Elastoplastic analysis of the effects of the advance of a double-track railway tunnel on the soil and on the tunnel support. In-situ testing and 2d modelling

Análisis elastoplástico de los efectos provocados sobre el terreno y el propio sostenimiento por el avance de un túnel de ferrocarril de vía doble. Ensayos in situ y modelización bidimensional

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Abstract
This study involves an elastoplastic analysis of the effects of the advance of a double-track railway tunnel on the soil and the tunnel support. The strength and deformation properties have been determined using in-situ testing with equipment designed and patented by the Soil Engineering Research Group at the University of Oviedo (GIITUO). The behaviour of the excavation face and the ideal trace are studied using a 2D model of the longitudinal cross section of the tunnel. In this work one study of the behaviour of the soil and the support during the different phases of excavation is given. From it, it is possible to say that in the initial phases of the excavation and with temporary support there are not significant deformations. Once the final support is applied, the maximum compressive stresses (7MPa) are concentrated in the crown zone and the maximum tensile stresses are in the base of the support of the top heading and the sidewalls (0.15-0.5 MPa).

Keywords: *In-situ* testing, FLAC 2D modelling, tunnel support, excavation length, elastoplastic analysis.

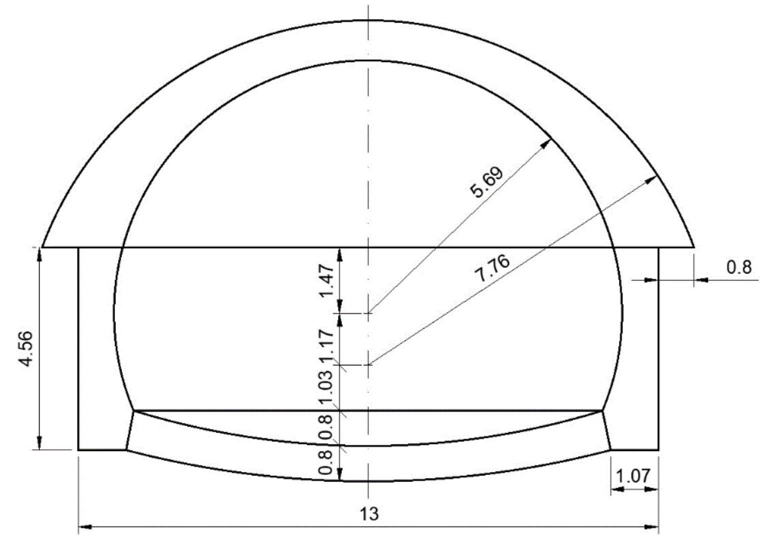
Resumen
En este trabajo se realiza un análisis elastoplástico de los efectos provocados sobre el terreno y sobre el propio sostenimiento por el avance de un túnel de ferrocarril de doble vía. Las propiedades resistentes y deformacionales del terreno se han determinado mediante ensayos "in situ" llevados a cabo con un equipo diseñado y patentado por el Grupo de Investigación de Ingeniería del Terreno de la Universidad de Oviedo (GIITUO). Con este procedimiento se garantiza la solidez en las simulaciones numéricas realizadas. En este trabajo se estudia el comportamiento del frente de excavación y el avance ideal mediante una modelización bidimensional de la sección longitudinal del túnel. Además, se ha llevado a cabo un estudio sobre el comportamiento del terreno y del sostenimiento a lo largo de las diferentes fases de la excavación. De él, se deduce que en las fases iniciales de la excavación y con el soporte temporal no hay deformaciones significativas. Una vez colocado el sostenimiento definitivo, las tensiones de compresión máximas (7 MPa) se sitúan en la zona de la corona y las tensiones de tracción (0.15 – 0.5 MPa) se sitúan en la base del sostenimiento del avance y en los hastiales.

Palabras clave: Ensayos *in situ*, modelización con FLAC 2D, sostenimiento de túneles, longitud de excavación, análisis elastoplástico.

This study describes the effects of the excavation and advance of a tunnel on the soil in which it is built and on its own support. The tunnel is a double-track high-speed railway tunnel located under the "Cerro de San Cristóbal" in the province of Valladolid, Spain. For the purposes of the analysis, the soil is regarded as homogeneous with elastoplastic behaviour and follows the Mohr-Coulomb failure criteria (Coulomb, 1776) because of using other advanced soil constitutive models hardly offer advantages over the Mohr-Coulomb model (Ruiz & Rodríguez, 2015). The strength and deformation parameters have been obtained by in-situ testing with equipment and a methodology developed and patented by GIITUO (Gonzalez-Nicieza, Álvarez-Vigil, Alvarez-Fernandez, Lopez-Gayarre, & Pizarro-García, 2011). They have been carried out a total of 6 tests distributed in three trenches. The subsequent characterization of the materials is carried out using a calibration model. The depth of the excavation is 50 m and the cross section of the tunnel is shown in Figure 1.

The Belgian tunnelling method, or traditional Madrid method, is used based on the execution of small top heading excavations in the keystone that are expanded and widened to form the vault. Supports are then installed using wooden props, metal struts and plates, before excavating the bench and applying its final support structure. The bench is divided into two half-benches (left and right), staggered by 8 m and finished by applying concrete to the sidewalls.

Figure 1. Cross section of the tunnel. Source: own elaboration, 2018.



The advance is carried out by excavating the soil and leaving a central column, so that, it is guaranteed that throughout the advance excavation there is always this protection column with a minimum length of 2 m.

The crown is initially supported with a 0.05 m layer of shotcrete. Metal panels and Bernold plates are then installed to act as casing for the pumped concrete used for the support structure. The total thickness of the concrete in the section is 0.80 m. The bench is excavated and surfaced in lateral trenches with a length of 4 m.

