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Methodology for real-time adaptation of tunnels support using the observational method

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Abstract. The observational method in tunnel engineering allows the evaluation in real time of the actual conditions of the ground and to take measures if its behavior deviates considerably from predictions. However, it lacks a consistent and structured methodology to use the monitoring data to adapt the support system in real time. The definition of limit criteria above which adaptation is required are not defined and complex inverse analysis procedures (Rechea *et al*. 2008, Levasseur *et al*. 2010, Zentar *et al*. 2001, Lecampion *et al*. 2002, Finno and Calvello 2005, Goh 1999, Cui and Pan 2012, Deng *et al*. 2010, Mathew and Lehane 2013, Sharifzadeh *et al*. 2012, 2013) may be needed to consistently analyze the problem. In this paper a methodology for the real time adaptation of the support systems during tunneling is presented. In a first step limit criteria for displacements and stresses are proposed. The methodology uses graphics that are constructed during the project stage based on parametric calculations to assist in the process and when these graphics are not available, since it is not possible to predict every possible scenario, inverse analysis calculations are carried out. The methodology is applied to the "Bois de Peu" tunnel which is composed by two tubes with over 500 m long. High uncertainty levels existed concerning the heterogeneity of the soil and consequently in the geomechanical design parameters. The methodology was applied in four sections and the results focus on two of them. It is shown that the methodology has potential to be applied in real cases contributing for a consistent approach of a real time adaptation of the support system and highlight the importance of the existence of good quality and specific monitoring data to improve the inverse analysis procedure.

Keywords: tunnel; observational method; inverse analysis; numerical modeling

1. Introduction

The excavation of a tunnel involves complex soil-structure interactions that imply the systematic use of bi-dimensional or tri-dimensional numerical models in the conception and design

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stages (Van Eekelen *et al*. 1997, Do *et al*. 2013, Clough *et al*. 1985). Also, the uncertainties related to the characteristics of the geomaterials found during the excavation increase the difficulties in the definition of the most appropriate excavation method and support system (Mollon *et al*. 2009a, 2009b, 2013).

There are two main approaches in the design of tunnels: the traditional and the one based on the observational method. In the traditional design, a single set of parameters is adopted for each geotechnical zone to calculate and predict the behaviour of the structure. Monitoring data is used to verify if the predictions are close to the observed response and normally no adaptation is made during construction. Therefore, and for safety sake, the characteristics of the ground are normally underestimated and therefore the construction cost is higher.

The observational method (OM) (Terzaghi and Peck 1948, Peck 1969 Wakita and Matsuo 1994) makes use of the monitoring data to adapt the project (the support and the excavation method) during the construction stage, which allows optimizing the final cost of the construction and address safety in a more rational way. In this context, numerical methods can be coupled with backanalysis techniques and using measurements carried out during the excavation it is possible to re-evaluate geomechanical parameters efficiently and update the predictions while the underground work is advancing. The new optimized set of parameters allows an improved construction of the project.

In backanalysis the complexity of the adopted numerical model is crucial. If the model is too simple it will not translate reality in a proper way and if it is very complex it can lead to prohibitive calculation times so a balance has to be achieved. In simpler cases (mainly in terms of geology) a 2D model can provide acceptable results and due to the lower calculation times it allows performing a wider range of parametric calculations if needed. In more complex cases, for instance in complex geologies or if the tunnel face behavior is highly important, a 3D model would be preferable but it would also increase considerably the computational costs. However, with the advances in computer science, coding of the backanalysis algorithms, parallel and distributed calculations, etc. calculation times that are nowadays prohibitive will decrease rapidly in a short period of time.

The application of the OM and the adaptation approach presents however some difficulties which mainly depend to the considered application: for each application, new criteria have to be defined. For instance there is no clear criteria of the observational limits after which the support or the excavation process should be re-evaluated. These observational limits depend on the case one is dealing with but at the same time their definition in many cases is very difficult mainly in the initial design stages. Moreover, the adaptation of the support in real time can be difficult since elaborated calculations of inverse analysis can be very time consuming and inconsistent with the time schedule for the development of the work.

In this work it is intended to contribute to improve the practical applicability of the OM in the real-time adaptation of the excavation sequence and support to the real ground conditions. In a first stage guidelines for surveillance and alert limits are established for measured displacements and stresses. Then, a practical methodology for the real-time adaptation of the excavation and support based on the OM is presented. The application of the methodology starts in the project phase where graphics are built based on parametric studies varying the support system, geomechanical parameters and advancement step. Each graphic corresponds to a different support system computed with a certain set of geomechanical parameters. The graphics allow one to know in each excavation stage the safety zone the work can be contextualized and also if the geomechanical parameters were correctly estimated. If not, another graphic can be used in order to check if with other support and/or more realistic parameters, the deformation and stresses limits are acceptable. This step will allow, if necessary, an adaptation of the project in real time.

When there are no alternative graphics and there is a need to re-evaluate the support and excavation methods, inverse analysis for the identification of better suited geomechanical parameters is carried out. Then, based on these parameters the project can be re-evaluated.

The graphics are only valid for underground works in consideration and involve a significant computation burden since many scenarios have to be considered. However, the construction of the graphics is also a way to plan and systematize the information produced by customary parametric calculations normally carried out in design stages. The graphics can then be more useful in the construction stages where normally less time for computation tasks is available.

2. Description of the methodology

2.1 Overview

To apply the methodology it is necessary, in a first step to collect a data set with information normally needed to carry out the project of an underground work based on the OM (Dias and Kastner 2005), namely: (a) information related with the section geometry and the excavation site (type of support profile, advance step, overburden); (b) information about the type of soil/rock formation obtained through the survey carried out in the project phase, namely geomechanical parameters; (c) definition of standard support and excavation schemes adapted to type of expected formation; (d) design of a monitoring plan (measuring devices, measurement frequency, distribution of measurement profiles,...) and establishment of surveillance and alert limits.

Based on this information, a large number of numerical calculations, considering different scenarios, are carried out, which allows the plot of graphics which summarizes in a consistent way the results of these parametric calculations. Each graphic is drawn for a certain section of the tunnel. A total of 4 zones are defined in these graphics which allow for the safety assessment of the analyzed section based on monitoring data.

