NUMERICAL ANALYSIS OF TWIN TUNNELS INTERACTION

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INTRODUCTION

The construction of twin tunnels has become a common practice mainly due to the needs to improve performance and safety. In the case of road and railway networks the construction of one tunnel branch per flow direction significantly increases the transfer speed and the safety of the commuters and is often compulsory, whereas for hydraulic tunnels one main reason behind twin tunnelling is the increase of discharge without the enlargement of the section area, which could lead to excavation difficulties under unfavourable geotechnical conditions.

In twin tunnels the support of the branch that is first constructed is influenced by the construction of the second one, while the second one is excavated in a distorted stress field. The interaction of twin tunnels has been recorded in many cases, especially in tunnels excavated in unfavourable geotechnical conditions, since the construction of the second branch may lead to the development of additional convergence, loads or even failures of the support shell of the first branch. Yet, many of the methodologies that are used in tunnel design - even the recent ones - have been formulated for single tunnels and do not take into account the interaction between the two branches.

This paper investigates the interaction between twin tunnels using three dimensional numerical analyses with the finite element code ABAQUS. More specifically, the influence of the interaction on the tunnel convergence and support pressure is examined for a wide range of geometrical parameters (pillar width) and geotechnical conditions (rock mass strength and deformability parameters, in-situ stress). This interaction is also illustrated in terms of convergence via construction data of tunnels in Egnatia Highway in northern Greece.

LITERATURE REVIEW

The interaction of twin tunnels has been investigated in literature through numerical and experimental methods, mainly in terms of tunnel convergence and surface settlements. Table 1 summarizes the parameters, the methods and the most significant results and conclusions of research papers on the specific research area.

AN EXAMPLE FROM EGNATIA HIGHWAY

In order to illustrate the interaction of twin tunnels the case study of Driskos tunnel in Egnatia Motorway in northern Greece is presented. All data have been derived from the Tunnel Information and Analysis System (Marinos et al., 2010). Driskos tunnel comprises of two branches with horseshoe section of ~12.5m equivalent diameter, it has a total length of 4600m and the maximum overburden height is ~220m.

Table 1. Literature review on twin tunnels interaction, a selection (W: net distance between the two tunnels-thickness of pillar).

REFERENCES	GEOTECHNICAL CONDITIONS	TUNNEL GEOMETRY	METHOD OF ANALYSIS	RESULTS - REMARKS
Terzaghi (1942) Ward & Thomas (1965)	Chicago clay London clay	W=0.425D, 0.6D	Measurements in tunnels in Chicago and London.	For W = 0.425D, 0.6D significant deformation were developed (0.1% and 0.12% of the radius in Chicago clay and London clay respectively).
Ghaboussi & Ranken (1977)	-	-	2D FE analysis	For W>2D, the displacements of each branch were almost identical to those of the single tunnel.
Adachi et al. (1993)	Sand	-	Laboratory tests	-
Fujita (1985) Fang et al. (1994) [from Ng et al. (2004)]	-	-	Empirical method	For W>1.7D, the interaction between the two branches is considered insignificant.
Addenbrooke & Potts (1996)	London clay	Circular cross-section	2D FE analysis	For W<D, the interaction between the two branches is significant. For W>>D the interaction tends to be insignificant.
Chang et al. (1996)	Sandstone slate	Horseshoe cross section (width 16m, height 11m)	Case history (Taiwan)	For W~2.5D, the tunnel interaction is significant.
Kim et al. (1998)	O.C. clay	-	Laboratory tests	For W>1.5D, the interaction of the two branches is insignificant.
Addenbrooke & Potts (2001)	Thames Gravel (c=0Kpa, φ=35^o), London clay (c=5KPa,φ=25^o)	D=4.146m, H=34m W=8, 12, 16, 32m	2D FE analysis	For W>7D, the interaction between the two branches is insignificant.
Koungelis & Augarde (2004)	Thames gravel (c=0, φ=35^o), London gravel (c=5Kpa, φ=35^o)	D=4.174m, H=14.36m W=0.5D-7D	2D FE analysis	For W>3-4D the interaction between the two branches is insignificant.
Kim S.H. (2004)	Clay (S_u=20KPa)	D=70mm, W=1.4÷2D	2D laboratory tests 2D FE analysis	For W>2D, the interaction between the two branches is insignificant.
Karakus et al. (2007)	Ankara clay (c=10KPa, φ=35^o)	D=7.0m, W=15.0m	Case history (Turkey)	For D=7m and W=15m~2D, deformation problems may develop.
Chen et al. (2008)	Sandstone, slate (c=340KPa, φ=24^o)	Circular cross-sections (D_1=12m, D_2=5m) Horseshoe cross-sections (A_1=110m², A_2=18m²) H=300m, W=12÷85m	2D FE analysis	For circular sections and W>4B* the interaction between the two branches is insignificant. For horseshoe sections and W>30m (W>2B*) the interaction between the two branches is insignificant. *B=the sum of the diameters of the two tunnels

