Expanding Underground. Knowledge and Passion to Make a Positive Impact on the World – Anagnostou, Benardos & Marinos (Eds)

Tunnel design guideline in cases of tunnel face pre-support by fiberglass nails and/or forepoling umbrella

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ABSTRACT: The present research aims to investigate the optimal design parameters of the tunnel face pre – support, where the conventional tunnel excavation method is implemented, by using forepoling umbrella and/or fiberglass nails. The research is based on numerical investigation, in both cases of shallow and deep tunnels, by usage of the Simulia ABAQUS FEM. During the numerical analysis, the tunnel face deformation (extrusion) and the tunnel face pre – support measures response were recorded, in occasions of different rockmasses and various tunnel overburden heights. From the results of the several design parameter, was specified the optimal length of the support measures in order to achieve the maximum effectiveness. For both tunnel face reinforcement with fiberglass nails and/or tunnel face pre – support by forepoling umbrella, were exported non – dimensional nomographs and expressions, in order to be useful for the tunnel designer.

1 INTRODUCTION

The stability of the tunnel face is one of the most critical factors for the safety of the staff working underground, the cost of the project but also the safety on the surface buildings when referring to swallow tunnels. For that reason, in weak grounds, it is essential to apply pressure on tunnel face constantly, either by the use of EPB/Slurry TBMs in mechanized tunneling or by various face support techniques (fiberglass nails, forepolling, ground treatment etc) in conventional excavations.

Whenever is required to pre-support the tunnel face in conventional excavations, with forepolling umbrella or fiberglass nails, shall ensure that the support measures are being installed in sound ground. The deformation of the surrounding ground of a tunnel always starts behind the tunnel face and it is manifested through the face extrusion, i.e. the axial face movement. The extension of the deformed/plasticized ground (plastic zone) is correlated to the ground mechanical characteristics and the assessment of the length of the plastic zone will determine the overlap length of the forepoles/fiberglass nails. A good estimation of the support measures length will ensure their proper functionality, transferring the loads to the healthy ground and minimizing the risk of failure.

Nowadays it is very common in the tunneling projects to estimate the length of the support measures by using the silo approach with the method of limit equilibrium (*Anagnostou and*

DOI: 10.1201/9781003348030-70

Kovari, 1994). In complicated ground conditions the 3D FEM analysis is offering more detailed information of the axial face movement (extrusion) which lead in more concrete solutions that will tackle unfavorable face failures.

2 NUMERICAL INVESTIGATION

In order to investigate the tunnel face response, in cases of tunnel face reinforcement by fiberglass nails and/or forepoling umbrella protection, three dimensional (3D) numerical analysis designed in the FEM program Simulia ABAQUS. The simulated tunnel had a non – circular shape with equivalent tunnel diameter D= 10m and the tunnel excavated in two phases; a) top heading with temporary invert and b) bench with final invert. The tunnel overburden height (H) set to 20 - 40m for cases of shallow tunnels and 100 – 200m for cases of deep tunnels, measured from the tunnel axis.

In the numerical models only the tunnel top heading excavated in order to record the amount of the tunnel face extrusion, as the length of tunnel face pre-support measure (fiberglass nails or forepoling umbrella) reduces.

In order to determine for the optimal length or the equivalent overlap length of the tunnel face pre – support measures, the tunnel pre–support measures installed from the early stages of the numerical analysis, with total length of 40m. The first 20m of the tunnel top heading excavated in one stage and then, the tunnel excavation advance was 1m. As the tunnel face pre–support length reduced, according to the tunnel face advance, the tunnel face average horizontal displacement (U_h) recorded. Moreover, in every excavation step as the tunnel face pre – support measure length reduced, the axial force (P_{axial}) in the fiberglass nails and the bending moment (M) in the forepoling umbrella, recorded according to the case of tunnel face pre – support by fiberglass nails and/ or forepoling umbrella. From the numerical investigation records, the main criterion for choose the optimal length of the tunnel face pre – support, is the tunnel face extrusion – displacement (U_h), when this value increases rapidly. Moreover, the previous assumption in cases of poor ground conditions, was verified too, when the axial force (P_{axial}) in the fiberglass nails or the bending moment (M) in the forepoling umbrella increases too.

