised excavation, however, seems to be minimum, with exception of the settlement behind the wall (Figure 4)). Varying the value of  $R_f$  from the reference value of 0.83

to 0.67 and 0.97 has resulted in a change of the surface settlement by about -8 and 6% respectively. For all the other cases, the difference remains below  $\pm 5\%$ .

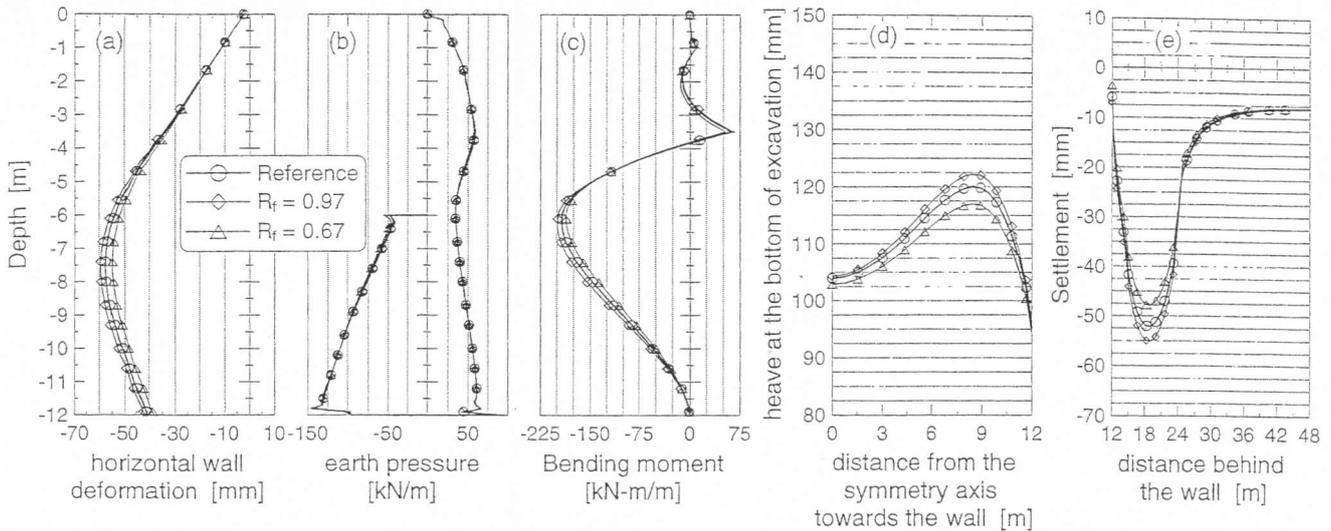


Figure 4. The effect of the variation of  $R_f$  on a) deformation of the wall, b) earth pressure, c) bending moment of the wall, d) heave of the bottom of excavation, and e) settlement at the surface

#### 5.4 The effect of the variation of the constrained modulus $E_{oed}$

As shown in Figure 5, a variation of the constrained modulus  $E_{oed}$  by about  $\pm 50\%$  its reference value, has resulted in a change of the maximum wall deflection by about -30 and 5% respectively. Similarly, a change of the heave by about -17 and -4%, a change of the surface settlement by about -24 and 2%, a change of the maximum earth pressure above the bottom of excavation by about -17 and 2%, and a change of the bending moment by about -23 and 3% respectively has been observed (Figures 5). Hence, the following conclusion may be drawn with regard to the response of the excavation to the change of  $E_{oed}$ .

- In all cases  $E_{oed}$  is more sensible to a change of value below the reference value than to value greater than the reference. It can be seen from Figure 6 that a reduction of the reference value of  $E_{oed}$  by 50% has caused a reduction of the wall and soil movements, the active earth pressure and the bending moment by about 17 to 30%, whereas increasing the reference value by same amount (50%) show no significant influence (2 to 5%). It seems that the ratio of  $E_{50}/E_{oed}$  is more important than the absolute value of the  $E_{oed}$ . For the reference case, this ratio becomes 1.1. If the  $E_{oed}$  is increased or decreased by about 50%, the ratio becomes 0.73 and 2.20 respectively. The ratio in the case of increasing  $E_{oed}$  is more closer to the reference ratio than the other way round. This might be the reason why the change of  $E_{oed}$  is more sensible to a value below the reference than above the reference value.
- Contrary to expectation, a reduced value of  $E_{oed}$  has resulted in a reduction of wall and soil movements.
- Figure 6b shows a reduced active pressure and an increased passive pressure for the case of  $E_{oed}$  smaller than the reference value. This again contradicts with the reduced wall movement that is discussed in (b). A reduced wall movement would have resulted a higher active pressure and lower passive pressure.
- A reduced active pressure on one side and an increased passive pressure on the other side has resulted in a reduced bending moment, which seems logical in respect to the given loading condition but not in a general sense.

