NUMERICAL SIMULATIONS OF STAGED TUNNEL CONSTRUCTION FOR IDENTIFYING CRITICAL PHASES IN DESIGN

SIMULAÇÕES NUMÉRICAS DA CONSTRUÇÃO FASEADA DE TÚNEIS PARA IDENTIFICAÇÃO DE FASES CRÍTICAS NO PROJETO

Yu, Xianbin, *Associate Professor, KUST, Yunnan, China; Post-doctorate, Centro de Geotecnia, IST, xianbin@ist.utl.pt*
Dinis da Gama, Carlos, *Centro de Geotecnia, IST, Lisboa, Portugal, dgama@ist.utl.pt*

ABSTRACT

The powerful capacity of numerical methods of structural analysis allows the possibility of simulating the different phases of advance in geotechnical works, being able to identify critical situations at the design stage.
This is particularly valid for tunnelling where many ground failures can be theoretically predicted before starting the process of construction, sometimes involving more dangerous conditions than the final situation.
It is suggested, by means of field examples, that tunnel designers should use staged construction modelling by finite element methods, in order to avoid those risks, especially for tunnels opened in soils and weak rocks.

RESUMO

A capacidade poderosa dos métodos numéricos da análise estrutural permite a possibilidade de simulação das diferentes fases das obras geotécnicas, sendo capazes de identificar situações críticas, ainda no âmbito da concepção e do projecto.
Isto é particularmente válido para a execução de túneis, onde muitas instabilidades do terreno podem ser previstas durante o processo da construção, por vezes envolvendo condições mais perigosas do que a situação final.
Sugere-se, com base em exemplos de aplicação, que os projectistas de túneis utilizem a modelação por elementos finitos na simulação das várias fases do projecto, a fim de evitar aqueles riscos, especialmente para os túneis abertos nos solos e em rochas brandas.

1. INTRODUCTION

It is well known that when a tunnel is excavated in weak rocks/soils, the state of plastic and failure may appear in the country rocks/soil. In the plastic state, the relationship between load (stress) and deformation (strain) values is no longer linear, and the intermediate state (loading path) will influence the ultimate state (failure). In most conditions, the excavation of an underground tunnel with large section is composed of several phases. When such a tunnel is excavated in weak rocks/soils, all intermediate excavation phases have to be considered in the stage of design, or else possible failure during intermediate phases would be neglected, causing serious problems during construction.

In this paper, numerical simulation results of a case study on a highway tunnel are presented to show the risk of neglecting the intermediate excavation phases. To illustrate those assumptions it was selected a highway tunnel located near Fundão, Portugal, with the length of 1650 m, 12m wide, and 10m high (before support). Although most parts of the tunnel were excavated in hard rocks, the North portal of that tunnel was opened in soils with extremely low strength, and so a series of difficulties were encountered during and after the process of excavation.

The designed cross section in the initial part of the tunnel is shown in Fig.1. According to the design, jet grouting through a series drilling holes was first performed before the excavation, forming a protection layer of about 50cm thick above the planed tunnel. At lower sides of the tunnel, jet grouting was also needed to reinforce the soil, as shown in Fig.1.

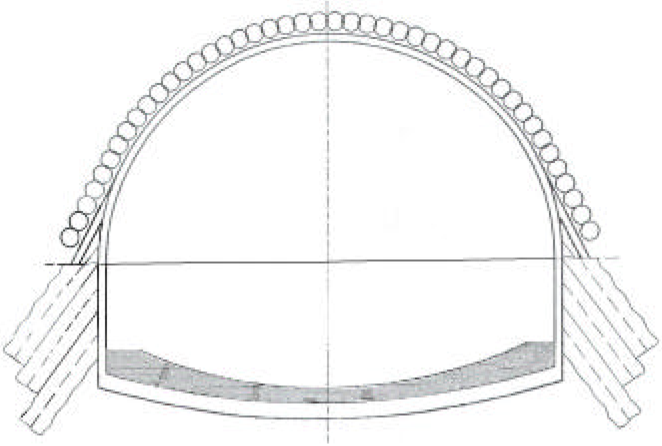
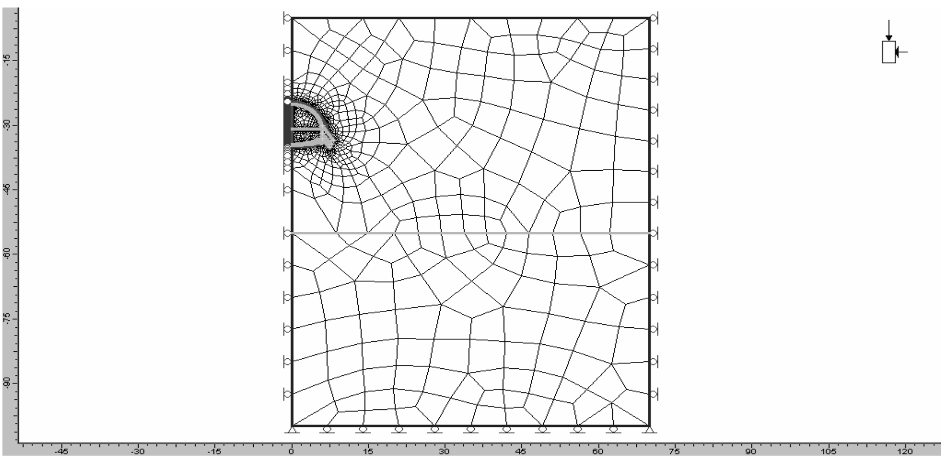