On Figure 1, is presented a characteristic example of shallow tunnel which excavated in a poor soil and the tunnel face is pre – supported by fiberglass nails. For that case the optimal length of the fiberglass nails (L_{FG}) is 6m, as the tunnel face defamation (U_h) and fiberglass nails axial force (P_{axial}) increases rapidly, when the fiberglass nails length is lower than 6m.

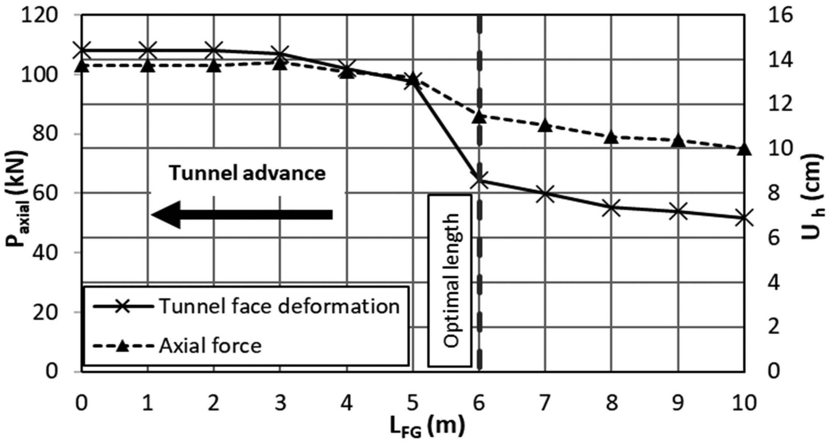

Figure 1. Chrematistic example, of estimation of the optimal fiberglass length (L_{FG}) in 6m, as the tunnel face deformation (Uh) and fiberglass nails axial force (P_{axial}) increases rapidly, for lower fiberglass length of 6m.

In the numerical models, the tunnel lining consist of 30cm thick shotcrete, placed 1m behind the excavation face.The shotcrete liner is modelled with linearly elastic 4-noded shell elements, with a relatively low initial concrete modulus (E = 15 GPa) to account for gradual concrete setting time during tunnel advance.

For shallow tunnels, the ground is modelled as a relatively stiff elastic - perfectly plastic material, with elastic modulus (E), yielding according to the Mohr-Coulomb criterion. The ground unit weight is 20 kN/m^3, soil cohesion (c') ranges between 20 and 50 kPa, soil friction angle (φ') ranges between 22,5° and 30° and the elastic modulus (E) ranges between 80 and 200 MPa. For deep tunnels, the rockmass is modelled as a linearly elastic - perfectly plastic material, yielding according to the Generalised Hoek-Brown failure criterion (Hoek et al, 2002). The examined cases for deep tunnels are weak or heavily fractured rockmass with unit weight 25 kN/m^3, intact rock properties σ_{ci} = 10 MPa and E_i = 2 GPa, Poisson ratio v = 0.33 and Geological Strength Index in the range GSI = 25 to 45 (three cases). In all cases, the horizontal stress coefficient (K_o) set to 0,5 and 1. Moreover, all examined cases are based on dry ground conditions, due to the fact thath drainage holes are used in tunnel excavation projects. Thus, the examined failure mechanism of the tunnel face, is based on dry ground conditions.

In cases of tunnel face reinforcement by fiberglass nails, two tunnel face reinforcement densities (p) examined in cases of shallow tunnels: p= 1 nail/1m^2, 1 nail/2m^2 and in cases of deep tunnels, three tunnel face reinforcement densities (p) examined: p= 1 nail/0,75m^2, p= 1 nail/1m^2 and 1 nail/2m^2. The yield load (F_y) of each fiberglass nail is 200 kN and the elastic modulus E= 40 GPa. The fiberglass nails in the numerical investigation, simulated as truss elements with elastic – perfectly plastic behavior.