The Explicit Finite Difference Method (EFDM) (Lu & Hwang, 2017) is used for the analysis, and it is implemented using the FLAC 2D software developed by the International Company Itasca (Itasca (Itasca Consulting Group), 2008). The study analyses the excavation and support of the tunnel to know the behaviour of the soil and the support of the tunnel during the different phases of the excavation.

To simulate the effect of the proximity of the front during the first three phases in which temporary support is placed, a tension-state deconfinement rate of 50% has been assumed (stress relaxation factor). Thus the charges on the support are similar to those that it would support when the front is near. The elements of the temporary support are wooden props of 25 cm diameter every 0.5 m. These elements have been simulated as bar elements. The definitive support consists of concrete and it has been simulated as an elastic material.

The following sections give a detailed description of the method and the results obtained.

With the end of analysing the effects produced by the construction of a tunnel, on the soil and on its own support, one on-site testing (penetration test) campaign is carried out to determine the terrain properties, both deformational and resistant. Subsequently and from the previously obtained characteristics, two modelling are carried out with the FLAC 2D software: a cross section of the tunnel with all phases of excavation and support; a longitudinal section of the tunnel.

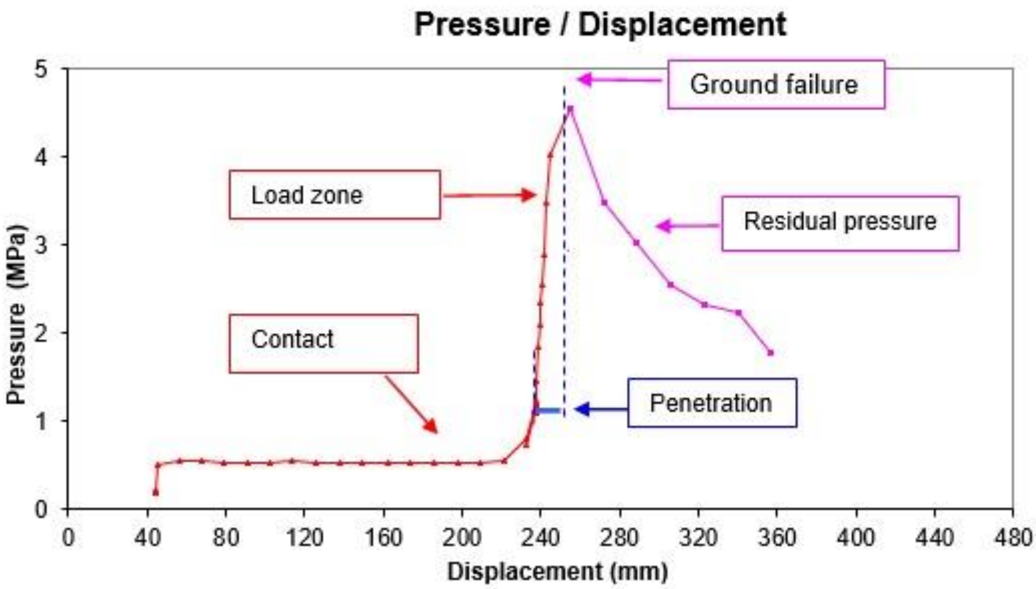
Penetration tests

The tunnel is located under the "Cerro de San Cristóbal" in the province of Valladolid, Spain. In the centre of the Duero Basin, it is filled with Tertiary and Quaternary sediments as well as limestone of lacustrine origin. Therefore, among its materials are sands, calcareous clays and saline mud, as well as limestone on the surface.

The geomechanical properties of the soil at the site of the tunnel excavation are determined using penetration tests that let plot the pressure-deformation curve for the soil. The tests are carried out by applying pressure to the soil using a pump-actuated hydraulic cylinder and the resulting deformation is measured as the load is applied. The cylinder is connected to a data capture system and is positioned perpendicular to the walls of the trench. The pressure, which is gradually increased until failure of the soil, is transmitted perpendicularly from the rod of the cylinder to the soil via the built-in hinge joint. When the breaking load is reached, a number of cracks appear and propagate rapidly, causing the trench walls to collapse. The equipment and measurement method were designed and patented (Gonzalez-Nicieza et al., 2011) based on the recommendations of the International Society for Rock Mechanics (Barton et al., 1979). They have been carried out 6 tests distributed in three trenches.

Figure 2 shows the pressure-displacement curve obtained for one of the tests, showing the failure pressure, the residual pressure and the penetration experienced by the hydraulic cylinder from the point it is installed until soil failure. The initial displacement corresponds to when the end of the hydraulic cylinder came into contact with the resisting stratum of the soil.

Figure 2. Penetration test curve using the hydraulic cylinder. Source: own elaboration, 2018.