Fig.1 – Designed section of the initial part of the tunnel

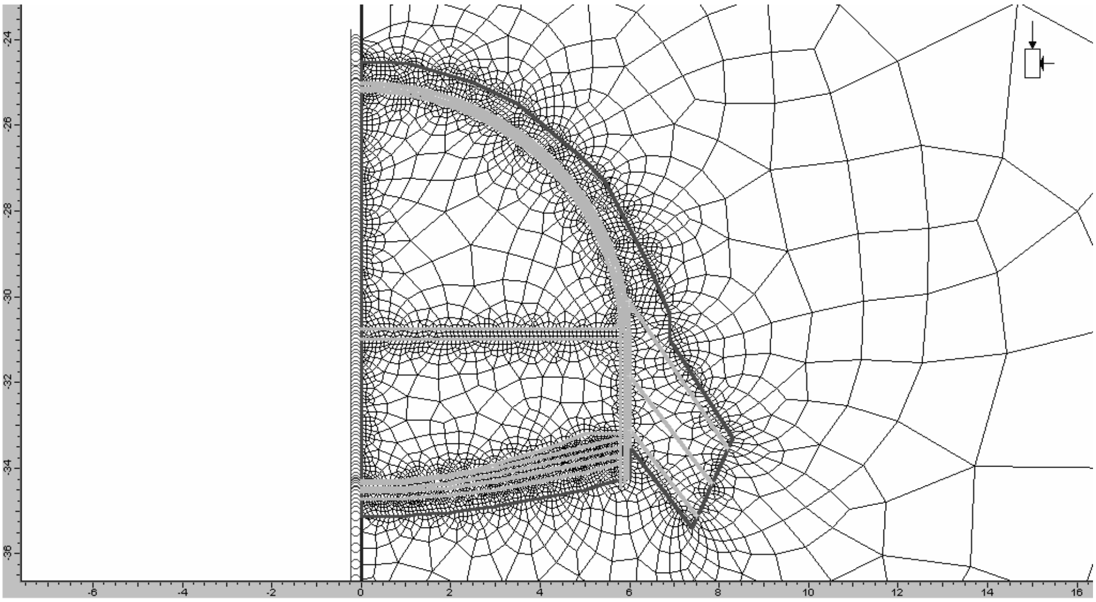
2. SIMULATION MODELS AND EXCAVATION SEQUENCE

One of the simulation models is presented in Fig.2. Because of the symmetry conditions, only half of the tunnel and the soils/country rocks are included in the model, and the size of this model is 70m wide and 100m high. The country rocks around the tunnel are all weathered granite, or granitic soil. Unweathered rocks existed 20m below the tunnel bottom and the vertical distance from the top of the tunnel to surface is also 20m.

Four-node quadrilateral elements were adopted in the FE simulations. In each simulation model, both the total number of elements and the number of the nodes were about 6000. Steel bars installed in the bottom concrete and in the jet grouting were included in the models, as shown in Fig.2 (b).



(a) The simulation model and the mesh



(b) Details of the mesh around the tunnel

Fig.2 – Simulation model and mesh utilized in the analyses

Since steel supports are not continuous at the axial direction of the tunnel, their function cannot be directly simulated with a 2-D finite element program. As the general function of the supports are to resist bending, their ability of sustain bending depends mainly on the moment of inertia. In the simulations involved with this paper, the size of the steel supports was considered equivalent to a steel plate with an constant thickness of 4cm along the axial of the tunnel.

