Geotechnical Aspects of Underground Construction in Soft Ground, Mair & Taylor (eds)
© 1996 Balkema, Rotterdam. ISBN 90 5410 856 8

Some Spanish experiences on measurement and evaluation of ground displacements around urban tunnels

C.S.Oteo
Polytechnic University of Madrid, Spain

C.Sagaseta
University of Cantabria, Santander, Spain

ABSTRACT: A review is presented of the current developments of evaluation and measurement of ground displacements around urban tunnels. The observations in actual cases are interpreted with simple methods, and the results are used to increase the available experience to be used in future works. Separate consideration is given to the evaluation of the ground loss and to the surface settlement distribution.

Observations are presented from a number of cases. Some details are given about the cases in Madrid.

1. INTRODUCTION

The evaluation of soil displacements due to tunnelling can be based on a variety of sources and methods, such as numerical analysis (F.E.M.), laboratory (centrifuge) tests, and observations in actual cases. All these methods have been used in the past four decades, particularly from the work of Peck and Schmidt (1969).

The main limitation of theoretical (F.E.M.) analyses and laboratory experimentation is the impossibility of adequate consideration of details of construction process. In the early seventies, the efficiency of finite element analyses was enhanced with the simulation of construction phases, by addition and removal of elements. In the eighties, a second major step was the consideration of three-dimensional effects near the tunnel face; 3-D modelling of the excavation process led to simplified methods for stress relief consideration in plane strain analyses (Kielbassa and Duddeck, 1991).

These advances have increased the accuracy of predictions from theoretical analyses. However, the remaining uncertainties have given rise to some methods of analysis, with a variable degree of empiricism, making use of the main results of all the above mentioned sources.

It is usually accepted that the problem can be split into two main parts, corresponding to the evaluation of: i) the amount of soil deformation, defined by the total ground loss at the tunnel or at the soil surface, and ii) the shape of the distribution of soil movements. Both problems will be addressed below separately.

2. GROUND LOSS

2.1 Ground loss vs. settlement volume

The absolute value of soil displacements is governed by the so-called ground loss, defined as the volume of soil enclosed between the final and initial (undeformed) positions of the tunnel wall. The term 'relative ground loss', v_s (%), refers to this volume expressed as a percentage of the excavated cross-sectional area.

Typical values of the ground loss for normal conditions are in the range 0.5-2% for stiff soils, increasing up to about 5% in soft soils. Obviously, higher values have been measured under difficult conditions.

The measurement of the ground loss in actual tunnels is a very difficult task, because a good part of the deformation takes place ahead of the tunnel face. So, the measuring devices inside the soil mass must be installed from the surface. They must also be close to the tunnel periphery, and so they are easily damaged by tunnel excavation. The points located below the tunnel invert are particularly difficult, because any instrument installed from the surface will be cut by the tunnel.

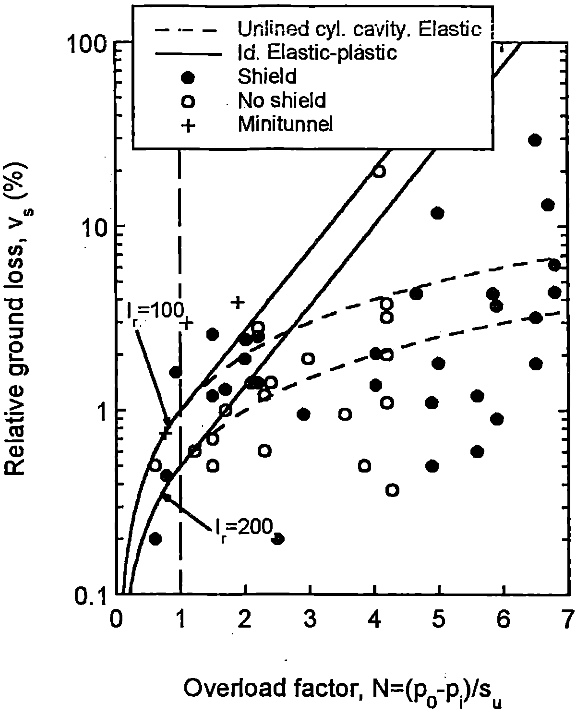

Fig. 1. Ground loss vs. overload factor (based on Clough and Schmidt, 1981)

