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Multi-criteria procedure for the back-analysis of multi-supported retaining walls

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ABSTRACT: A numerical back-analysis procedure for multi-supported deep excavations is proposed based on the optimization of several indicators, taking in account the forces in the struts and the differential pressures derived from the wall displacement. The evaluation of the procedure is performed on 1 g small scale laboratory experiments (Masrouri 1986) on semi-flexible retaining walls embedded in Schneebelli material. The proposed numerical procedure was applied on an excavation with 2 passives low stiffness struts. The resulting Hardening Soil Model parameters are further used to back-calculate the 14 different tested configurations. The results are compared with the classical methods, SubGrade Reaction Method, Finite Element analysis with Mohr Coulomb model with parameters proposed by Masrouri (1986) and with the back-analysis using Hardening Soil Model parameters based on triaxial tests results.

1 INTRODUCTION

Numerical back-analysis of in situ monitoring results of multi-supported deep excavations is generally extremely complex (Hashash & Whittle 1996, Finno & Calvello 2005, Delattre 1999): soil characteristics can be heterogeneous or determined with a low degree of confidence, the different stages of the excavation can be difficult to reproduce in a 2D numerical approach, key mechanical parameters can be unknown (for example the actual stiffness of the strut-to-wall contact) and the number of measured quantities such as wall displacements, settlements and strut forces is generally too small to perform a comprehensive comparison between the actual behavior and the numerical results.

Thus the full validation of a back-analysis numerical procedure (including in particular the choice of the constitutive law and the determination of all the required parameters) is rarely directly possible on real case histories. Therefore, the numerical procedure proposed in this paper is validated on 1 g small scale laboratory experiments performed by Masrouri (1986) on semi-flexible retaining walls embedded in Schneebeli material (mixture of steel rods of different diameters representing in 2D the behavior of a cohesionless soil). The 14 considered experiments correspond to a retaining wall, whose length and mechanical properties are kept constant, supported by one or two levels of active or passive steel struts with various axial stiffness and prestressing.

Even for such simple comprehensive laboratory experiments, usual design method like SubGrade Reaction Method (SGRM) or classical limit equilibrium methods do not capture all the observed behaviors and test results. It is thus necessary to propose a unified numerical procedure to back calculate with an acceptable degree of confidence, all the results of the excavations tests of Masrouri (1986).

2 EXPERIMENTS AND COMPARISON WITH CLASSICAL/SGRM CALCULATIONS

The experiments correspond to small scale 2D models of flexible retaining walls (Figure 1).

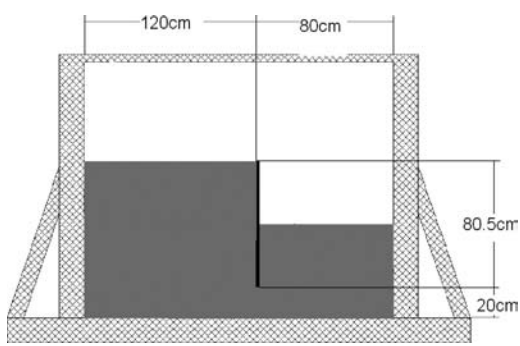

Figure 1. Masrouri's experimental set up.

Table 1. Summary of Masrouri (1986) experiments.

	First strut		Second strut	
Experiments	Prestressed (kN/m.ml)	Stiffness (kN/m.ml)	Prestressed (kN/m.ml)	Stiffness (kN/m.ml)
B1	2.133	83333	(not used)	
B2	3.025	83333	(not used)	
B3	0.208	816	(not used)	
B4	2.1	816	(not used)	
B5	3	816	(not used)	
B6	0.191	404	(not used)	
B7	2.016	404	(not used)	
B8	2.916	404	(not used)	
B10	0.383	83333	0.333	83333
B11	1.330	83333	2.691	83333
B12	0.366	816	0.400	816
B13	2.075	816	2.700	816
B14	1.733	816	4.041	816

Schneebelli 2D analogic soil was used: this material offer a good repeatability and enables to build a homogeneous 2D soil model with quick handling for the experiments. However Schneebeli materials have some inconvenients: the unit weight is close to 6.5 kN/m^3, the angle of friction is smaller than that of most of the soils (21°) and it only presents a dilatant behavior.

