Comparison of different limit state design approaches of retaining structures

Comparaison des différentes techniques de détection pour l'état limité de la capacité portante des fouilles

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ABSTRACT

The choice of the different design approaches has a substantial effect on the limit state analysis of geotechnical structures. In particular, the soil-structure interaction behaviour is complicated in supported excavations, since the actions and resistances due to the earth pressure on the excavation walls are dependant on the deformations and has to be taken into consideration with different partial safety factors.

The limit state conditions of excavations had been investigated according to the German approach with consideration of the national annex to the EC 7-1 and the national recommendations for "Excavations" EAB (2006). The determination of the required embedment depth and the corresponding section forces are of special importance in the design of the retaining walls in the ultimate limit state (ULS) according to EC 7-1 and DIN 1054:2005. For a practical application, this has been illustrated in the paper on the basis of a numerical and analytical analysis of an idealized excavation. The results are compared with other alternative design approaches of the EC 7-1.

RÉSUMÉ

Le choix de la technique de détection a des conséquences considérables sur les considérations d'état limite des structures géotechniques. L'interaction de bâtiment avec le sol est particulièrement problématique, parce que les effets et des résistances suite à la pression de terre sur la paroi de la fouille sont ici de façon dépendante de la déformation et pas avec des coefficients de sécurité partiels saisir clairement.

Les considérations d'état limite avec des fouilles conformément à la procédure allemande étaient présenté compte tenu de l'annexe nationale à l'EC 7 et de l'EAB (2006). La détermination de la profondeur de la longueur murale dans le sol et les grandeurs d'intersection résultant dans l'état limite de la capacité portante (GZ 1) conformément à DIN 1054:2005 et/ou. l'ultimate limit state (ULS) après EC 7 présente un intérêt particulier. Celui-ci a été illustré à une fouille simple dans le travail sur la base d'une analyse numérique et analytique pour une application pratique. Les résultats sont comparés à d'autres techniques de détection alternatives.

1 INTRODUCTION

The new generation of European code of standard for geotechnical structures EN 1997-1:2005-10 (EC 7-1) recommends three different design approaches for the ultimate limit state (ULS) based on the partial safety factor concept, from which the governing approach shall be specified in the respective national annex.

It has already been mentioned several times in the literature, that the different design approaches with the corresponding partial safety factors may not always lead to the same and a comparable level of safety with the old global safety concept, which was used in some member countries of the European Union (see for e.g. Vogt et al. 2006), Schweiger 2006). The application of the partial safety factor concept of the new standard is particularly critical to soil-structure interactions, for e.g. supported excavations. This is because the soil is involved both on the action as well as on the resistance side and a strict separation of actions (or the effect of actions) and resistance is demanded in the new code.

The effects of the different design approaches of the EC 7-1 and the corresponding partial safety factors on the calculation of the ultimate limit state (ULS) has been described in the paper using an idealized supported excavation. The basis of this analysis is the finite element method, which supplies a reliable prediction of displacements of soil-structure interaction problems for the serviceability limit state (SLS), but gives substantial deviations for the ULS for different design approaches. The numerical results are also compared with analytical calculation results.

2 ULTIMATE LIMIT STATE (ULS) DESIGN APPROACHES ACCORDING TO EN 1997-1

For the verification of the ultimate limit state (ULS), three design approaches are given by EC 7-1, whereas the governing design approach has to be specified and included in the respective national annex.

The design approaches (Table 1) differ in the application of the partial safety factors to actions, soil strength and resistances (Tables 2 and 3). Moreover, the design approaches, which apply partial safety factors to permanent actions, are particularly problematic for numerical computations, because the active earth pressure due to soil weight, for example, is not an input parameter but a result of the computation.

The design approach 1 (DA 1) includes two combinations from which the unfavourable one shall govern the design.

In DA 1-1 the partial safety factors are applied only to the characteristic actions F_k, whereas they are mainly applied to the characteristic soil strength parameters in DA 1-2. In both cases the calculation is carried out with design values. Thereby, the respective unfavourable deviations in the actions and soil strength parameters will be taken into account. The application of DA 1-1 to the numerical analysis of supported excavations is not simple, because of the consideration of the action from active earth pressure resulting from soil own weight with the partial safety factor.