In cases of tunnel face protection by forepoling umbrella, forepoling tubes are modelled with horizontal beams spanning the tunnel crown at an angle 120 degrees (60 degrees at each side of the crown) with spacing S = 0.50m. The beams are modelled as elastic – perfectly plastic with elastic modulus E = 200 GPa, Poisson ratio ν = 0.25 and yield stress 235 MPa. The following two types of forepoling beams are used in the analysis: Ø114.3/100.3 and Ø168.3/154.

The forepoling tubes and the fiberglass nails are added in the FEM model as beams and truss elements respectively. Due to the complexity of the 3D FEM model, the accuracy of the interface strength between these structural elements and the surrounding ground may be limited.

In order to corelate the conclusions of the present investigation for the optimal tunnel face pre – support length with the ground conditions, the tunnel face stability parameter (Λ_f) used from *Georgiou et al. 2022*, based on the following formula:

$$\Lambda_f = 3.8\left(\frac{\sigma_{cm}}{\gamma H\sqrt{1+(2/3)K_o}}\right)\left(\frac{H}{D}\right)^{0.35} \tag{1}$$

where (σ_{cm}) is the soil/rockmass strength; γ is the ground unit weight; Ko is the horizontal geostatic stress coefficient; H is the tunnel overburden height from the tunnel axis and D is the tunnel face equivalent diameter. Based on the previous equation and research of *Georgiou et al. 2022*, in cases of $\Lambda_f < 1$, the tunnel face is potential unstable. The present research use the value of $\Lambda_f = 1$ as the upper bound where the tunnel face pre-support measures are necessary. tunnel face pre – support by forepoling umbrella.

2.1 *Forepoling umbrella overlap length*

One of the most critical parameters on the tunnel face support by forepoling umbrella, is the overlap length between two continues forepoling umbrellas. As the overlap length is reduces, the tunnel face core tends to fail when the geotechnical conditions are poor and the geostatic load is high. Moreover, there is a critical failure surface, similar with cases of slope stability, where the tunnel face will slide and collapse. When the forepoling umbrella tubes has lower length than the expected failure surface, the tunnel face will collapse. Thus, the forepoling umbrella overlap length must be higher of the expected failure surface.

From the numerical investigation, it was observed that the tunnel forepoling umbrella minimum length (overlap length) in order the tunnel face to be stable depends on the ground conditions and tunnel geometrical characteristics. On the other hand, the impact of the forepoling umbrella stiffness on the overlap length has minor influence on the overlap length. Moreover, the combined tunnel face pre – support by forepoling umbrella and tunnel face reinforcement by fiberglass nails, has a minor reduction on the forepoling umbrella overlap length, especially in cases of shallow tunnels.

At Figure 2, is presented the correlation between the optimal overlap length of the forepoling umbrella (L_{FP}) and the ground conditions via the tunnel face stability parameter (Λ_f), based on the following formula:

$$L_{FP} = 0,35D\ \Lambda_f^{-1,2} \tag{2}$$

where L_{FP} is the optimal overlap length of the forepoling umbrella; D is the equivalent tunnel diameter; and Λ_f is the tunnel face stability parameter (Equation 1).

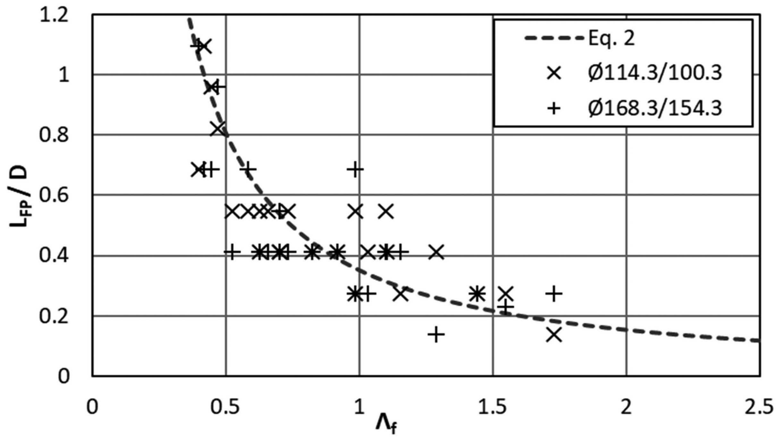

Figure 2. Correlation between the optimal overlap length of the forepoling umbrella (L_{FP}) and the tunnel face stability parameter (Λ_f) for different forepoling tubes stiffness.

