Chapter

Design and Construction for Tunnel Face Stability: Theoretical and Modeling Approach

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Abstract

Tunneling is considered to be among the most important projects in all countries worldwide. However, interspersed, some tunnels rise to problems of instability during excavation. This chapter is a case study of the tunnel of "Djebel El Kantour" which is part of the East–West Algerian Highway. Face stability is the most critical problems that affect the subject of our research. This study is carried out via analytical and numerical methods based on the instability relationship, characteristics of the ground and the geometry of the tunnel, to draw conclusions and recommendations for overcoming this problem.

Keywords: replace stability, tunnel face, convergence, confinement, soil characteristics, excavation

1. Introduction

The instability problem of the tunnel face has occurred during the construction of several tunnels in the world (STEFANO Tunnel 1984, TASSO Tunnel 1988, VASTO Tunnel 1991, recognition gallery of St Martin-de-La Porte 2001). This problem has been reported many times in Algeria (Algiers Metro Tunnel 2000, Djebel El Kantour Tunnel in Skikda 2010 and Djebel El Ouhach Tunnel in Constantine 2011). Several studies have been conducted in this regard: [1–3], and various stabilization techniques of tunnels face were used:

1. Fiberglass method of FIT (injection tube) was applied for the first time in 1988 HST Link Roma – Fiorenze(Italy);

2. The technique of shotcrete.

The tunnel face is located at kilometer point (KP 230 + 586.5), which is located inside the tunnel of «Djebel El Kantour". The main problem in this tunnel is the instability of the face which was reported since the beginning of the project, especially in northern the tunnel because of two key factors: the quality of the ground (Marly sandstone clay) and low coverage. In this research finite element simulation were conducted using the software ANSYSE in order take into account the staged construction technique for better estimation the vertical and longitudinal deformations, as well as the failure mechanisms to the front of the tunnel face.

To solve this problem, many techniques have been used to stabilize the face, but the technique did not yield the expected results. Therefore, the prime contractor applied a new technique called the Fiberglass Technique (FIT). However, it proved inadequate in this case, due to its high cost and the limited effectiveness. This is mainly due to the poor mastery of the excavation method, which in turn, was not suitable to this type of rocks. In this study, we try to demonstrate, via a numerical modeling tool, the relationship between the attack section and the deformation field on the Tunnel face.

2. Case study. Djebel El Kantour tunnel

T4 tunnel is located in the north-east of the department of Constantine. It crosses Djebel El Kantour from south to north with a total length of 2500 m. Its cover is higher than 15 m, and reaches 224 m in maximum points (**Figure 1**).

The section of the tunnel was chosen according to the geometric characteristics, the geological and geotechnical data tests performed on the ground in question as well as the height of the cover. To take into account the natural conditions of the surrounding terrain, an arched profile has been adopted to ensure the stability of the structure and the service conditions during the constructions process.

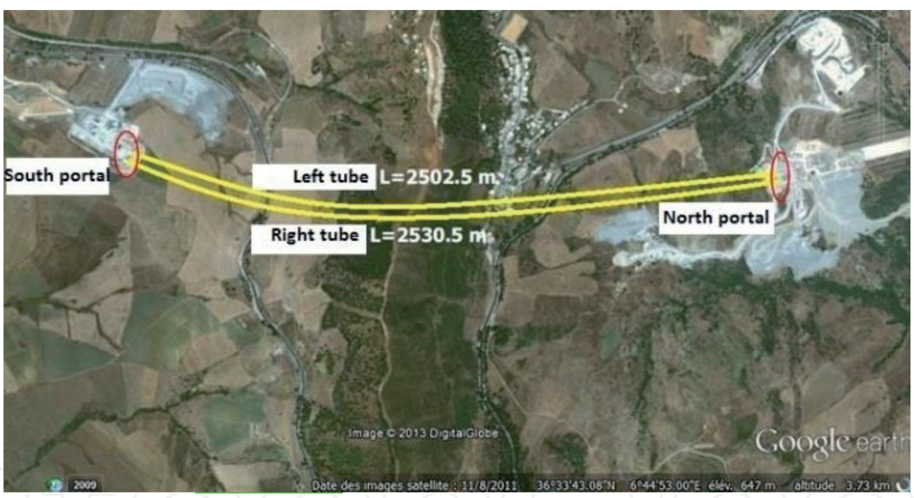

Figure 1.
Position of the tunnel T4.