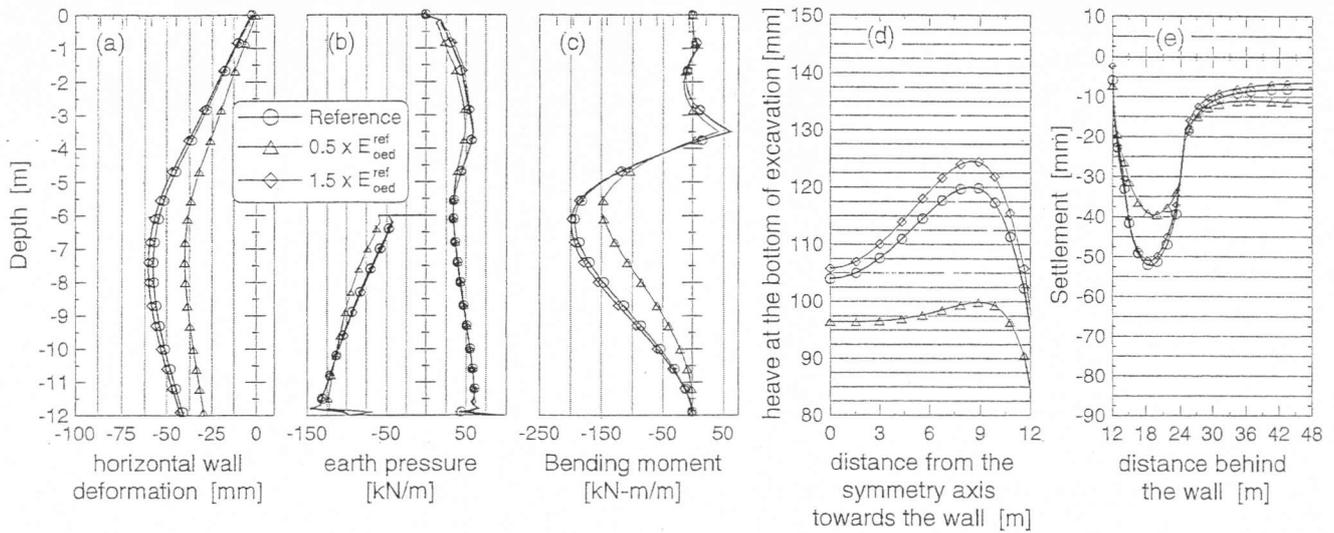


Figure 5. The effect of the variation of  $E_{oeed}$  on a) deformation of the wall, b) earth pressure, c) bending moment of the wall, d) heave of the bottom of excavation, and e) settlement at the surface

### 5.5 The effect of the variation of the un/reloading modulus of elasticity $E_{ur}$

The reference value of  $E_{ur}$  was directly taken from triaxial test result and it is equal to  $5.9 \cdot E_{50}^{ref}$  (Gebreselasie, 2003). Lowering the reference value to  $3 \cdot E_{50}^{ref}$ , which is usually recommended in practice with the absence of a test result, and further lowering the reference value to  $2 \cdot E_{50}^{ref}$  have resulted in an increase of the displacement of the toe of the wall by about 27 and 63% respectively. Similarly, a change of the heave of the bottom of excavation by about 75 and 150%, a change of the surface settlement by about -15 and 30%, a change of the active pressure above the bottom of excavation by about -16 and 22% respectively, and an insignificant change of the maximum bending moment (below 2.5 %) are calculated (Figure 6). The earth pressure below the excavation level on both active and passive side also shows no significant change relative to the reference value. Contrary to the expectation, the settlement at the surface for the reduced values of  $E_{ur}$  is less