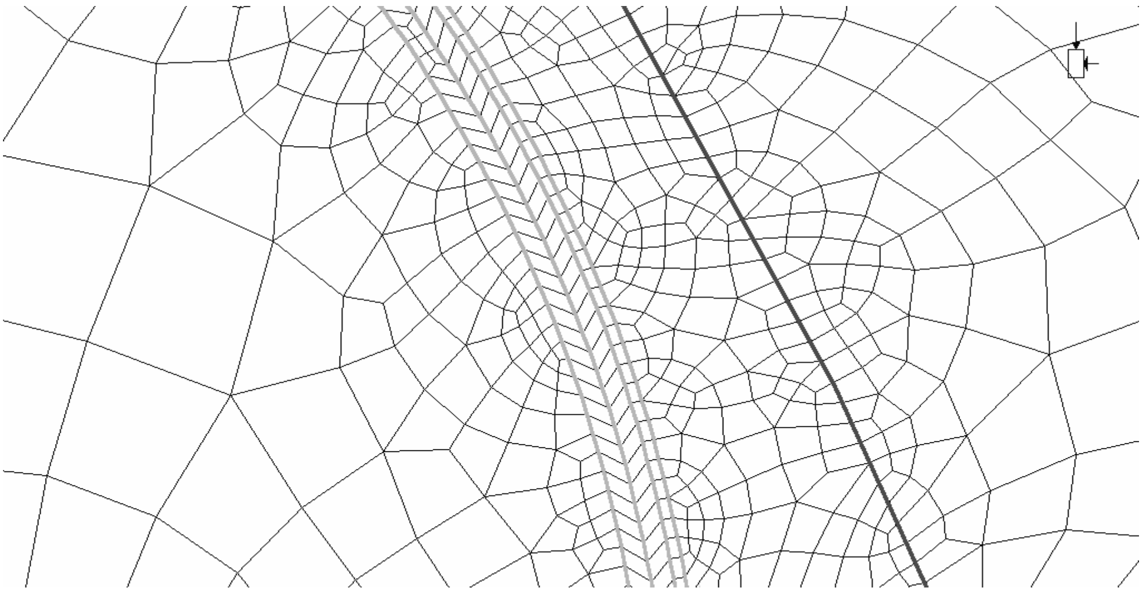


Fig.3 – Details of the mesh (in the arch).The materials included in Fig.3 (from left to right): (1) Soil within the tunnel (to be excavated); (2) shotcrete arch; (3) Steel arch; (4) Jet grouting; (5) Soil outside the tunnel.

Table 1 – Mechanical parameters of the materials used in the FE simulations

		Test results from IST	Data indicated in the design	Adopted in the simulations
Soil	Young's modulus (MPa)	5.8	150	5.8; 8; 40; 150
	Poisson's ratio	0.3	0.3	0.3
	Cohesion (kPa)	18	50	50
	Friction angle (°)	14	7.5	7.5
	Volumetric weight (KN/m^3)	20	20	20
Jet grouting	Young's modulus (MPa)	1000		1000
	Poisson's ratio	0.3		0.3
	Cohesion (kPa)	0.4		0.4
	Friction angle (°)	35		35
	Volumetric weight (kN/m^3)	23		23
Shotcrete	Young's modulus (MPa)	20,000		20,000
	Poisson's ratio	0.25		0.25
	Cohesion (kPa)	2		2
	Friction angle (°)	40		40
	Volumetric weight (kN/m^3)	25		25

One problem in the simulations is the difference between the mechanical parameters from the tests performed by IST and those adopted by the designer. Table 1 shows the two data groups.

The initial stresses that existed in the rocks/soil are important data which influence the simulation results. In the tunnel design documents, measurement data on initial stress values are not available. However, the geologic model assumed that the soil where the tunnel was excavated is formed by a landside, so it is very weak and an initial stress with large values is impossible, so it was assumed in both the design and the numerical simulations that the stress field in the soil is mainly due to gravity, i.e., its vertical component, P, is greater than horizontal one, Q, and the ratio $K_0=Q/P$ was initially 0.5. However, other values of K_0 were also considered in the simulations.

For the sake of reinforcement, steel bars were considered in concrete invert arch at tunnel floor and also at the lateral jet grouting columns. In the simulation model, the former ones are simulated by equivalent reinforced concrete and the later ones are simulated as individual metal bars, as shown in Fig. 3.