From the previous graph, is summarized that for poor ground conditions (meaning $\Lambda_f < 1$), the forepoling umbrella overlap length must be equal or higher of the tunnel diameter (D).

2.2 *Forepoling umbrella response*

In cases of tunnel face pre – support by forepoling umbrella, one of the most critical parameters for the support system design is the expected maximum bending moment on the forepoling umbrella tubes, in order not to yield. The maximum acceptable plastic bending moment ($M_{,pl}$) on a forepoling umbrella can be calculated via the following equation, based only on the geometrical characteristics of the forepoling umbrella and its stiffness.

$$M_{pl} = \frac{1}{6}\left(d_{ext}{}^3 - d_{int}{}^3\right)\sigma_y \tag{3}$$

where d_{ext}= external diameter of the forepoling tube; d_{int}= internal diameter of the forepoling tube; and σ_y = yield strength of the forepoling tube.

From the numerical investigation, it was observed that the tunnel overburden height (H), by meaning of the total stress on the forepoling umbrella tubes has a significant impact on the tube loading, as in cases of lower overburden height, the pipe loading is lower, than in cases of high tunnel overburden height.

Moreover, the numerical investigation shows that the lateral earth pressure ratio at rest (K_o) has significant impact in the maximal developed bending moment (M_{max}) of the forepoling umbrella. In cases where the lateral earth pressure ratio at rest $K_o = 1$ (hydrostatic conditions), the maximal bending moment is higher (equal to double) than in cases of $K_o = 0.5$. On the following Figure, is presented the maximal developed bending moment (M_{max}) on the forepoling umbrella for a shallow tunnel, which is excavated in a poor soil for different values of lateral earth pressure ratio at rest $K_o = 0.5$ and 1.

From the graphon Figure 3, the maximum bending moment on the forepoling umbrella tubes, is recorded on the tunnel crown tubes than on the side tunnel area tubes, for both cases of lateral earth pressure ratio at rest K_o =0,5 and 1.

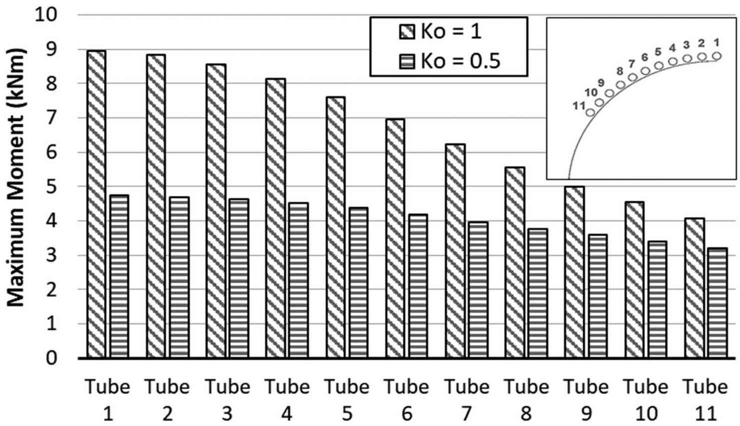

Figure 3. Maximum bending moment on the forepole tubes around the tunnel perimeter for a shallow tunnel in a poor soil, with different values of lateral earth pressure ratio (K_o).

Based on the records of the numerical investigation for both shallow and deep tunnels, the maximum expected bending moment (M_{max}) on the forepoling umbrella is correlated with the ground strength and the tunnel geometrical parameters. Previous researches as *Oreste & Peila - 1998* and *Dias -2013*, can estimate the maximum bending moment on the forepole tubes or the maximum pressure on them, based on analytical solutions using the spring – stiffness method.