As a consequence, most references to the measured value of the ground loss are not taken at the tunnel itself, but at the soil surface, corresponding to the volume of settlements, i.e., the area enclosed by the transverse profile of surface settlements. Under the assumption of undrained deformation, both quantities would be identical, but in actual cases some volume change takes place, and the settlement volume is only a fraction of the ground loss at the tunnel. Cording and Hansmire (1976) report values about 70% for this fraction in some carefully instrumented sections of Washington Metro. Similar values were obtained by Sagaseta and Oteo (1974) in elastic finite element analyses of unlined tunnels for H/D in the range 1-8, for Poisson ratios of 0.1 to 0.4, increasing to unity for incompressible soil (μ=0.5).

2.2 Evaluation

Given the limitations of theoretical analyses commented on the preceding section, some simplified approaches were necessary. The authors' approach (Sagaseta and Oteo, 1974; Oteo and Moya, 1979) was to evaluate the ground loss in each case by considering the tunnel unlined, and the soil in elastic condition. Then, a reduction factor, Ψ, was applied to the soil displacements. This reduction factor was obtained by matching the results of observations in actual cases.

This method gave reasonably good results in stiff soils. The reduction factor, Ψ, varies with the lining stiffness, as well as advance rate, time for lining setup. etc. For good tunnel conditions, with a high degree of deformation control (high advance rate, shield tunnelling, or early placement of support, closed invert, etc.), Ψ is of about 0.4-0.5. For closed shields, with immediate tail grouting, the value can be of 0.2-0.3. On the other hand, for less careful construction, or in the case of stops in the excavation, Ψ can reach a value of 1,0.

From the above approach, the assumption of linear elasticity in the analysis requires further justification. The basic idea is that in soft soils, the elastic limit is soon reached, and plastic deformation takes place. This deformation would be very large, but it is limited by the lining. At the end, the pressure on the lining will keep the soil, if not in elastic state, with limited plastic strains. For an approximate evaluation, it can be reasonably accurate to assume elastic behaviour.

This can be easily shown by a simple analysis using the overload factor (Peck, 1969):

$$N = \frac{p_0 - p_i}{s_u} \qquad (1)$$

where p_0 is the total overburden pressure at the tunnel axis level and p_i the inner tunnel pressure, when it exists. Fig. 1, redrawn from Clough and Schmidt (1981) with some additions, shows some measured values of ground loss in clays, correlated with N. In the same figure, the results of unlined cylindrical cavity contraction are shown. The rigidity index, I_r=G/s_u, is taken as 100 and 200, covering the range for actual soils. The analysis is performed assuming pure elasticity (dashed lines) and fully elastic-plastic behaviour. Some interesting trends can be observed:

- Ground loss increases with overload factor.
- There is no significant difference between shield and no-shield tunnels. However, only shield tunnelling appear to be feasible for N>4.
- For overload factors lower than about three, the measured ground loss agrees well with the elastic-plastic analysis of unlined cylindrical cavity. For higher N values, this analysis is an upper bound.
- For all the range of overload factors, the elastic solution seems to reproduce reasonably well the resulting values of the ground loss.

