Influence of constitutive model and EC7 design approach in FE analysis of deep excavations

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1. INTRODUCTION

Advances in computer hardware and, more importantly, in geotechnical software over the past ten years have resulted in a widespread application of numerical methods in practical geotechnical engineering. These developments enable the geotechnical engineer to perform very advanced numerical analyses at low cost and with relatively little computational effort. Commercial codes, fully integrated into the PC-environment, have become so user-friendly that little training is required for *operating* the programme. They offer sophisticated types of analysis, such as fully coupled consolidation analysis with elasto-plastic material models. However, for performing such complex calculations and obtaining sensible results a strong background in numerical methods, mechanics and, last but not least, theoretical soil mechanics is essential. The potential problems arising from the situation that geotechnical engineers, not sufficiently trained for that purpose, perform complex numerical analyses and may produce unreliable results have been recognized within the profession and some national and international committees have begun to address this problem, amongst them the working group AK 1.6 "Numerical Methods in Geotechnics" of the German Society for Geotechnics (DGGT) and working group A "Numerical Methods" of the COST Action C7 (Co-Operation in Science and Technology of the European Union). The results of the work of the latter can be found in Potts et al. (2002). One of the main goals of AK 1.6 of the DGGT is to provide recommendations for numerical analyses in geotechnical engineering. In addition benchmark examples have been specified and the results obtained by various users employing different software have been discussed in Schweiger (2002). At present the influence of the constitutive model on results of simplified practical problems, such as deep excavations, tunnels and deep foundations, are investigated. In order to avoid the influence of implementation details only one fe-code (Plaxis) is used in the first step. Some results of this ongoing study are presented in the first part of the paper whereas a comparison of different EC7 design approaches employing numerical analysis is discussed in the second part of the paper.

2. DEEP EXCAVATION BENCHMARK EXAMPLE

2.1 Problem definition and calculation steps

The basic geometry of the investigated deep excavation is depicted in Figure 1. In order to study the effect of different constitutive models for various ground conditions four different (homogeneous) soil conditions are assumed, namely a loose to medium dense sand, a dense sand, an overconsolidated clay and a soft soil. Only the results for the dense sand and the soft clay will be discussed in this paper. For simplicity the wall (EA = 2.53E06 kN/m, EI = 3.02E4 kNm^2/m) and the strut (EA = 1.5E06 kN/m) have been assumed the same for all ground conditions, only the length of the wall and drainage conditions vary depending on the soil layer assumed. Wall friction was taken as 2/3 of the friction angle of the soil in all cases. The soil parameters have

been determined based on experimental results which can be considered to be representative for the respective soil.
The following calculation steps have been performed, but only results for the final stage are presented here.

Step 0: Initial stress state ($\sigma'_v = \gamma.h$, $\sigma'_h = K_0\,\sigma'_v$, $K_0 = 1 - \sin\varphi'$)
Step 1: Apply surcharge load
Step 2: Activate wall (wished-in-place), set displacements to zero
Step 3: Excavation to level -2.0 m
Step 4: Activate strut at level -1.5 m
Step 5: Lowering of GW-Table to -6.0 m inside excavation (only for example in dense sand)
Step 6: Excavation to level -4.0 m
Step 7: Excavation to level -6.0 m

For the excavation in the sand layer a deep hydraulic barrier is assumed at the level of the base of the wall and thus no seepage flow is considered (see Figure 1). The analysis considering the soft soil layer has been performed under undrained conditions and it has been assumed that the water is excavated simultaneously with the soil and no modifications to the groundwater conditions are made. The original GW-table is assumed to be at -3.5 m for the sand and at -2.0 m for the clay (Figure 1). The clay above the water table has been modelled as drained material.

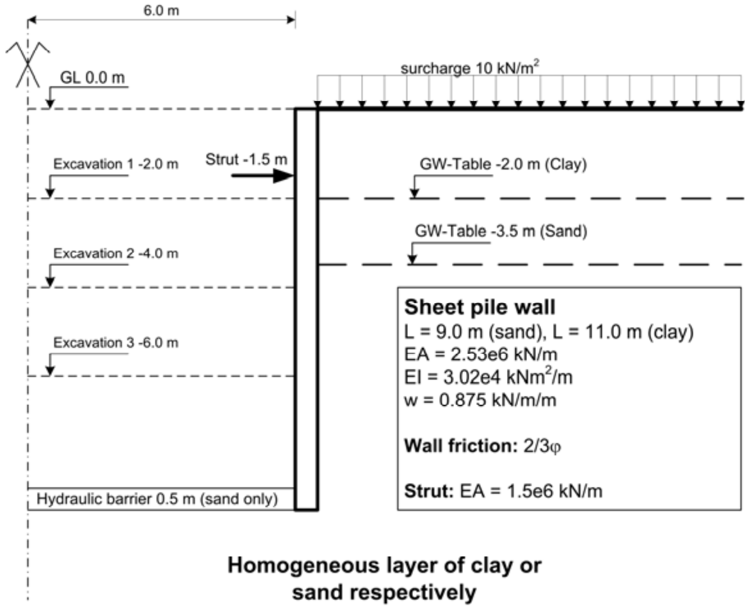


Figure 1 Geometry of benchmark example

2.2 Constitutive models and parameters

2.2.1 Sand layer

For the excavation in the sand layer three different constitutive models have been employed, namely the simple Mohr-Coulomb failure criterion (MC), the standard Plaxis Hardening Soil model (HS), which is a double hardening plasticity model, and finally the Hardening Soil Small model (HSS) which is the extension of the latter to account for small strain stiffness (Benz, 2007). The parameters used in the analysis are listed in Tables 1 to 3. Strength parameters are the same for all models but stiffness parameters are different. They are stress dependent in the HS and

HSS model, strain dependent in the HSS model in the small strain range but constant in the Mohr-Coulomb model. The average value of loading and unloading stiffness as used in the HS models have been assigned as stiffness in the latter.