Using the appropriate graphic it is possible to situate the safety scenario where the section is located at that instant in terms of the pre-defined deformation and stresses limits. Based in the identified scenario it is possible to define if an adaptation of the support is needed. Also, it is possible to verify if the geomechanical parameters used to construct the graphic are correct or not. If the parameters were underestimated another graphic should be used. If this graphic is not available then inverse analysis for the identification of proper parameters and validation of the support using this new set together with numerical models can be carried out. If the parameters are correct or were overestimated it is necessary to analyze if an adaptation of the support is needed. If no adaptation is needed the process stops, otherwise inverse analysis must be carried out based on this set of parameters and parametric analysis should be conducted together with the numerical models to perform the calculation of a new support system. The overall methodology is represented in Fig. 1. In this context, the application of the methodology involves four main steps:

- Step 1: collection of data (information about the formations obtained by surveys carried out in the excavation face and monitoring data);
- Step 2: determination of scenarios through the use of the graphics built during the project stage and evaluation of the geomechanical parameters adopted for their construction;
- Step 3: depending on the scenario identified in step 2, it may be necessary to use another

graphic to validate the previous scenario or decide to apply inverse analysis to validate or adapt the support;

- Step 4: the use of inverse analysis to accurately assess the geomechanical parameters to validate or optimize the adopted support.

The use of inverse analysis to identify proper geomechanical parameters in the framework of the methodology is carried out in the following cases (Fig. 2):

(a) If an adaptation or reinforcement of the support is needed and it is intended to optimize its design for the next excavation stages. This optimization is carried out using the geomechanical parameters identified by inverse analysis and parametric studies;

(b) If actual geomechanical parameters are better than the ones used to draw the graphic and no alternative graphics are available. This will allow evaluating the real scenario and validate or not the adopted support. If the scenario is far from the allowable limits a lighter support or a larger advance step can be adopted.

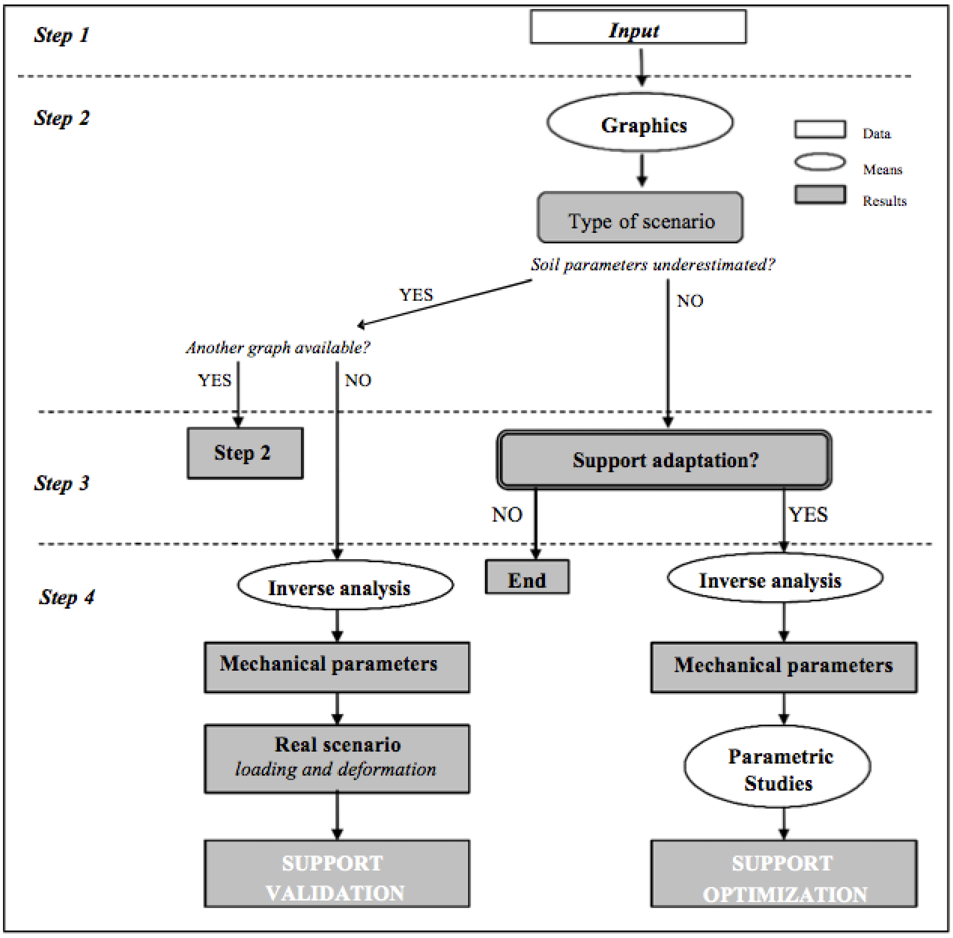

Fig. 1 Proposed methodology for the support adaptation

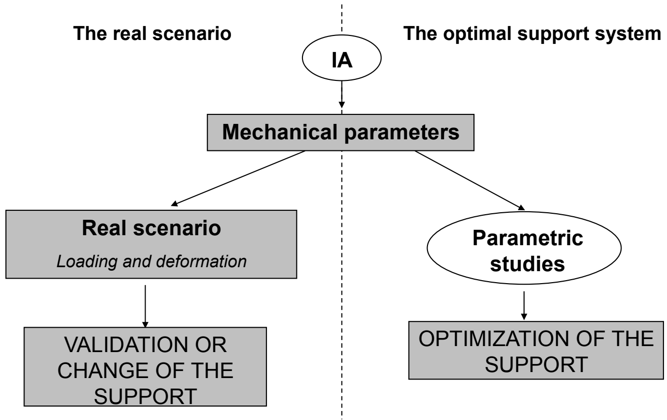

Fig. 2 Scheme of how to use inverse analysis in the scope of the developed methodology

In practical terms the methodology should be constrained by:

- Support design changes that does not imply additional costs;
- The numerical model should be simple enough to avoid high computational cost and complex enough to be representative mainly in complex geological sites;
- Definition of the backanalysis process and the relevancy of constitutive soil models and geomechanical parameters, namely which parameters to identify and their number.

2.2 Surveillance and alert limits

In the developed methodology two types of limits are considered, namely the deformation limit (U/r) where U is the convergence of the walls and r is the equivalent radius of the tunnel, and the stress limit (σ / σ_{adm}) where σ is the stress in a given structural element and σ_{adm} the allowable stress for the element.