While maintaining the same geometrical and mechanical characteristics for the wall ($EA = 1.2\ 10^6$ kN/m and $EI = 14.4$ kN/m^2/m), a wide range of configurations (1 or 2 struts, with different prestressing and stiffness) was considered, see Table 1. The phases used in all the excavations were planned as follows;

1. strut cases: 10 cm of excavation, installation of the first strut at −5 cm from the top and prestressing, then 3 excavations of 10 cm each till −40 cm, then several excavations of 5 cm until failure is obtained.
2. struts cases: the same procedure as for the 1 strut cases is followed until the excavation reaches −40 cm from the top, then the 2nd strut is installed at the level −25 cm and prestressed. The step-wise excavation (by increment of 5 cm) is resumed until failure occurs.

The strut to wall contact is hinged in order to prevent bending moment to be transmitted to the struts. For each excavation phase, the horizontal displacements of the top or bottom of the wall are measured. The curvature is also measured in 26 locations (both sides on the wall) allowing to determine with a reasonable accuracy the differential pressure acting on the wall from the top to the bottom (polynomial approximation performed by the Palpan program created by Boissier *et al.* 1978). Photographs were also taken to determine the displacement fields with a stereophotogrammetric technique.

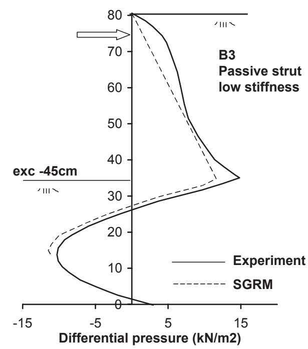

Figure 2. Differential pressure calculated with SGRM compared to experimental values: – Case B3 – low stiffness passive strut (−45 cm excavation level).

Load cells determine the forces on the cylindrical struts.

2.1 *Classical and SGRM calculations*

The whole series of experiments were first compared with classical (modified Blum method) and SGRM calculations (Terzaghi 1955). The subgrade reaction modulus used in RIDO calculations (Fages 1996) is increasing with depth in the following manner:

$K_h = K + K'\sigma_v$. with $K = 0$, $K_h = K'\sigma_v$

Classical methods calculation totally neglects the influence of the stiffness of the wall, construction steps, stiffness and prestressing on the struts, arching effect, etc. On the opposite SGRM explicitly considers the stiffness of the wall, the construction steps and the stiffness of the strut.

In the case of one passive strut with low stiffness (tests B3, B6), the SGRM method reproduces in an acceptable manner the differential pressures results (Figure 2).

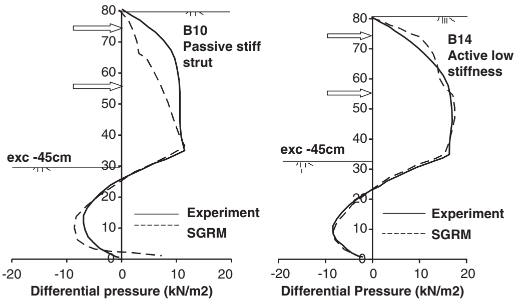

Figure 3. Calculated (SGRM Method) and measured differential pressure for B10 and B14 at −45 cm of excavation.

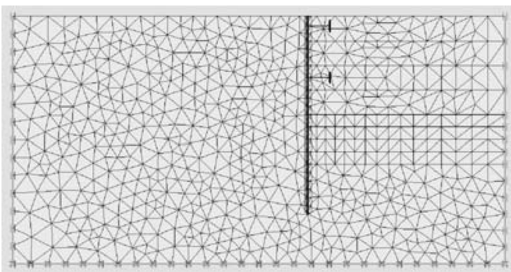

Figure 4. Mesh used for the Plaxis calculations of 2 struts supported walls.

With one stiff or active strut or in the case of excavations with 2 struts, both the SGRM and the limit equilibrium methods fail to reproduce the experiment differential pressures (Figure 3a – Test B10). In some cases, a good description of the pressure diagrams seems to be obtained (Figure 3b – Test B14). It actually results from two errors compensating each other: the over estimation of the pressure induced by prestressing and the lack of ability to reproduce the arching effect.

2.2 *Finite element back calculation with Mohr Coulomb model*

It appears that the classical and SGRM methods do not, even in simple cases of excavation, accurately describe all the observed results. A finite element approach is therefore proposed.

The finite element calculations were performed with Plaxis V8.2, the model represents a vertical slice of Masrouri's experiment. The mesh is composed of triangular 15 nodes elements (Figure 4).