In the two groups of data (test results and those adopted by designer) shown in Table 1, the values of Young's modulus are completely different. Simulation results show that this difference has great influence in the stability of the tunnel, as described below.

Practical excavation phasing was composed of the following steps:

(a) Roof jet grouting columns in advance (12m long in the direction of tunnel axis);
(b) Excavating the upper part of the tunnel;
(c) Supporting the upper tunnel section with steel and shotcrete;
(d) Excavating the lower part of tunnel section;
(e) Supporting and finishing the operations.

3. THE INLUENCES OF YOUNG'S MODULUS OF SOIL AND INITIAL STRESS

Upon finite element processing of several tunnel excavation steps, principal stress contours of s_1 and s_3 from simulation results are presented in Figs.4 and 5. From these figures, the following results may be obtained:

- Stress values at points located far from the tunnel are very small.
- Stress values in most part of the jet grouting columns and at the shotcrete layer are also small.
- High stress concentrations appear at the steel arch, shotcrete and bottom concrete arch.

It was found that when K_0=0.5 and E_{soil} (the Young's modulus of soil) =150MPa, the stresses in the steel arch and shotcrete layer are much smaller than their strength values. For example, the largest compressive stress in the shotcrete arch is 10MPa and the tensile stress in the shotcrete arch is always smaller than 0.1MPa, and so no failure would appear there.

For lower values of E_{soil}=8MPa and smaller, however, the stresses in the steel arch and shotcrete layer arch are much greater. For example, the largest compressive stress within the shotcrete is as high as 53MPa and the largest tensile stress is 20.3MPa, both larger than its strength values, meaning that failure is unavoidable.

When E_{soil}=40MPa, the stress values within the shotcrete are between the two conditions described above. For example, the largest compressive stress in the shotcrete layer is 27MPa, which is over its strength value, while the tensile stress is smaller than 0.8MPa.

The above results show that E_{soil} value influences the induced stress very much. In situ excavation works show that failure did appear during and after the excavation, meaning that the E_{soil} value adopted in design is not suitable and the value obtained from tests at IST is adequate. Fig.6 shows the maximum stress values in steel arch, shotcrete layer and bottom concrete arch.

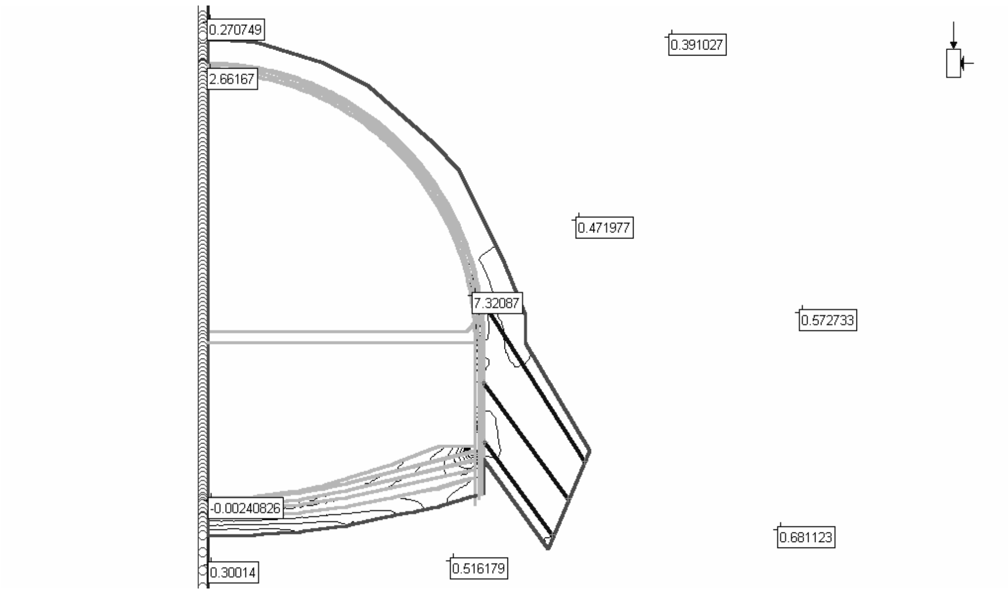


Fig.4 – Contours of s_1 after the excavation (E_{soil}=150MPa)