Table 1. Design approaches for ULS according to EC 7-1:2005

Design Approach	Actions		Soil strength		Resistances	eq. no.
DA 1-1	A1	+	M1	+	R1	(1)
DA 1-2	A2	+	M2	+	R1	(2)
DA 2	A1	+	M1	+	R2	(3)
DA 3	A1[a] or A2[b]	+	M2	+	R3	(4)

[a] Structure (STR).
[b] Geotechnical (GEO).

Table 2. Partial safety factors according to EC 7-1:2005 for retaining structures

Actions			Soil strength			Resistances			
	A1	A2		M1	M2		R1	R2	R3
γ_G	1.35	1.00	$\gamma_{\varphi'}$	1.00	1.25	$\gamma_{R;e}$	1.00	1.40	1.00
γ_Q	1.50	1.30	$\gamma_{c'}$	1.00	1.25				

Table 3. Partial safety factors according to DIN 1054:2005 (would be the national annex to EC-7) and recommendations for excavations EAB (2006) for load case 2 (LC 2)

Actions			Soil strength			Resistances			
	A1	A2		M1	M2		R1	R2	R3
γ_G	1.20	1.00	$\gamma_{\varphi'}$	1.00	1.15	$\gamma_{R;e}$	1.00	1.30	1.00
γ_Q	1.30	1.20	$\gamma_{c'}$	1.00	1.15				

The governing combination for the design approach 2 (DA 2) requires partial safety factors on the actions or effects of actions and on ground resistances. There are two possible variations in DA 2. According to Frank et al. (2004), the partial safety factors are applied to the characteristic actions F_k at the beginning of the calculation for DA 2, whereas they are applied to the characteristic effects of actions E_k at the end of the calculations for DA 2*. The design approach DA 2* is specified in the German national annex to the EC 7-1 as obligatorily and makes the computation possible with characteristic actions.

In the design approach 3 (DA 3), a distinction is made between the actions from the structure (STR) and geotechnical actions (GEO). For the calculation of retaining structures, e.g. supported excavations, the limit state GEO is generally decisive, since the strength of the soil determines the resistance. Here the partial safety factors are applied to the actions or the effects of actions and to the soil strength parameters. For supported excavations, this combination corresponds to the design approach 1-2 (DA 1-2).

The partial safety factors for ULS according to German national standard DIN 1054:2005 and EAB (2006) are given in Table 3. These partial safety factors are dependent on load cases, which in turn depend on combination of actions and the safety class. The partial safety factors according to EC 7-1 correspond comparatively to the load case 1 (LC 1), i.e. persistent design situation. However, supported excavations are considered as a temporary construction according to DIN 1054:2005 and EAB (2006) and hence they are classified as LC 2, i.e. transient design situation.

The comparison of different design approaches in this paper contains the analysis of the excavation with the partial safety factors for the LC 1 (EC 7-1) as well as for load case LC 2 (DIN 1054:2005; EAB (2006)).

3 PRACTICAL APPLICATION ON IDEALIZED EXCAVATION

3.1 *General*

The effect of the different design approaches on the determination of the necessary embedment depth and the resulting design section forces, e.g. bending moments and strut forces, are presented based on an idealized excavation supported by a single strut. Both numerical and analytical computations have been conducted for the ultimate limits state (ULS).

The geometry and loading condition of the idealized excavation is shown in Figure 1. The surcharge load 10 kN/m^2 is assumed to be a permanent load and the 50 kN/m^2 as a variable load. A bearing layer is assumed at a depth of 20 m below ground surface.

3.2 *Numerical analysis*

Although the main focus lies on the determination of the required embedment depth for ULS condition and

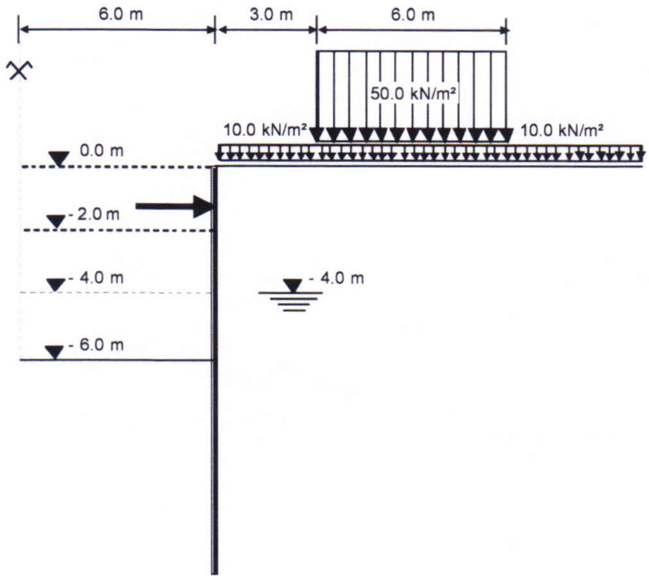

Figure 1. Idealized excavation.