Parameter		**Meaning**	**Value**
γ	[kN/m³]	Unit weight (unsaturated)	18
γ_{sat}	[kN/m³]	Unit weight (saturated)	20
φ'	[°]	Friction angle (Mohr-Coulomb)	41
c'	[kPa]	Cohesion (Mohr-Coulomb)	0
ψ	[°]	Angle of dilatancy	15
ν_{ur}	[-]	Poisson's ratio unloading-reloading	0.20
E_{50}^{ref}	[kPa]	Secant modulus for primary triaxial loading	30 000
E_{oed}^{ref}	[kPa]	Tangent modulus for oedometric loading	30 000
E_{ur}^{ref}	[kPa]	Secant modulus for un- and reloading	90 000
m	[-]	Exponent of the Ohde/Janbu law	0.55
p_{ref}	[kPa]	Reference stress for the stiffness parameters	100
K_0^{nc}	[-]	Coefficient of earth pressure at rest (NC)	1-sin(φ')
$\sigma_{Tension}$	[kPa]	Tensile strength	0

Table 1 Parameters for Hardening Soil Model (sand)

Parameter		**Meaning**	**Value**
E_0	[kN/m³]	Small-strain Young's modulus	270 000
$\gamma_{0.7}$	[kN/m³]	Reference shear strain where $E_{sec}=0.7E_0$	0.0002

Table 2 Additional Parameters for Hardening Soil-small Model (sand)

Parameter		**Meaning**	**Value**
γ	[kN/m³]	Unit weight (unsaturated)	18
γ_{sat}	[kN/m³]	Unit weight (saturated)	20
φ'	[°]	Friction angle (Mohr-Coulomb)	41
c'	[kPa]	Cohesion (Mohr-Coulomb)	0
ψ	[°]	Angle of dilatancy	15
ν	[-]	Poisson's ratio unloading-reloading	0.25
E	[kPa]	Young's modulus	60 000
$\sigma_{Tension}$	[kPa]	Tensile strength	0

Table 3 Parameters for Mohr-Coulomb Model (sand)

2.2.2 Clay layer

For the clay layer the Plaxis Soft Soil model has been used in addition to the Hardening Soil models and the Mohr-Coulomb model. The Soft Soil model is a modification of the well known

Modified-Cam-Clay model (MCC) incorporating a Mohr-Coulomb failure criterion and allowing for a modification of the volumetric yield surface in order to improve K_0-predictions. The parameters for all models are listed in Tables 4 to 7, for details of the models see Brinkgreve (2002).

Parameter		Meaning	Value
γ	[kN/m^3]	Unit weight (unsaturated)	15
γ_{sat}	[kN/m^3]	Unit weight (saturated)	16
φ'	[°]	Friction angle (Mohr-Coulomb)	27
c'	[kPa]	Cohesion (Mohr-Coulomb)	15
ψ	[°]	Angle of dilatancy	10
ν_{ur}	[-]	Poisson's ratio unloading-reloading	0.20
E_{50}^{ref}	[kPa]	Secant modulus for primary triaxial loading	4 300
E_{oed}^{ref}	[kPa]	Tangent modulus for oedometric loading	1 800
E_{ur}^{ref}	[kPa]	Secant modulus for un- and reloading	14 400
m	[-]	Exponent of the Ohde/Janbu law	0.90
p_{ref}	[kPa]	Reference stress for the stiffness parameters	100
K_0^{nc}	[-]	Coefficient of earth pressure at rest (NC)	1-sin(φ')
$\sigma_{Tension}$	[kPa]	Tensile strength	0

Table 4 Parameters for Hardening Soil Model (clay)

Parameter		Meaning	Value
E_0	[kN/m^3]	Small-strain Young's modulus	100 000
$\gamma_{0.7}$	[kN/m^3]	Reference shear strain where $E_{sec}=0.7E_0$	0.0003

Table 5 Additional Parameters for Hardening Soil-small Model (clay)

Parameter		Meaning	Value
γ	[kN/m^3]	Unit weight (unsaturated)	15
γ_{sat}	[kN/m^3]	Unit weight (saturated)	16
φ'	[°]	Friction angle (Mohr-Coulomb)	27
c'	[kPa]	Cohesion (Mohr-Coulomb)	15
ψ	[°]	Angle of dilatancy	0
ν	[-]	Poisson's ratio unloading-reloading	0.30
E	[kPa]	Young's modulus	9 350
$\sigma_{Tension}$	[kPa]	Tensile strength	0

Table 6 Parameters for Mohr-Coulomb Model (clay)