The proposed limit values (Table 1) for U/r were derived from the average values of convergences observed in five French tunnels cases: Toulon, Foix, Tartaiguille, Lambesc and Schirmeck tunnels (Eclaircy-Caudron 2009) where the convergence of the tunnel walls was limited by an appropriate support implemented in time. These limits range approximately between 0.1% and 1% according to the stiffness of the support (Allagnat 2005). The value of these deformation levels can be fine-tuned for each specific case if the geotechnical survey and/or experience allow a better understanding of the real ground behavior and after the calculation of numerical models.

The lower limit corresponds to a value below which it is appropriate to modify the excavation process or lighten the adopted support profile. A surveillance limit corresponds to a value beyond which a more detailed analysis of the observational data and its evolution must be made and, if the

Table 1 Definition of three types of deformation limits

Limit classification	Low	Surveillance	Alert
U/r	0.25%	0.5%	1%

alert limit is reached the construction must be modified and the structure reinforced before continuing (support profile changes, changing the length of excavation...).

Stress limits depend on the maximum allowable capacity for each structural element. In the case of a mixed support of the walls, composed by shotcrete and steel truss, the stress existent in the truss and in the concrete are compared respectively with the allowable stress of the steel and the concrete if the two stresses are measured. For structural elements only the alert limit exist and if installed stresses reach this limit the construction must be altered and the structures reinforced.

2.3 Construction of the graphics and their application in the field

The presented graphics refer to a specific tunnel section having known depth of cover and geometry, excavated through a known technology, having specific supports, surrounded by a rock/soil mass described by a specific constitutive model (Hejazi *et al*. 2008). So it will be necessary to recreate them if anyone of these parameters change.

The three main stages of the graphics construction are:

(1) The definition of the nature and values of the surveillance and alert limits for the structure;
(2) The identification of different scenarios that may be encountered for different support schemes and excavation sequences during the construction of the structure. The set of states corresponding to each situation is determined from numerical calculations performed with two sets of parameters (the most probable and the most adverse set of characteristics for each type of formation) and other possible combinations of parameters (range of geometrical parameters for each profile: overburden, advance step...). At this stage many numerical calculations are required;
(3) Graphical representation of all scenarios by support profile and set of geomechanical parameters. Other steps are previously necessary, like the geotechnical characterization before the beginning of construction, which will allow defining the types of soils and their location along the tunnel

Each graphic is obtained for a specific section and for each support profile and geomechanical parameters set with the help of numerical calculations carried out with varying characteristics of overburden and advance step. Since a large number of calculations are needed, in this stage 2D calculations are better suited so that computational times are not prohibitive.

Each graphic includes the advance step in the abscissa axis between two pre-defined limits. In the ordinates it is represented a relative displacement in the keystone or in the sidewall, namely the difference between the calculated value of the displacement at a distance x from the tunnel face and the displacement value calculated upon the installation of the support ($Um = u(x) - u(d)$) (Fig. 3).

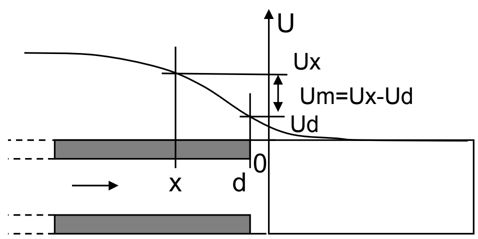

Fig. 3 Scheme for the Um calculation (ordinates of the graphics)

Then, curves representing the thresholds of deformations and stresses are plotted. To draw these curves a set of numerical calculations are performed. Keeping constant the remaining parameters (support profile, geomechanical parameters and advance step), the overburden is varied until the different selected thresholds of deformations *U*/*r* and stresses are obtained and the correspondent Um values for each threshold are plotted in the graphics. For instance, to obtain the 0.25% threshold line for a given advance step, the overburden is varied until this value of *U*/*r* is obtained and the correspondent Um is then plotted in the graphic. Several points are then obtained by varying the advance step and used to draw the curves (Fig. 4).

This procedure also allows obtaining "overburden lines" to which correspond the expected displacements *Um* for a given advance step and overburden (dotted lines in Fig. 5). These lines allow, by comparing the real measurements of Um with the ones predicted by the graphic, to evaluate if the geomechanical parameters were correctly evaluated in the project stage.

For each support profile a total of four graphics are obtained. Two using the two sets of geomechanical parameters, namely the most probable and the most conservative though realistic set; and for each set of parameters, one for each main displacements, specifically in the sidewall and in the keystone. Fig. 4 shows an example of a graphic for a certain support profile.

In the graphics, a total of four curves that define four zones (or scenarios) are obtained.

- Zone 1: the lower limit is not exceeded (*U*/*r* < 0.25%) so the support can be changed to a lighter one;
- Zone 2: the lower limit is exceeded but the surveillance deformation limit is not reached (*U*/*r* > 0.25% and *U*/*r* < 0.5%). In this case, optimization of the support may be carried out.