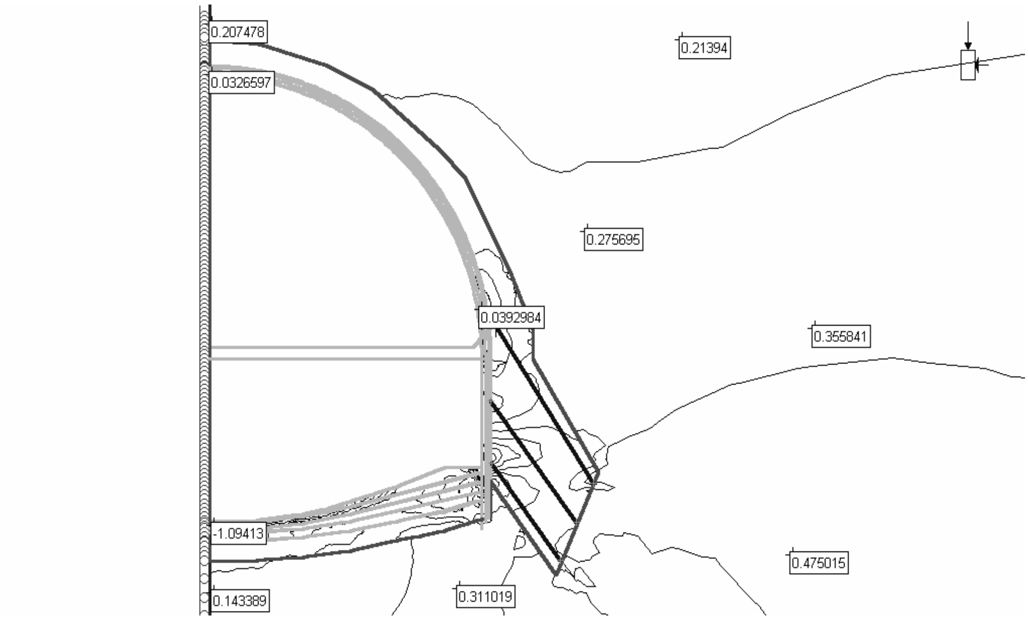


Fig.5 – Contours of s 3 after the excavation (E_{soil}=150MPa)

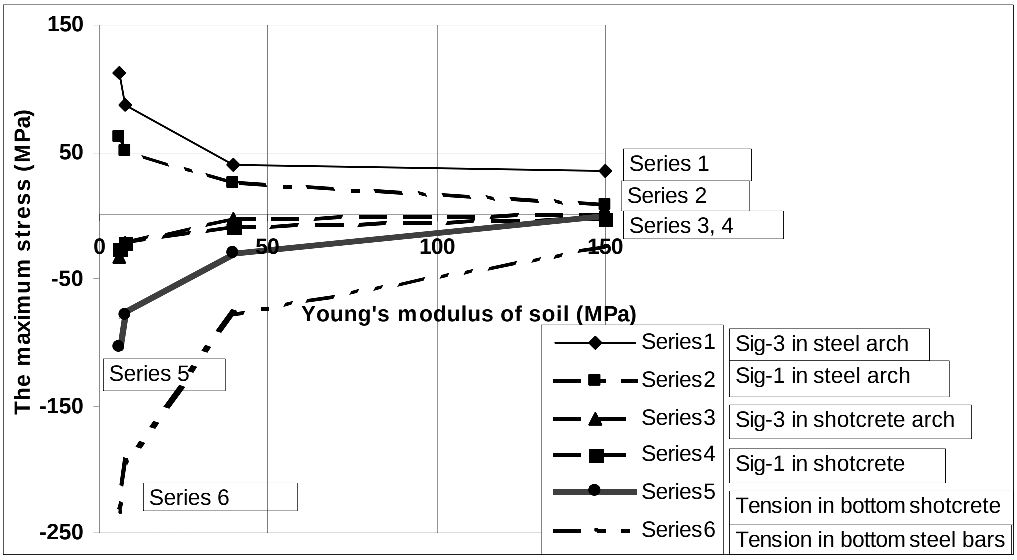


Fig.6 – Maximum principal stress values around the tunnel vs. Young's modulus values of soil

4 THE INFLUENCE OF THE EXCAVATION PHASES

4.1 Support time

Excavation results show that in the North portal of the tunnel, the support with shotcrete was necessary as soon as any part of the face section is excavated, or else collapse occurred in a few hours.

Simulation results show that if the support is not applied within the same phase of excavation, great deformations will appear. For example, when E_{soil} is 150MPa, the vertical deformation at the central roof (downwards) is as high as 22m when the shotcrete is not supplied in a same excavation phase. When E_{soil} is 8MPa or below, the calculation iterations diverge, meaning that collapse occurs in the soil. These results coincide with practical excavation observations.