3. Geological and geotechnical parameters of the study

The design of the tunnel was performed on the basis of a geological and geomechanical survey conducted from recent geotechnical survey includes the following investigations:

- Geological surveys conducted by geologists' experts;
- A recognition campaign by core drilling, in situ and laboratory tests.

It noted that any study must lead to the acquisition of the following information with the maximum possible degree of reliability, considering the wide range of technical resources currently available in the geological survey field:

- Structural geological and hydrogeological conditions and the natural stress state of the ground to be tunneled;

- The physical characteristics, the strength and deformability of the geological bodies affected by the excavation:

- The hydrogeological conditions in the rock mass.

Geology of the area crossed by the tunnel is essentially of Cretaceous age (Telliennes Tablecloths) and consists of marl and limestone in the form of strongly folded and sheared blocks (**Figure 2**). These are covered by Quaternary deposits consisting of clays, silts and conglomerates.

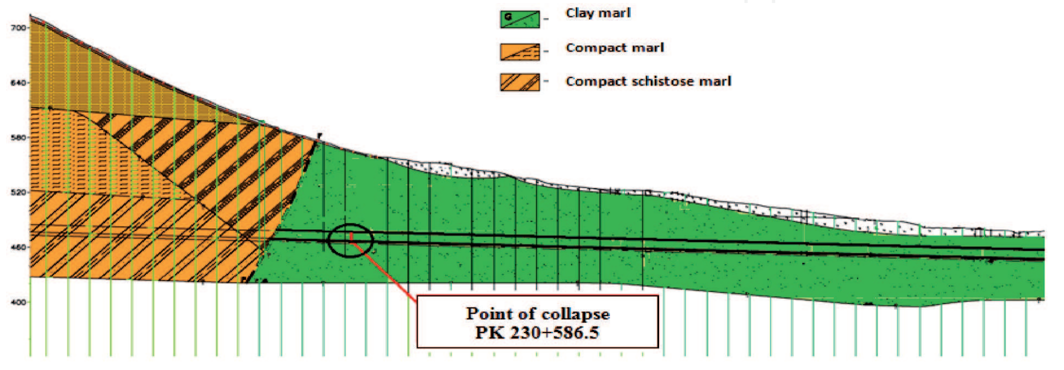

Figure 2.
Plan and geological section of the T4 tunnel and the collapse point.

N	Parameters	Coefficients	Rating
1	Resistance to uniaxial compression	R0 $(R0 < 0.1\,MPa)$	0
2	Rock Quality Designation (RQD)	(115–3.3Jv) 0–25 very poor	3
3	Spacing of discontinuities	$< 600mm$	5
4	Conditions of discontinuities	Slickensided surfaces; 3–10; 10–20; 0.1–5 $> 5mm$;< 5 mm; highly altered	6.5
5	Groundwater	Damp	10
6	Rating adjustment for discontinuity orientations	Digging against the dip	−12
Total rating			12.5

Table 1.
Results according to the RMR classification system in the PK 230 + 586.5.

Total rating	<21
Class	V
Average stand up time	30 min for 1 m span
C « KPa »	<100
$\varphi(°)$	<15
Description	Very poor rock

Table 2.
Total-rating results.

γ (kN/m^3)	20
E (MPa)	200
C (kPa)	50–160
φ(°)	25

Table 3.
Geotechnical parameters.

The design of the tunnel was carried out on the basis of geological and geotechnical studies. The results of RMR classification are presented in **Table 1**.

The table below (**Table 2**) shows the value of the rock classification (Rock Mass ratings) determined after application of the Total rating.

In our case, the rocks are of marl-clay-sandstone type (**Table 3**).

4. Collapse problem of the tunnel face at PK 230 + 586.5

This tunnel consists of two tubes spaced 22 m. The problem of collapse occurred on the southern side of the tunnel in the right tube. The RMR classification results obtained on the Southern side during the day that preceded the crisis was similar to the classification results of the opposite side of the tunnel, which also suffers from the same problem of instability, but with a technique linked to the soil stabilization.

As we dig in the initially stable soil, the preexisting stress state has changed. Indeed, the stress on the excavation contour vanishes: the decompression phenomenon. This change in the stress state appears only in an area surrounding the Tunnel face: the influence area of the face. It extends over a length towards the front edge which is of the same order of magnitude as the diameter of the tunnel according to the measurements performed on several displacement starts [4, 5].