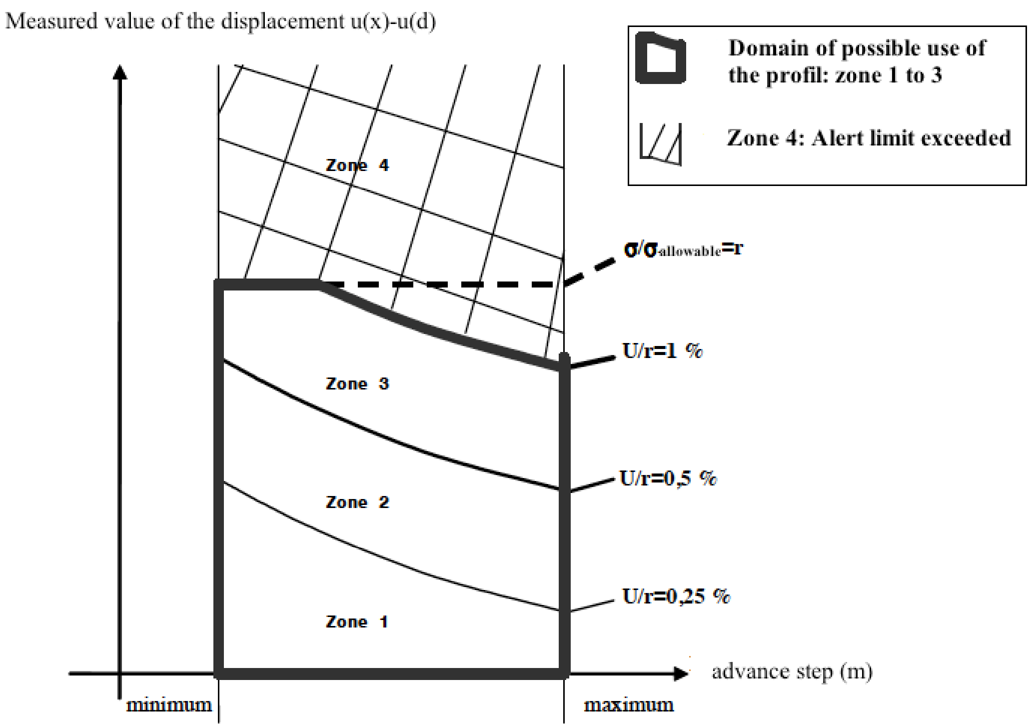

Fig. 4 Example of a graphic for a certain support profile

However, attention should be paid that in this case the scenario does not cross for Zone 3.

- Zone 3: surveillance and lower limits are exceeded and the deformation of the section is important (U/r > 0.5%). In this case attention must be paid and it is necessary to confirm that the section does not crosses to zone 4 in the graphic;
- Zone 4: the alert level in stress and in deformation is exceeded such that the safety of the structure is threatened. Another support system should be adopted for the following excavation steps.

In practical terms the objective is to determine the scenario in which the work is at a given construction time by registering the measured values on the graphic correspondent to the used support profile, advance step and adopted set of geomechanical parameters. This scenario is translated by a point in the graphic as it is shown in the example of Fig. 5 (point A).

So the first step in the application of the graphics is to choose the appropriate one considering the type of support used and a set of geomechanical parameters (the most probable or the most conservative one since both are available). Then considering the advance step used on site the measured displacement is plotted in the graphic allowing the identification of the scenario. In the example of Fig. 5 a deformation of 4.5 mm is observed correspondent to an advance step of 2 meters. Therefore reaching zone 3, it would be necessary to increase the frequency of measurements and to pay attention to the scenarios changes.

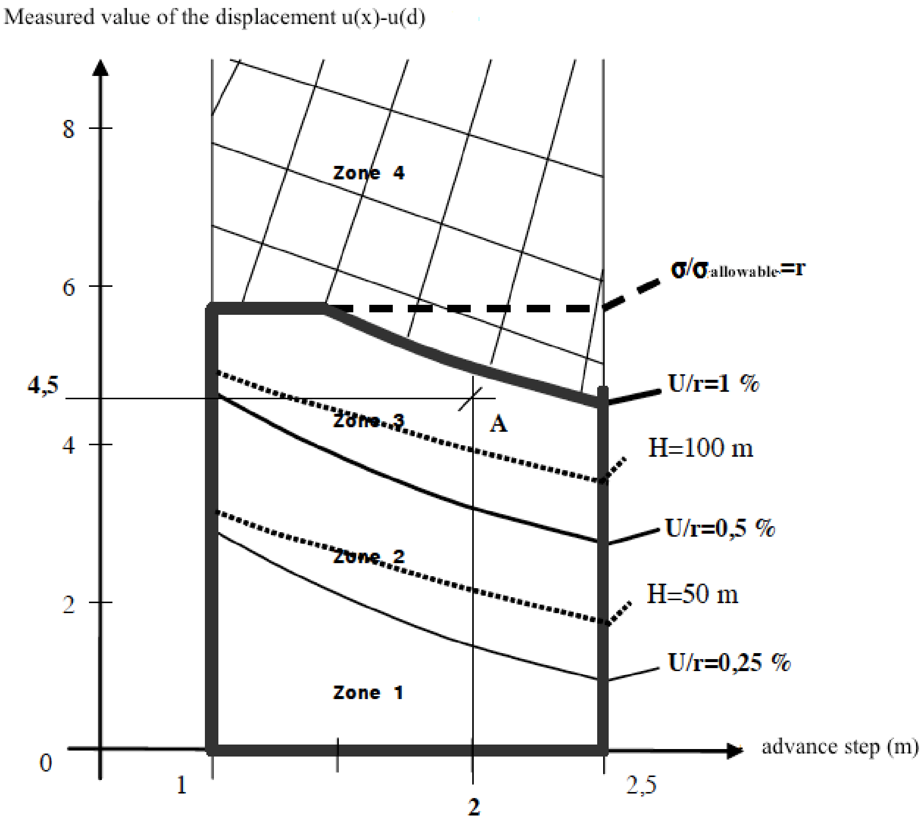

Fig. 5 Example of the graphics use

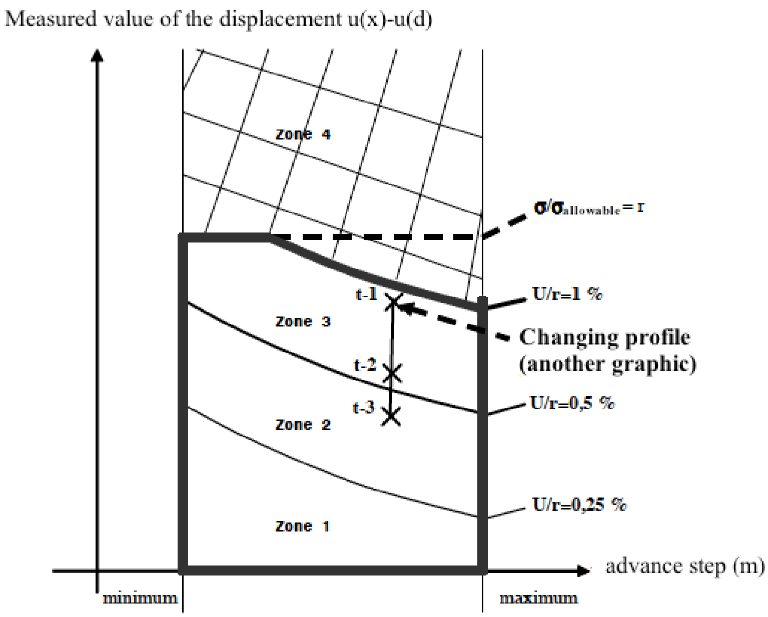

Fig. 6 Example of scenarios changes over time

The graphic also provides indications on the geomechanical parameters adopted for the excavated formation. To perform this evaluation it is necessary to compare the overburden indicated in the graphic and the real overburden of the tunnel. If the real value of the overburden is higher than the value of the overburden read in the graphic, then the real geomechanical characteristics of the formation are of higher quality than the ones considered for the graphic construction. If the difference is large the scenario is no longer valid. In this case it is necessary to use another graphic if available or carry out an inverse analysis in order to identify more accurately the geomechanical parameters.