In the field, deformations appeared within some minutes and even a few hours after the excavation was accomplished, which is much different from the strains generated along time or creep, and can be regarded as elastic and/or plastic deformations. So the shotcrete support supplied within one or two hours after excavation is still an “immediate” one and in this case was essential to excavation short term stability.

It should still be noticed that elastic deformations appear as soon as the tunnel is excavated except in the part near the working face where elastic deformation was partly restrained by the surrounding rock/soil. This means that in practical conditions, shotcrete support, even when quickly applied, cannot completely prevent the elastic deformation to occur.

4.2 Stresses at the top of shotcrete layer

Excavation phases also influence the stress values at the central top of the shotcrete lining. When Young’s modulus of the soil is 8MPa and the whole section of the tunnel is excavated in one phase, the maximum tensile stress that appeared within shotcrete was 17.5MPa. When the tunnel is excavated in two phases (upper and lower part, as shown in Fig.2), the maximum tensile stress in the shotcrete after the first and the second excavation phases are respectively 31.3 and 20.2MPa. The results under different values of E_{soil} are shown in Fig.7.

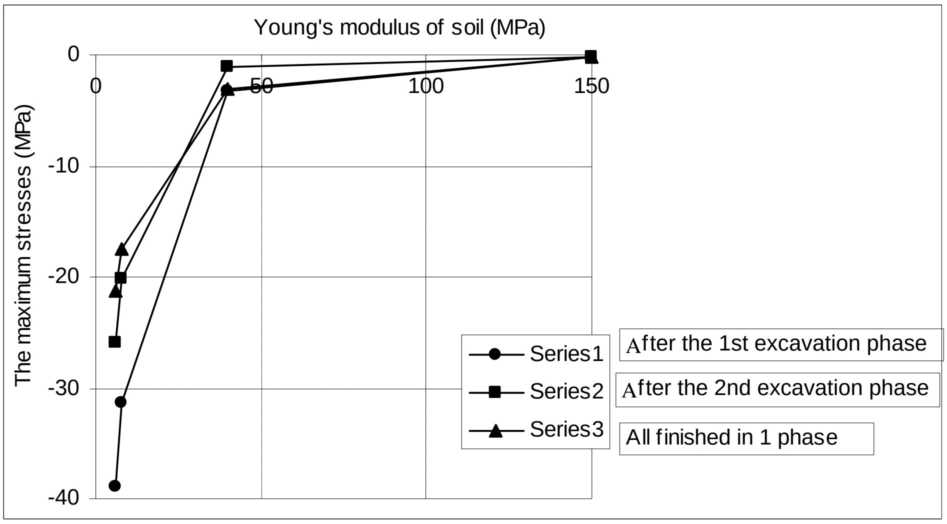


Fig.7 - Maximum tensile stresses at top of shotcrete vs. Young’s modulus of soil.

4.3 Stress concentrations at sharp corners

It is well known that stress concentrations are induced at sharp corners in underground cavities. In many conditions, the stress values are so large that plastic deformation and local failure appear at the locations of such stress concentrations. From the results of numerical simulation, the stress values at sharp corners depend on the initial stress and mechanical properties of rocks/soils, but also varies with mesh size and element numbers, with higher order and small size elements giving higher values to calculated stresses.

When the total section of an underground tunnel is excavated within one phase, the largest values of principal stress s_1 and s_3 at sharp corner of the shotcrete arch are respectively 38 and -3.9MPa, meaning that local failure may appear there.

When the excavation is achieved in phases (upper part and lower part, as shown in Fig.2b), as the real excavation works did, stress concentrations also appear at the sharp corner of the first excavation phase, while such an effect will not appear when the tunnel is excavated in one phase. Furthermore, the largest stress values of s_1 and s_3 at the upper sharp corners are respectively 240MPa and -470MPa, much greater than the values at the sharp corner of the second phase (lower corner).

After the second excavation phase, the stress values at the upper corner decrease to 115MPa and –220MPa (tension) respectively. Such high stress values will make local failure, influencing the stress distribution and deformation of the later phases.

The above results are obtained when E_{soil}=8MPa but the variations of tensile stress at upper and lower sharp corner for different values of E_{soil} are shown in Fig.8 and Fig.9. From Fig.8, it can be observed that wherever the value of E_{soil}, the tensile stresses at the upper sharp corner after the second excavation phase are much smaller than those after the first phase.

