Numerical analysis of an experimental tunnel

Analyse numérique d'un tunnel expérimental

F. RODRIGUEZ-ROA, Professor of Civil Engineering, Pontificia Universidad Católica de Chile, Chile

SYNOPSIS This paper presents the numerical analysis of an experimental tunnel excavated in the "blue marls" of Sevilla, Spain, as part of the works undertaken in the year 1975 for the construction of the metropolitan railway system for that city. The numerical analysis was developed using a two-dimensional finite element model which can account, among other features, for field anisotropy, the geostatic stress field existing prior to excavation of the tunnel, and the different stages of its construction. A very good degree of approximation was obtained when comparing the numerical values given by the model with the field measurements; such degree of approximation is more remarkable when one considers all the difficulties usually associated with the collection of good quality in situ data in underground works.

INTRODUCTION

Among the works undertaken for construction of the subway system in the city of Sevilla, Spain, it was necessary to carry out a detailed geotechnical engineering study which included the construction of an experimental tunnel bored in the "blue marls" formation, typical of the Guadalquivir river valley. The experimental tunnel was constructed in order to predict the difficulties which could occur during construction of the Metro stations and tunnels.

The experimental tunnel was driven from an existing 25-m-diameter and 35-m-deep shaft excavated for access to a future station (Uriel & Oteo, 1976, 1977, 1979). The 26.1-m-long tunnel was advanced in 2 to 3 m long rounds and lined close to the excavation face with rings of brick masonry. The geometry and dimensions of a cross section are shown in Figure 1.

A program of field instrumentation was developed in order to measure the total pressures of the ground against the lining, to evaluate lining movements and to control the vertical displacement at the top of the marls stratum. (Uriel & Oteo, 1979).

GROUND PROPERTIES

From the geotechnical exploration program the following stratigraphic profile was determined:

"A" Horizon: A 15-m-thick quaternary deposit consisting mainly of dense silty sands with interstratifications of sandy silts and fine sand.

"B" Horizon: An approximately 13-m-thick stratum of very dense quaternary gravels.

"C" Horizon: Made up of a miocenic "blue marls" formation. This is a stratum of blue-gray, medium to highly plastic clay, with frequent horizontal discontinuities resulting from fissures in the marls and from thin layers of sandy silts. This stratum continues to depths of, at least, 65 m.

The water table is located about 4.5 m under the ground surface.

The different horizons of the ground and the location of the tunnel can be seen in Figure 2.

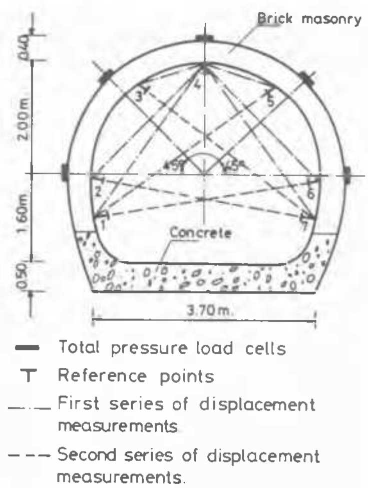

Fig. 1 Experimental Tunnel. Cross Section (Uriel and Oteo, 1979).

MATHEMATICAL MODEL

The two-dimensional plane-strain analysis considers a linear elastic and orthotropic medium. The principal axis of anisotropy are coincident with the vertical (y) and horizontal (x) directions. In practice, when considering a cross section far from the tunnel heading, the hypothesis of plane strain is reasonably real.