In order to estimate the expected maximum bending moment (M_{max}) on the forepoling tubes, the present research correlate the ratio M_{max}/M_{pl} from Equation 3, with the ground strength and the tunnel geometrical parameters, using the following formula:

$$\frac{M_{max}}{M_{pl}} = 0,004 N_{ground}{}^{-1,3} \tag{4}$$

$$N_{ground} = \left(\frac{\sigma_{cm}}{\sigma_v}\right)\left(\frac{H}{D}\right)^{-0,5} \tag{5}$$

where N_{ground} = non – dimensional ground factor; σ_{cm} = soil or rockmass strength; σ_v = vertical stress on the tunnel axis; H = tunnel overburden height; and D = tunnel diameter.

In the graph on Figure 4, is presented the correlation between the expected ration M_{max}/M_{pl} and the factor N_{ground} over the numerical investigation results, using the Eg. 5.

From Figure 4, it is obvious that in cases of poor ground conditions (low values of N_{ground}), the expected loading of the forepoling umbrella by means of the bending moment, increases rapidly

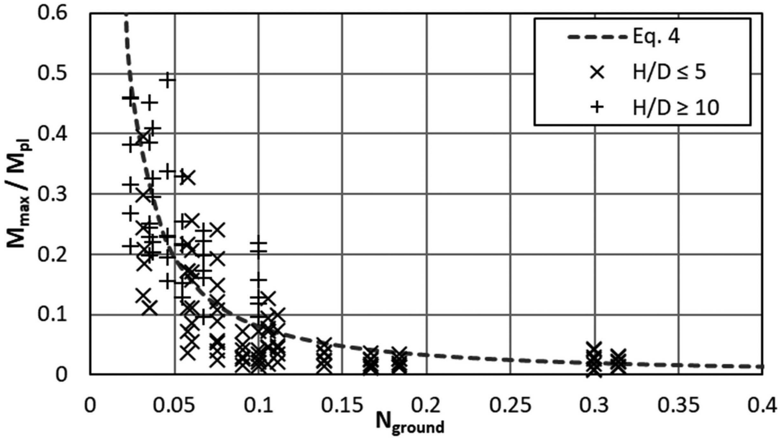

Figure 4. Correlation between the ratio M_{max}/M_{pl} and the non – dimensional ground factor (N_{ground}) for shallow and deep tunnels, using the Equation 4.

as the tunnel face tends to fail due to the increment of the tunnel face extrusion. On those cases, the forepoling umbrella has a significant impact on the increment of the tunnel face stability, due to the reduction of the tunnel face extrusion and reduction of the vertical stress (σ_v) on the tunnel face core. In addition, the maximum ratio M_{max}/M_{pl} on our numerical analysis reaches about 60% and refers to poor rockmass conditions and high overburden stress.

3 TUNNEL FACE REINFORCEMENT BY FIBERGLASS NAILS

3.1 *Fiberglass nails overlap length*

The main purpose of the fiberglass nails is the increment of the horizontal stress (σ_h) in front of the tunnel face in order to minimize the tunnel face extrusion. When the ground is excavated the horizontal stress in the tunnel face tends to be zero. Due to the reduction of the horizontal stress (σ_h) and the active vertical stress (σ_v) the tunnel face behave in similar way to the triaxial shear test, where the reduction of the horizontal stress (σ_h) increases the rapid failure of ground, while the vertical stress (σ_v) is maintained. On the other hand, when the horizontal stress (σ_h) increases, as a result of the tunnel face reinforcement, the potential failure of the tunnel face tends to decrease.

From the analyses carried out for the shallow tunnels (H<50m), aroused that the minimum overlap length of the nails (L_{FG}), depends on the mechanical characteristics of the soil and the intensive loading conditions. For the grounds investigated, it was observed that supporting the tunnel face with fiberglass nails affected drastically the decrement of surface subsidence. Even in occasions with low density the subsidence reduced a lot. While concluded that independently of K_o, the decrement of extrusion when installing nails with density of ρ = 0,5 nail/m^2 it is significantly higher from a density of ρ = 0,25 nail/m^2.