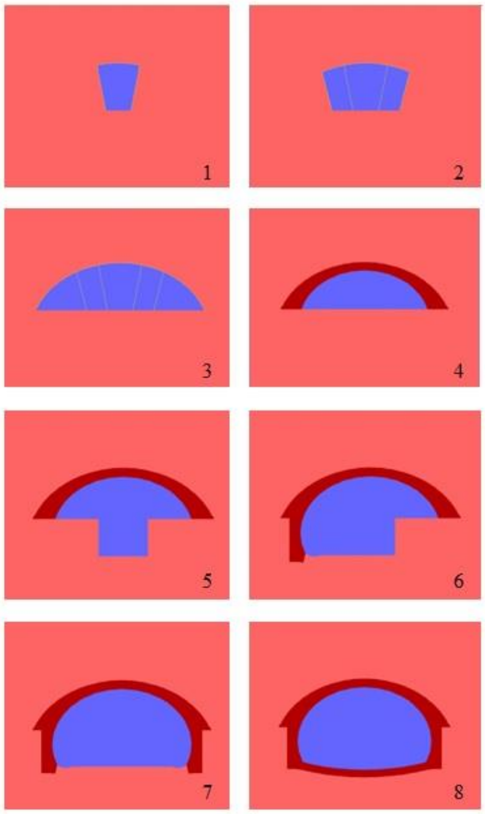


A simple calibration, with a calculated model generated using numeric methods, makes it possible to accurately determine the strength and deformation properties of the tested soil (Basarir, 2006; Funatsu, Hoshino, Sawae, & Shimizu, 2008). For that, the model uses different values of the variables (cohesion, friction, deformation moduli) and determines the pressure-displacement curve for the different cases. The properties of the soil are obtained comparing the theoretical curves with the real test values. The greater the similarities between the two curves, the closer the real values are to the properties used in the model.

2d cross section modelling

Once the properties of the soil are obtained, they are modelled a cross section of the tunnel and several construction phases (Arnau & Molins, 2011; Karakus, 2007; Negro & Marlísio, 2017) by mean of the FLAC 2D software (Itasca (Itasca Consulting Group), 2008). These phases are simulated to provide detailed knowledge of the main effects of the excavation and support structure. In Figure 3 are shown the simulated phases: advance of central pilot of top heading (1), excavation and support of the first expansion of the top heading (2), excavation and support of the final expansion of the top heading (3), application of final concrete support structure for the top heading (4), excavation of the central bench(5), excavation and support of the left bench (6), excavation and support of the right bench (7), and excavation and support of the invert (8).

Figure 3. Modelling phases: Source: own elaboration, 2018.



The stress state of the soil is varied to simulate the effect of the distance of the face with respect to the cross section of the tunnel at different stages of the model (Álvarez-Fernández, González-Nicieza, Argüelles, & Álvarez-Vigil, 2011). To simulate the effect of the face closed to the analysed section, corresponding to the drive phases with temporary support structures, it is used a value of half of the lithostatic pressure (confinement level of 50%). The real lithostatic pressure value is used for the analysis in those phases that have the final concrete support.

The temporary support is simulated using wooden props with a diameter of 0.25 m on the walls of the successive trenches. The props are spaced at 0.5 m intervals along the walls and are modelled using bar elements. A Young modulus of 10 GPa is used for the timber. The final support structure is simulated using concrete with a thickness that varies along the section and it is modelled as an elastic material.

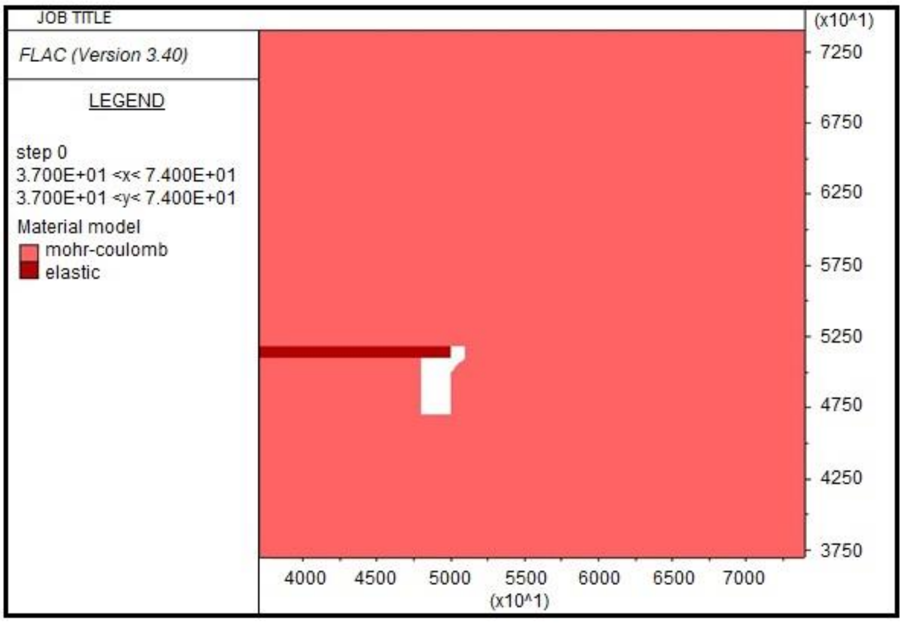
In this type of model, it is extremely important to note the distance between the face and the simulated section, since the stresses that are generated depend on this variable. In turn, the stresses determine how the support responds, especially the provisional supports. When the temporary support is installed, the face is extremely close to the supported zone and the stresses in this zone will be lower than when the face moves further away. In this case, the final concrete support will already have been applied.

Longitudinal 2d modelling

The end of the model is to determine the optimal excavation length. Figure 4 shows a close-up of the geometry of the longitudinal model. A trace of 50 m with a structure gauge of 4.22 m, a depth of 50 m, and a thick concrete support of 1 m. The soil and the filling (Mohr-Coulomb model) are represented in pink and the support (elastic behaviour) is represented in red. It shows the small excavation of the top heading in the crown, leaving a prop for reinforcement. No support work has been carried out in this small drive of the excavation.