With this basis, Sagaseta and Oteo (1974) performed parametric analyses with finite elements for elastic soil and unlined tunnels. The resulting maximum surface settlement was roughly independent of the tunnel depth, and depends only on the soil elastic parameters and the tunnel diameter:

$$s_{max} = \Psi \cdot \frac{\gamma D^2}{E} \cdot (0.85 - \mu) \qquad (2)$$

3. TRANSVERSE SETTLEMENT PROFILE

3.1 Error curve

Once the ground loss is determined, the second step is the evaluation of the lateral distribution of settlements. For this purpose, the use of the error curve, proposed by Peck (1969) and Schmidt (1969), has become the most efficient tool. This curve is:

$$s(x) = s_{max} \cdot e^{-\frac{x^2}{2.i^2}} \qquad (3)$$

where s(x) is the settlement at a distance "x" from the centerline, s_{max} is the maximum (centerline) settlement, and "i" is the abscissa of the point of inflection of the curve. At this inflection point, the settlement is $0.61.s_{max}$.

Another property of eq. (3) is that the area enclosed by the curve (settlement volume) is:

$$V_s = \sqrt{2\pi} \cdot i \cdot s_{max} \cong 2.5 \cdot i \cdot s_{max} \qquad (4)$$

If the ground loss is already known, this means a relationship between the two parameters, "i" and s_{max}, defining the curve.

The abscissa of the inflection point, "i", was determined by Peck (1969) in a number of cases. The results were plotted against the tunnel depth, H, with both "i" and H scaled by the tunnel diameter, D (or radius, R). The selection of these variables reflects the implicit assumption that the tunnel depth was the main factor controlling the lateral spreading of the settlements. The points corresponding to the actual tunnel showed a large scatter, and a set of lines were plotted separating zones for different soil conditions.

During the past 25 years, considerable attention has been paid to further refinements of the prediction of the value of "i". In particular, the initial points were all for H/D<4. When new points for deeper tunnels have been added to the plot, the division in zones for different soil types is not clear. All the points fall in a band, with some tendency to have larger values of i/H for clays than for sandy soils.

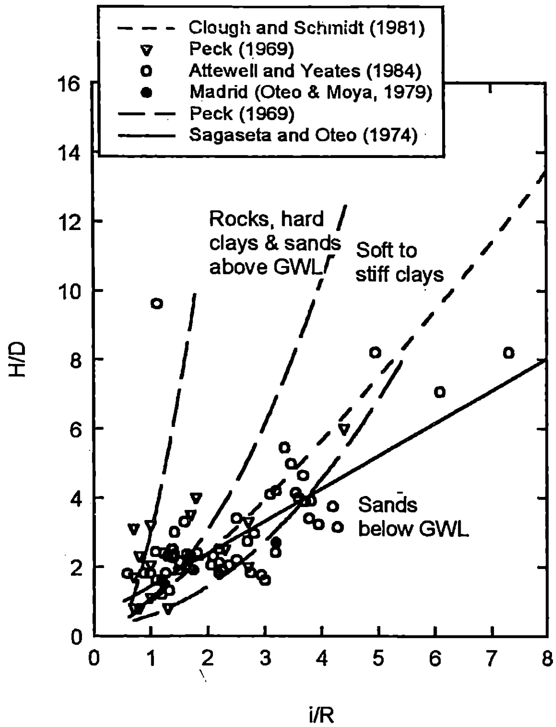

Fig. 2. Error curve. Location of inflection point

Fig. 2 (Oteo and Sagaseta, 1982) shows the initial suggested zones for soil types, together with new cases, and the results of elastic finite element analyses for unlined tunnel, as commented above. These analyses resulted in a unique relationship between i/R and H/D, with a very small influence of the Poisson ratio:

$$\frac{i}{R} = 1.05\frac{H}{D} - 0.42 \qquad (5)$$

Clough and Schmidt (1981) have proposed another expression:

$$\frac{i}{D} = 0.5 \cdot \left(\frac{H}{D}\right)^{0.8} \qquad (6)$$

For the usual range of H/D, these relationships give similar values of i/H, between 0.35 and 0.50.

3.2 Theoretical approach

The use of the error curve is based on its capability for reproducing the actual patterns of settlement profiles, but there is no theoretical reason demonstrating that settlement distribution are governed by such an expression.