Table 4. Construction steps

phase	Description
1	initial stress ($K_0 = 0.5$)
2	wall installation
3	surcharge load application
4	excavation to a depth of −2.0 m
5	strut placement at level −1.5 m, and further excavation to a depth of −4.0 m
6	groundwater lowering excavation to a depth of −6.0 m (bottom of excavation)

Table 5. Material parameters for Mohr-Coulomb Model (MC)

φ' [°]	c' [kN/m^2]	ψ [°]	E [kN/m^2]	ν [°]
38	0.1	6	$20000\sqrt{z}$	0.2

Table 6. Material parameters for Hardening Soil Model (HSM)

φ' [°]	c' [kN/m^2]	ψ [°]	E_{oed} [kN/m^2]	E_{50} [kN/m^2]	E_{ur} [kN/m^2]	ν_{ur} [−]
38	0.1	6	$20000\sqrt{z}E_{oed}$		$4\,E_{50}$	0.2
p_{ref} [kN/m^2]	R_f [−]	K_0^{NC}[−]	γ [kN/m^3]	γ_{sat} [kN/m^3]	R_{inter} [−]	m [−]
100	0.9	0.384	19.0	21.0	1.0	0.55

hence an elastic-perfectly plastic soil model is sufficient, a more advanced constitutive soil model is also used for comparison purpose.

A plain-strain state is assumed for the calculation and the wall installation is neglected (wished in place). The wall is assumed to behave elastic with E = 30000 MN/m^2, d = 0.80 m and $\nu = 0.18$. The strut is also assumed elastic with axial stiffness of EA = 1500 MN/m. The construction phases followed in the numerical analysis are outlined in Table 4.

The size of the computation model is selected in accordance to the recommendation of the working group "Numeric in Geotechnics" of the German Society of Geotechnical Engineers (Meissner 2002). The finite element code used for the numerical analysis is called PLAXIS v8.2. Two constitutive soil models are used in the analysis, namely, the Mohr coulomb model (MC) and the Hardening Soil Model (HSM). Details about the constitutive soil models can be found in Brinkgreve (2004). The soil parameters for the Mohr coulomb model (MC) and the Hardening Soil Model (HSM) are summarized in Table 5 and 6 respectively. Figure 2 shows the computation model and the finite element mesh.

The contact between the wall and the soil is represented by interface elements at both sides of the wall. A separate material set is organized for the interface elements, in which the strength parameters are adopted from the surrounding clusters after reducing them by

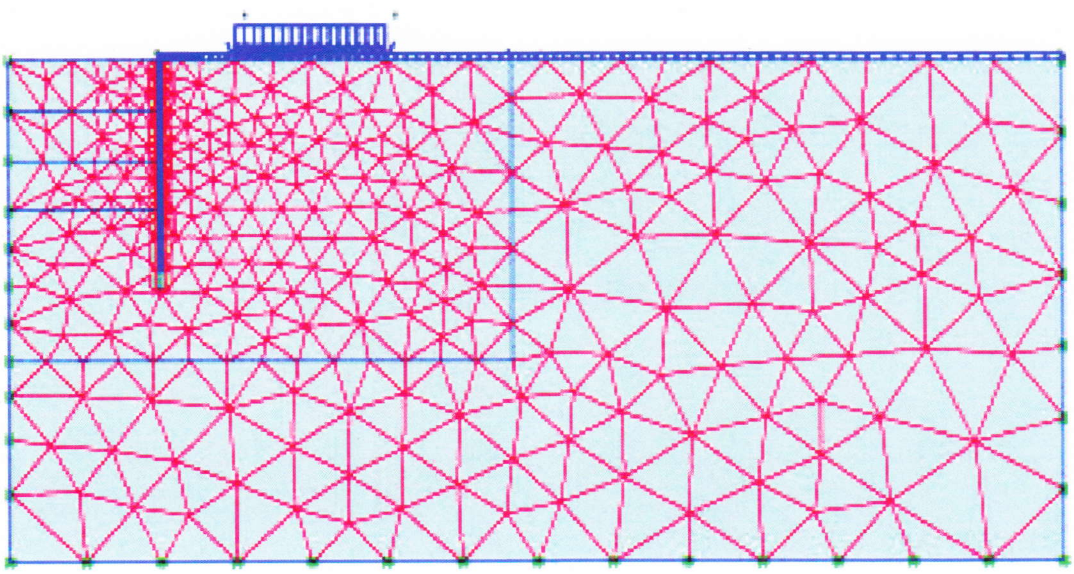

Figure 2. The Finite element mesh (15-noded triangles).