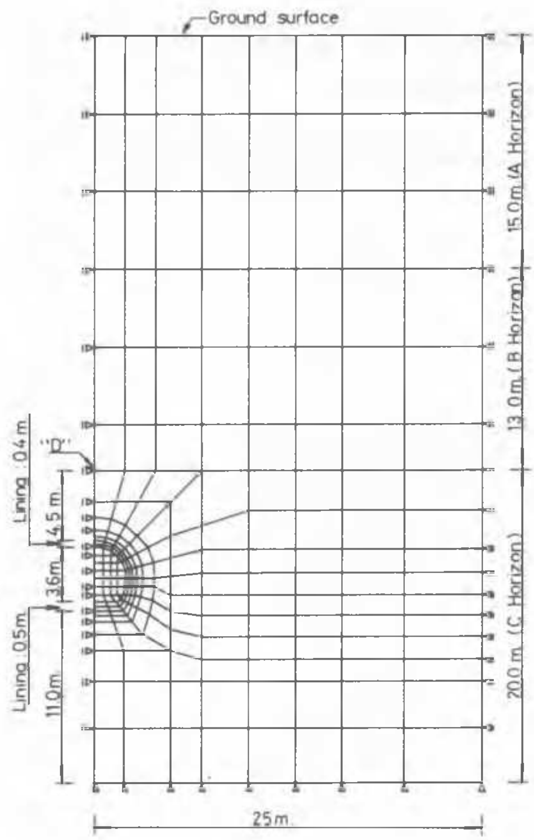

Fig. 2 Finite Element Mesh Used in the Analysis

Mean values of the physical properties representative of the different materials were determined from field and laboratory tests and are shown in Table I.

TABLE I

Material Properties Used for the Analysis *

Material	Young's Modulus E_x (KPa)	E_y (KPa)	Poisson's Ratio (ν)	Unit Weight (KN/m^3)
A Horizon	18,633	18,633	0.30	18.14
B Horizon	25,497	25,497	0.30	20.59
C Horizon	343,232	137,293		20.59
Brick	2,941,995	2,941,995	0.15	17.65
Concrete	9,806,650	9,806,650	0.15	21.57

* Notes:

(i) The upper part of the A horizon was assumed saturated by capillarity.

(ii) The shear modulus "G_{xy}" and Poisson's coefficients "ν_{yx}" and "ν_{xx}" associated to the C horizon were back calculated with the numerical model.

(iii) The lateral earth pressure coefficient, K_0, assumed in the model was the one given by an elastic analysis for a horizontal layered medium (Rodríguez-Roa, 1984).

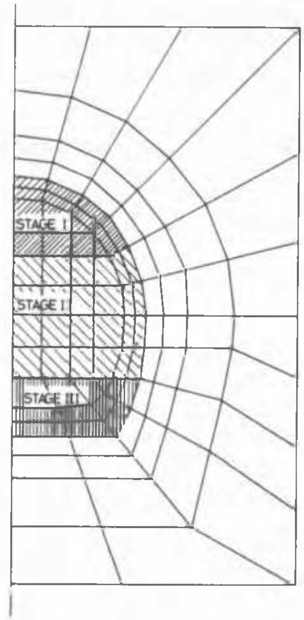

Fig. 3 Construction Stages Considered in the Model

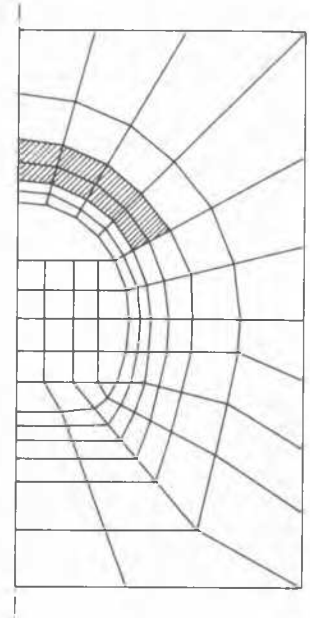

Fig. 4 Distribution of Marls Altered During Stage I

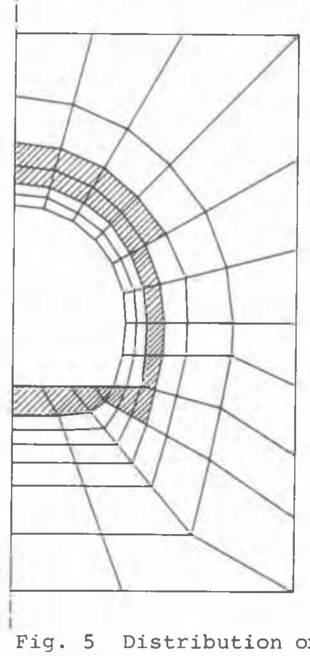

Fig. 5 Distribution of Marls Altered During Stage II

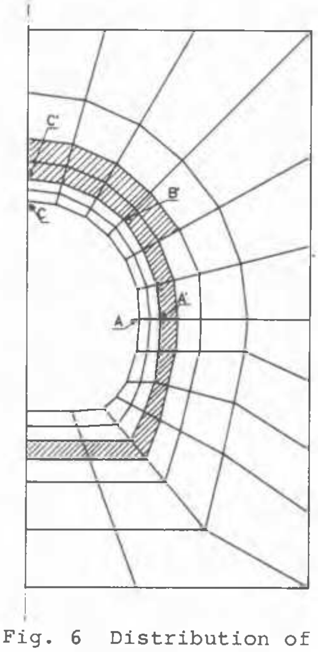

Fig. 6 Distribution of Marls Altered During Stage III

It was also considered that the presence of mortar between the bricks and the marls would produce an infinitely rough interface.