Sagaseta (1987) has presented a theoretical analysis of soil deformation due to ground loss. The approach is based on solutions for incompressible irrotational fluid flow. This method has given very good results for analysis of deep penetration problems (Baligh, 1985). By combining fluid flow with elastic solutions for the half space, Sagaseta (1987) proves that in the case of a loss of ground at a point inside the soil, the displacements at the surface are twice the displacements that would occur if the sink (ground loss) was in an infinite space, with no free surface. The case of an infinite space is trivial, because the conditions of incompressibility and spherical symmetry about the sink, determine a radial field of displacements.

As a result, the settlements at the soil surface are:

$$s(x) = s_{max} \cdot \frac{1}{1+\left(\frac{x}{H}\right)^2} \tag{7}$$

Equation (7) predicts a settlement profile having the correct shape, with a maximum for x=0 and an inflection point, and tending to zero at the infinite. However, the lateral spreading is much larger than observed, with an inflection point (s/s_{max} =0.61) at i/H=0.8, which is about twice the observed values. The main reason for this discrepancy is the assumption of incompressibility. Sagaseta (1988) suggests a possible way to incorporate soil volumetric strains, based on the solutions for cylindrical cavities in dilatant soils. The corrected expression for the settlements is:

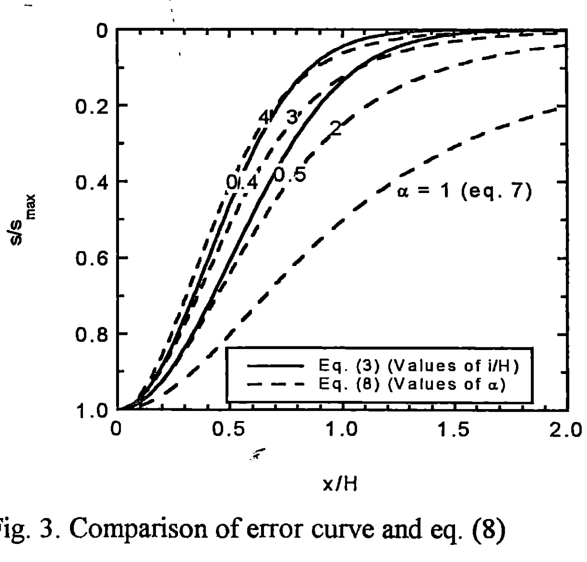

Fig. 3. Comparison of error curve and eq. (8)

$$s(x) = s_{max} \cdot \frac{1}{\left[1+\left(\frac{x}{H}\right)^2\right]^{\alpha}} \tag{8}$$

where the exponent α incorporates the volumetric strain effects (α>1 for dilatant soils and α<1 for contractive soils). The capability of eq (8) for reproducing realistic settlement profiles can be assessed by the abscissa "i" of the point with $s=0.61.s_{max}$ (inflection of the error curve):

$$\frac{i}{H} = \sqrt{\frac{1}{(0.61)^{1/\alpha}} - 1} \tag{9}$$

For α= 2-3, i/H results in the usual range 0.4-0.5. In fig. 3 a comparison of eqs (3) and (8) is presented. With the appropriate values of the parameters, they give practically identical results.

4. HORIZONTAL DISPLACEMENTS

The horizontal displacements are less frequently controlled in tunnel construction. At the surface, they are of the order of 1/2 to 1/3 of the settlements. Again from finite element analyses, Sagaseta and Oteo (1974) proposed a ratio of 0.3 between the maximum horizontal displacement, u_{max}, and the maximum settlement, s_{max}.

The horizontal displacements are zero at the centerline and reach a maximum at a point located near the inflection of the settlement profile. In fig. 4 a simplified scheme is presented, commonly used to predict the extent of potential damage.