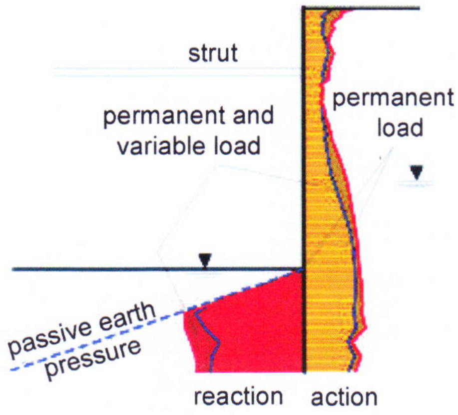

Figure 3. Horizontal stresses without hydrostatic water pressure from numerical analysis using MC for DA 2* and LC 2 according to EAB (2006).

a factor of 0.5 ($\delta_{a,p} = 0{,}5\varphi'$), where as the stiffness parameters are adopted as it is.

For the numerical computation of the excavation for ULS, the verification of the soil reaction (resistance) on the passive side of the wall is decisive for the determination of the required embedment depth. Sufficient safety is verified, if the limit state condition

$$B_{h,d} \leq E_{ph,d} \tag{5}$$

is fulfilled, where $B_{h,d}$ is the design value of the horizontal component of the resultant reaction force and $E_{ph,d}$ is the design value of the horizontal component of the passive earth pressure. The soil reaction force can be calculated by integrating the soil reaction pressure from the numerical analysis on the passive side of the wall.

The determination of the embedment depth using the design approach DA 2* has been described exemplary in the following. The DA 2* demands to enter the calculation using the characteristic effects of actions and soil strength parameters. The soil reaction force can then be determined from Eq. (6).

$$B_{h,d} = \gamma_G \cdot B_{Gh,k} + \gamma_Q \cdot B_{Qh,k} \tag{6}$$

The variable load part in Eq. (6) is given by Eq. (7) as follows:

$$B_{Qh,k} = B_{h,k} - B_{Gh,k} \tag{7}$$

To verify a sufficient safety for ULS against failure of the soil reaction on the passive side of the wall, the design value of the soil reaction force must always be smaller or equal the design value of the passive earth pressure in ULS (Eq. (8)).

$$E_{ph,d} = \frac{E_{ph,k}}{\gamma_{R,e}} \tag{8}$$

The characteristic passive earth pressure can be determined assuming a curved sliding surface, a wall friction of $\delta_p = -0.5\varphi'$ and a passive earth pressure coefficient of $K_{pgh} = 7.862$ according to Sokolovsky/Pregl:

$$E_{ph,k} = \int \sigma_{v,k} \cdot K_{pgh} dz \tag{9}$$

The required embedment depth will be optimised iterative until the mobilization factor $\mu = 1.00$ is reached (Eq. (10)).

$$\mu = \frac{E_{ph,d}}{B_{h,d}} \tag{10}$$

Fig. 4 represents an example of the relative shear stresses and the plastic points for an embedment depth t = 2.35 m and $\mu = 0.99$ using the design approach DA 2* and partial safety factors for load case LC 2 (EAB (2006)).

The numerical analysis using the other possible design approaches except DA 1-1 is generally carried out with design values of actions or/and soil parameters, whereby the soil reaction force determined numerically is compared with passive earth resistance in the ULS for all design approaches.

The required embedment depths and the design values of the resulting section forces for the different design approaches and for $\mu = 1.0$ are shown in Table 7. For excavations, the design approach DA 3 is similar to DA 1-2.

3.3 *Analytical analysis*

The idealized excavation is also analyzed using a conventional analytical program for the different design approaches. Here the partial safety factors can be applied on the actions or effect of actions and soil properties both before and after the calculation, since the earth pressure is usually determined in the ULS according to the classical earth pressure theory. The classical active earth pressure distribution is converted to an equivalent pressure diagram according to EAB (2006) (Fig. 5), in order to take in to account the interaction between the soil and the retaining structure.