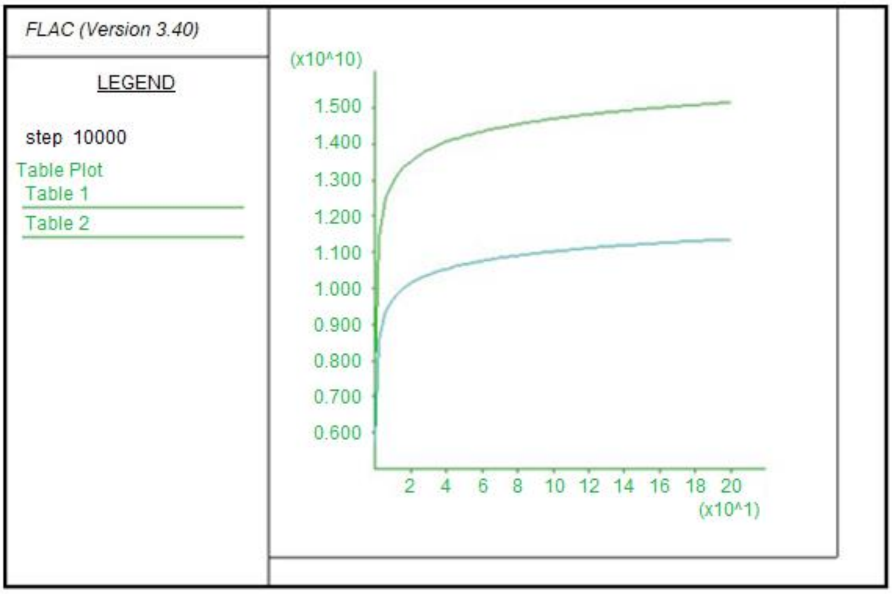
At general, in 2d modelling, the open cavity is assumed to be infinite in the direction perpendicular to the plane of the model. As such, a modification has been made in order to consider the existence of sidewalls. The tunnel has also been modelled using a filling material with properties that make it possible to reproduce the same subsidence recorded in the advance of the cross section model (0.04 m).

Figure 4. 2D Longitudinal model of the tunnel. Detail of the advance. Source: own elaboration, 2018.



To simulate the setting and hardening of the concrete, the elastic modulus of the support has been varied depending on the distance to the face. In figure 5, the green line represents the variation of the volumetric deformation modulus (K) and the blue line represents the change in the shear modulus (G), both of them in Pa. The curves have been obtained based on the recommendations of the Spanish structural concrete standard EHE-08 (Fomento, 2008) for varying the elastic moduli and unconfined compressive strength of quick-setting concrete over time.

Figure 5. Change in the elastic modulus of the concrete due to the face distance. Source: own elaboration, 2018.



The initial stress state was taken as being gravitational. The ratio between horizontal and vertical stresses is 0.5.

Results

The results obtained in the penetration test are described below, as well as the two modelling carried out.

Results from penetration tests

Table 1 shows the strength and deformation properties obtained in the in-situ testing. Each of the values specified in the table corresponds to the average value obtained from four tests carried out at different points located in six zones distributed along the trace of the tunnel. The table also contains the average values of the strength and deformation parameters obtained in the characterisation of the soil based on the calibration of the tests. These values are used to numerically simulate the tunnel construction process.

Table 1. Properties obtained from the in-situ testing. Source: own elaboration, 2018.

		Deformation properties		Strength properties	
Test identifier	Materials	K (Pa)	G (Pa)	c (Pa)	ϕ (º)
Z1E1	Clays	2.9 e8	1.3 e8	1.5 e5	20
Z1E3	Clays	8.7 e8	3.9 e8	1.5 e5	27
Z1E4	Clays	2.9 e8	1.3 e8	1.75 e5	20
Z2E1	Clayey sand	3.8 e8	1.7 e8	1.75 e5	22
Z2E2	Clayey sand	3.8 e8	1.7 e8	1.5 e5	20
Z3E3	Clays	7.5 e7	3.0 e7	3.5 e4	20
Material	Sp. weight (kN/m^3)				
Soil	21	2.9 e8	1.34 e8	1.5 e5	22
Weak stratum	21	7.5 e7	3.0 e7	3.56 e5	20

From the table, it is possible to day that the tunnel passes through one type of material, and there is a weak stratum with a thickness of 5 m immediately above the crown of the tunnel.

Results from the 2d cross section modelling

The most significant results of each of the phases of the 2D model under analysis are described and analysed below.

Phase 1: Advance of central pilot of top heading

This phase simulates excavation and support using wooden props. The analysis of this first phase of excavation and support with temporary elements allows to know if the elements work correctly.

Figure 6 shows the horizontal displacements along the X axis that are generated for the excavation and its surrounding area. They are represented using colours that correspond to the isovalues or zones with the same value (in m) indicated in the legend to the left of the figure. The horizontal displacements are located on the walls and the crown of the excavation and reach a maximum value of 0.0075 m, tending to close the open cavity.

The values of the vertical displacements are shown in Figure 7. In this case, positive values indicate swelling and negative values subsidence. The maximum value of 0.04 m, located on the crown of the excavation is negative. It should be recalled that this zone is located in weaker material.

Figure 8 shows the plastic state of the excavation and its surrounding area. The zones in red have suffered plasticization processes from shear stresses, resulting in permanent deformation. These zones are restricted to the immediate area surrounding the sidewalls and the crown, directly above the area where the temporary wooden props are located. This is due to the fact that the props tend to penetrate slightly.

Regardless, given the displacements that have been described, these are deformations of limited importance, since the model stabilises after this phase. It should be noted that the wooden props of the sidewalls support a load of 154 kN, which is below the critical load for buckling.

Figure 6. Horizontal displacements in phase 1. Source: own elaboration, 2018.