The usual methods for calculating the tunnel's, Tunnel face stability are resulting from experimental studies [6], extrusion testing in laboratory [7] Semi-empirical and theoretical which mainly the approach of calculating the rupture [8, 9].

In our case, the experiment shows that the ruptures of the Tunnel face can mobilize important volumes of ground.

The first systematic studies on the face instability of the tunnels dag in the soft soil carried out by [10] were used to characterize the stability conditions starting from a stability parameter coefficient (**Figure 3**).

The stability coefficient N is defined in [10].

$$N = \frac{\sigma\_s - \sigma\_T}{Q\_u} + \frac{\gamma.(C+R)}{Q\_u} \quad (1)$$

Where;

γ: density of the rock.

Q u: Shearing resistance (**Figure 4**).

The **Figure 5** gives an indication of the relation between the amount of real stability and awaited deformations.

In our case, the parameters required for the calculation of the stability coefficient N are the following:

- The surface load; $\sigma_s = 14.10^5\,\text{Pa}$ and $\sigma_T = 0$;

- The cover C = 25 m;
- The tunnel radius R = 5 m. $\gamma = 2.10^4\ \mathrm{N/m^3}$; Qu =3,1.105 Pa;

The obtained result for the stability coefficient is N = 6.45, which means that the tunnel face is unstable.

In our case, the Tunnel face is unstable; therefore, we try to find the value of the pressure of supporting σ_T suitable to be applied in order to decrease this state towards an Elastoplastic deformation.

The interval of σ_T is calculated starting according to the following formula:
$N \epsilon$ [2 untill 4]

$$2 \le \frac{\sigma_s - \sigma_T}{Q_u} + \frac{\gamma.(C+R)}{Q_u} \le 4 \tag{2}$$

$$2 \le \frac{14.10^5 - \sigma_T}{3{,}1.10^5} + \frac{2.10^4.(25+5)}{3{,}1.10^5} \le 4$$

The required σ_T an Elastoplastic deformation the following:

$$13{,}17.10^5\,\mathrm{Pa} \ge \sigma_T \ge 6{,}97.10^5\,\mathrm{Pa}$$

In this case, the pressure to be exerted on the Tunnel face will lie between the two following values:

$$\sigma_T \epsilon\,[\approx 0.7\ \text{untill} \approx 1]\ (\mathrm{MPa}).$$

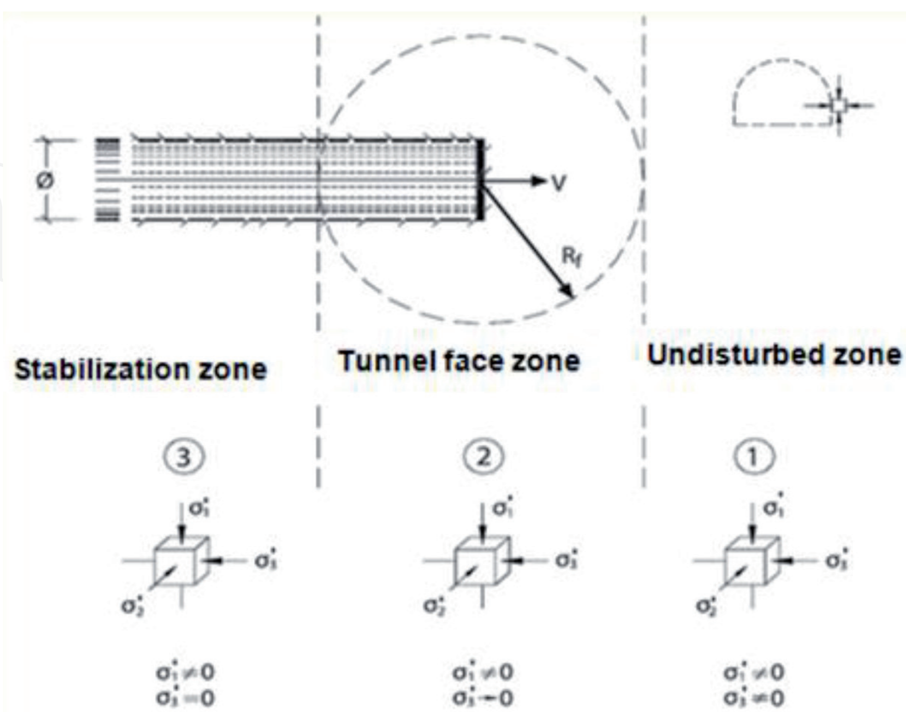

Figure 3.
Formation of three characteristic zones during the digging of a tunnel (Lunardi 1993) [11].