Another possible use of the graphic in the adaptation process includes an analysis of trends over time. Measurements made at instant t can be connected to previous ones allowing the analysis of the scenario evolution. Fig. 6 shows an example of a path where, for example, at time *t*-3 the scenario is in zone 2, and the displacement evolution in relation to instant t-2 is small. However, at time *t*-1, even though the scenario is still in zone 3 there was a considerable change compared to *t*-2, then an alert warning is reported since this path may conduct to zone 4. This type of analysis permits the detection of anomalies and the implementation of pro-active actions or urgent modifications.

The construction of these graphics allow taking advantage of current parametric studies normally carried out in the design stages in a systematic manner in order to, together with pre-defined surveillance and alert limits, establish a functional tool to support the application of the OM in real-time.

3. Description of the case study

The "Bois de Peu" tunnel is a road tunnel with two tubes, the upward tube (M) with length of

511 m and the downward (D) with 521 m. The difficulty of this project was not due to its length, but due to soil heterogeneity and properties. An exploration gallery, permitted to highlight the presence of eighteen geological units. Among these units, four types of materials were distinguished, namely: limestone, marl, clay and interbeddings of marl and limestone. The used excavation method was full section with explosives for rigid formations and with road header for the softer ones (Eclaircy-Caudron 2009).

Four support profiles were defined in the design phase, but two other profiles were added during the excavation phase (P1, P2, P2bis, P3, P4 and P4bis). To apply and test the methodology a total of 4 sections of the tunnel were analyzed: 3 on the M tube and 1 on the D tube. These sections were selected based on the quality of available measurements.

The M1 and M4 sections are located in the upward tube and the support profile type is P2 composed by steel truss type HEB200 with shotcrete between the truss but not at the excavation face. The advance step is 1.75 m ± 0.75 m. Section M6 is also located in the M tube and the profile type is P4 formed by shotcrete and steel truss HEB200 and the advance step is 1.5 m with a pre-support by umbrella vault. The last section, D4 located at the downward tube has the same support profile than M6.

4. Application of the methodology

The methodology was applied in four sections with different geological features and implemented monitoring system. Two sections (M1 and M4) have only traditional measures namely convergences and leveling and the other two (M6 and D4) have more specific ones including displacement measurements in the tunnel face and stresses in the truss.

Table 2 resumes the fundamental issues concerned with the sections and all types of measurements made during the excavation. The application of the methodology was carried out firstly through the identification of scenarios and then, inverse analysis techniques were used.

Two optimization algorithms, a deterministic and a probabilistic, were used in the inverse analysis calculations. Backanalysis plays a very important role in the methodology. Therefore, the use of two different algorithms is intended to highlight the importance of its choice in the results of backanalysis and therefore in the results of the developed methodology. These algorithms were coupled with the computational code in order to perform the identification process in a more efficient way. The deterministic method was the one provided by the software SiDolo (SiDolo 2003). It uses a hybrid algorithm, which combines two traditional optimisation techniques, namely, a gradient based algorithm and a variant of the Levenberg-Marquardt method to accelerate convergence when the process is close to the solution (Eclaircy-Caudron 2009).

The other optimization algorithm used was an Evolution Strategy (ES). The ES algorithms are search procedures that mimic the evolution of the species in natural systems. The ES work with populations of candidate solutions, requiring only data based on the objective function and constraints, and no derivatives or other auxiliary knowledge. ES work directly with the real representation of the decision variables (in this case a set of geomechanical parameters) in which an individual is a vector of real numbers (the decision's variable) and represents a potential solution for the optimization problem. They search the solution from an initial population (a set of points) normally generated at random and constraints are normally handled eliminating the points outside their range (Costa 2006, Miranda *et al*. 2010, Moreira *et al*. 2013). The error function used with both algorithms is a simple least squared error that measures the differences between

Table 2 Key features of the analyzed sections

Section	M1	M4	M6	D4
Type of formation	Marls-Limestone	Marls	Soft marls	Soft marls
Support profile	P2	P2	P4	P4
Advance step (m)	1.75	2	1.5	1.25
D_{face}(m) [1]	1.3	3	3.5	3.84
Overburden (m)	25	29	17	22
Type of measures	Conv. +Leveling	Conv.	Conv.+ Leveling + Radial disp.+ Deformations in the tunnel face	Conv.+ Leveling +Deformations in the tunnel face
Max. convergence in the sidewall (mm)	30	5	8	25
Max. displacement in the keystone (mm)	13	-	6	14
Max. deformations in the tunnel face (mm)	-	-	25	40

measured and computed displacements.

The results are presented only for two of the sections, namely sections M4 and M6. Section D4 present similar results to section M6 and the application of the methodology to section M1 showed that no adaptation of the support was needed.

For both sections the linear elastic perfectly plastic constitutive model with a failure criterion of Mohr-Coulomb's type was adopted. The parameters chosen to identify were the ones that showed higher influence on the observed displacements in preliminary calculations. The 2D numerical models were developed using CESAR-LCPC and the 3D model was developed using FLAC3D.

4.1 Section M4

Fig. 7 shows the graphic constructed for this section for the analysis of displacements in the wall. The point correspondent to the section at the considered instant is also plotted. The convergence measures of section M4 (5 mm) locates this section in the scenario of zone 2, which corresponds to an overburden of 35 m, which almost corresponds to the real case for this section ($H = 29$ m). Therefore, the geomechanical properties were slightly overestimated and an adjustment of the support profile can be performed.