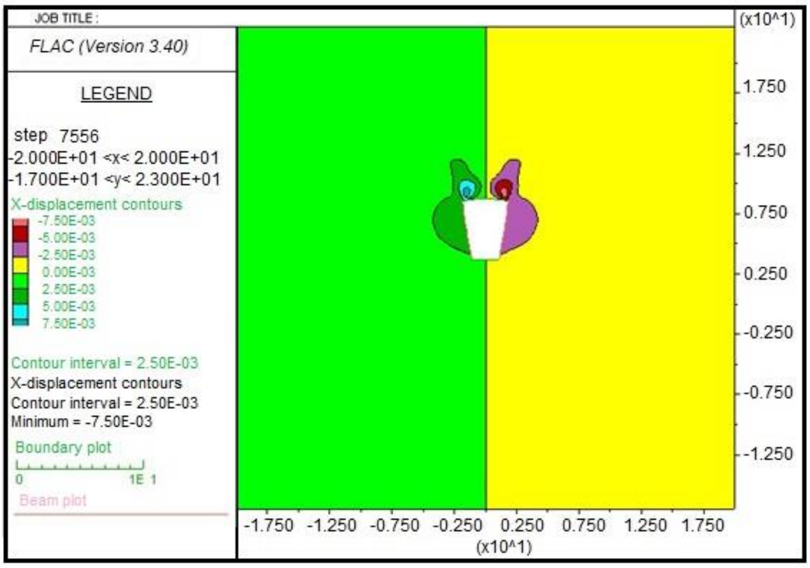


Figure 7. Vertical displacements in phase 1. Source: own elaboration, 2018.

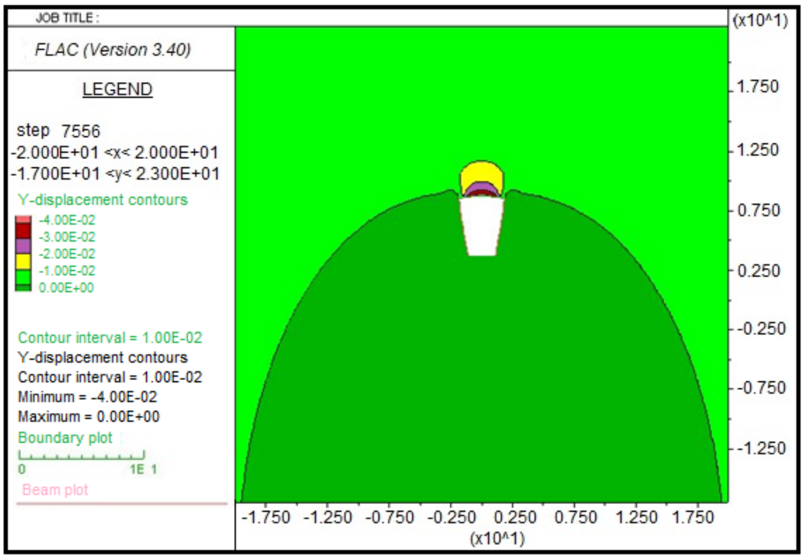
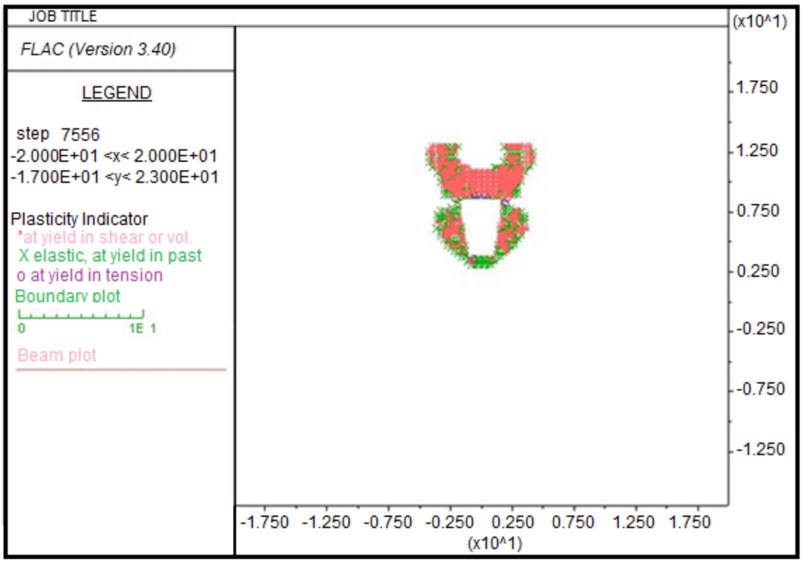


Figure 8. Plastic state for phase 1. Source: own elaboration, 2018.



Phase 2: Excavation and support of the first expansion of the top heading

This phase involves excavation and support works of the first expansion of the top heading. The expansion is executed in two trenches, to the left and to the right of the top of the vault. Wooden props are used to provide temporary support.

The maximum horizontal displacements of 0.015 m after this new phase of excavation are double those in the previous phase and are located on the crown. With respect the vertical displacements, the crown of the excavation suffers negative displacements with a maximum value of 0.05 m at the top of the vault, and 0.02 m for the expanded zones. Swelling of 0.01 m is observed along the full length of the floor of the excavation.

The analysis about the axial force on the temporary supporting elements shows a load of 240 kN in the props installed in the previous phase and a load of 270 kN in the props of the expansion.

Phase 3: Excavation and support of the final expansion of the top heading

In this phase, the excavation of the top heading of the tunnel is completed with new expansions to the left and to the right. The maximum horizontal and opposite displacements are located in the zone between the crown and the sidewalls. They have values of 0.04 m, which tend to close the open cavity. The vertical displacements have increased with respect to the previous phase. In the central part of the crown, the negative displacements reach a maximum of 0.06 m, with values of 0.02 m and 0.04 m to the sides. On the floor, there is swelling of 0.02 m, rising to 0.04 m at specific points in the areas immediately surrounding the support elements.