Parameter		**Meaning**	**Value**
ν	[kN/m³]	Unit weight (unsaturated)	15
γ_{sat}	[kN/m³]	Unit weight (saturated)	16
φ'	[°]	Friction angle (Mohr-Coulomb)	27
$\mathbf{c'}$	[kPa]	Cohesion (Mohr-Coulomb)	15
ψ	[°]	Angle of dilatancy	0
ν_{ur}	[-]	Poisson's ratio	0.20
κ^*	[-]	Modified swelling index	0.0125
λ^*	[-]	Modified compression index	0.0556
$\mathbf{K}_0^{nc}$	[-]	Coefficient of earth pressure at rest (NC)	1-sin(φ')
$\sigma_{Tension}$	[kPa]	Tensile strength	0

Table 7 Parameters for Soft Soil Model (clay)

2.3 Results for homogeneous sand layer

Figure 2 shows the lateral displacement of the sheet pile wall for the final excavation stage. It is observed that the MC model predicts the smallest displacements but of course this strongly depends on the chosen elasticity modulus. HS and HSS model show similar behaviour but including small strain stiffness effects reduces the maximum displacement slightly. It should be mentioned at this stage that the results from the HSS model may be quite sensitive on the choice of the parameter $\gamma_{0.7}$ (which is the shear strain at which the maximum small strain shear modulus is reduced to 70%) but reasonable values based on literature data have been chosen in this studies. A similar trend is observed for bending moments (Figure 3).

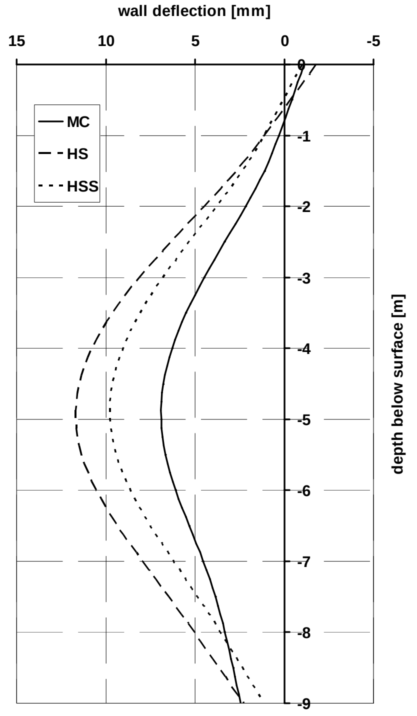


Figure 2 Horizontal wall displacements (sand)

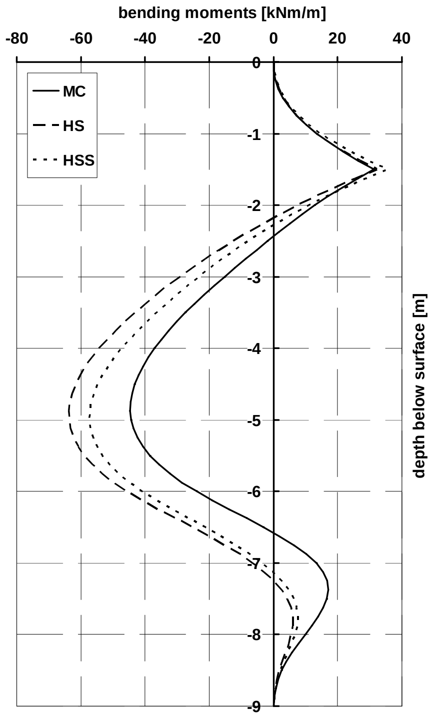


Figure 3 Bending moments (sand)

The notable difference between the simple and the advanced models become apparent when examining surface settlements behind the wall (Figure 4). The MC model shows unrealistic heave whereas the advanced models show the expected settlement, the maximum values being approx. 50% of the maximum horizontal displacement.

Strut forces obtained are -84 kNm/m for the MC model and -102 and -107 kNm/m for the HS and HSS model respectively. Figure 5 shows earth pressure distributions and the differences between the models are generally not significant. The difference in strut forces is also reflected in the stress path ($\frac{1}{2}(\sigma_x + \sigma_y)$ vs $\frac{1}{2}(\sigma_x - \sigma_y)$) of a soil element behind the wall at strut level (Figure 6) and the differences between the models are again not significant except for the last excavation step.

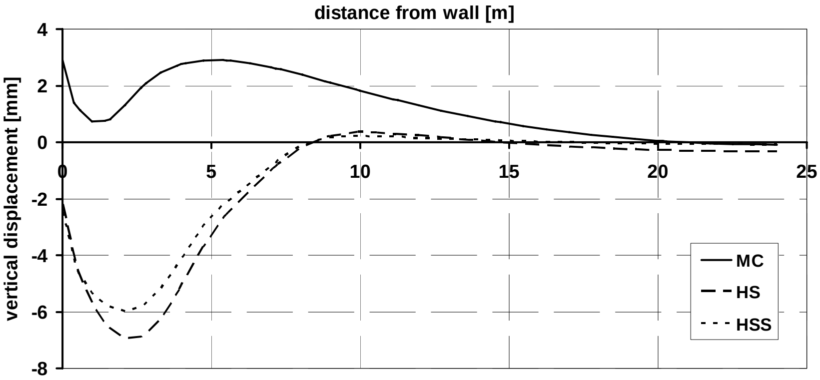


Figure 4 Vertical surface displacements behind the wall (sand)