On the other hand, in deep tunnels (H≥ = 50m) it was observed that the tensile force of the nails was increased as the depth increased, due to the increased σ_3 in the bigger depths. The nails' tensile strength was challenged, thus it was selected to use denser installation pattern with 1.25 nails/m^2. Same conclusion on the effect of the tunnel face reinforcement density for both shallow and deep tunnels, verified from 3D numerical analysis on *Georgiou, D. (2021)* research.

On both shallow and deep tunnels ascertain that the minimum overlap length depends on the in-situ geotechnical conditions and the height of the overburden, and of course this cannot be constant, due to the many factors that affect it.

At Figure 5, is presented the correlation between the optimal overlap length of the fiberglass nails (L_{FG}) and the ground conditions via the tunnel face stability parameter (Λ_f), based on the following formula:

$$L_{FG} = 0,5\ D\,\Lambda_f{}^{-0,8} \tag{6}$$

where L_{FG} is the optimal overlap length of the fiberglass nails; D is the equivalent tunnel diameter; and Λ_f is the tunnel face stability parameter (Equation 1).

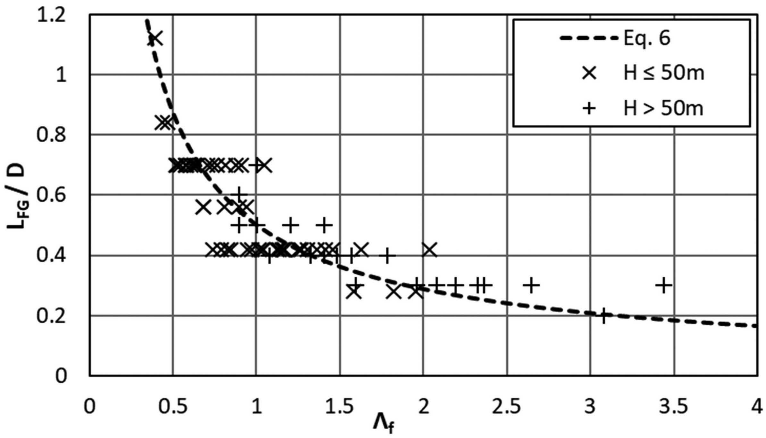

Figure 5. Correlation between the optimal overlap length of the fiberglass nails (L_{FF}) and the tunnel face stability parameter (Λ_f) for different forepoling tubes stiffness.

The best fitting curve of the Equation 6 on Figure 5, is drawn in records of lower values of tunnel face reinforcement density, as the present research doesn't investigate the effect of the tunnel face reinforcement density on the optimal length of the fiberglass nails. In the examined densities of the present research, the differences between the optimal length of the fiberglass nails for different tunnel face reinforcement densities, are not significant.

3.2 *Fiberglass nails response*

From the numerical investigation the maximal axial force in each fiberglass nail was recorded, as it is one of the most critical parameter in the tunnel face reinforcement design, in order the nails to be workable and not yielded. The expected maximum tensile load on the tunnel face fiberglass nails, due to the tunnel face extrusion, can be described by the working level (WL) of them where is the ratio of the expected tensile stress on fiberglass nails over the maximum tensile strength of them.

Based on the investigation records, the maximum working level (WL) of the fiberglass nails can be correlated with the ground strength and the ultimate tensile strength of each nail, using the following formula:

$$WL = \frac{\sigma_y}{F_u} 100\% = 4 W_{FG}{}^{-0,7} \tag{7}$$

where WL is the maximum working load of the tunnel face fiberglass nails; σ_y is the maximum expected tensile stress on the fiberglass nails; F_u is the ultimate tensile stress of each nail; and W_{FG} is the fiberglass nail reinforcement parameter, which is calculated as follow:.

$$W_{FG} = \left(\frac{P_{u,FG}}{\sigma_h}\right)^{0,6} \left(\frac{p_o}{\sigma_{cm}}\right)^{-0,5} \left(\frac{H}{D}\right)^{-0,5} \tag{8}$$

where $P_{u,FG}$ is the total tunnel face pressure from the fiberglass nails reinforcement (total applied force of fiberglass nails/tunnel face area); σ_h is the horizontal field stress (lithostatic); p_o is the geostatic field stress (lithostatic); σ_{cm} is the ground strength; H is the tunnel overburden height; and D is the tunnel equal diameter.