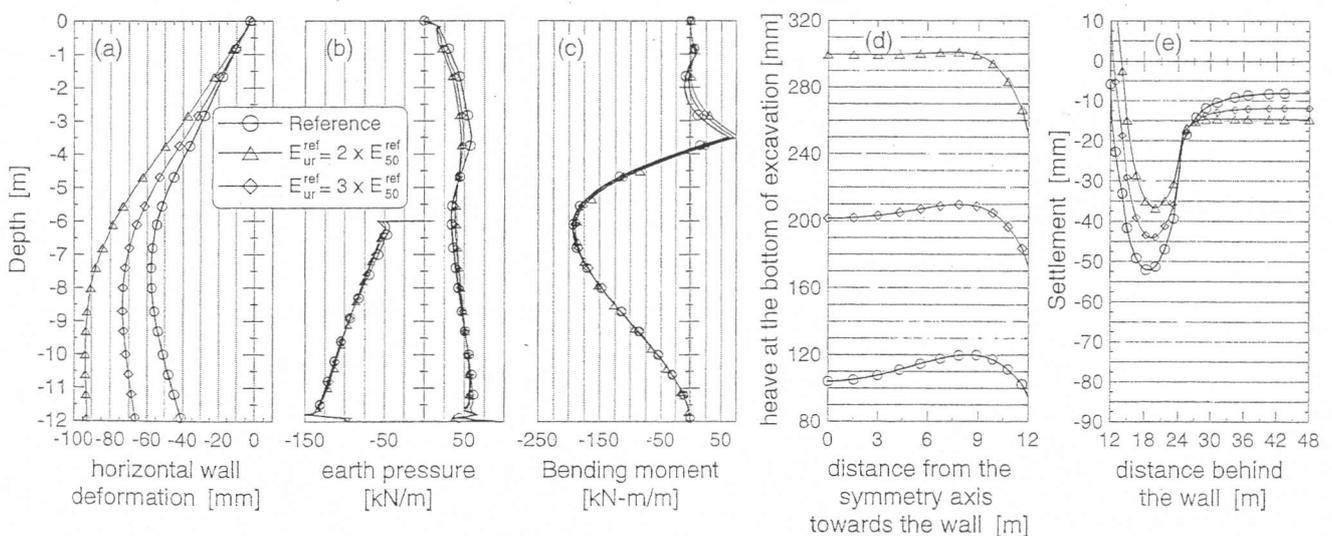


Figure 6. The effect of the variation of  $E_{ur}$  on a) deformation of the wall, b) earth pressure, c) bending moment of the wall, d) heave of the bottom of excavation, and e) settlement at the surface

than that from the reference value. This is mainly due to the upward displacement of the wall. The whole soil body seems to heave upwards due to lower values of  $E_{ur}$ .

## 5.6 The effect of the variation of the secant modulus of elasticity $E_{50}$

Figure 7 shows the effect of the variation of  $E_{50}$  by  $\pm 50\%$  from its reference value. These variations of  $E_{50}$  have resulted in a change of the maximum wall deflection by about 45 and -24% respectively. Similar change of the heave by about 21 and 11%, the surface settlement by about 71 and -37%, the maximum earth pressure above the bottom of excavation by about 19 and -15%, and the maximum bending moment by about 27 and 18% respectively has been observed.

At first glance, it seems that an increased wall movement should result in higher passive resistance, because the soil is more close to the passive limit state. However, as the numerical study of the mobilisation of the passive resistance (Gebreselassie, 2003; Gebreselassie & Kempfert, 2005) also shows, the passive resistance is lower for lower values of the modulus for a given displacement of the wall and keeping the shear parameter constant. This is exactly what one can observe in Figure 7. Lower value of  $E_{50}$  leads to higher wall movement but a lower passive resistance and vice versa.