The tunnel was driven through the Ionian flysch formation, which lithologically comprises alternations of sandstones and siltstones. Due to a major thrust in the tunnel area all the geological formations are disturbed or folded with a respective decrease of their properties. Moreover shear zones, frequently oriented parallel to the bedding planes, were crossed. In these zones the rock mass has chaotic structure of isolated lensed blocks of hard rock “floating” within a soft clayey-silty matrix resulting to very low strength and deformability parameters. A more detailed description of the geological and geotechnical conditions can be found in Marinos et al. (2006). As a result of the poor rock mass quality and the high overburden, the tunnel experienced in specific section severe failures with maximum convergence more than 300mm. Figure 1 shows the development of the convergence measurements from a Monitoring Station (MS) in the northern branch which was first excavated. After the excavation of the Top Heading of the first branch the convergence tends to a maximum value of 50-60mm. Yet, the approach of the second branch at the area of the Monitoring Station leads to an increase of the convergence, which has a rapid component of ~40mm (~25days and ~50m=5D advance of the second branch) and a smoother one of 30mm (~110days, the MS is not influenced by the advance of the second branch).

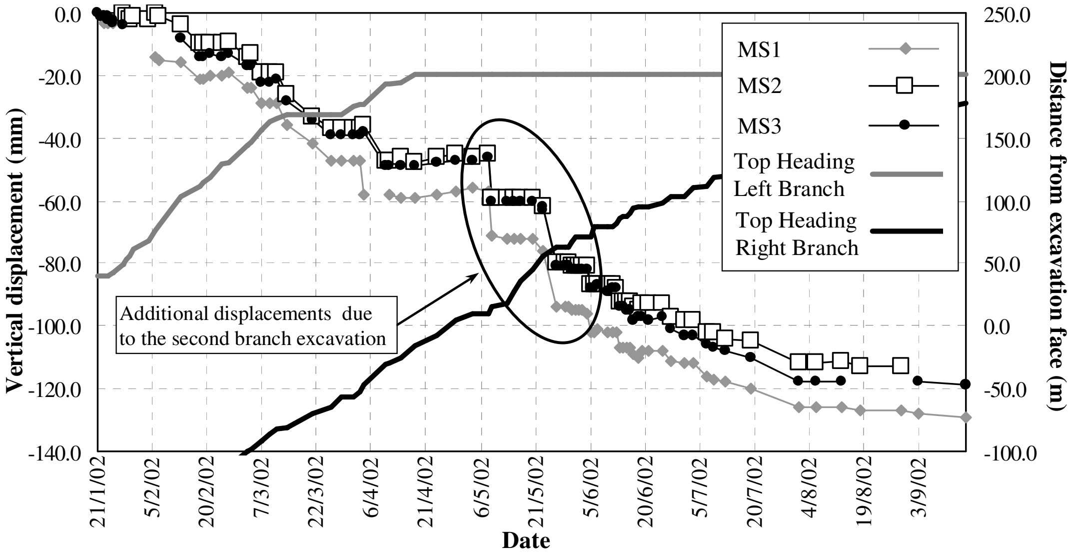

Figure 1. Construction measurements of the vertical displacements in the northern branch of Driskos Tunnel in Egnatia Higway related with face advance (Data from TIAS Database, Marinos et al., 2010).