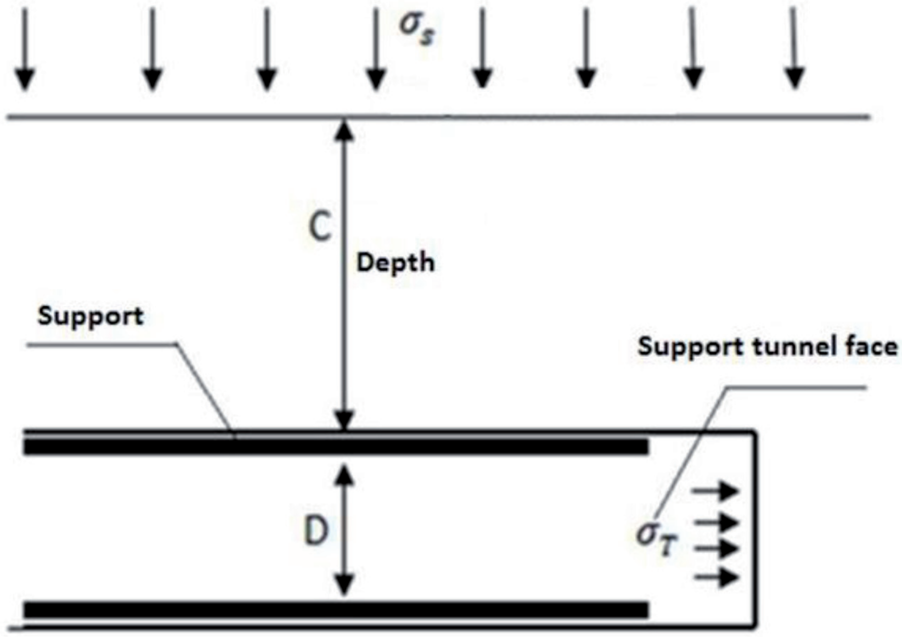

Figure 4.
Tunnel face schematization.

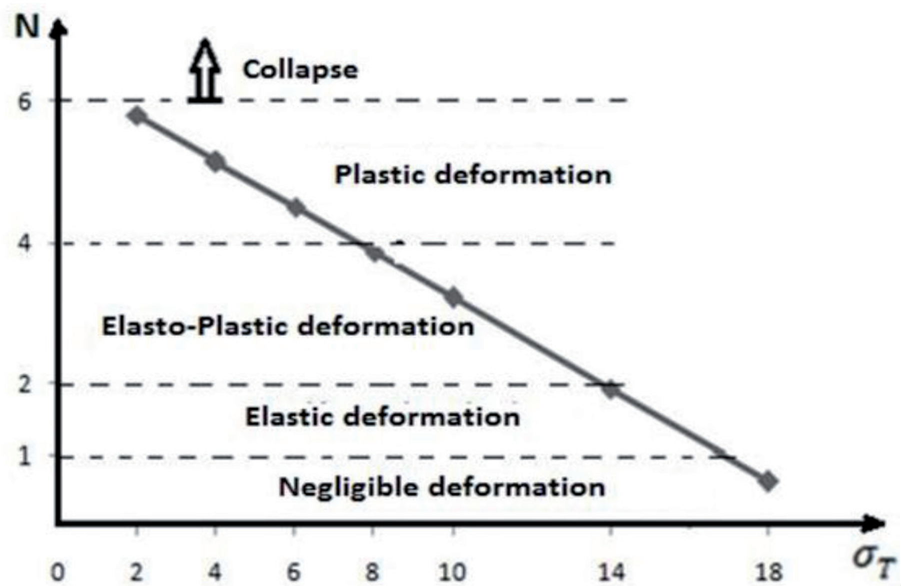

Figure 5.
Relation between the stability coefficient and the face supporting according to [12].

The state of stress in the ground is considerably greater than the strength properties of the material even in the zone around the face. For this consideration based for the results of the diagnosis phase, the techniques to be applied for the application of the supporting pressure on the Tunnel faces are as follows:

- Shotcrete;
- Fiber glass bolting.