The plastic state of the model after this phase shows a full zone surrounding the excavation with plasticization from shear stresses, which, at general, are not permanent. At some points, in the zones between the props on the floor of the excavation, there are points that have suffered plasticization from tensile stresses.

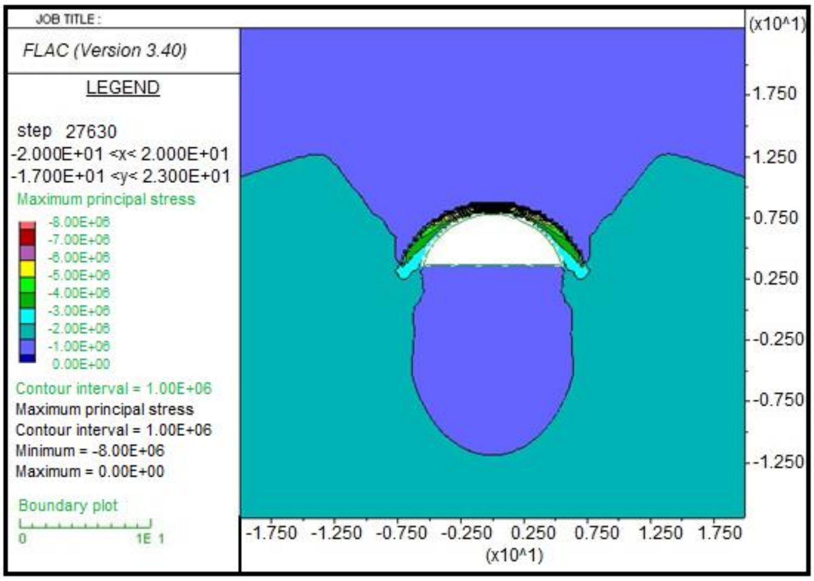
The axial force over the temporary support elements at the top of the vault at the end of this phase is (250 kN), and the props of the first expansion are subject to a smaller load (240 kN) than in the previous phase.

Phase 4: Application of final concrete support structure for the top heading

In this phase, the temporary support from the wooden props is replaced by the final concrete support. The drive face is moving away, meaning it is necessary to reinitialise the stresses in the model. The stresses to which the new support will be subjected are now double those supported by the temporary structure as a result of their proximity to the face.

Figure 9 shows the compressive stresses of the model. The maximum compressive stresses of around 8 MPa are located on the crown, on the face where the concrete is in contact with the soil. The tensile stresses are located at isolated points on the inner face of the concrete of the crown. The values are extremely low, with maximums of 0.07 MPa.

Figure 9. Compressive strength distribution around tunnel in phase 4. Source: own elaboration, 2018.



Once the concrete support is in place and the stresses are reinitialised, for the majority of the concrete, there are maximum horizontal displacements of 0.01 m towards the outside of the excavation. In the lower part of the support, i.e. in the lower zone of the sidewalls and the surrounding soil, the maximum displacements are 0.02 m towards the outside of the excavation. The maximum horizontal displacements for the model of 0.04 m appear in the crown zone.

The vertical displacements throughout the concrete are negative, with a value of 0.10 m. In the soil located above the support, the vertical displacements are also negative, with a maximum value of 0.15 m, as a result of the softer stratum located above the crown. Due to the deformation of the soil, there are also negative displacements along the excavation floor, albeit of a lesser magnitude, with a value of 0.05 m.

Analysing the plastic state of the model after this phase, it can be seen that the plasticization, that has occurred in the areas surrounding the excavation, is not very relevant since the model stabilises without any difficulty at the end of the phase.

Phase 5: Excavation of the central bench

The bench is excavated in three trenches. The first excavation is the central zone, which does not require support. The compressive stresses supported by the concrete as a result of this new phase are similar to those in the previous phase, with a maximum value of 7 MPa. The tensile stresses are restricted to the base of the elephant foot of the concrete and have extremely low values, with a maximum of 0.25 MPa.

The horizontal displacements after this new phase exhibit little variation with respect to the previous phase. However, the excavation of the central bench resulted, in its walls without support, displacements of 0.02 m towards the inside of the cavity. The vertical displacements are also extremely similar to those in phase 4.

Furthermore, the full area surrounding the excavation has suffered from minor plasticization caused by shear stresses. There are isolated zones on the walls of the central bench that have been plasticised as a result of tensile stresses.

Phase 6: Excavation and support of the left bench

In this phase, the left bench is excavated and supported using concrete. The maximum values of the compressive stresses of 7 MPa are still located in the support of the crown zone. In the new zone that is supported, the compressive stresses are between 3 MPa and 4 MPa. The tensile stresses are around 0.25 MPa, although the zone supported in this final phase has two zones with tensile stresses of 2.25 MPa. The horizontal and vertical displacements are similar to those in the previous phases.