Thus, it becomes evident that there is a significant interaction between the two branches, since the excavation of the second leads to significant additional convergence of the first one. It is noted that if the section shape had lower curvature or the support shell was more rigid, then the additional convergence would be decreased and a significant component of the interaction would be expressed through the increase of the support loads.

DESCRIPTION OF THE NUMERICAL ANALYSES

The problem was investigated via 3D numerical analyses using finite element code ABAQUS. Three different models were constructed in order to simulate three values of pillar width W, which is the net distance between the two tunnels (W=0.5D, 1.0D, 2.0D). The section of the tunnels was assumed to be circular with diameter D=10m and the overburden height equal to 4D=40m. In all cases the tunnels were excavated in one phase with excavation step equal to 1.0m (the total excavation length was 50m) and they were supported with 20cm of shotcrete. The left branch was excavated first and then followed the excavation of the second one. Therefore the results of the left branch before the excavation of the right one are representative of a single tunnel. Hence, in the paper the left branch before the excavation of the right is referred as Singe Tunnel (SIT), after the excavation of the second one as First Tunnel (FIT) and the right one as Second Tunnel (SET). Figure 2 shows a section and a perspective view of one of the models (W=D).

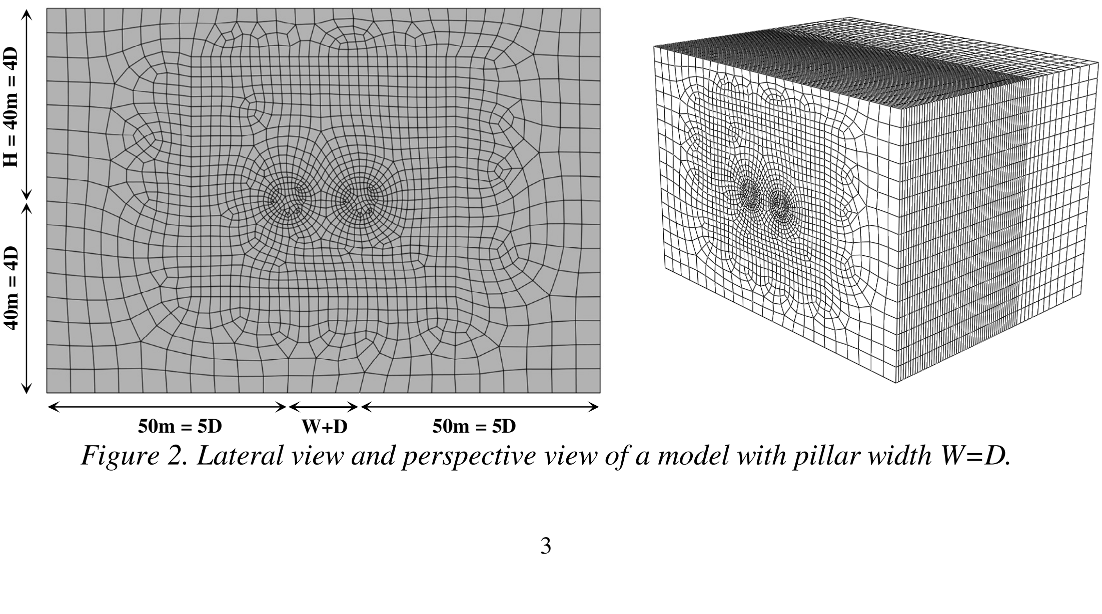

Figure 2. Lateral view and perspective view of a model with pillar width W=D.

The soil was simulated with hexahedral, eight-noded, solid elements and the support with quadrilateral, four-noded shell elements. The total number of elements was about 110.000, for each model. The ground was modeled as isotropic linearly elastic - perfectly plastic material following the Mohr-Coulomb failure criterion. The initial selection of the parameters was made in terms of the Hoek-Brown, which better describes rock materials. The equivalent Mohr-Coulomb parameters were determined using the methodology proposed by Hoek et al. (2002) and the rock mass modulus was calculated according to Hoek et al. (2002) and Hoek & Diederichs (2006). In all cases the geostatic stress ratio was taken K=0.70, the rock mass unit weight γ=0.025kN/m^3, the disturbance factor D=0, the Modulus Ratio MR=350 and the dilatancy angle $\delta=\varphi/6$. A summary of the model parameters used in the analysis is given in the Table 2. The shotcrete shell was modeled as an isotropic linear elastic material with deformation modulus E_c=20GPa.