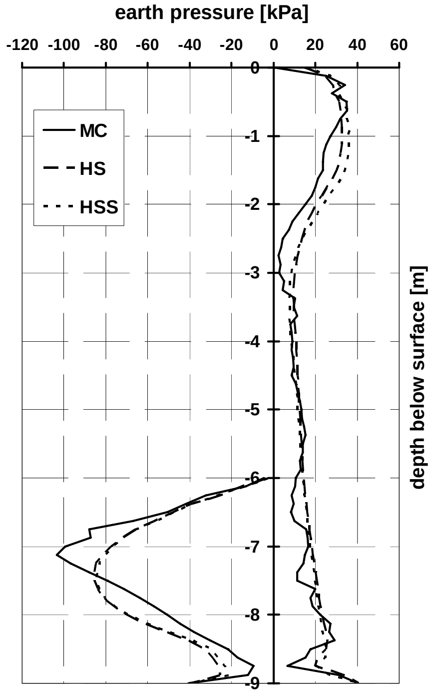


Figure 5 Earth pressure distribution (sand)

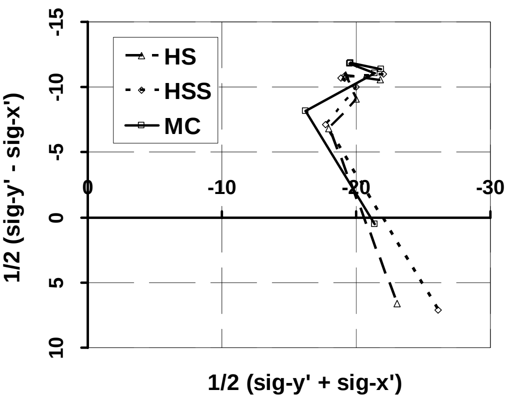


Figure 6 Stress path in soil element at strut level (sand)

2.4 Results for homogeneous clay layer

Figures 7 to 9 depict lateral wall displacements, bending moments and surface settlements for the wall, now 11 m long, in the soft soil. Here the difference between HS and HSS model are larger (but again this depends to a large extent on the value chosen for $\gamma_{0.7}$). The MC model shows a different shape of wall deflection, namely an almost parallel movement of the bottom half of the wall, which is significantly different from the other models. This behaviour leads also to differences in the bending moments, an important aspect for the design.

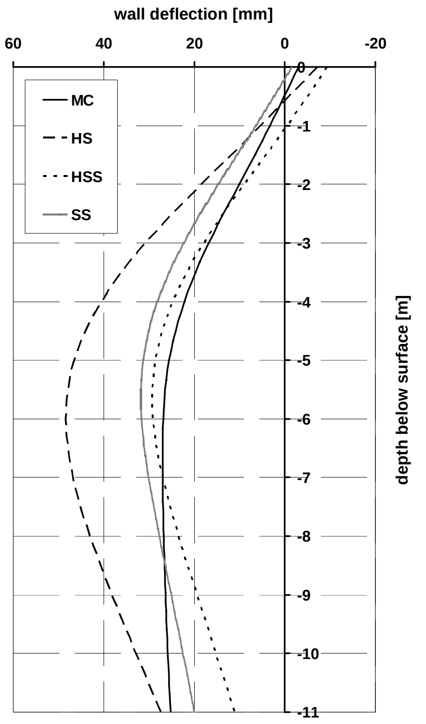


Figure 7 Horizontal wall displacements (clay)

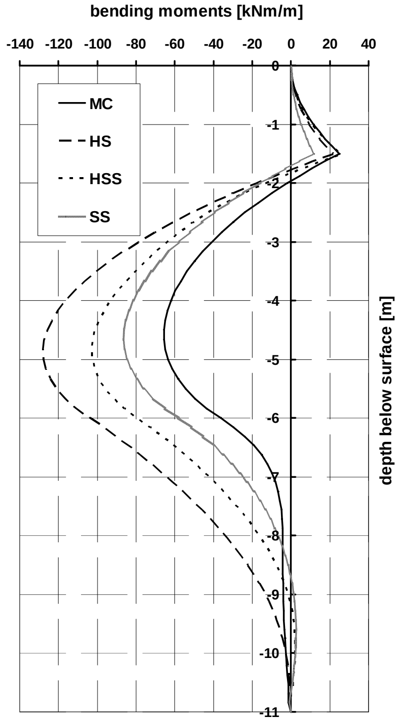


Figure 8 Bending moments (clay)

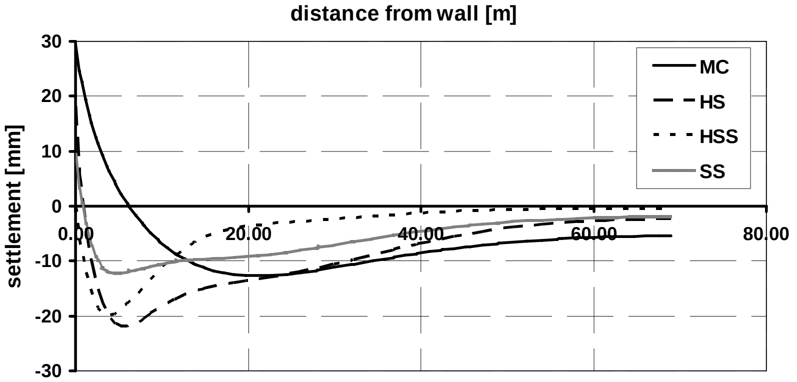


Figure 9 Vertical surface displacements behind the wall (clay)

