Geotechnical Aspects of Underground Construction in Soft Ground – Yoo, Park, Kim & Ban (Eds)
 ISBN 978-1-138-02700-8

Physical and numerical experimental study of stress-strain state of the structure lining of a shallow tunnel

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ABSTRACT: This article refers to research carried out to determine the stress-strain state of the structure lining of a shallow tunnel subjected to an increase of the forces acting on the ground surface, using two different methods and independent, allowing it to compare the results obtained from the magnitude of the maximum stress in the structure and the safety factor for that effort. The Two methods used during the investigation that developed compromise: first, building a physical model in a test tunnel type 1g, which was properly instrumented to record the variables of interest during the test, increased load. The second was developed with a numerical model using the finite element method, which simulated the increase of the forces acting on the ground surface. In particular, the results presented in detail for two points in the section of the lining tunnel: the top or vault and lateral or hastial. The comparison of the results obtained by the two methods conclude that for the present study the result of the maximum value of internal stress in the structure calculated from numerical modeling showed a difference of 1.64 times in excess compared with the record in the physical test. Regarding the analysis of the safety factor FS for maximum overload condition applied it was found that the numerical modeling method exceeds 1.6 times the value obtained in the physical test.

1 INTRODUCTION

Currently in the design stages, the main possibilities to study the stress-strain state of tunnel structure under geostatic loads can be grouped in: theoretical solutions, mathematical modeling and experimental physical models.

Thanks to the advancement in computing and programming the application of the finite element method FEM has allowed to determine the stress-strain state by discretization of the continuum tunnel structure, course is also based on an elastic or elasto-plastic matrix of rigidity established in the numerical model. Some of the recent work in this area is that presented by Abu-Krisha (2007), who performed an investigation of the deformation and internal stresses in the lining of an inner circular tunnel section of 10.8 m and varying the depth of emplacement of tunnel. Other cases of application of this method can be seen in the works of Chen & Ruan (2007), (John et al. 2009), Nakthong & Suwansawat (2009).

It is important to note that it is necessary to be clear about the benefits and limitations of numerical analysis, as we clarify (Bieth et al. 2007) the analysis through the use of software based on FEM helps to perform sensitivity analyzes in search to optimize designs. However, always keep the uncertainty about the actual behavior of the ground. Also Potts (2003) shows that there are limitations in the use of numerical modeling of three basic aspects: limitations caused by the constitutive models of material, limitations in their application in nonlinear models and the dependence of the results on user decisions who created the model. On the other hand, as affirmed Muir (2004) "The physical modeling is a way to study particular aspects of behavior of the prototypes." The results of the scale models must respect the laws of similarity, ie, they must attend the scale factors and dimensional analysis (Langhaar 1954). Research on scale models in physical tests for the study of the stress-strain state in tunnel problems can be seen in the works of (Ahn et al. 2006) and results which compare the results of physical testing with numerical models are observed in the works of (Kim et al. 2007) and Torres & Nieto (2009). In recent years this type of research has become highly relevant in the development of research on tunnels.

The importance of estimate the stress-strain state of the lining tunnel during the design stages is to allow the calculation of the safety factor for the structure, and solve the problem in the most approximate to the real behavior of the tunnel that can get a technical and financial proposal optimized structure, which for projects of very long tunnels has a major impact on the final cost of the project.

Table 1. Input soil parameters.

Parameter	Name	Value
Type of material behavior	Type	Drained
Soil weight above phr. level (kN/m^3)	γ_{unsat}	19,187
Young's modulus (kN/m^2)	E_{ref}	13.000
Poisson's ratio	ν	0.3
Cohesion (kN/m^2)	C_{ref}	1.0
Friction angle (Degree)	Φ	31.0
Dilatancy angle (Degree)	Ψ	1.0
Coefficient of lateral earth pressure	K	0.485

Table 2. Input lining tunnel parameters.

Parameter	Name	Value
Type of behaviour	Material type	Linear–elastic
Volumetric Weight (kN/m^3)	γ	16.00
Young's modulus (kN/m^2)	E_{ref}	842.70
Poisson's ratio	ν	0.1
Compression strength (kN/m^2)	R_c	416.70

2 STUDY OF THE TUNNEL MODEL TEST AND NUMERICAL MODELLING

The following are the three fundamental stages in the development of research.

2.1 *Preliminary stage*

Initially a series of laboratory tests on artificial soil mix were performed to know their geomechanical properties. To define the materials, the percentages for mixing and compaction energy were used by the guide chart of artificial soil mixes created by GeoLab – Geotechnical Laboratory Models at La Salle University, were to use a mixture of sand – bentonite and oil was determined. Also through triaxial and the direct shear tests were found basic geomechanical characteristics soil to a behavior model type Mohr-Coulomb.

Also the ring material forming the circular lining of the tunnel was defined and characterized. The material is composed of a mixture of sand, water and gypsum with a reinforcement wire gauge galvanized mesh 0.38 mm, the mechanical properties lining are given in Table 2.