Fig.9 shows that when the intermediate excavation phases are neglected, the stress values appearing at the lower sharp corner are even much different from those where the intermediate phases are included. For example, when E_{soil} is 5.8MPa and all excavations are finished in one phase, the maximum tensile stress occurring at the shotcrete of lower sharp corner is –3.2MPa; while when the excavations are conducted in different phases, the maximum stress value there is as high as –53MPa, 16 times greater than the former one. These data demonstrate that if attention is not paid to the results of the intermediate phases, failure and collapse happening during these intermediate phases would be the neglected, which leads to serious risks at the design stage of the tunnel.

4.4 Tensile stresses at tunnel bottom

The reinforcement at the tunnel bottom is another work concerned with the excavation phases. According to the design, an invert arch is placed at tunnel floor as shown in Fig.2, and 40cm thick shotcrete is to be applied as soon as possible. Later on, a reinforced concrete arch with 40 to 60cm thickness is built on top of the shotcrete.

Simulation results also show that the excavation sequence has great influence on the stress values of the floor shotcrete and reinforced concrete. When all excavation operations are finished within the same phase, the maximum value of stresses in the bottom reinforced concrete and the steel bars installed in the concrete are respectively 36 and 326MPa. If the tunnel excavation is achieved in two phases (upper and lower part of the tunnel), and the reinforced concrete arch is constructed in the same phase as the shotcrete, the maximum value of tensile stresses in the reinforced concrete and in the steel bars are respectively 18 and 158MPa, about half of the former data.

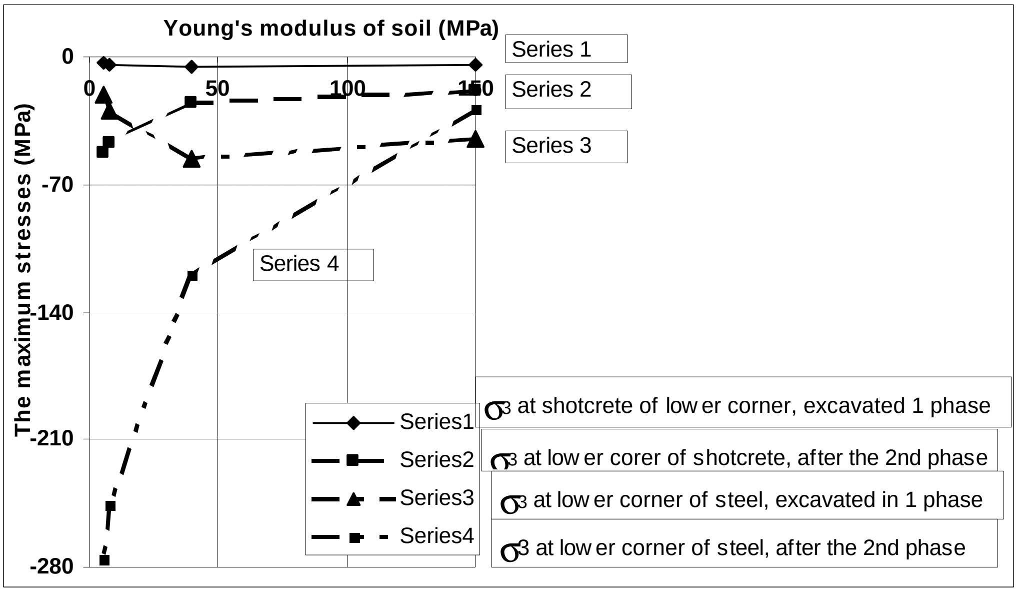


Fig.8 – Maximum tensile stresses at upper sharp corner vs. Young's modulus of soil