On Figure 6 is presented the correlation between the maximal expected working level (WL) of the tunnel face fiberglass nails with the fiberglass nail reinforcement parameter (W_{FG}), using the Equation 7 and based on numerical investigation records.

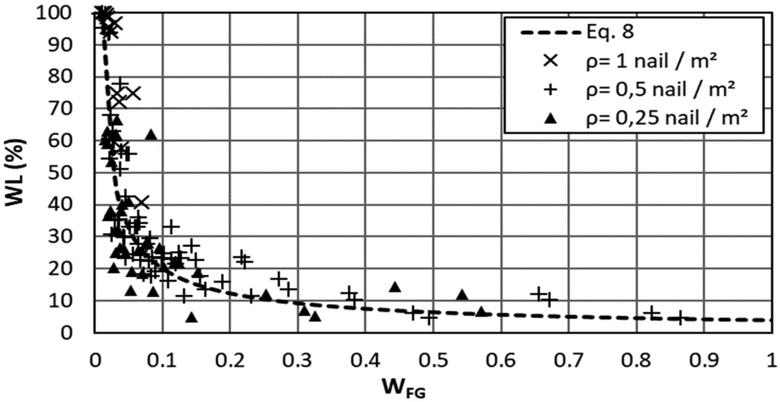

Figure 6. Correlation between the expected working level (WL) of the tunnel face fiberglass nails and the fiberglass nail reinforcement parameter (W_{FG}) for different tunnel face reinforcement densities (ρ).

4 CONCLUSIONS

The present research investigate the effect of the tunnel face pre – support design parameters by using forepoling umbrella or fiberglass nails, in the reduction of the tunnel face extrusion and the increment of the tunnel face stability. From the numerical investigation records, is observed that the optimal geometrical characteristics of the pre – support measures, depends mainly on the ground conditions and less in the pre – support mechanical properties. Thus, the optimal geometrical characteristic of the tunnel face pre – support measures can be calculated from the analytical formulas and the nomographs of the present investigation, which correlate the design parameters of the tunnel face pre – support with the ground conditions. Moreover, the design loads of the tunnel face pre – support measures can be estimated from the analytical formulas, which are presented in the present research.

In addition, from the numerical investigation it was observed that in cases of using a combination of both forepoling umbrella and fiberglass nails for the tunnel face pre – support, the effect on the reduction of the optimal overlap length either of the forepoling umbrella or the fiberglass nails, it is not significant and the authors suggest to use Equations 2 & 6 for the optimal length of them.

REFERENCES

Anagnostou, G., & Kovári, K. (1996). Face stability conditions with earth-pressure-balanced shields. Tunnelling and underground space technology, 11(2),165–173.

Diakoumi E., “Numerical analysis for the optimum application of tunnel face support elements in shallow tunnels”, MSc Thesis, NTUA Greece (2020).
Dias, D., & Oreste, P. (2013). Key factors in the face stability analysis of shallow tunnels. American journal of applied sciences, 10(9), 1025.
Georgakopoulos G., “Numerical analysis for the optimum application of tunnel face support elements in deep tunnels”, MSc Thesis, NTUA Greece (2021).
Georgiou, D. (2021). Numerical investigation of the tunnel face stability (Doctoral dissertation), NTUA, Athens.
Georgiou, D, et al. (2022). 3D Numerical Investigation of Face Stability in Tunnels with Unsupported Face. Geotechnical and Geological Engineering, 40(1),355–366.
Malandraki A., “Design of the tunnel face pre-support by forepoling umbrella”, MSc Thesis, NTUA Greece (2021).
Oreste, P. P., & D. Peila. “A new theory for steel pipe umbrella design in tunnelling.” Proceedings of the World Tunnel Congress. Vol. 98. 1998.
Peila, D. “A theoretical study of reinforcement influence on the stability of a tunnel face.” Geotechnical & Geological Engineering 12.3 (1994): 145–168.