Inverse analysis was applied in order to more accurately determine the geomechanical parameters and both inverse analysis techniques were used. Two different sets of parameters were established with three and four parameters to identify, namely: E (deformability modulus), c' (effective cohesion), φ' (effective friction angle) and λ (unconfinement rate). The Poisson coefficient and the dilatancy angle were supposed known (0.3 and 0° respectively). Table 3 shows the parameters sets to identify in each of the 4 tests that were carried out. The "initial values"

[1] D_{face}: Distance between the measured section and the tunnel face

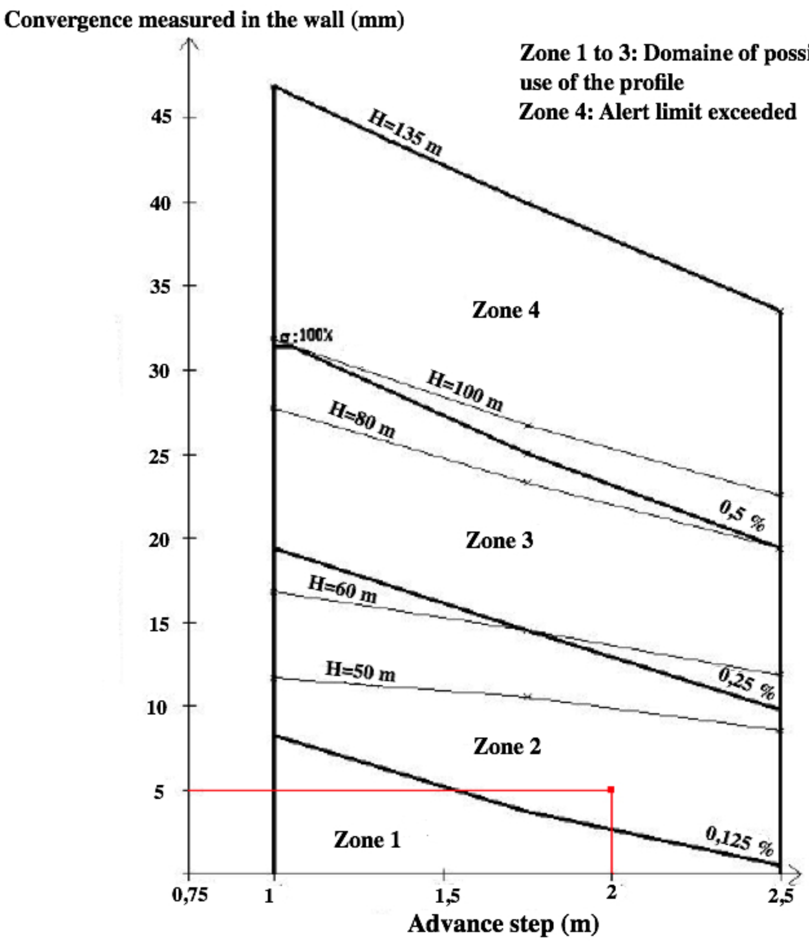

Fig. 7 Use of the graphic to evaluate the scenario for section M4

shown in this Table are only used by SiDolo since this kind of algorithms need an initial guess to start the search (the ES only needs the range of variation). The range of variation is a restriction that is valid for both algorithms. A 2D plain strain numerical model was used to simulate the section.

Table 4 shows the results of the inverse analysis for this section. It was not possible to identify a single set of geomechanical parameters due to the existence of only convergence measures in this section. Based in theoretical calculations it was concluded that complementary measures were needed, otherwise the problem is ill-posed (Eclaircy-Caudron 2009). The type and quantity of available monitored data and also the algorithm chosen in backanalysis play a central role in the success of the process. Eclaircy-Caudron (2009) showed that in a similar case to section M4 it would be needed, for instance, to add displacement along extensometers to the convergence measures to increase the possibility of obtaining a unique solution.

The optimized sets of parameters vary considerably from calculation to calculation; however, some sets present considerable lower error function values. For sets of three parameters the ES algorithm identified the following with the lowest error value: E = 102.16 MPa, c' = 247.06 kPa

Table 3 Initial values of the parameters for the inverse analysis

Test	Unknown Parameters	Range variation	Initial values		
			a	b	c
1	E (MPa)	$750 < E < 1600$	$E = 1200$	$E = 1500$	$E = 800$
	c' (kPa)	$210 < c' < 750$	$c' = 500$	$c' = 700$	$c' = 220$
	λ	$0.8 < \lambda < 0.9$	$\lambda = 0.86$	$\lambda = 0.89$	$\lambda = 0.81$
2	E (MPa)	$200 < E < 1600$	Equal to case 1		
	c' (kPa)	$100 < c' < 750$			
	λ	$0.7 < \lambda < 0.9$			
3	E (MPa)	$100 < E < 1600$	Equal to case 1		
	c' (kPa)	$75 < c' < 750$			
	λ	$0.6 < \lambda < 0.9$			
4	E (MPa)	$100 < E < 1600$	$E = 1200$	$E = 1500$	$E = 800$
	c' (kPa)	$75 < c' < 750$	$c' = 500$	$c' = 700$	$c' = 220$
	λ	$0.6 < \lambda < 0.9$	$\lambda = 0.86$	$\lambda = 0.89$	$\lambda = 0.81$
	φ' (°)	$25 < \varphi' < 43$	$\varphi' = 38$	$\varphi' = 38$	$\varphi' = 38$