Table 2. Analyses parameters

Parameters	Symbols	Values	Units
Geological Strength Index (Marinos & Hoek, 2000; Marinos et al., 2005)	GSI	10 - 40	-
Uniaxial compressive strength of intact rock	σ_{ci}	8 - 20	MPa
Geomaterial parameter m_i	m_i	6	-
Rock mass strength (Hoek et al., 2002)	σ_{cm}	0.31 - 2.17	MPa
Rock mass equivalent cohesion	c	0.039 - 0.170	MPa
Rock mass equivalent friction angle	φ	23 - 42	(^{o})
Rock mass equivalent uniaxial compressive strength, $\sigma_c=2\cdot c\cdot \tan(45+\varphi/2)$	σ_c	0.12 - 0.80	MPa
Rock mass deformation modulus	E_m	85 - 2515	MPa
Poisson's ratio	ν	0.30	-

NUMERICAL ANALYSES RESULTS

The main objective of the specific analyses was to investigate the differentiation of the support pressure and the displacements due to the interaction of the twin tunnels. All the exports and the results that are presented correspond to the "characteristic section" of the tunnel, which is the section that the displacements and the pressure in the longitudinal direction have practically converged to the final values.

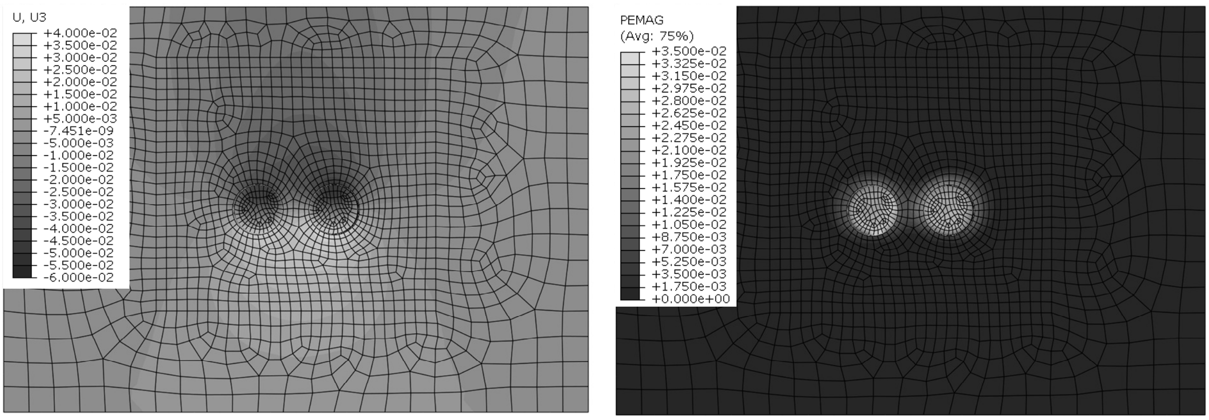

Figure 3. Distribution of vertical displacements and plastic deformaion in the case of twin tunnels excavation (GSI=10, σ_{ci}=8MPa, m_i=6, W=0.5D, E_m estimation method: Hoek & Diederichs, 2006).

The contour lines are not symmetrically developed over each tunnel, since the maximum values are observed in the side of the pillar. Due to the small distance between the two tunnels in the case presented the whole pillar is plasticized. However, the rigid support of the tunnels does not allow the development of large additional convergence.