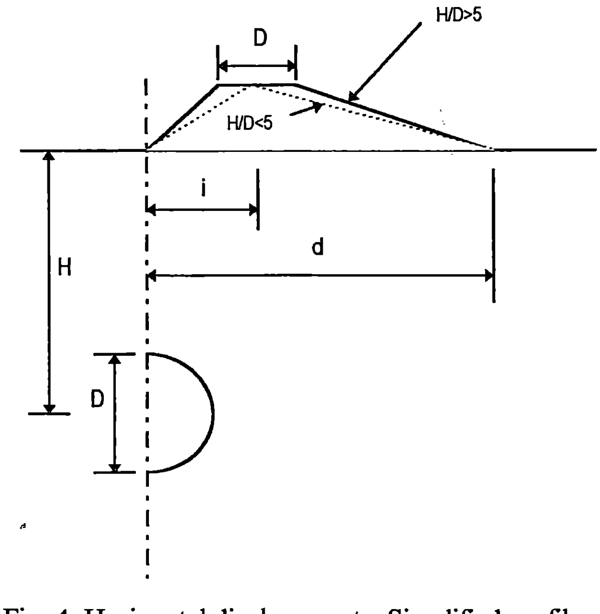

Fig. 4. Horizontal displacements. Simplified profile

Some authors (Attewell and Yeates, 1984) have postulated that the horizontal displacement is so that the displacement vector is addressed to the tunnel axis. It is worth noting that in the theoretical solution of Sagaseta (1987), this condition holds exactly for the points at the soil surface. As a consequence, it can be applied to eqs (8) or eq. (3) to give a complete description of the surface movements.

5. EXPERIENCES IN MADRID

5.1 Ground conditions

Most of the urban area of Madrid is settled on Tertiary (Pliocene) deposits, with a total thickness of about 200 m., formed from erosion of the nearby granitic mountains. The following units are defined, according to the fines content (<#200 ASTM):

- *Arenas de miga*, clayey sands, with fines content less than 15%
- *Arenas tosquizas*, with fines from 15 to 40%.
- *Tosco arenoso*, sandy clays, with 40-60% of fines.
- *Tosco*, silty clays, with 60-80% of fines.
- *Tosco arcilloso*, clays, more than 80% of fines.

These materials are stiff, with unconfined compressive strength increasing with the fines content, up to 1.5 to 2.0 MPa (fig. 5). These strengths mean that these materials can be classified as "indurated soils". The deformation moduli (fig. 6), determined with pressuremeter, are also very high, up to 1300 MPa. The soil surface has a gentle and uniform slope towards the South, and in the Southern part of the city, the underlying Miocenic materials appear at the surface. They are stiff fissured gray or green marly clays, locally known as *peñuelas*. In some parts, they have a significant sulfate content (gypsum). The unconfined compressive strength is generally higher than 2 MPa, increasing with gypsum content.

5.2 Subsidence analysis

The main issues for tunnelling in Madrid have been in relation to the extensions of the Metro. In the sixties and seventies, several new lines (lines VI, IX) were constructed and others were prolonged.

Fig. 7 shows the reduction factor, Ψ, for the settlements (cf. Sec. 2.2), obtained as a function of the tunnel advance rate. Some measurements in Caracas Metro have also been included. All the cases correspond to a tunnel diameter of about 9 m. and depth to diameter ratio, H/D, of 2 to 3.

Referring the transverse profiles of settlements and horizontal displacements, fig. 8 shows the values of the abscissa "i" of the maximum displacement, and of the lateral extent of the horizontal displacements, "d" (cf. fig. 4). As can be seen, the ratio i/H is of about 0.5, and d/i about 3.0.