For the settlement trough behind the wall (Figure 9) the same can be observed as in the previous section, namely that the MC model produces significant heave adjacent to the wall and – in this case due to undrained conditions – settlements in the far field (the lateral model boundary for this analysis was placed at a distance of 75 m from the wall). The calculated settlement troughs can be generally considered as too wide with the exception of the HSS model and this is a consequence of taking into account small strain stiffness effects. Figures 10 and 11 show the development of excess pore pressures at a depth of approx. -4.3 m below surface behind the wall (Figure 10) and approx. -1 m below excavation depth in front of the wall (Figure 11) for all calculation phases. The maximum differences between the models are in the order of about 20%.

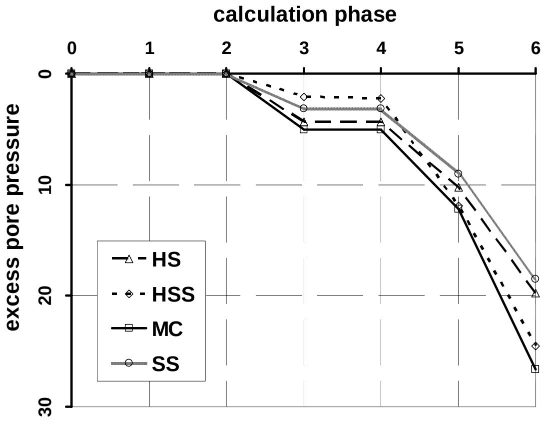


Figure 10 Development of excess pore pressures at soil element -4.3 m below surface behind wall

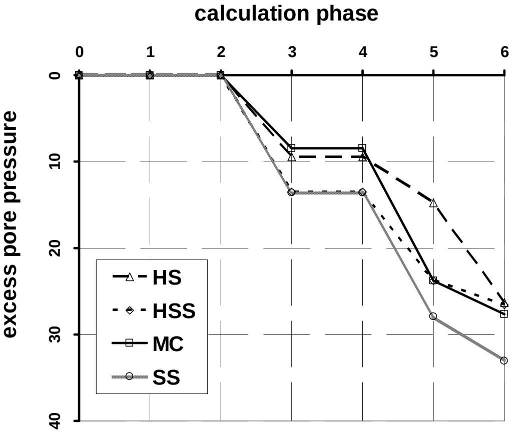


Figure 11 Development of excess pore pressures at soil element -1 m below excavation in front of wall

2.5 Discussion of results

The results for both soils clearly emphasize the well known fact that elastic-perfectly plastic constitutive models such as the Mohr-Coulomb model are not well suited for analysing this type of problems and more advanced models are required to obtain realistic results. Although reasonable lateral wall movements may be produced with simple failure criteria with appropriate choice of parameters, vertical movements behind the wall are in general not well predicted, obtaining heave in many cases instead of settlements. Even with strain hardening plasticity models in general a tendency of predicting too wide settlement troughs can be observed. When small strain stiffness behaviour is included in the model narrower settlement troughs are usually obtained being more in agreement with observed behaviour in the field. As the goal of the study presented here was to qualitatively highlight the differences in results with respect to the constitutive model no quantitative comparison with in situ measurements has been provided. However, the performance of the Hardening Soil model as compared to in situ measured behaviour for five different excavations in soft soil has been presented in Schweiger & Breymann (2005).

3. APPLICATION OF EC7 DESIGN APPROACHES WITH FEM

3.1 EC7 design approaches

The second example briefly addresses the influence of the design approach according to Eurocode7 when performing ULS-design with finite elements. Eurocode7 allows for three different design approaches DA1 to DA3 which differ in the application of the partial factors of safety on actions, soil properties and resistances. They are given in Tables 8 and 9 for all three approaches. It is noted that 2 separate analyses are required for design approach 1. The problem which arises for numerical analyses is also immediately apparent because DA1/1 and DA2 require permanent unfavourable actions to be factored by a partial factor of safety, e.g. the earth pressure acting on retaining structures. This is of course not readily taken into account in numerical analyses because the earth pressure is not an input but a result of the analysis. EC7 also states that instead of putting a partial factor on actions the factor can be put on the effect of the action, e.g. the bending moment or strut forces. This is commonly referred to as DA2*. This is straightforward with finite elements because the analysis is performed with characteristic loads and parameters and the partial factor is introduced at the end of the analysis when design bending moments are defined. Each country defines in its National Annex which design approach has to be used in the respective country and they may differ depending on the problem. Austria has decided to use DA2* for deep excavation problems and a number of other countries have done the same. However, in contrary e.g. to Germany, Austria allows the use of DA3 when design is based on numerical analysis. To the author's knowledge France has chosen a similar strategy and the UK will use DA1 which is basically a combination of DA2 and DA3 with the less favourable result to be used. It is beyond the scope of this contribution to elaborate on the advantages and disadvantages of each of the approaches but some discussion can be found e.g. in Simpson (2000), Bauduin et al. (2000), Bauduin et al. (2003) and Schweiger (2005).