This asymmetry is also expressed in terms of the pressure around the section of the tunnels. Figure 4 illustrates the distribution of the pressure around the First and the Second Tunnel. In both branches the maximum values are observed in the inner side (First Tunnel $\theta=90^o$ & Second Tunnel $\theta=270^o$). It is obvious that the increase of the pressure is very large for the First Tunnel, whereas the difference between the Single and the Second Tunnel is relatively small. The reason is that the First Tunnel is already excavated and supported when the second one approaches. Therefore the potential displacements induced by the excavation of the Second Tunnel cannot be developed without restraint due to the rigid support of the First Tunnel, leading to a significant increase of the imposed pressure. On the other hand the second tunnel is excavated in a distorted stress field with a pre-existing plastic zone in the case of poor geotechnical conditions, which leads to a small increase of the pressure. It should be noted that if the rigidity of the support shell is reduced (horseshoe section, elastoplastic behaviour of shotcrete) then the additional load would be decreased with a corresponding increase of the convergence.

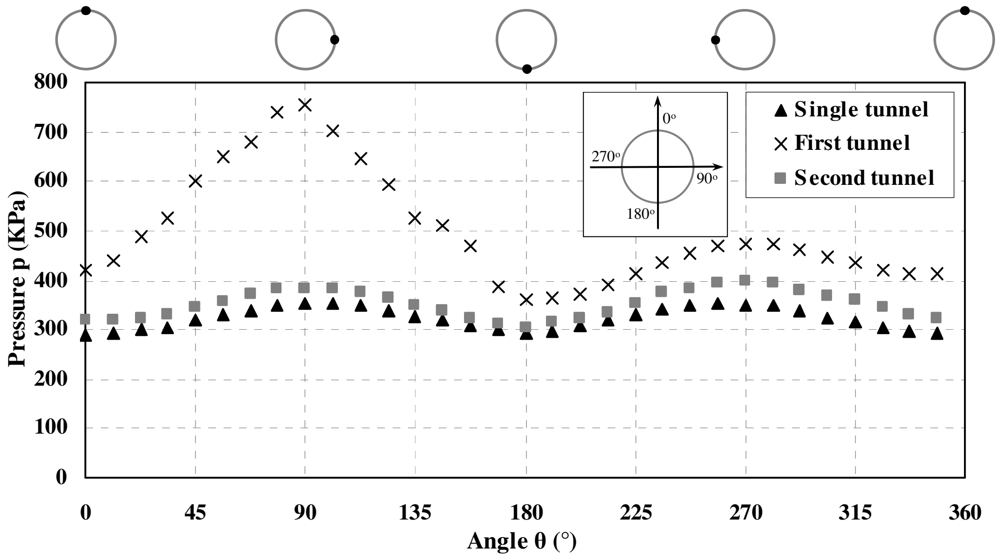

Figure 4. Distribution of support pressure around the tunnel section for the cases of Single, First and Second Tunnel (GSI=10, σ_{ci}=8MPa, m_i=6, W=0.5D, E_m estimation method: Hoek & Diederichs, 2006).

In order to investigate more thoroughly the evolution of this Figure 5 presents the development of the average pressure for a specific section of the First Tunnel, which is located 1.0D from the end of the excavation and 4.0D from the edge of the model during the analysis. In the 40th step the specific section is excavated and supported and the imposed pressure from the surrounding rock mass starts to increase as the excavation face advances. At the end of the 50th step the pressure has practically converged to a value of 320kPa, for all the cases of pillar thickness, W. Thence the excvation of the Second Tunnel begins but the pressure of the specific section remains constant since the distance of the specific section from the face of the Second Tunnel is very large. The distance that the Second Tunnel begins to affect the considered varies from 1.5D to 2.5D according to the value of W. Yet, in all cases it is obvious that when the distance is 1.0D (step 80) the slope of the curves increases rapidly until the point where the face of the Second Tunnel passes next to the under study section (step 90). This section of the curves (step 80 - step 90) which is between the two inflection points shows that the most significant effect on the first tunnel is due to the pre-convergence of the second one. After the Second Tunnel is constructed, the rigid shell, does not allow the development of significant convergce and consequently the additional load on the First Tunnel is small.