Table 4 Parameter identification results for section M4

Test	Parameters	Optimized values			
		Gradient method			ES method
		a	b	c	
1	E (MPa)	$E = 750$	$E = 750$	$E = 750$	
	c' (kPa)	$c' = 210$	$c' = 713.4$	$c' = 210$	
	λ	$\lambda = 0.8$	$\lambda = 0.8$	$\lambda = 0.8$	
		Err. = 1.43	Err. = 1.47	Err. = 1.43	
2	E (MPa)	$E = 200$	$E = 200$	$E = 200$	
	c' (kPa)	$c' = 100$	$c' = 722.2$	$c' = 100$	
	λ	$\lambda = 0.7$	$\lambda = 0.7$	$\lambda = 0.7$	
		Err. = 0.15	Err. = 0.53	Err. = 0.15	
3	E (MPa)	$E = 101.9$	$E = 101.9$	$E = 152.7$	$E = 102.16$
	c' (kPa)	$c' = 518.1$	$c' = 720.1$	$c' = 135$	$c' = 247.06$
	λ	$\lambda = 0.6$	$\lambda = 0.6$	$\lambda = 0.6$	$\lambda = 0.61$
		Err. = 7.59×10^{-5}	Err. = 8.10×10^{-5}	Err. = 9.30×10^{-3}	Err. = 2.44×10^{-6}
4	E (MPa)	$E = 101.9$	$E = 101.9$	$E = 344.1$	$E = 340.46$
	c' (kPa)	$c' = 490.1$	$c' = 712.6$	$c' = 129.2$	$c' = 75.00$
	λ	$\lambda = 0.6$	$\lambda = 0.6$	$\lambda = 0.6$	$\lambda = 0.85$
	φ' (°)	$\varphi' = 38.6$	$\varphi' = 38.1$	$\varphi' = 26.5$	$\varphi' = 25°$
		Err. = 7.43×10^{-5}	Err. = 8.10×10^{-5}	Err. = 9.21×10^{-2}	Err. = 4.36×10^{-3}

and $\lambda = 0.61$. Using four parameters the gradient algorithm identified two sets with very close error values (8% difference), namely: $E = 101.9$ MPa, $c' = 490.01$ kPa, $\lambda = 0.6$, $\varphi' = 38.6$ and $E = 101.9$ MPa, $c' = 712.6$ kPa, $\lambda = 0.6$, $\varphi' = 38.1$. These results point out to a stable set of optimized parameters with exception of c'. The error function seems to be more insensitive to the variation of this parameter and a large interval between 247.06 kPa and 712.6 kPa was obtained with similar error values. However, the remaining parameters are considerably stable around a certain value so parametric studies for the support validation could be carried out varying the value of c'.

Based on these results the adopted support could be validated or changed to a lighter one. This could be done fixing E, λ and φ' with values around 102 MPa, 0.6 and 38.5°, respectively and carrying out a parametric study on the influence of c' in the support needs by varying it between, for instance 250 kPa and 710 kPa. This highlights how the OM can be used to tunnel support during construction.

4.2 Section M6

For section M6 that uses the support profile P4, no graphic was available, so the use of inverse analysis was also required. It was carried out using 2D and 3D numerical models and the parameters chosen to identify in this case were E, c' and φ'.

In 2D, the used model was axisymmetric in order to analyze the evolution of the deformations of the tunnel face. The observational points were the axial displacements in nine points and were the only monitoring data used with this model (Fig. 8).

The bolting was simulated by the introduction of a cohesion reinforcement in the zone before the tunnel face and the mixed support (steel truss and shotcrete) by a homogenized section (Dias *et al*. 2002, Wong *et al*. 2004, Dias and Kastner 2005, Dias 2011, Oreste and Dias 2012). The mesh extends over a 154 m length in the axial direction and 77 m in the transverse direction and the initial stress field is isotropic. The excavation phases considered in the model are very similar to

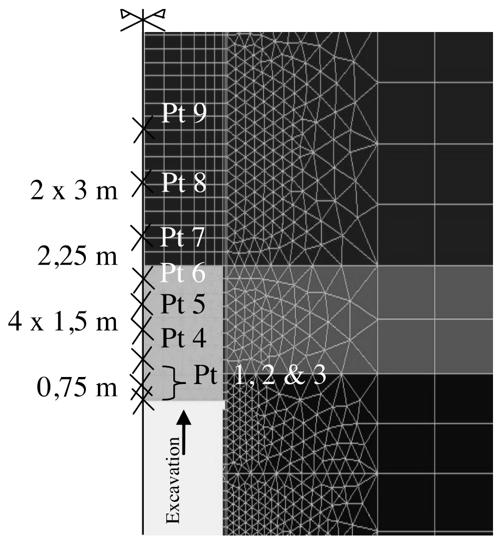

Fig. 8 Location of the measurement points considered in the inverse analysis process of section M6 with the 2D model

the ones considered in the project phase. A linear elastic perfectly plastic model with a Mohr-Coulomb failure criterion was adopted.

For the 3D model an anisotropic field stress was adopted with an earth pressure coefficient of 0.7. Bolts in this case were modeled individually as structural elements. All the available measures were considered in this 3D model, namely: vertical displacements in the keystone, horizontal displacements in the wall, radial displacements along an extensometer and longitudinal displacements at the tunnel face (Fig. 9).

Fig. 10 shows the computed deformations with both models in the tunnel face and the measured values using the design geomechanical parameters. The adjustment of the curves is satisfactory but the models have more difficulties to simulate the actual behavior near the tunnel face, which is probably due to the existence of a soil softening near the wall, which is not taken into account in

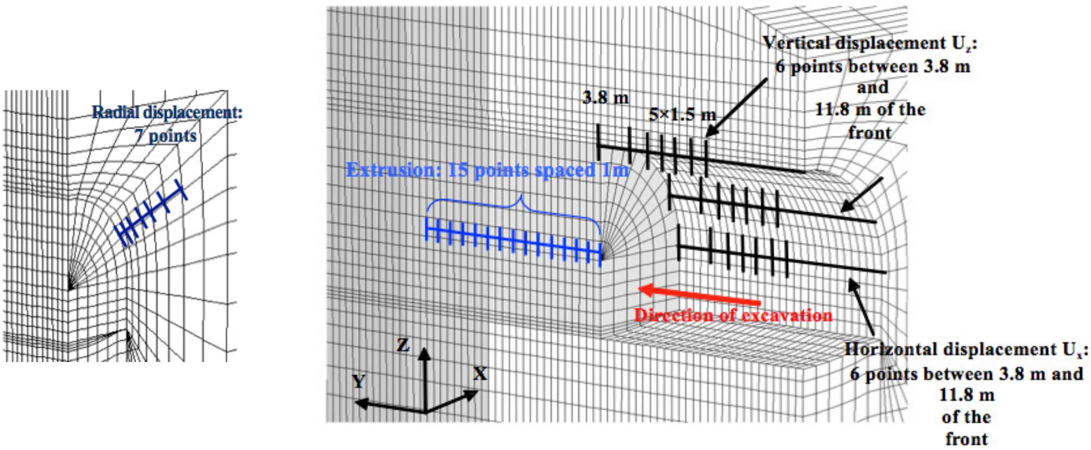

Fig. 9 Measurements considered in the inverse analysis process of section M6 with the 3D model