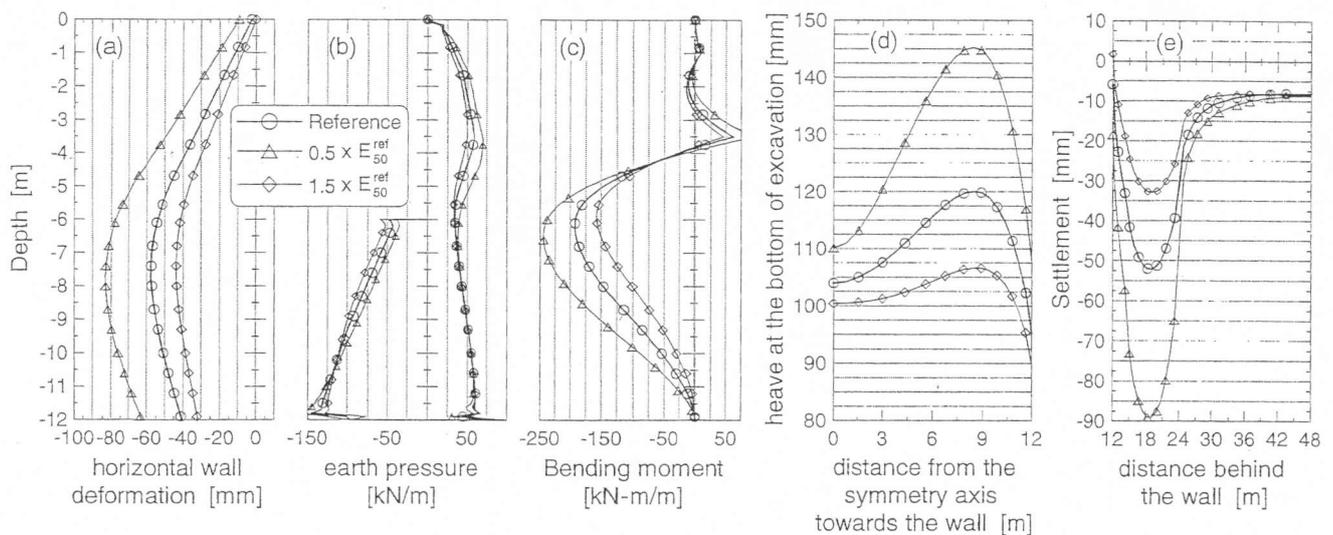


Figure 7. The effect of the variation of  $E_{50}$  on a) deformation of the wall, b) earth pressure, c) bending moment of the wall, d) heave of the bottom of excavation, and e) settlement at the surface

## 6 Summary

A sensitivity study of the soil parameters for HSM has been conducted based on an idealised excavation in normally consolidated cohesive soils in order to study the influence of these parameters on the performance of an excavation. The result of the study may be summarised as follows:

- $E_{50}$  seems to lead the role of influencing the wall displacement, the earth pressure, the bending moment and the settlement behind the wall. Due to the non-linearity, however, increasing its value does not necessarily produce the same effect as the other way round. Its influence on bottom heave is limited only to the heave near the wall and its influence ceases towards the middle of the excavation.  $E_{ur}$  plays a dominant role on the heave of the excavation and the displacement of the wall toe, but it has insignificant influence on the bending moment.  $v_{ur}$  has only an effect on the bottom heave and settlement at the surface.  $R_f$  shows a negligible effect on all the cases.
- $K_0^{nc}$  value may affect the deformations, bending moment and the earth pressure, although

the magnitude of its influence is moderate as compare to the triaxial state of stress.  $K_0^{nc}$  value towards the lowest limit is more sensitive than varying its value towards the upper limit, although the difference between the reference value and the extreme values is almost the same.

- The sensitivity study of the  $E_{oed}$  shows that the ratio  $E_{50}/E_{oed}$  is more important than the absolute value of  $E_{oed}$ .

## 7 List of Symbols and Abbreviations

$E_{50}^{ref}$ = secant modulus at 50% of the failure stress and at effective reference pressure of $p^{ref}$	$R_f$ = ratio of the stress at failure and the ultimate stress
$E_{oed}^{ref}$ = constrained modulus at $p^{ref}$	$K_0^{nc}$ = coefficient of the earth pressure at rest for normally consolidated soils
$E_{ur}^{ref}$ = un/reloading modulus at $p^{ref}$	$\nu_{ur}$ = Poisson's ratio for un/reloading
$E$ = modulus of elasticity	$\nu$ = Poisson's ratio
$\gamma_{sat}$ = saturated unit weight of soil	$m$ = exponent in the power law
$\varphi'$ = effective angle of internal friction	HSM = Hardening Soil Model
$\delta$ = wall friction	MCM = Mohr-Coulomb Model
$c'$ = effective cohesion	

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