For these calculations, the same characteristics of the Schneebelli material were used ($c = 0$, $\phi = 21°$, $\gamma = 65\,kN/m^3$), taking into account for the soil-wall interface an interface factor $R_{int} = 0.55$ (related to the friction angle value noted by Masrouri).

Masrouri (1986) estimated the elastic modulus at 4500 kPa with an increment of 26830 kPa/m after

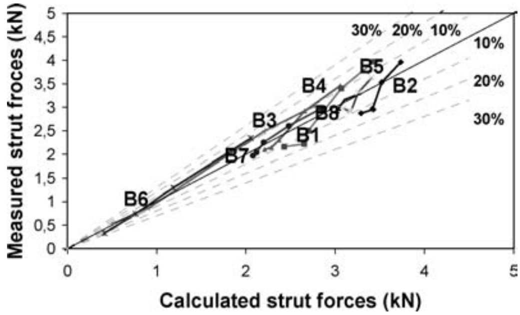

Figure 5. Strut forces for cases B1 to B8 with 1 strut – variation during the last 4 excavation phases before failure.

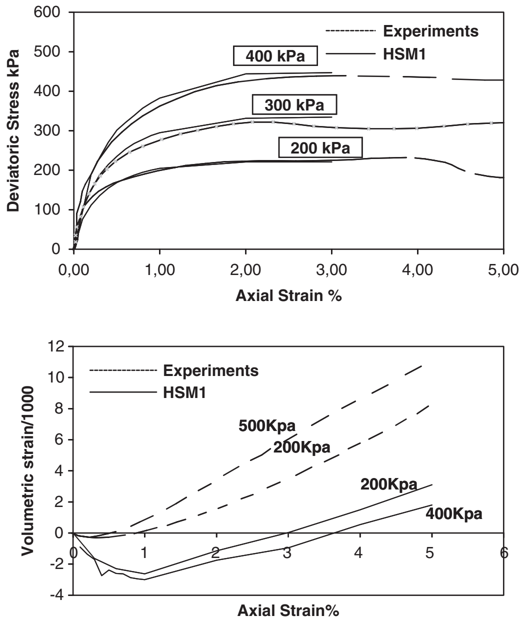

Figure 6. Simulation with HSM1 parameters of the biaxial tests with 200, 300 and 400 kPa.

0.6 m of depth (values back calculated on test B4 with 1 active strut of low stiffness).

In all the presented calculations the difference with the measured strut forces and wall displacements do not exceed 20% (see Figure 5 for the 1-strut cases B1 to B8).

A sensibility analysis is performed considering all the parameters of the Mohr Coulomb model. It appears that the differences between calculated and experimental results can not be satisfyingly reduced (especially in the cases with 2 struts). Therefore a more sophisticated constitutive law is required (Figure 6): the Hardening Soil model.

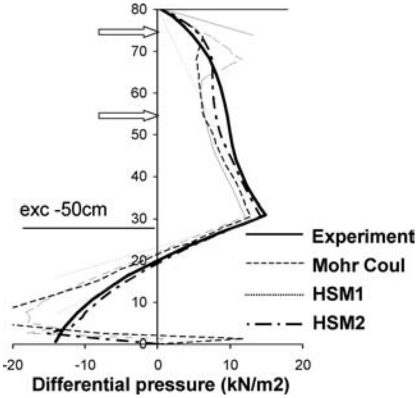

Figure 7. Differential pressure for test B12 and excavation at −50 cm: comparison of the experimental values with the results of SGRM, FE with Mohr Coulomb law and Hardening Soil Model (HSM1 and HSM2).

2.3 *FE back-calculation with Hardening soil model parameters based on triaxial tests*

Hardening soil model (Shanz *et al.* 1999) can capture soil behavior in a very tractable manner. The values of the different parameters were first fitted on the biaxial tests results (under a 200 Kpa confining pressure) performed on 20 × 10 cm samples by Kastner (1982). The 200 kPa confining stress is greater than the mean stress generally observed in Masrouri experiments, but in a first approach it was considered as more representative of the soil behavior than the biaxial tests using lower confining pressures: the size of the sample and the high value of the unit weight of Schneebelli material induces a non homogeneous stress state in the sample that could greatly affect the accuracy of the results. The obtained set of parameters noted HSM1 in the sequel provides a satisfying description of the biaxial test with 200 kPa or more confining stress (Figure 6).