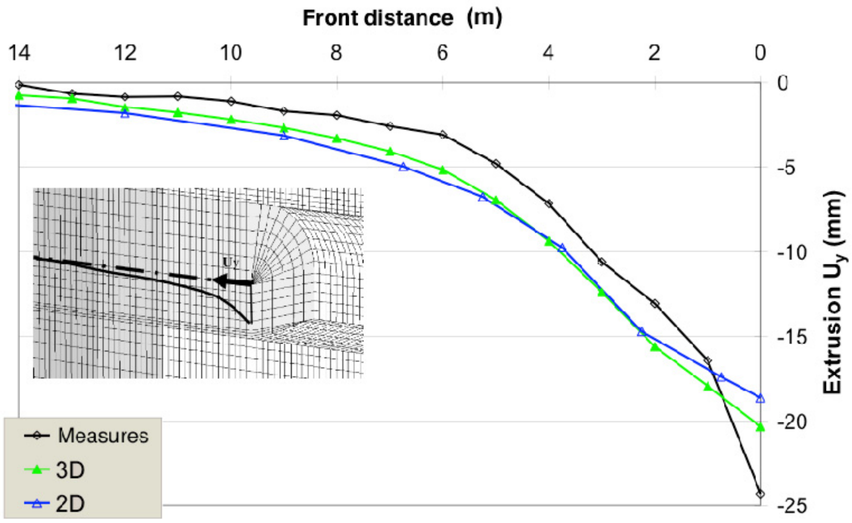

Fig. 10 Measured deformations at the tunnel face and comparison with the corresponding curves of the 2D and 3D models

Table 5 Comparison between the results of the inverse analysis for section M6

	Project values	2D	3D
E (MPa)	80	160-220	198-220
c' (kPa)	25-40	25-50	33-48
φ' (°)	13-17	22-28	24-28

the considered constitutive model. The results pointed out to more favorable geomechanical characteristics than the ones considered in project stage.

A great number of tests were carried out for this case considering many different ranges of variations for the parameters and initial values in 2D and 3D. Due to this large number of cases and the use of a 3D model only SiDolo was used to limit computation time since this algorithm, even though less robust than the ES, is more efficient since it takes less calculations to converge. In Table 5 a summary of all the tests is presented in the form of variation ranges defined based on the obtained results of the calculations. It can be concluded that no unique solution have been found. The identification process allowed however to identify a range of variation for each parameter. Identifications carried out using the 3D model can further reduce the range of variation found in 2D. In fact, the use of the 3D model together with additional monitoring data allowed fine-tuning the results and a considerable narrowing of the interval ranges of variation.

It can be concluded that the involved formation has much better geomechanical quality than defined in the project stage based on the geotechnical surveys. This is mainly true for the deformability modulus since the inverse analysis showed that this parameter is at least twice the value considered in the design stage. Also for the friction angle the difference is considerable.

Therefore, as in section M4 the support could be changed to a lighter solution since as referred, the geomechanical quality of the involved formation is much better than expected and considered during design. Parametric studies could be carried out using the interval ranges identified in the backanalysis process to reach this goal. In this sense, the contribution of the inverse analysis in the use of the OM to adapt the support needs in real time is clear in this case.

5. Conclusions

In this work a methodology to assist the excavation and support adjustment in underground works based on the principles of the observational method and inverse analysis was presented. The methodology starts with the definition of surveillance and alert limits for displacements and stresses in structural elements. A proposal for these limits was introduced; however it can be adapted for other cases based on experience or other type of knowledge.

In the project stage graphics are built based on the characteristics of the project and numerical calculations. This stage of the methodology allows to organize and systematize current parametric studied developed in the project stage in a tool that can be useful during construction. These graphics allow, using monitoring data obtained during construction, to define the safety level in which the section can be contextualized at any given moment. Also it is possible to analyze the evolution of the safety levels in time. In this sense, safety is evaluated in a more direct and rational way and quick measures can be adopted if necessary. When there is no graphic correspondent to the real situation, inverse analysis is carried out in order to properly identify the geomechanical parameters to be used in a possible adaptation of the support.

The developed methodology was applied to the “Bois de Peu” tunnel which was characterized by considerable uncertainties concerning the heterogeneity of the soil. The methodology was applied in four sections and the results focus on two of them, namely sections M4 and M6. In both cases, inverse analysis of geomechanical parameters was carried out and two optimization algorithms were used for this task.

Through the application of the methodology, it was concluded that section M4 was stable but the geomechanical properties were slightly overestimated therefore an adjustment of the support profile could be performed. Inverse analysis was applied to identify geomechanical parameters values closer to the real ones but it was not possible to identify a single set of geomechanical parameters due to the existence of only convergence measures in this section. However, analyzing the results, mainly the calculations with lower error function value, the identified parameters only slightly varied with exception of cohesion. Using the identified parameters the adapted support could be validated or changed to a lighter one.

For section M6, no graphic was available, so the use of inverse analysis was also required. 2D and 3D numerical models were used and also no unique solution has been found. The identification process allowed however to identify a range of variation for each parameter which were considerably narrowed with the 3D model together with additional monitoring data. It was concluded that the formation has much better geomechanical quality than defined in the project. With this new set of geomechanical parameters identified by the inverse analysis the support could be adapted.

There are some differences in the results obtained using the two algorithms. The ES is normally more robust, i.e., in most of the cases it allows to identify better solutions or in other words solutions with lower error function values. On the other hand, it needs a substantial amount of calculations which can be a problem mainly when computationally heavy 3D models are used. In this sense, SiDolo shows to be more efficient providing a solution with a lower number of calculations. Therefore, the choice of the algorithm to use should be subjected to the time available for the task and the level of accuracy that one wants to have.

The results of the inverse analysis in both sections point out to the importance not only of the number but also the type of measurement carried out in the monitoring of underground structures in the results of inverse analysis calculations. This issue should be considered when defining the monitoring plans if it is intended to implement identification procedures.

In conclusion, with this application it was highlighted the potential use of the developed methodology for the real time adaptation of the support system and also the importance of good quality and specific monitoring data to enhance inverse analysis procedures.

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