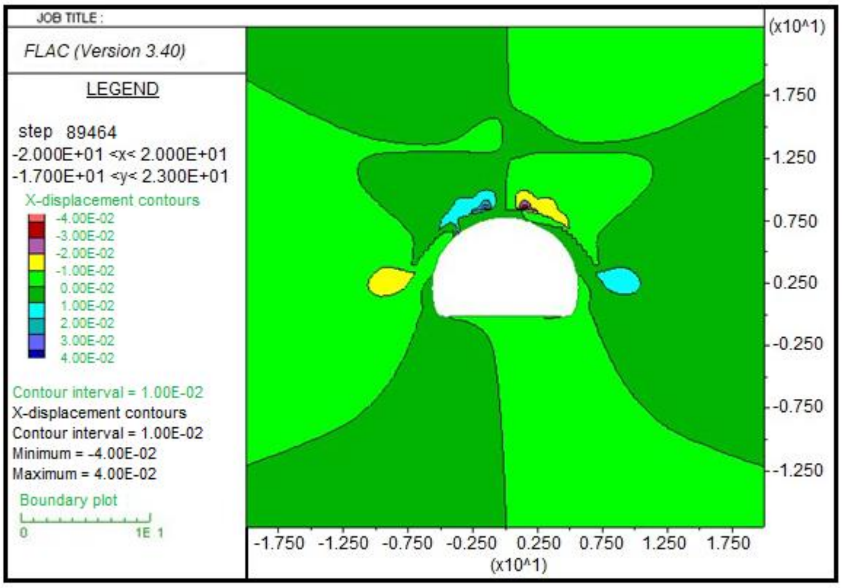
The plastic state of the model is also practically identical to the previous phase. The full area surrounding the excavation has suffered less plasticisation by shear stresses. There are also some zones in the unexcavated bench zone that have been plasticised as a result of tensile stresses. Regardless, the deformations suffered by the soil and the support are not significance, since the model stabilises at the end of this phase.

Phase 7: Excavation and support of the right bench

This phase completes the bench work of the tunnel. The maximum compressive stresses are 7 MPa, located, once again, on the concrete of the crown. Stresses on the sidewalls reach a maximum of 3 MPa. Tensile stresses occur on the concrete of the bench, with an average value of 0.5 MPa, reaching 2.5 MPa on the internal face of the support.

The horizontal displacements in this phase are given in Figure 10. In the top heading, the displacements in the concrete are 0.01 m and diverge, causing the expansion of the cavity. The values of the displacements reach a maximum of 0.02 m at the base of the support for the top heading. In the bench zone, the displacements reach 0.01 m and converge towards the inside of the excavation. The maximum displacements of the model remain in the crown, with values of 0.04 m.

Figure 10. Horizontal displacements in phase 7. Source: own elaboration, 2018.



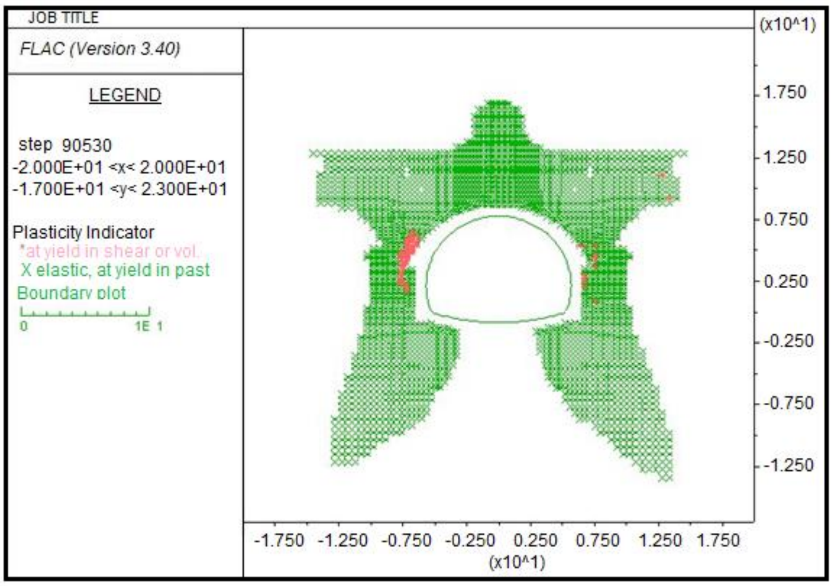
The vertical displacements and the final plastic state of this phase are extremely similar to previous phases. The full area surrounding the excavation is plasticised, although the deformations are unimportant and can be supported by the model.

Phase 8: Excavation and support of the invert

This phase finalises the excavation and support of the tunnel. The distribution and magnitude of the compressive and tensile stresses are identical to the previous phase, however there are some new zones of the support of the invert that are subject to tensile stresses of approximately 0.5 MPa. The horizontal and vertical displacements are similar to those in phase 7.

The plastic state of the model after this final phase is shown in Figure 11. The full area surrounding the tunnel is plasticised, with the exception of the zone below the invert. As is the case in the previous phases, the plasticization does not progress, with the model stabilising at the end of the phase.

Figure 11. Plastic state at the end of phase 8. Source: own elaboration, 2018.



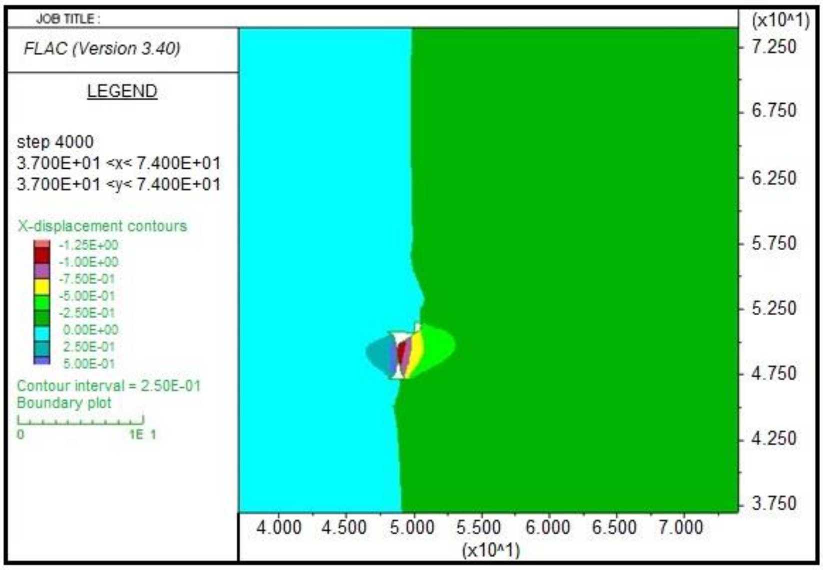
Results from the longitudinal 2d modelling

In the model has been considered two possible excavation lengths in advance:

- Model A with an excavation length of 1 m.
- Model B with an excavation length of 0.5 m.