More recently, two parallel tunnels, 20 m. wide, 30 m. deep, and separated 40 m. between axes, have been constructed in motorway M-40 in Madrid,

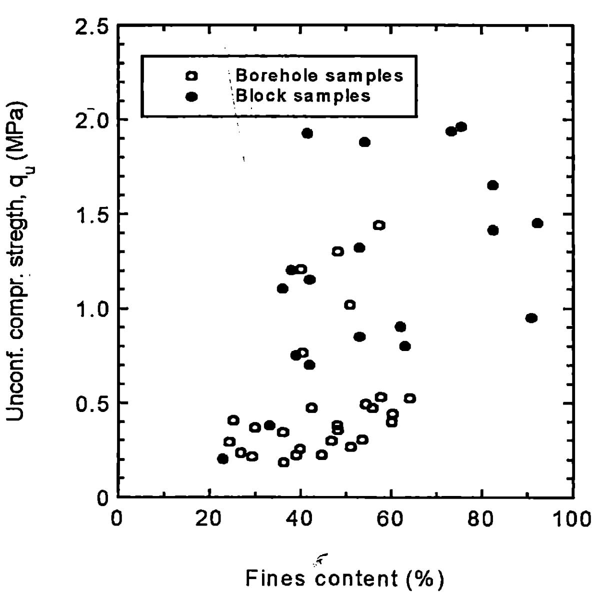

Fig. 5. Madrid. Upper Pliocene deposits. Unconfined compressive strength

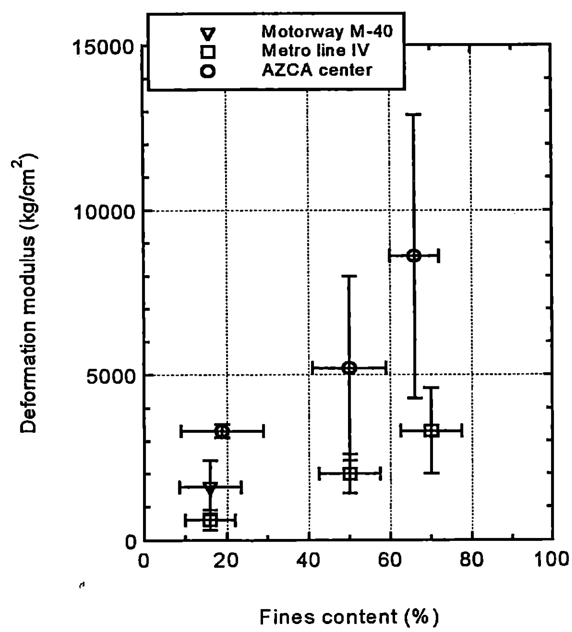

Fig. 6. Madrid upper Pliocene deposits. Deformation moduli.