The continuum was discretized by using a mesh made up of 213 isoparametric cuadrilaterals and 3 constant strain triangular elements (see figure 2).

The construction of the tunnel was simulated in a simplified manner, considering 3 stages as shown in figure 3:

STAGE I: Excavation, temporary support and permanent masonry lining of the top heading.

STAGE II: Bench excavation and springline masonry lining.

STAGE III: Invert excavation and concrete lining.

The three-dimensional effect is more pronounced during the first stage because, as each top heading round is excavated, the exposed rock ring is supported not only by the temporary support but also by the lining already in place and by the excavation face. Therefore the hypothesis which is implicit in the simulation of Stage I excavation seems reasonable and quite realistic. Such hypothesis assumes that top heading excavation and lining occur simultaneously.

It is often found in practice that a certain thickness of rock behind the lining experiences a modification from its pre-excavation condition, either as a result of damage produced by the excavation process or due to stress removal. Such changes should be reflected in the model by a modification of the corresponding parameters. The extent and degree of alteration of this zone may be quite variable depending on the ground properties, in situ stresses and construction procedures. For each of the construction stages herein considered, figures 4, 5 and 6 show the development of the altered zone which was assumed in the analysis.

The mean physical properties representative of the altered rock zone were adjusted with the model.

RESULTS

Several cases were studied depending on the values assumed for the unknown in situ material properties. One of the most relevant conclusions of the parametric studies was the high sensitivity of the computed results, particularly of displacements, on the values of the shear modulus and Poisson's coefficient associated to the direction normal to the bedding planes. This high sensitivity was quite favorable for evaluating the magnitude of such variables.

The computed values of contact pressure and lining displacement which were closest to the field measurements were obtained for the following parameters:

a) Marls in their natural state:
E_x = 343,232 KPa; E_y = 137,293 KPa;
G_{xy} = 14,710 KPa; ν_{yx} = 0.15; ν_{xx} = 0.40

b) Marls disturbed by the excavation:
E_x = 98,066 KPa; E_y = 39,227 KPa;
G_{xy} = 7,355 KPa; ν_{yx} = 0.15; ν_{xx} = 0.40

Of particular interest among these parameters are those associated to the marls in their natural state. The low value obtained for the shear modulus must be emphasized. Such low shear modulus is probably a result of the horizontal discontinuities which have been mentioned previously.

The results obtained with the numerical method are compared in Table II to the field measurements collected in an instrumented test section of the tunnel. The location of the points analyzed is shown in Figs. 2 and 6.

TABLE II

COMPARISON OF COMPUTED AND MEASURED VALUES

Point Analyzed	(1)	(2)	(3)	(4)	(5)	(6)
D	-0.015	-0.018				
C	-0.011	-0.007				
C'					481	412
B'					461	412
A			-0.002	-0.006 to -0.020		
A'					314	0 to 59

where:
(1): Computed vertical displacement (m)
(2): Measured vertical displacement (m)
(3): Computed horizontal displacement (m)
(4): Measured horizontal displacement (m)
(5): Computed pressure on the lining (KPa)
(6): Measured pressure on the lining (KPa)

Except for the ground pressure on the lining at the springline, where a significant difference results, the good approximation obtained with the model either in displacement or stress values is really encouraging. However, Uriel and Oteo (1979) show that large variations occurred in the measured values of those springline pressures. Ground pressures exceeding 160 KPa, which are closest to the values obtained with the finite element model, were measured in other test sections of the tunnel. Also it is recognized that a more rigorous modelling of the construction Stage II should have included appropiate interface elements at the base of the lining in order to reproduce the relative movements between the lining and the foundation ground which were observed in the field. Obviously, such procedure would have reduced the computed values of ground pressure at the springline.