This set of parameters was further used to back calculate the whole series of Masrouri's experiments (14 cases). It appears that these calculations do not reproduce well the test results in term of strut forces or differential pressures on the wall (Figure 7 represents only the results of differential pressures).

3 PROPOSED BACK ANALYSIS PROCEDURE

The aim of this procedure is to find the proper set of parameters for the constitutive soil model (Mohr Coulomb or Hardening Soil models) considered in the Finite Element simulations. Considering one particular test configuration, the parameters will be first obtained from the minimization of indicators based on differential pressures and struts forces errors. The resulting set of parameters will then be confronted with the results of the biaxial tests and of 14 configurations considered experimentally by Masrouri (1986).

$$E_{pd} = \frac{\int_0^z |P_1 - P_2| dz}{\int_0^z P_1 . dz}$$

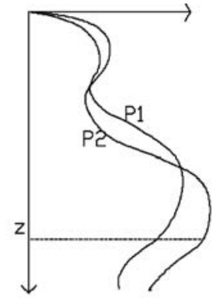

Figure 8. Differential pressure curves for indicator explication.

3.1 *Definition of indicators*

In order to consider the main features of the retaining wall behaviour, two indicators were defined: E_{sm} is related to the strut forces and E_{pd} related to the differential pressure on the wall (linked to the displacement profile).

$$E_{sm} = \frac{E_{b1} + E_{b2}}{f_1' + f_2'} \quad (1)$$

where $E_{b1} = f_1 - f_1'$ and $E_{b2} = f_2 - f_2'$ and with $f_{1(2)}$ and $f'_{1(2)}$ the strut force calculated or measured in the 1st (2nd) strut level.

The E_{sm} indicator is based on the error of the sum of struts forces: therefore an error on the strut force f_1 can be compensated by f_2.

The E_{pd} indicator (Figure 8) takes into account the absolute value of the difference between the measured and calculated differential pressures (respectively P_1 and P_2), divided by the integral of the measured differential pressure P_1. Integrals are calculated from the top of the wall to 10 cm below the final excavation level.

E_{pd} is the main indicator while the possible inaccuracy of the strut force measurement especially at the beginning of the excavation makes the E_{sm} indicator a second validation indicator.

3.2 *Parameter optimization for Mohr Coulomb model based on a 2 strut excavation test*

The determination of a second set of parameters for Mohr Coulomb model is performed by fitting the final results of a 2 struts excavation case (excavation level −50 cm). The particular case of test B12 (corresponding to passive struts with low stiffness) was selected because it clearly appears that the SGRM fail

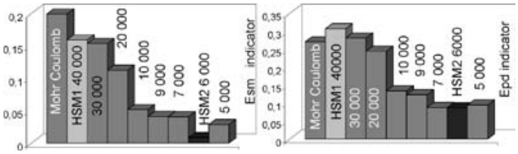

Figure 9. Variations of indicators E_{sm} and E_{pd} with E_{50}^{ref}.

to reproduce the differential pressure diagram obtained in such configuration.

Only one independent parameter was used for fitting the results, the elastic modulus E. The other parameters are kept constant (c, ϕ, γ ...). The final value of E is determined through the optimization of the indicators E_{sm} and E_{pd}. Actually, a linear variation of E with the depth is considered:

$$E = E_0 + \alpha z \quad (2)$$

Both the initial value of the elasticity E_0 and its variation with depth α were considered in the optimization procedure. It appeared that none of these parameters could be modified to improve the description of the experimental results, indicating that the Mohr Coulomb model is unable to reproduce important features of the soil behaviour involved in the global behaviour of the retaining wall.

3.3 *HSM model optimized by fitting on a 2 strut excavation test*

The determination of a second set of parameters for Hardening Soil Model (noted HSM2 in the sequel) is performed by fitting the final results of the same 2 struts excavation case (B12 excavation level −50 cm) as in section 3.2.

Only one parameter was used for fitting the results, the reference stiffness modulus E_{50}^{ref}. The other parameters are either constant (c, ϕ, γ ...) or keep a direct relationship with E_{50}^{ref}. For example:

$$E_{oed}^{ref}=1.5\,E_{50}^{ref} \quad \text{and} \quad E_{ur}^{ref}=3\,E_{50}^{ref} \quad (3)$$

The final value of E_{50}^{ref} is determined through the optimization of the indicators E_{sm} and E_{pd}. Considering the accuracy of the experimental results (in particular the procedure leading to the differential pressure diagrams), E_{50}^{ref} is determined with a precision of 1000 kPa (Figure 9).