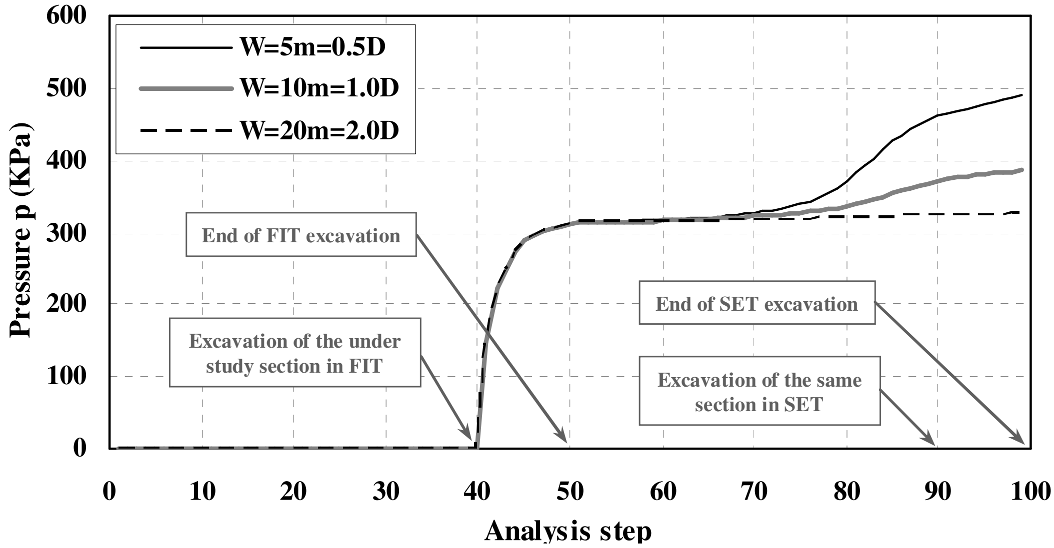

Figure 5. Development of the average support pressure in three analyses with geotechnical parameters GSI=10, σ_{ci}=8MPa, m_i=6 (E_m estimation method: Hoek & Diederichs, 2006) and W=0.5D, 1.0D, 2.0D. The results correspond to the section which is 4.0D from the model edge and 1.0D from the excavation face.

The results for all the analyses are presented in the following figure. Increase of the pillar width leads to a significant decrease of the interaction and the value of the ratios $p_{m,FIT}/p_{m,SIT}$ and $p_{m,SET}/p_{m,SIT}$. The increase of the support for the First Tunnel has a significant differentiation as the geotechnical conditions change, with a maximum relative increase of 60% for the average pressure in the case of W=0.5D. On the contrary the sensitivity of the ratio $p_{m,SET}/p_{m,SIT}$ to the geotechnical conditions is relatively small since the values vary from 1.0 to 1.1. Taking into account the remarks derived from Figure 4 it is evident that for the First Tunnel this increase is higher (up to 120%) for the section in the pillar side and lower (up to 35%) in the outer side of the section.