Finally, the distribution of internal forces in the lining and interface stresses given by the model at the end of construction are shown in Figures 7 and 8.

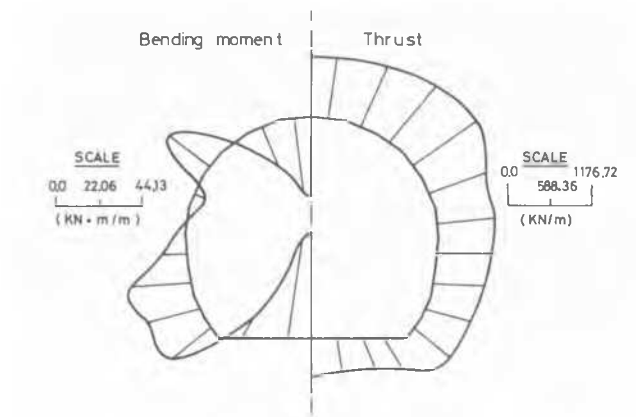

Fig. 7 Lining Thrust and Bending Moment at the End of Construction of the Tunnel.

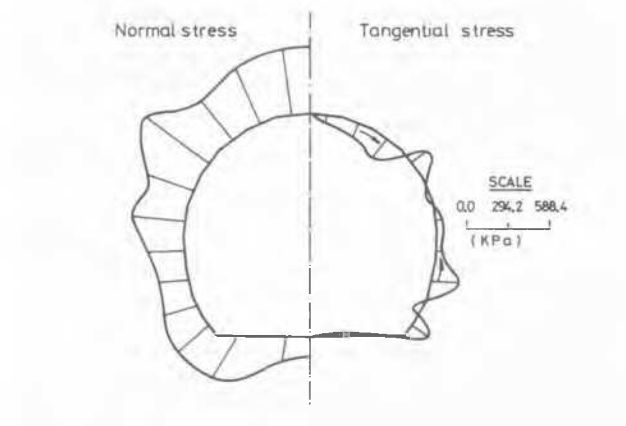

Fig. 8 Contact Stresses over the Lining at the End of Construction.

CONCLUSIONS

The results obtained with a finite element model were compared with field measurements from an experimental tunnel in Sevilla, Spain. The degree of approximation reached was really good.

The model parameters representative of the miocenic blue marls of Sevilla, which could be helpful for future projects located in this geologic formation, are the following:
E_x = 343,232 KPa; E_y = 137,293 KPa;
G_{xy} = 14,710 KPa; ν_{yx} = 0.15; ν_{xx} = 0.40; where "x" and "y" are the horizontal and vertical directions.

The low value adjusted by the model for the shear modulus associated to the direction normal to the bedding planes can be explained by the horizontal discontinuities observed in the marls.

The excellent agreement obtained between the computed results and the field measurements has brought to light the enormous potential of appropiate numerical methods for the design of underground structures.

ACKNOWLEDGMENTS

Financial support for the research summarized in this paper was obtained through grant DIUC 33/82 of the Catholic University, Chile.

The collaboration of Mr. Ricardo Rodríguez in preparing the computer program is greatly appreciated.

The field measurements were obtained by the "Laboratorio de Carreteras y Geotecnia José Luis Escario" of Madrid, Spain, and they were kindly given to the author by Professor Carlos Oteo.

REFERENCES

Rodríguez-Roa, F. (1984). Solicitaciones de diseño en estructuras subterráneas, Escuela de Ingeniería, Universidad Católica, DIE 2/84, pp. 1-203, Santiago, Chile.

Uriel, S., and Oteo, C. (1976). Propiedades geotécnicas de las margas azules de Sevilla. Proc. 1er Simposio Nacional sobre Rocas Blandas, A-9, 1-12, Madrid, Spain.

Uriel, S., and Oteo, C. (1977). Empujes y movimientos en una pantalla circular excavada con lodos bentoníticos, Boletín del Laboratorio del Transporte y Mecánica del Suelo (124), 3-14, Madrid, Spain.

Uriel, S., and Oteo, C. (1979). Measurements in an experimental tunnel bored in the Sevilla Blue Marls and recomendations for the subway design. Proc. 4th Int. Conference on Rock Mechanics, 2, 697-703, Montreux, Switzerland.