design approach	permanent unfavourable	variable
DA1/1	1.35	1.50
DA1/2	1.00	1.30
DA2	1.35	1.50
DA3-Geot.	1.00	1.30

Table 8 Partial factors for actions according to EC7

design approach	tanφ'	c'	undrained shear strength	passive resistance
DA1/1	1.00	1.00	1.00	1.00
DA1/2	1.25	1.25	1.40	1.00
DA2	1.00	1.00	1.00	1.40
DA3-Geot.	1.25	1.25	1.40	1.00

Table 9 Partial factors for soil strength properties and resistances according to EC7

3.2 Example Specification

A simple example (see Figure 12) has been chosen to demonstrate the difference obtained for design bending moments and strut forces when different design approaches are used. In addition a comparison between a conventional analysis and finite element calculations are shown. As in this example only the Ultimate Limit State is considered and displacements under working load conditions are of no interest a simple Mohr-Coulomb model has been employed. This choice has a consequence for the resulting bending moments and strut forces as has been shown in chapter 2. However, in order to be comparable with the conventional analysis the application of a Mohr-Coulomb model is justified.

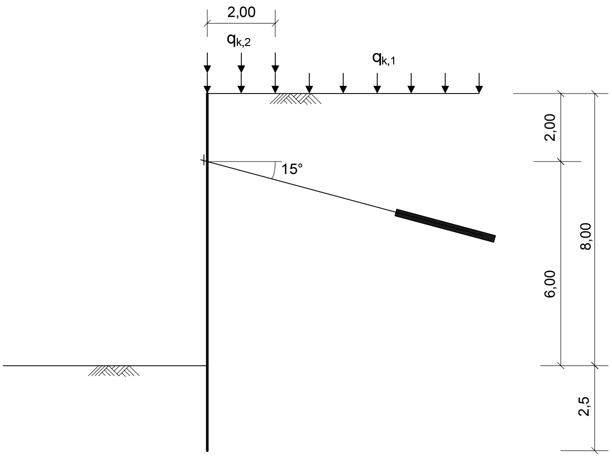


Figure 12 Geometry of example for comparison of EC7 design approaches

Parameter		Meaning	Value
γ	[kN/m³]	Unit weight	18
φ'	[°]	Friction angle (Mohr-Coulomb)	35
c'	[kPa]	Cohesion (Mohr-Coulomb)	0
ψ	[°]	Angle of dilatancy	0
ν	[-]	Poisson's ratio unloading-reloading	0.30
E	[kPa]	Young's modulus	65 000
$\sigma_{Tension}$	[kPa]	Tensile strength	0

Table 10 Parameters for Mohr-Coulomb Model used in FE-analysis

Fall friction was assumed to be 2/3 of the soil friction angle and according to ÖNORM 4434 the earth pressure applied is 50% at rest and 50% active and a redistribution was assumed in the conventional analysis (see Figure 13). The earth pressure is the only permanent action in this example and two design situations (DS1 and DS2) have been considered. They differ in the variable load, for DS1 only $q_{k,1}$ = 20 kPa is active and for DS2 in addition $q_{k,2}$ = 30 kPa is considered (see Figure 12). It should be noted that different partial factors have been specified in Austrian's National Annex for different design situations and these will be taken into account in this comparison.

3.3 Conventional analysis based on ÖNORM 4434

Although DA2* is the design approach to be used for ULS design of deep excavations in Austria an exception is made when numerical methods are employed. In this case DA3 can also be used. In order to highlight the consequences of this decision the example shown in Figure 12 is analysed for both design approaches by means of conventional and numerical methods. According to ÖNORM B4434 the earth pressure distribution for design situation 1 is shown in Figure 13 for DA2* (i.e. earth pressures are calculated based on characteristic soil properties) and in Figure 14 for DA3, where earth pressures are calculated based on factored soil properties. The static system for the analysis follows also from Figure 14.

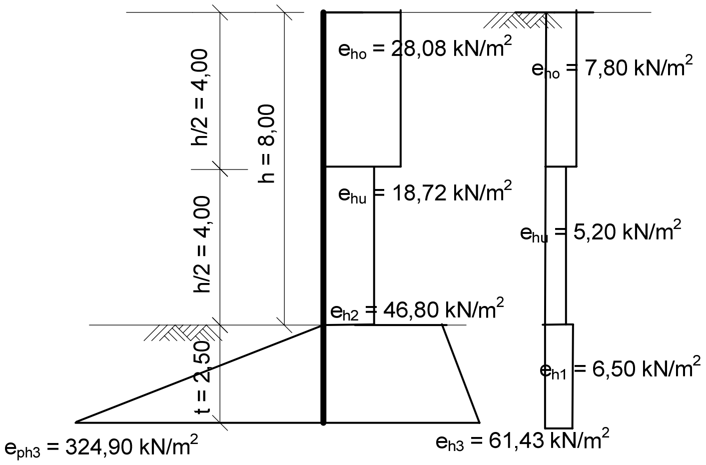


Figure 13 Redistributed earth pressure for DA2* (DS1)