Both methods assure the rigidity of the core of ground ahead of the face, and therefore the conditions of stability in that ground, have a decisive effect on that deformation response and determine how an arch effect is triggered and consequently the tone of the stress–strain response in the whole tunnel.

On the other hand, they are difficult to apply this theoretical approach in the domains of soft rocks, flysch and soils, give insufficient consideration to the effects of natural stress states and the dimensions and geometry of an excavation on the deformation behavior of a tunnel and fail to take account of new constructions systems [13].

However, it does not give adequate consideration to the construction stages and therefore it does not constitute a fully integrated method of design and construction. To do this, our analysis of the deformation response continued using the numerical modeling which is able to consider stress states in the ground that are not of the hydrostatic type, which take due account of gravitational loads and which also calculate the effects which the various construction stages have on the statics of a tunnel by simulating the real geometry of lining structures and the sequence and the distance.

5. Numerical analysis

In engineering practice, different design methods tend to be used; in this study, advanced numerical modeling was used due to its ability to predict vertical and longitudinal deformations; as well as the failure mechanisms at the front of the tunnel face [14, 15]. It can indeed be used to simultaneously take into account constraints and anisotropic materials, tunnel advance stages and any pre-containment and cavity containment intervention. In this work, the use of a calculation code through finite element method according to the execution situation [16–18]. The numerical parameter used in the simulation as already mentioned in the **Table 3** resulting from the geotechnical investigation of the zone in question, where the behavior criteria used in the simulation is Drucker-Prager criterion, which as a generalization of the Mohr–Coulomb criterion for soils. The criterion is based on the assumption that the octahedral shear stress at failure, it depends linearly on the octahedral normal stress through material constants. The results indicate that the action of the surrounding terrain on the tunnel based on the attack section R. The main input parameters are the mesh network of elements which determines the domain to which the analysis applies, the geomechanical properties of each element, the surrounding conditions and the loads acting (**Figure 6**).

Numerical simulations allowed obtaining practical results of the radial and longitudinal displacements in the figure and table below:

5.1 Longitudinal displacement

The following diagram shows the extrusion of different attack section based on the distance in front of the face (**Figure 7**).

5.2 Radial displacements vertical displacements

The vertical and longitudinal deformations and changes of the critical zone that occur in the tunnel are linked to attack section R. When the maximum value of the attack section “R” is equal to 5 m, the corresponding Maximum Vertical Displacement (Ux) is equal to 0.53 m, and Longitudinal Deformation in front of the Tunnel face can reach a maximum of 65 m.

In the case where the radius R = 3.5 m attack, the maximum vertical displacement (Ux) is 0.2 m. So, it is 3 times less than the previous case, and similarly for Longitudinal Deformation in front of the Tunnel face which does not exceed 50 m. it is shown that simulation results are consistent with the observed extent and those obtained in literature. (**Figure 8**).