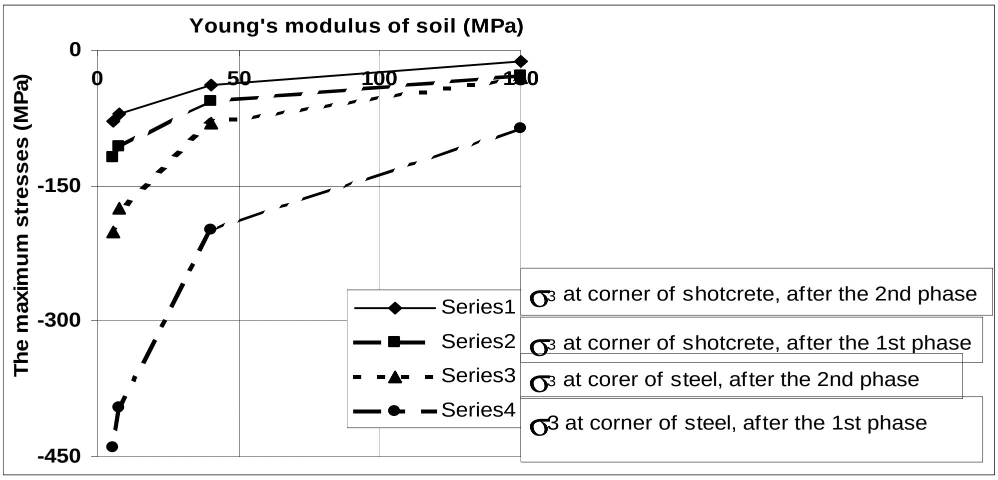


Fig.9 - Maximum tensile stresses at lower sharp corner vs. Young's modulus of soil

However, when the excavations and support are applied in 3 phases (excavations of the upper and lower tunnel, and the concrete reinforcement at the bottom), the maximum tensile stresses appearing in the bottom concrete and the steel bars are respectively 0.2 and 1.1MPa, which are much smaller than their strength values.

These data show that when intermediate excavation phases are neglected in the design while it is in fact practiced in the excavation works, large errors on the stress values appearing in the country rocks/soils are unavoidable, and high stress values and local failures may be neglected, although these failures really exist. As a consequence, while the conclusions from the design stage may indicate a safe situation, many collapses may happen in the actual tunnel, as confirmed in the present case study.

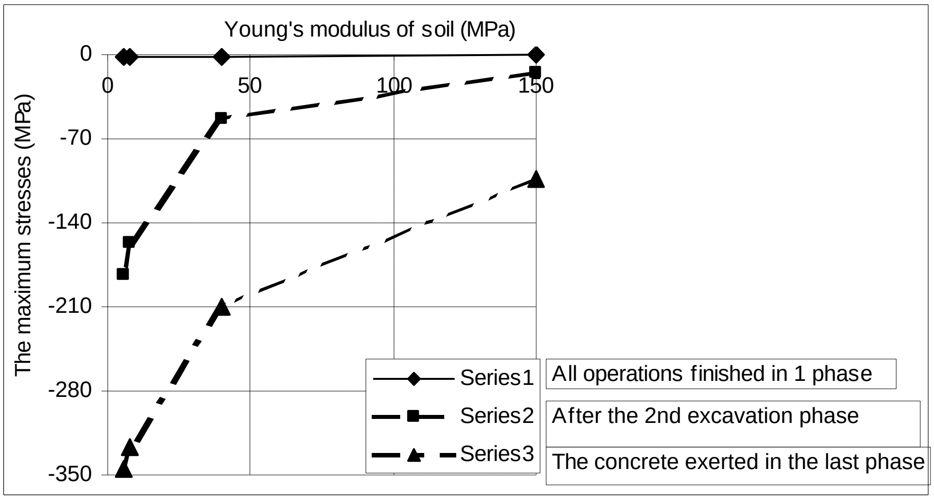


Fig.10 – The maximum tensile stresses in the bottom concrete vs. Young's modulus of soil

5 CONCLUSIONS

Numerical simulations with the finite element method were conducted to analyse a series of collapses in the construction of a highway tunnel in order to evaluate the influence of the excavation phases and the other factors on the process. It was observed from these simulations that intermediate excavation phases may have great influence on the induced stress values, and these cause failure/collapse of the rocks/soils around the tunnel. When total excavation of the tunnel is achieved within one single phase, however, the induced stress values at these critical positions would be much smaller and thus the real failure/collapse are neglected.

Therefore, tunnel designers should use staged construction modelling by accurate numerical simulations, in order to avoid those risks, specially when tunnels are to be opened in soils and weak rocks.

ACKNOWLEDGEMENT

The authors thank EPOS for the permit to use data from their files on the Gardunha tunnel.

REFERENCES

[1] Arakida, K. et al.: The report of tunnel digging by a new type of pre-lining method. *Progress in Tunnelling after 2000*, Proceedings of the AITES-ITA 2001 World tunnel Congress, Teuscher, P. and Colombo, A. (Ed.), Milan-Italy, 10-13th June 2001.

[2] Sano, N. Et al., Approach for design method of the umbrella method, *ISRM 8th International Congress on Rock Mechanics*, pp1367-1370, Proceedings, Volume 3, Fujii T. (Ed.), Tokyo, Japan 1995.