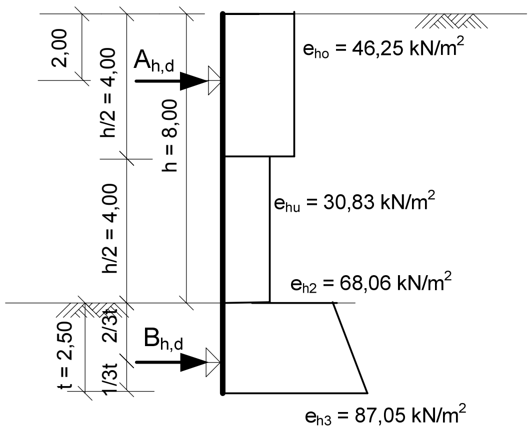


Figure 14 Redistributed earth pressure for DA3 (DS1) and static system for conventional analysis

According to DA2* bending moments and strut forces are calculated for permanent and variable loads based on characteristic soil strength properties and loads. These, characteristic, structural forces are then multiplied by the respective partial factors to obtain the design forces. For example the strut force for design situation 1 is obtained as

$$A_{d,h} = A_{Gh,k}\gamma_G + A_{Qh,k}\gamma_Q$$

with

$A_{d,h}$ horizontal component of design anchor force

$A_{Gh,k}$ horizontal component of characteristic anchor force due to permanent load

$A_{Qh,k}$ horizontal component of characteristic anchor force due to variable load

γ_G partial factor for permanent actions (or effect of actions)

γ_Q partial factor for variable actions (or effect of actions)

For design situation 1 the factors are $\gamma_G = 1.35$ and $\gamma_Q = 1.50$ (National Annex Austria).
For design situation 2 the factors are $\gamma_G = 1.20$ and $\gamma_Q = 1.30$ (National Annex Austria).

It should be mentioned that with this type of analysis design approaches DA2 and DA2* yield the same design forces.

When using DA3 (partial factor on soil strength and on variable loads) the calculation result is already the design strut force and design bending moment.

For this particular example the following results are obtained from the conventional analysis.

DA2* / DS1

Design strut force: 272 kN/m
Design bending moment: 228 kNm/m

DA2* / DS2

Design strut force: 267 kN/m
Design bending moment: 197 kNm/m

DA3 / DS1

Design strut force: 253 kN/m
Design bending moment: 211 kNm/m

DA3 / DS2

Design strut force: 266 kN/m
Design bending moment: 194 kNm/m

From these results it is obvious that the chosen design approach has an influence on the results as expected but it is worth noting that in this particular case the differences are negligible for design situation 2 (DS2). This is caused by the fact that different partial factors are valid for DS1 and DS2 and that the ratio of permanent to variable load is different. As will be seen from the next section tendencies are different when applying numerical methods.

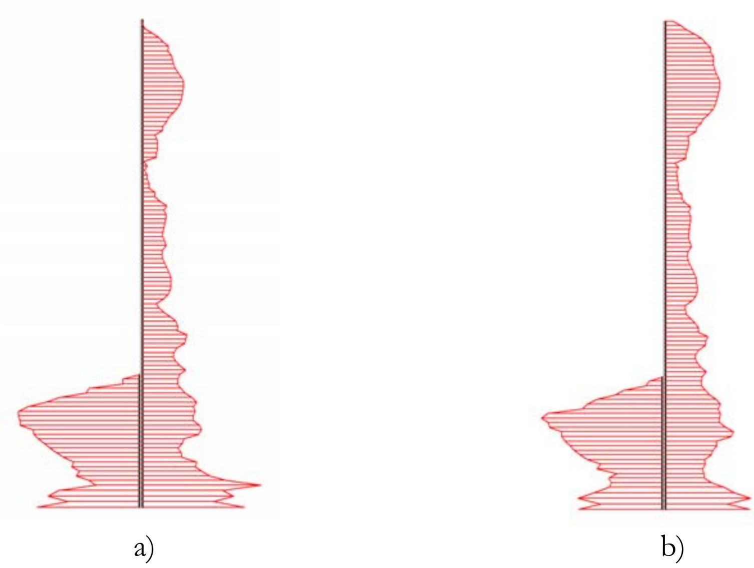
Finally it should be mentioned that in some countries (e.g. Germany) the factor of safety of passive failure is checked with the same logic as applied to the strut force. Thus the resultant (characteristic) support force $B_{h,k}$ (see Figure 14) is treated as effect of actions and therefore multiplied with partial factors γ_G and γ_Q respectively. This force has to be equal or smaller than the theoretical passive resistance divided by $\gamma_R = 1.4$ (partial factor on resistance).