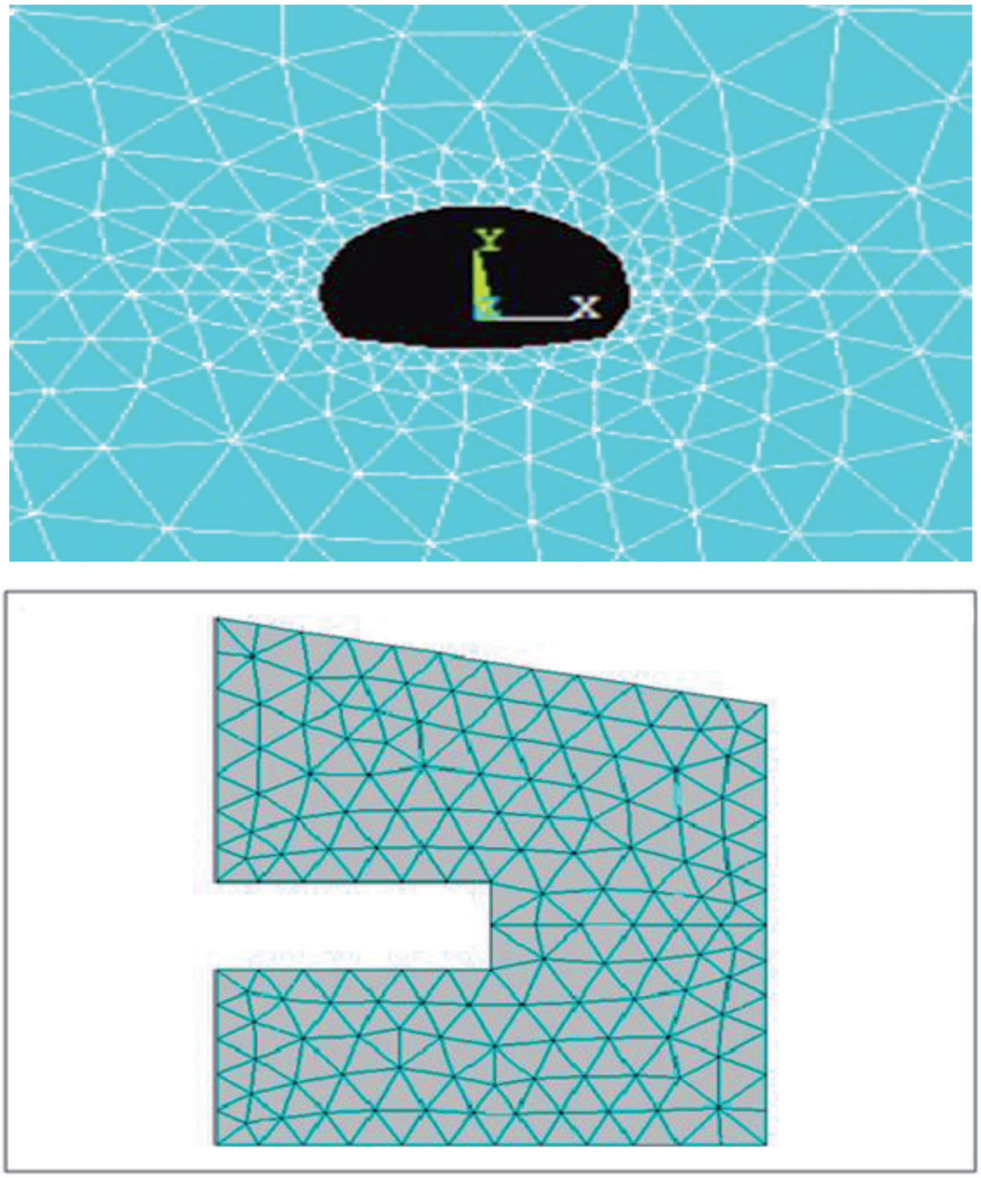

Figure 6.
Model and meshing.

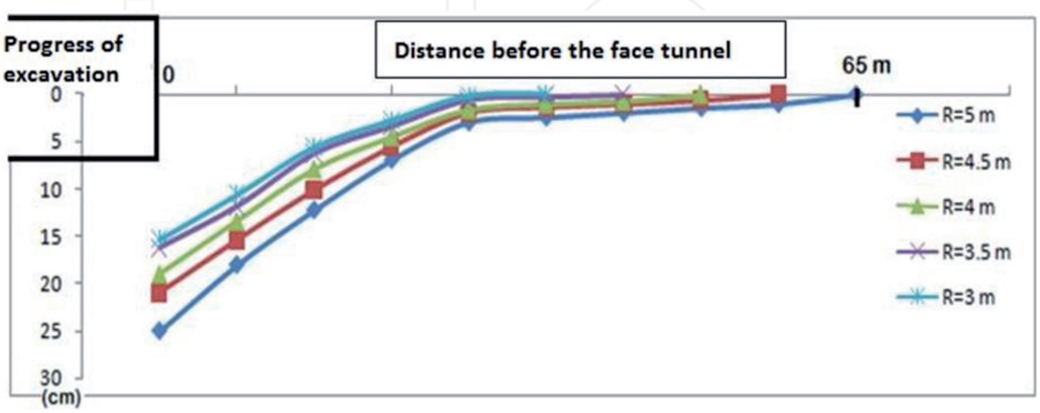

Figure 7.
Uz displacement curves based on the attack section R.

5.3 Horizontal displacements

We studied the horizontal displacement of the solid rock in vertical section for all cases of attack sections.