The iterative determination of the required embedment depth can be done in the same manner as above using Eq. (10) and mobilization factor μ. The results of the analytical analysis are given in Table 8. Contrary to numerical analysis, there are no differences between the results of DA 2 and DA 2*.

Table 7. Embedment depth and design section forces from numerical analysis for LC 2

Design approach		t [m]	$M_{max,d}$ [kNm/m]	$A_{h,d}$ [kN/m]
DA 1-1	HSM	1.10	84.4	223.6
	MC	1.53	91.8	145.9
DA 1-2	HSM	1.35	77.5	289.9
	MC	1.77	88.8	184.9
DA 2	HSM	1.96	141.8	198.5
	MC	2.00	117.3	133.6
DA 2*	HSM	2.64	210.5	199.0
	MC	2.35	137.8	132.4
DA 3	HSM	1.35	77.5	289.9
	MC	1.77	88.8	184.9

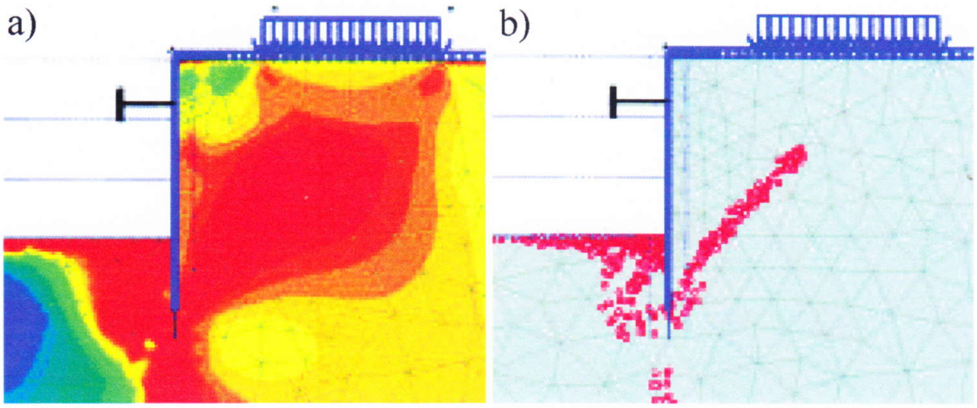

Figure 4. The development of a) relative shear stresses and b) plastic points based on DA 2*.

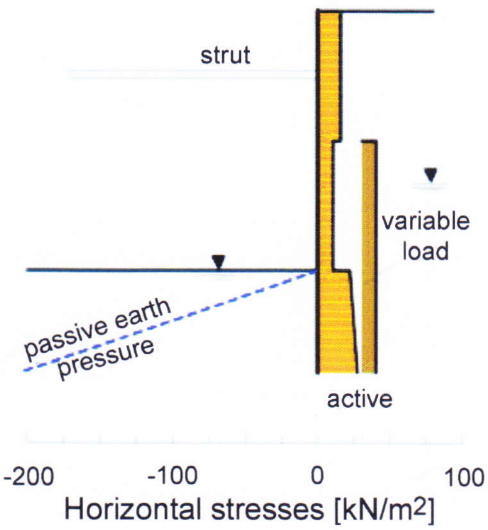

Figure 5. Horizontal earth pressures without hydrostatic water pressure for DA 2* and LC 2.

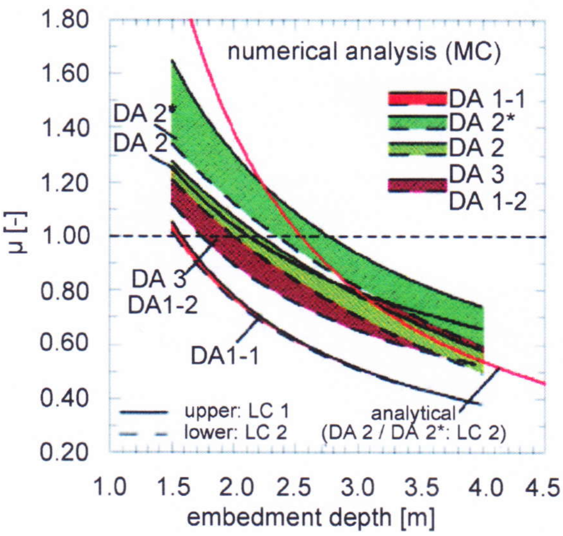

Figure 6. Determination of embedment depth using Mohr-Coulomb soil model.