The value of $E_{50}^{ref} = 6000$ kPa is chosen because it appears to minimize both indicators. HSM2 is further used to simulate the biaxial tests with low confining stress (50 and 100 kPa). Figure 10 shows a good agreement with the experimental results, even though the latter can be affected by the non-homogeneity of the initial stress state in 20 cm × 10 cm samples.

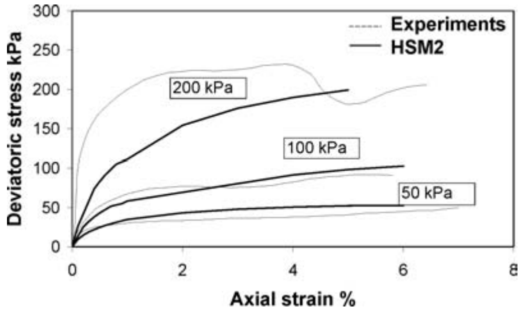

Figure 10. q-ε_1 biaxial curves: experimental results and calculated values with HSM2.

The proposed back analysis procedure shows its ability to verify or justify the HSM parameters that accurately describe the biaxial test results.

3.4 *Back calculation of the 14 configurations tested by Masrouri (1986)*

The HSM2 set of parameters is now used to back calculate the 14 different tests (Table 1). Figures 11 and 12 present the differential pressure diagrams obtained with HSM2 and other methods and the ratios f/f' of the calculated to the measured strut forces respectively for tests B1 to B8 (1 strut) and tests B10 to B13 (2 struts).

The strut forces and the differential pressure are well represented compared to the classical methods, the SGRM method or Finite element analysis with a simpler constitutive model (Mohr Coulomb). The effect of the prestressing of the strut is well reproduced by that procedure (comparing B3 and B5 in Figure 11 or B10 and B11 in Figure 12), as well as the arching effect (case of B10 in Figure 12) and the influence of the strut stiffness (comparing B1 and B7 in Figure 11).

The proposed back analysis procedure and the resulting set of parameters HSM2 show their efficiency in all the configurations (unlike the SGRM where the arching effect and the prestressing on the struts are not well reproduced).

3.5 *Summary*

Figure 13 presents a comparison of the values obtained for the selective indicator E_{pd} in 3 cases of excavation and for the SGRM, Mohr Coulomb, HSM1 and HSM2 models:

– B3 corresponds to a single passive and low stiffness strut
– B12 uses 2 passive low stiffness struts
– and B10 2 passive rigid struts.

In all of these 3 cases, it appears that the HSM2 set of parameters clearly minimizes the error between experimental and numerical results.