$$B_{d,h} = B_{Gh,k}\gamma_G + B_{Qh,k}\gamma_Q \leq E_{ph,k} / \gamma_R$$

3.4 Numerical analysis

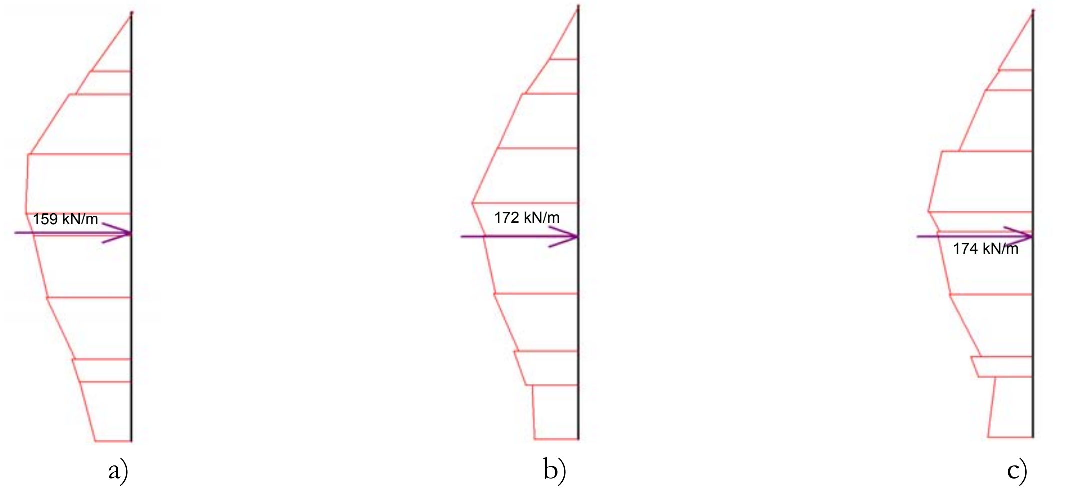
It has been mentioned that DA2 is not applicable for deep excavations when using numerical methods because the permanent unfavourable action (= the earth pressure) cannot be factored as it is a result of the analysis. However, applying the partial factor on the effects of the actions is of course possible with numerical analysis because a standard calculation is performed with characteristic input parameters and resulting characteristic internal forces are multiplied by the partial factors for actions leading to design forces. DA3 is straightforward with numerical methods because soil strength parameters are factored and the analysis is then performed in the usual way with reduced strength of the soil and increased variable loads when appropriate. This has been done for this example and results are presented in the following.

Figure 15 shows the calculated earth pressure on the active side for DA2* and design situation 1. Although some earth pressure redistribution can be observed at strut level the distribution is different to what is assumed in classical analysis. Figure 16 depicts mobilised passive pressures for DA2* and Figure 17 for DA3. It should be noted that integration of the passive pressure in Figure 16 yields the force which can be used to check against passive failure as described above. In the same way as in conventional analysis bending moments and strut forces obtained from DA2* have to be multiplied by the respective partial factors whereas in DA3 design bending moments and strut forces are obtained from the analysis directly.



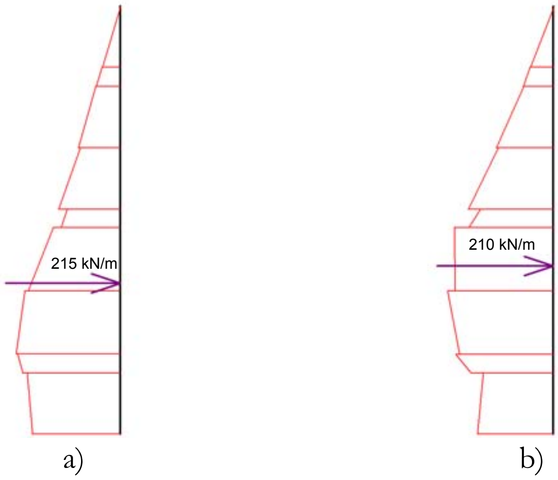
a) b)

Figure 15 Calculated earth pressure from FE-analysis DA2* a) no surcharge b) surcharge (BS1)



a) b) c)

Figure 16 Mobilised passive pressure from FE-analysis DA2* a) no surcharge b) surcharge (BS1), c) surcharge (BS2)



a) b)

Figure 17 Mobilised passive pressure from FE-analysis DA3 a) BS1, b) BS2

The following results for bending moments and strut forces are obtained from the FE-analysis:

DA2* / DS1

Design strut force:	215 kN/m
Design bending moment:	167 kNm/m

DA2* / DS2

Design strut force:	241 kN/m
Design bending moment:	167 kNm/m

DA3 / DS1

Design strut force:	234 kN/m
Design bending moment:	208 kNm/m

DA3 / DS2

Design strut force:	250 kN/m
Design bending moment:	190 kNm/m

It can be observed that the FE-analysis yields lower bending moments and strut forces as compared to the conventional design analysis for both design situations and both design approaches. In conventional analysis DA2* results in higher structural forces as compared to DA3 whereas the opposite trend is observed in the FE-analysis. This can be attributed to nonlinear soil behaviour and the influence of the relative stiffness of soil and support which is ignored in conventional calculations. It is emphasized that this trend cannot be generalized because of aforementioned reasons and indeed for other examples a different trend has been observed, i.e. bending moments and strut forces from numerical analysis have been found to be larger than those obtained from conventional calculations.

3.5 Summary on design approaches

It has been shown that the different design approaches specified in EC7 lead to different bending moments and forces in wall and struts supporting a deep excavation, which could be expected. This is true for conventional as well as numerical analyses. All design approaches can be used in combination with numerical methods although it has to be pointed out that DA2 is possible only in form of DA2*, i.e. partial factors of safety have to be applied to the effect of the action. Application of DA3 is straightforward with numerical methods because the relevant partial factors are applied before the calculation and consequently the analysis is performed with design parameters. Whether DA2* or DA3 (or DA1) is to be used has to be specified in the National Annex, each one having its advantages and disadvantages. Arguments for and against, and in particular a discussion on the merits of using DA1, can be found in Simpson (2007).

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