This stage also defined the number and type of measurement sensors to install on the physical model of the tunnel, and also defined the magnitude of the maximum load to be applied to transmit low levels of stress to guarantee the work structure of the material in the elastic range and compressive stresses in the rings.

2.2 *Physical model of the tunnel and load test*

The tunnel model was built in the GeoLab, the physical model in this research is considered the prototype

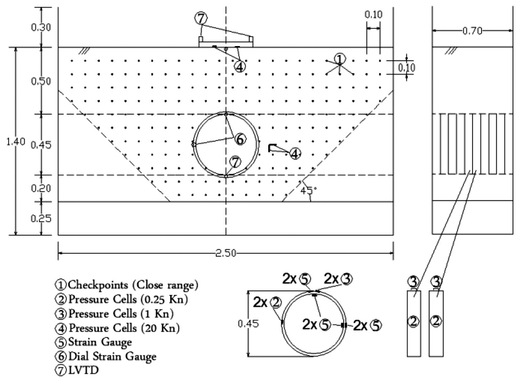

Figure 1. Measurement sensors installed in the physical model.

for comparison with the numerical method, where the construction process recreates the basic steps of the construction of a tunnel by TBM with closed cutting head, the model dimensions are limited by the capacity of the used experimental wall, figure 1 shows the geometry of the tunnel has a coverage of H = 0.50 m of material, the structure of the tunnel of circular shape has an internal diameter of 0.38 m and external 0.43 m, the width of the model is 2.5 m and the horizontal depth of 0.7 m, thereby guaranteeing the boundary conditions in the cross section analyzed and a stress planar task. The tunnel is formed by 10 rings of 0.075 m thick each.

The sensors in the model contemplated: in rings 4 and 5 located in the center of the tunnel length were installed pressure cells of 0.25 kN/m^2 in the external face of the hastial and on the vault of tunnel pressure cells of 1 kN/m^2; in the internal and external fibers of the rings were installed strain gauges to control of its strains; movement controls were installed in the ring 5 an LVDT on the hearth tunnel and two displacement sensors on both the hastial and in vault tunnel. Once finished with the installation of the rings of the tunnel, a metal plate of 0.4 m × 0.4 m was installed on model surface; when properly preparing the contact between the plate and the ground to ensure the smooth transfer of load, also on two corners of the plate were installed LVTD to control vertical movement of the plate and under the plate were installed two pressure cells of 20 kN/m^2 for monitoring the load distribution; also at a distance of 0.1 m from the outside of the hastial of the tunnel (approx. ring 5) were installed on the soil mass two pressure cells of 20 kN/m^2: one horizontally and the other vertically. Additionally, on the acrylic windows faces of the model checkpoints displacements for the soil mass were installed.

Once the performance was checked on all of the sensors and measuring instruments, we proceeded to make a load test in two cycles. Initially loaded onto the plate weights progressively up to a value of 416 kg, and gain variations on the records of the sensors installed in the rings, with this first cycle search in some measure ensure contact between the ground and the external

Figure 2. Images of the model and physical test.

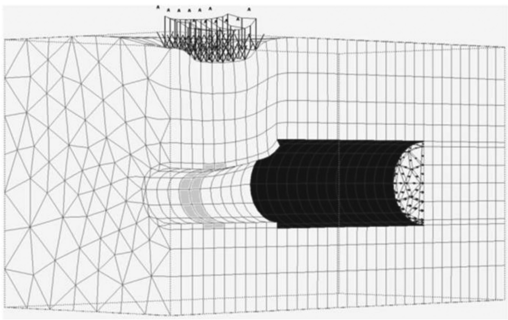

Figure 3. Three-dimensional numerical model.

contour lining tunnel. After downloading up to 252 kg and the second charge cycle was made to a maximum weight of 558 kg on the plate, of this second cycle the data and records were obtained of changes in contact pressures and strains in the fiber rings with strain gauges and in the similar control points the values were averaged.

Thus was obtained stress-strain state of the lining and in particular the results for the recharge cycle were studied and the determined structural safe factor for the maximum weight applied on the plate corresponding to 558 kg.

2.3 *Numerical modeling with F.E.M.*

In the computer room of GeoLab, we constructed a mathematical model of the physical test employing the finite element method, using the software Plaxis 3D Tunnel V2 – figure 3.

Experimental test was simulated through 35 stages of calculation, the first 15 phases correspond to the length excavated for the advancement of the TBM shield in the model (1.125 m), stages 16 to 25 correspond to the advances and positioning of rings in the physical model (10 rings) and stages 26 to 35 correspond to charge-discharge and charge cycle conducted in the experimental physics model.