Table 8. Embedment depth and design section forces from analytical analysis for LC 2

Design approach	t [m]	$M_{max,d}$ [kNm/m]	$A_{h,d}$ [kN/m]
DA 1-1	2.16	137.0	117.1
DA 1-2	2.50	145.9	119.2
DA 2	2.54	149.3	119.8
DA 2*	2.54	149.3	119.8
DA 3	2.50	145.9	119.2

4 COMPARISON

Figure 6 shows the effect of the different design approaches on the determination of the required embedment depth. The results using the partial safety factors for LC 1 (EC7-1) and for LC 2 (EAB 2006) are shown in the figure.

As shown in Figure 6, the embedment depth for $\mu = 1$ varies between 1.53 m and 2.35 m for the case of partial safety factors for LC 2 (EAB (2006)) and between 1.55 m and 2.80 m for LC 1 (EC7-1). The application of the partial safety factors to the soil parameters and the variable actions (DA 1-2 and DA 3) and exclusively to the actions (DA 1-1) supplies the smallest embedment depths. For these design approaches, the partial safety factor for passive earth resistance $\gamma_{R,e} = 1$. It has be noted that, it is numerically not possible to compute using the design values of actions according to the design approaches DA1-1, since the active earth pressure due to soil own weight is not an input, rather it is a result of the computation.

The comparison of the analytical analysis, which requires an embedment depth between 2.16 m and 2.54 m, provides the best agreement with the results of the numerical analysis for the design approach DA 2*. The difference between analytical and numerical embedment depths according to DA 2/DA 2* for LC 2 (EAB (2006)) is about 7.5 % (Figure 6).

The Application of the Hardening Soil Model supplies substantial deviations in the required embedment depth as shown in Figure 7. The embedment depth varies between 1.20 m and 3.40 m for the case of partial safety factors for LC 1 (EC 7-1). The design approach DA 2* leads to a 4 % deviation from the analytical analysis.

The Mohr-Coulomb Model does not consider the state of the stresses in unloading and reloading conditions. To include the effect of the unloading stiffness

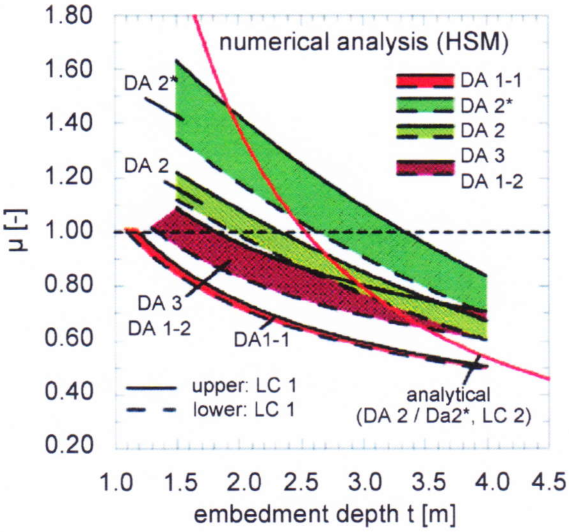

Figure 7. Determination of embedment depth using Hardening Soil Model.

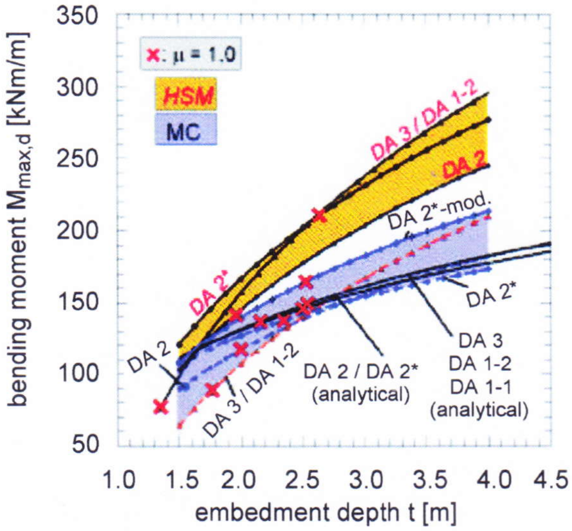

Figure 9. Design bending moments for LC 2.