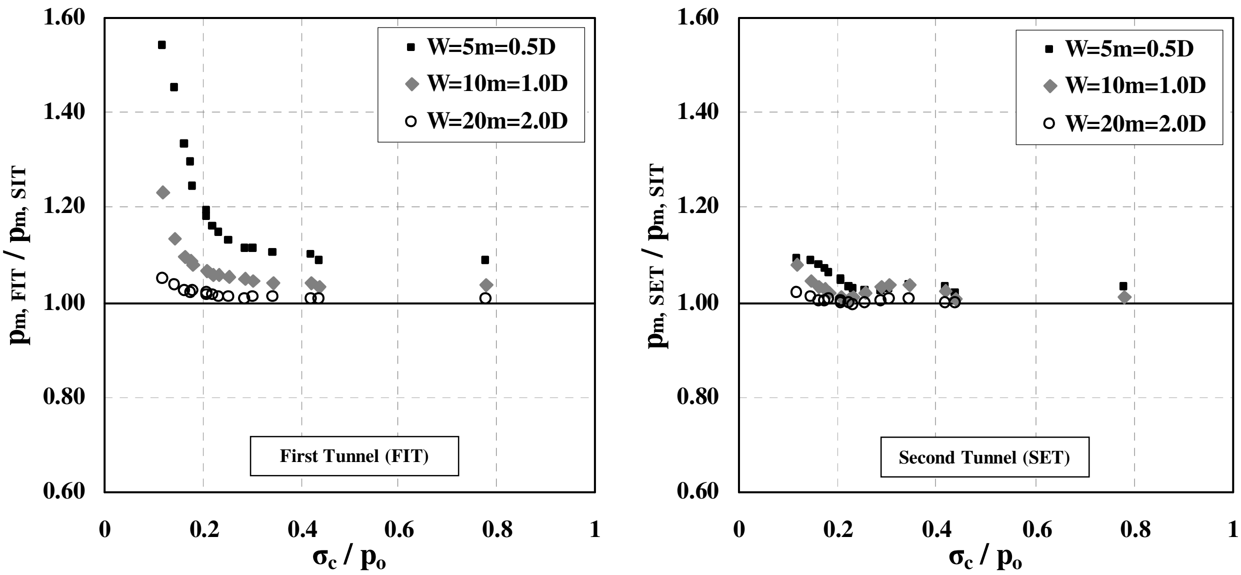

Figure 6. Distribution of the average pressure on the support shell of the First and Second Tunnel vs the geotechnical conditions ration σ_c/p_o. The results are normalized in respect with the corresponding pressure values of the Single Tunnel (E_m estimation method: Hoek & Diederichs, 2006).

From the curves in Figure 6 the combinations of geotechnical conditions and pillar width where the interaction between the two branches becomes negligible can be estimated. The marginal value for this, was assumed to be the relative differentiation of 5% for each of the studied parameters seperately. These combinations are plotted in Figure 7. The fitted curve can be used for the estimation of the minimum pillar width that leads to negligible interaction according to the geotechnical condition at the level of the tunnel.

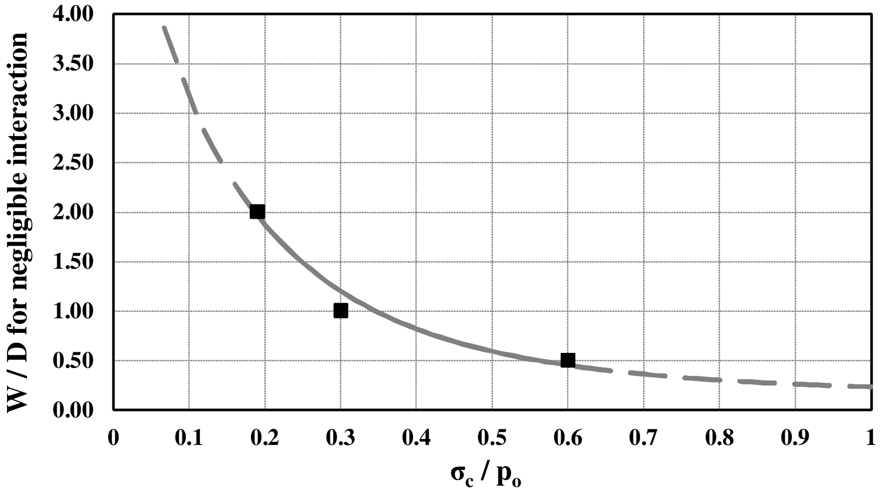

Figure 7. Estimation of the pillar width that leads to neglible interaction between twin tunnels in terms of the support pressure of the First Tunnel vs the geotecnical conditions.

CONCLUSIONS

The interaction between two tunnels had proved to be a significant issue in tunnel engineering since in few cases single tunnels are constructed and at the same time many of the available design methodologies do not take into account this interaction. The investigation of this issue in the present paper was based on the 3D numerical analyses with finite element code ABAQUS. Therefore the results are not subjected to the admissions of a specific method for the estimation of the pre-convergence.

The results of the numerical analyses have shown that the interaction of the two branches differentiates the shape and the value of the displacements and support loads. This differentiation was more significant in terms of the support pressure since the rigid shell of the tunnels did not allow the development of large additional displacements. The increase of the average pressure varies from 10% to 60% for the First Tunnel and up to 10% for the Second Tunnel, the sections more stressed being the inner ones in the pillar side. This increase of the pressure can be estimated via the proposed normalized diagrams as a function of the geotechnical conditions and the pillar width.

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