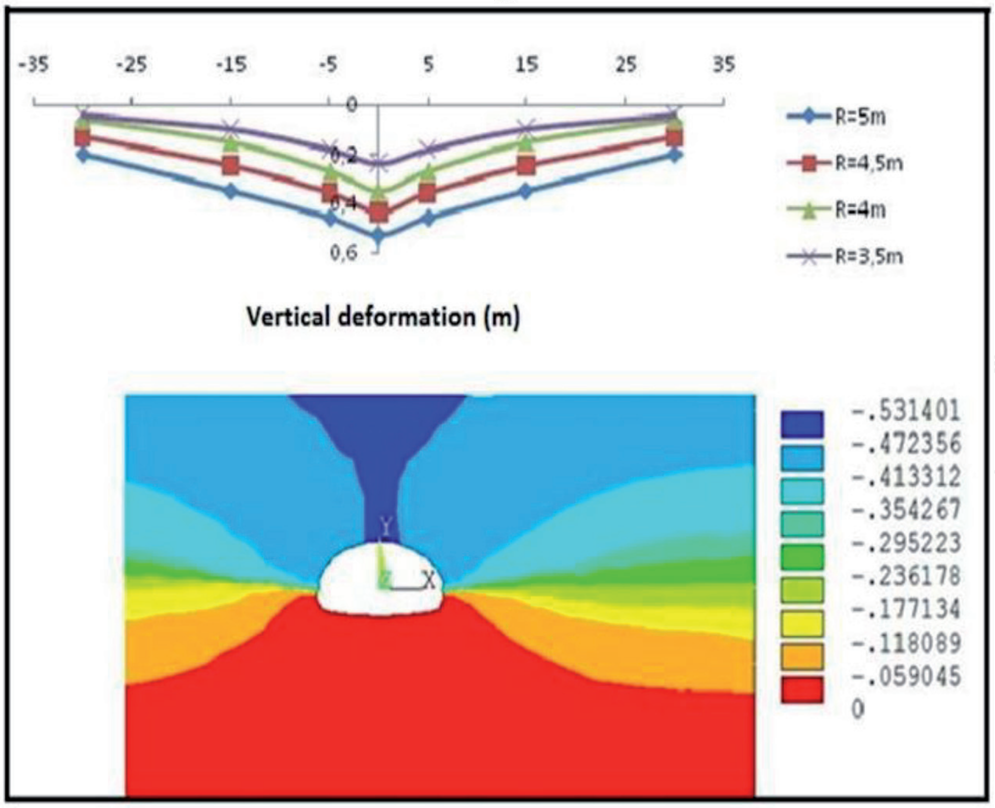

Figure 8.
Ux vertical displacement curves based on the attack section R.

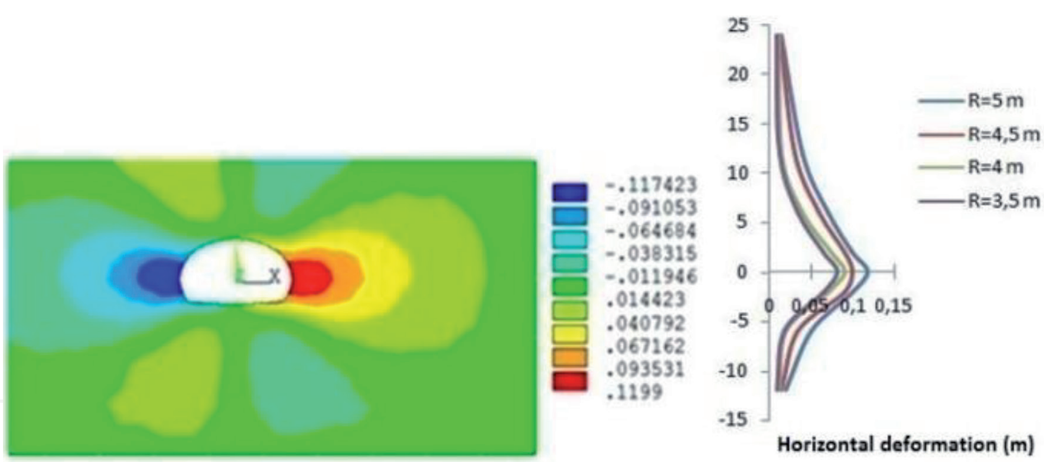

Figure 9.
Uh horizontal displacement curves based on the attack section R.

The (**Figure 9**) shows the horizontal displacements along a vertical section for various attack sections R.

The maximum horizontal displacement Uh is the edge of the excavation to tend towards zero displacement at a considerable distance from the tunnel. With a difference between the displacements, values of every driving section R compared to another.