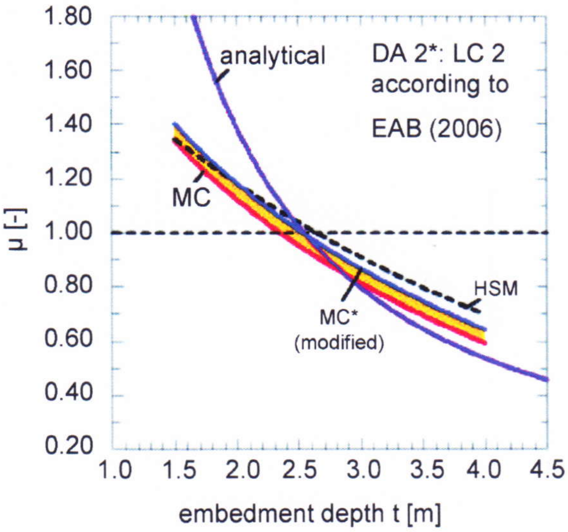

Figure 8. Comparison of different soil models for DA 2* and LC 2.

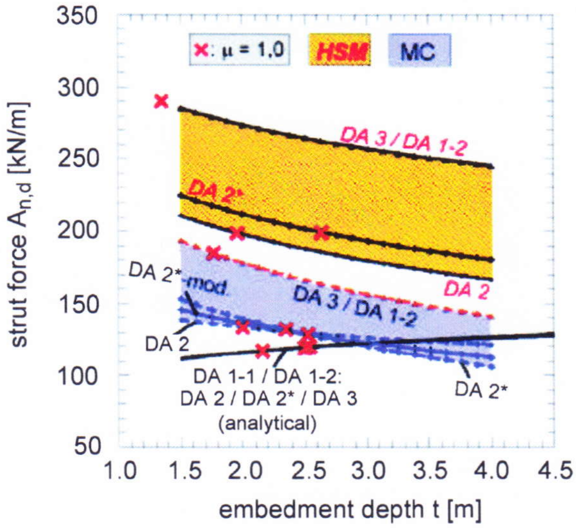

Figure 10. Design strut forces for LC 2.

of the soil on the excavation side, which is assumed to be 3 to 5 times higher than that of the primary loading stiffness, additionally computation had been carried out with modified stiffness of soil ($E_{u,r} = 4$ E) on the passive side. Fig. 8 shows the results of the modified analysis for DA 2* and LC 2 (EAB (2006)). It appears from the Fig. 8 that a good agreement with the analytical analysis can be achieved as a result of the unloading stiffness in the calculation using the MC model.

Figure 9 and 10 show design section forces, whereby the values for $\mu = 1.0$ are indicated with a "cross" symbol. Here the effects of the different design approaches become particularly clear. The numerical analysis with HSM supply generally higher section forces. A good agreement with analytical result can only obtained in the case of DA 2*, which requires characteristic values for the calculation and the partial safety factors are introduced afterwards.

4 CONCLUSIONS

The application of the different design approaches for temporary supported excavations has a substantial influence on the determination of the required embedment depth and the design section forces for ULS. The numerical analysis based on the design approach DA 2* has proven to supply reasonable results and a good agreement with analytical results. The advantage of DA 2* is that the analysis of supported excavation is possible using analytical and numerical methods with out any modifications and it maintains the old national global safety requirements.

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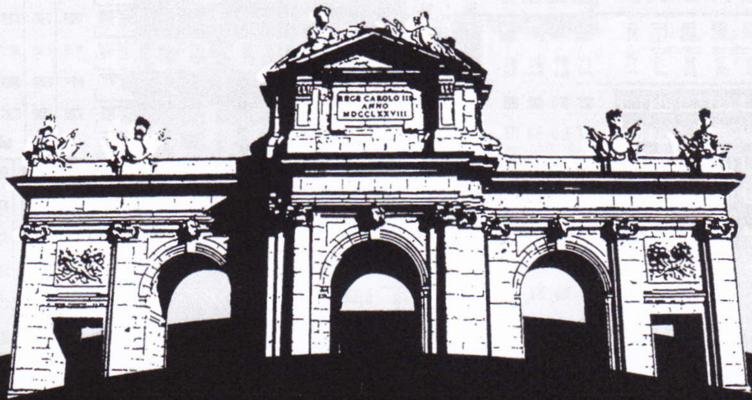

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