Figure 12 shows the isovalues of the horizontal displacement obtained for model A. It can be noticed that, after 1 m of advance, the central prop is completely unstable and it suffers more than 1 m of displacement towards the inside of the tunnel. Besides, the displacements tend to increase indefinitely until the cavity is completely closed.

Figure 12. Horizontal displacements for model A (advance of 1 m). Source: own elaboration, 2018.



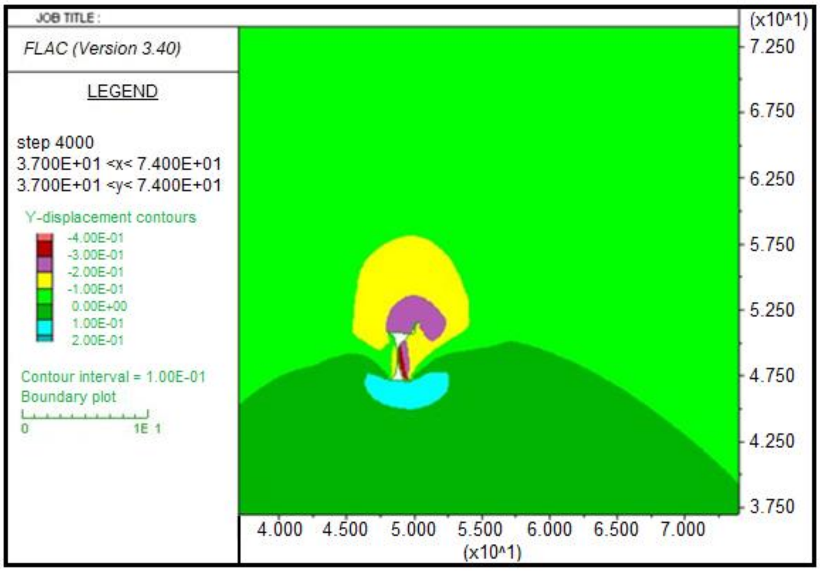
The analysis of the vertical displacements shows that the material above the prop suffers a significant subsidence of the order of 0.30 m, while, the floor of the small cavity that has been opened experiences swelling of 0.20 m. These displacements affect the surface, with subsidence of more than 0.03 m.

Furthermore, the full area surrounding the excavation is completely “ruptured” due to the large-scale deformation caused by the shear and tensile stresses. A small plasticised zone can be observed on the surface, however, this is not significant since there is no continuity with the failure phenomena of the inside.

The horizontal displacements experienced by model B in the zone surrounding the excavation after advancing 0.5 m are smaller than in model A, although the values remain extremely high (1 m towards the inside of the cavity). Furthermore, the movements do not tend to stabilise, but progress indefinitely until the open cavity is closed. This situation makes the level of risk extremely high during the work.

The vertical displacements clearly show that, in spite of reducing the length of the excavation from 1 m to 0.5 m, the scale of the subsidence and swelling is the same: subsidence of 0.30 m in the excavation and swelling of 0.20 m in the floor (Figure 13).

Figure 113. Vertical displacements for model B with one excavation length of 0.5 m. Source: own elaboration, 2018.



Similarly, the effect of the excavation on the surface is equal to model A, since on the surface, in the zone immediately above the excavation, there is a subsidence of 0.032 m.

With respect the plasticization, the model B presents a morphology and a layout similar to model A too. The zone immediately surrounding the excavation ruptures completely as a result of the shear and tensile stresses. Furthermore, the plasticization that occurs on the surface is negligible and lacks continuity with the inside.

Conclusions

- The excavation and support of the tunnel have been modelled using reliable soil properties determined beforehand using in-situ penetration tests. Two types of materials are thus defined: a general soil to which the average properties of the first five tests are assigned and a weak layer of about 5 m of power with the properties obtained from the Z3E3 test.
- All the phases of the cross section modelling have been carried out with a weak stratum above the crown of the excavation. This stratum makes the tunnel weaker and increases the margin of safety.
- The effect of the distance of the face on the cross section model during the first three phases of construction, in which a temporary support is used, has been simulated using a stress relaxation factor.
- Replacing the temporary support with the final concrete eliminates the stress relaxation factor in the cross section model. This change equates to a large distance between the face with respect to the zone subject to the final support.
- The analysis of the initial phases of the excavation and support with temporary elements shows that the model stabilises without significant deformation, meaning that the support of the wooden props works correctly.
- Once the final concrete support has been applied, the maximum compressive stresses (7 MPa) are concentrated in the crown zone.
- The tensile stresses in the concrete are concentrated at the base of the support of the top heading and the sidewalls. Their value is low, oscillating between 0.15 MPa and 0.5 MPa although tensile stresses of 2.5 MPa can occasionally appear in internal zones of the concrete of the sidewalls.
- Even though the excavation lengths in the crown of the top heading are small, the instability of the prop makes work of this sort unsafe for the considered soil.
- The vertical displacement on the surface can affect the existing structures in the zone (e.g. buildings, piping, aquifers).
- The reduction in subsidence is a result of increasing the density of the panels used as the main support, which is infeasible due to the limited bearing capacity of the soil.

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