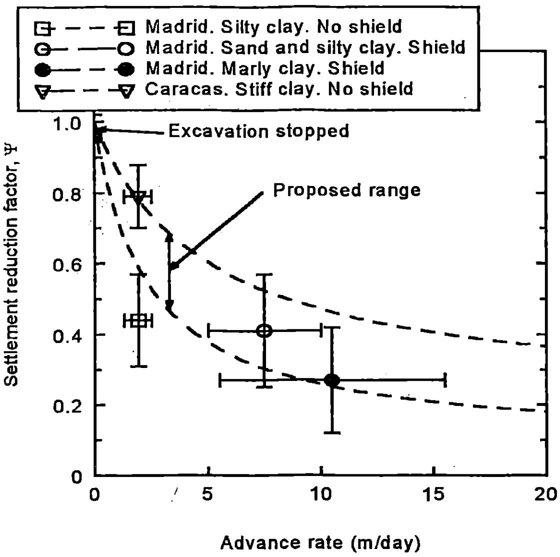

Fig. 7. Subsidence reduction factor, Ψ

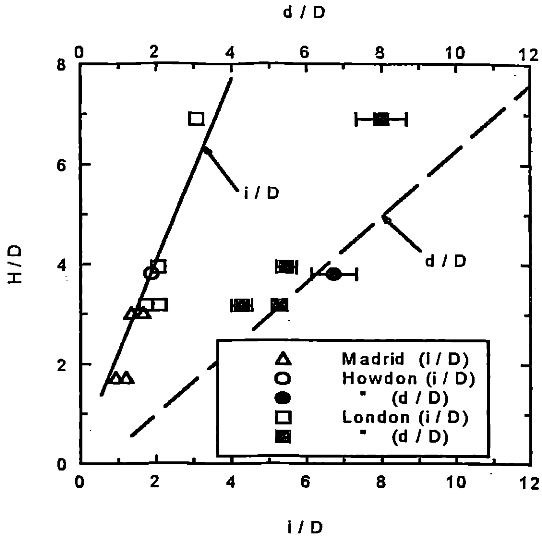

Fig. 8. Measurements of the extent of horizontal displacements.

through the Upper Pliocene deposits (*arenas de miga* and *arenas tosquizas)*. Each tunnel was constructed in stages: first, two side drifts were excavated and shotcreted, and then the vault was constructed with the aid of mechanical precutting ("premill"). The maximum settlements were 8 mm above the first tunnel and 13-15 mm. above the second tunnel, with a relative ground loss of 0.75%.

The first tunnel was analyzed using eq (2), with a Young modulus of 230 MPa and a Poisson ratio of 0.2. The reduction factor, Ψ, was taken as 0.6-0.7 for the side drifts and 0.35-0.40 for the vault. The resulting maximum settlement is 8.5 mm. For the second tunnel, the same parameters were taken. The interaction between parallel tunnels was considered with the coefficients derived by Ghaboussi et al. (1978). The calculated maximum settlement was 14.5 mm., in accordance with the measurements.

6. REFERENCES

Attewell, P.B. and J. Yeates 1984. Tunnelling in soil. In *'Ground movements and their effects on structures'* (P.B. Attewell and R.K. Taylor, eds.) 132-215. Surrey Univ. Press.

Baligh, M.M. 1985. Strain Path Method. ASCE J. Geot. Engg. Div. **111**:GT9,1108-1136.

Clough, G.W. and B. Schmidt 1981. Design and performance of excavations and tunnels in soft clay. In *'Soft Clay Engineering'* (P. Brand and P. Brenner, eds.), 569-634. Elsevier, Amsterdam.

Cording, E.J and W.H. Hansmire 1976. Displacements around soft ground tunnels. Gen rep. Session VI: Tunnels in soils. Proc. 5th. PACSMFE. Buenos Aires. **4**,571-633.

Ghaboussi, J., R.E. Ranken and M. Karshenas 1978. Analysis of subsidence over soft ground tunnels. Proc. Int. Conf. Subsidence. ASCE. Flo, 182-196.

Kielbassa, S. and H. Duddeck 1991. Stress-strain fields at the tunnelling face. Three-dimensional analysis for two-dimensional technical approach. Rock Mech. and Rock Engg **24**,115-132.

Oteo, C.S. and J.F. Moya 1979. Estimation of the soil parameters of Madrid in relation to the tunnel construction. Proc. 7th. ECSMFE. Brighton, U.K. **3**,239-247.

Oteo, C.S. and C. Sagaseta 1982. Prediction of settlements due to underground openings. Numer. Models in Geomech. (R. Dungar, G.N. Pande and J.A. Studer, eds.). Zurich, Switzerland. 653-659.

Peck, R.B. 1969. Deep excavations and tunnelling in soft ground.. Proc. 7th ICSMFE. Mexico. State-of-the-Art Volume, 225-290.

Sagaseta, C. 1987. Analysis of undrained soil deformation due to ground loss. Géotechnique **37**:3,301-320.

Sagaseta, C. 1988. Author's reply. Géotechnique **38**:4,647-649.

Sagaseta, C. and Oteo, C.S. 1974. Análisis teórico de la subsidencia originada por la excavación de túneles. Simp. Nal. Túneles. Madrid. **1**,II-10.

Schmidt, B. 1969. Settlements and ground movements associated with tunnelling in soil. Ph.D. Thesis. Univ. of Illinois.