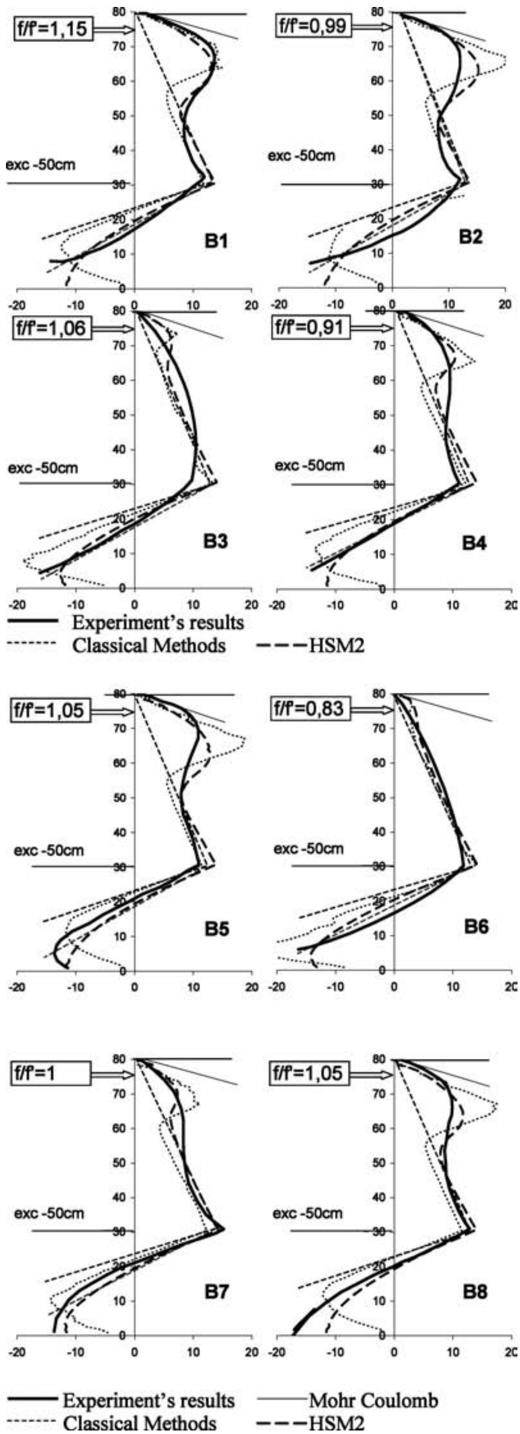

Figure 11. Differential pressure diagrams and strut forces obtained with HSM2 (excavations with 1 strut at −50 cm) compared with experiments, Mohr Coulomb, and classical methods results.

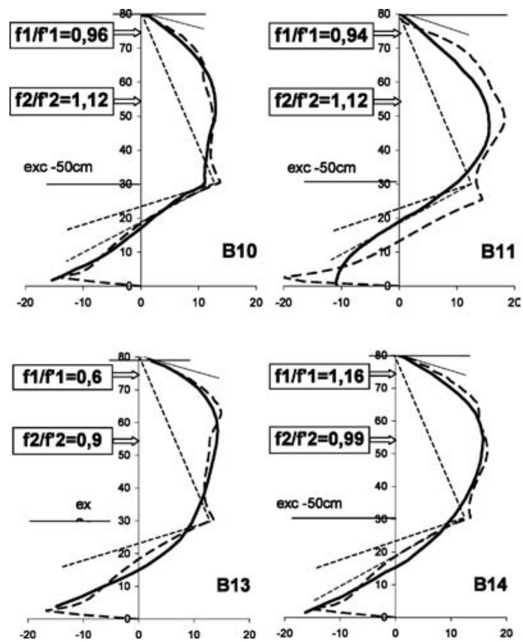

Figure 12. Differential pressure diagrams and strut forces obtained with HSM2 (excavations with 2 struts at −50 cm) compared with experiments and classical methods results.

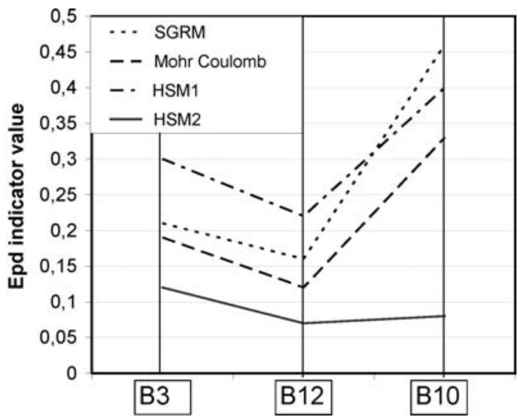

Figure 13. E_{pd} indicator calculated for tests B3, B12, B10 (excavation −50 cm), for the SGRM, HSM1, HSM2 and Mohr Coulomb calculation.

With the same constitutive model (HSM1 and HSM2) and with the same parameters except E_{50}^{ref}, the error is divided by 3 to 5.

4 CONCLUSION

A comprehensive series of 14 small scale experiments on flexible retaining walls with different strut stiffness and prestressing is used to validate a numerical back calculation using the Hardening Soil Model. The final set of parameters HSM2 is fitted on one single test (B12 at the final excavation level). The proposed model is based on the simultaneous minimization of two indicators E_{sm} and E_{pd} respectively related to the strut forces and differential pressure diagram. The verification of the proposed back analysis procedure showed that the HSM2 model gives the most acceptable description of the differential pressures and forces on the struts in all of the 14 tested configurations compared to the SGRM method or a Finite Element approach with either Mohr Coulomb model or Hardening Soil model with parameters based on biaxial tests (HSM1).

Further developments will include the verification of the ability of proposed back-analysis procedure to determined the HSM parameters not only on E_{sm} and E_{pd} at the final excavation level but also on intermediate levels, for example the first excavation step after the installation and prestressing of the lower strut.

Despite the already mentioned difficulties, further validation of the procedure on well-instrumented excavation sites will also be tested.

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