For analysis of data nodes and stress points on the mathematical model were sought, which correspond as closely to the coordinates of points on the instrumented ring during physical testing. Thus, the analysis during the comparison of the results was greatly simplified and as in the previous stage of research was obtained stress-strain state of the structure for the phases on the recharging cycle and calculated structural safety factor

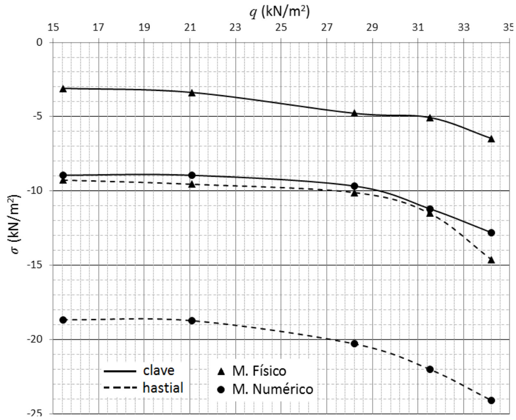

Figure 4. Internal stresses in the lining to the charging cycle.

corresponding to the applied load of 558 kg on step 35 of mathematical modeling.

3 RESULTS

Figure 4 shows the results obtained by the two methods to study the evolution of the internal media stress for the control points in the section of the tunnel – in the vault and in its hastial, during the recharging cycle is which is in the range between 252 kg and the maximum weight applied during the test 558 kg and that in terms of surface load is calculated according to equation (1).

$$q = \frac{F}{A} \tag{1}$$

where $q =$ Surface load (kN/m^2); $F =$ Weight applied to the plate (kN); $A =$ Area of the loading plate (m^2).

As shown in Figure 4, where maximum weight of 558 kg is applied, which generates a surface load of 34.2 kN/m^2, the maximum stresses in the section are located in the hastial of the tunnel. The resultant stresses in the physical model are calculated from strains measurement with the strain gages and calculated by using the equation (2) for the internal and external fiber of the tunnel lining after the obtained values of stress were averaged, meanwhile stress at points of interest to the mathematical model are obtained directly from the results provided by the program.

$$\sigma = E \cdot \varepsilon \tag{2}$$

where $\sigma =$ Stress (kN/m^2); $E =$ Young's modulus (kN/m^2); $\varepsilon =$ Strains.

The maximum internal stress is obtained from the numerical model and has a value of $-24{,}10$ kN/m^2, (with the physical model was an internal stress $-14{,}61$ kN/m^2).

Figure 4 shows that the difference in magnitude of the stress between the vault and tunnel hastial in the

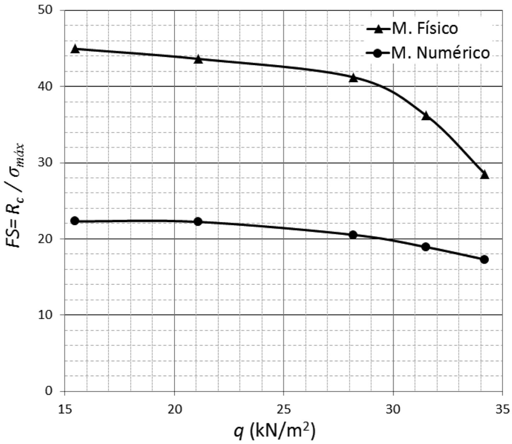

Figure 5. Variation of safety factor depending on the incremental stress.

numerical model is approximately 10 kN/m^2 and this difference is more or less constant during the charging cycle, the same behavior is observed in physical readings of the experimental model which maintains a difference of magnitude of stresses in the section of about 6 kN/m^2 between the same checkpoints.

To calculate the FS – structural safety factor of the tunnel, we compared the resistance to compression of the material. R_c determined in the preliminary stage of the research, with maximum average stress on the tunnel section determined by each of the two different study methods, Figure 5 shows the results.

In Figure 5 the FS values decrease as the value in surface charge is increased, because obviously also increase the internal stresses within the structure section, also show that for a given surface load value within the range of observation the lowest FS value is obtained from the numerical model, and the highest value of FS from the physical model. For the most extreme point on the surface load corresponding to a maximum weight applied on surface of 558 kg, we have calculated FS = 28.5 for the physical model and FS = 17.3 in the numerical model.

If the pattern continues for each of the curves of Figure 5 the fault condition, i.e. for FS = 1,0 is reached in the physical model for a load of 39.61 kN/m^2 and the numerical model to a load 51.39 kN/m^2 and the two curves intersect at a load value of 37.23 kN/m^2 with which the FS reaches a value of 15.6.

4 CONCLUSIONS

From the comparative analysis of the results, we conclude that in terms of internal stresses in the section of the tunnel structure, physical experimental test will have a maximum value of −14.61 kN/m^2 located in the hastial of the tunnel, with the calculation from the numerical model obtained 1.64 times difference most with the results of physical testing. The rigidity of plane surface for load application must be considered.

In future experiments, is suggested applying the load plane with a fluid in a flexible device in order to guarantee that load will be uniformly distributed.

As for the structural FS comparative results for the condition of maximum load 34.2 kN/m^2, can be concluded that the evaluation in the resistance obtained by result of numerical modeling method exceeds 1.6 times the value obtained by physical test.

The fault condition differs between the two methods by about 12 kN/m^2, i.e. there is a difference between around 23% compared to the numerical and experimental physics test at the fault condition.

New questions are open towards future research to compare the physical test results with other numerical methods, for example, based on discrete element method mainly due to the granular nature of the experimental testing ground.

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