6. Conclusion and recommendations

To ensure the stability of the Tunnel face and minimize its stress concentrations and deformations; the exploitation of this study's results with the inclusion of

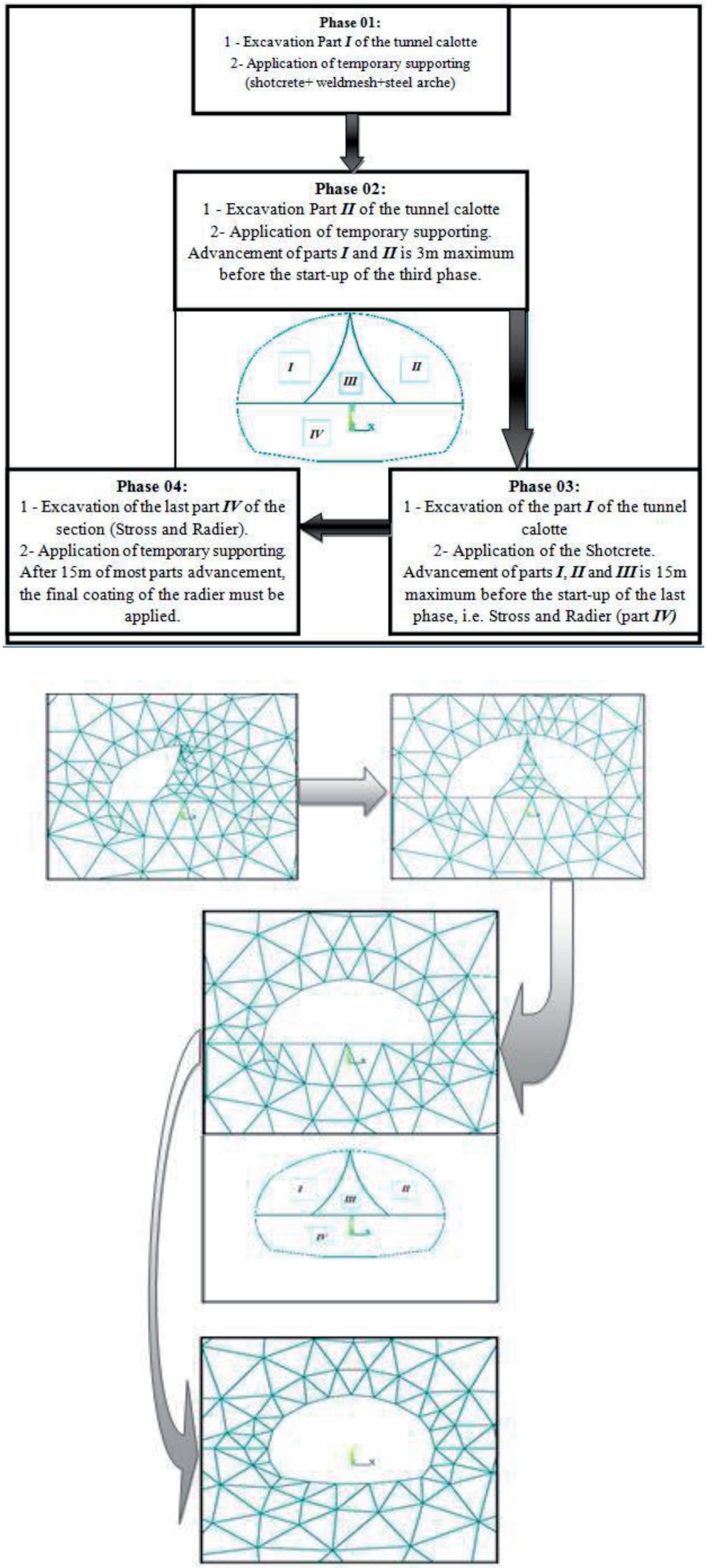

Figure 10.
Staged construction with divided section.

geometric and geological conditions traversed by the tunnel, we recommend the following points:

- The change in the advancement method in divided sections when terrain features are insufficient to ensure the necessary stability to the face. This method is adopted when the excavation cannot be performed in full section.

- The chart below (**Figure 10**) shows the progress detailed in the proposed divided section.

Applying pressure on the supporting σ_T on the Tunnel face to reduce its deformation, of which values interval should be:

$$\sigma_T \epsilon \left[\approx 0{,}7 \text{ untill} \approx 1{,}3 \right] \ (\text{MPa}).$$

It is suggested that the technique proposed maybe used for construction considerations under complication geological condition which satisfactory effect in engineering practice.

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