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The Excavation of a Shallow Tunnel Below a Commercial Centre

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ABSTRACT:
The excavation of a shallow tunnel passing below a commercial centre is described. At design stage, a numerical estimation of induced movements was performed and criteria for allowable movements were set. During construction, induced subsidence was monitored and compared to forecast values.

1 PROJECT DESCRIPTION

The excavation of a single track horse-shoe section type shallow tunnel (radius of crown R=3.05m, sidewall height h=3.8m) passing below a big commercial centre has recently been completed in Pontevedra (NW Spain, 2001). The total length of the tunnel is 776m, of which 124m were below the commercial centre. The tunnel will be part of the new railway line connecting Marin's Harbour to Pontevedra's city centre, financed by the Regional Authority Xunta de Galicia.

In the 1920s, the construction of the tunnel had been attempted without success, and had been abbandoned due to a collapse that took place after barely fifty metres. The project was forgotten and the portals became no longer visible. In the 1980s a commercial centre was built, laying partially on top of the abbandoned tunnel.

Upon request of the owner of the commercial centre, TYPSA was asked to study the project and make the necessary adjustments to the construction technique in order to guarantee the safety of the excavation below it.

The aim of the new project was to reopen the portals, and to restart excavation from the collapse point, after all necessary reinforcement works of the old tunnel had been completed. However, it was soon found out that both tunnels were slightly eccentric, so that a decision was taken to fill the abbandoned tunnel with low quality concrete.

Figure 1 shows a schematic view of the project. The new tunnel crosses below the commercial centre along a diagonal axis, and the abbandoned gallery is slightly eccentric with respect to the new tunnel. Overburden with respect to the crown of the tunnel is as low as 12.5m.

The commercial centre is a two floor high reinforced concrete (RC) building showing great isostaticity, made of prefabricated elements. Foundation is achieved through cast-in concrete footings, with very little reinforcement. Unloading of goods takes place through a platform that lies between columns no.4 and no.5 of pillars. The actual building façade corresponds to column no.5, and rows N and O. Spacing between aligned pillars is 7.50m. It was decided to reinforce the footings of the pillars before starting excavation (Fig.2).

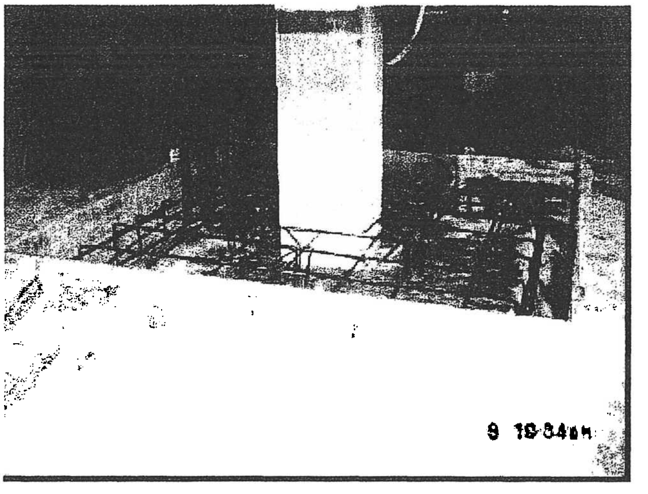

Fig. 2 Reinforcement of footings

2 GROUND CONDITIONS

The project area belongs to the Herzinian Granitic Rock Mass, and is characterised by the coexistence of weathered granite and gneiss and sound to very sound rock near the surface. Weathering is the results of high pluviometry.

In the project area, the average expected depth of highly weathered rock can roughly be estimated in 15m to 20m. In order to confirm this assumption, a complementary geotechnical study was carried out, which led to the geotechnical cut shown in Fig.3. Since overburden was almost constant, behaviour classes responded to the classification proposed by ISRM (1977), which is based in fact on the degree of weathering of the rock mass, with its lower bound set to class V. Water was found in

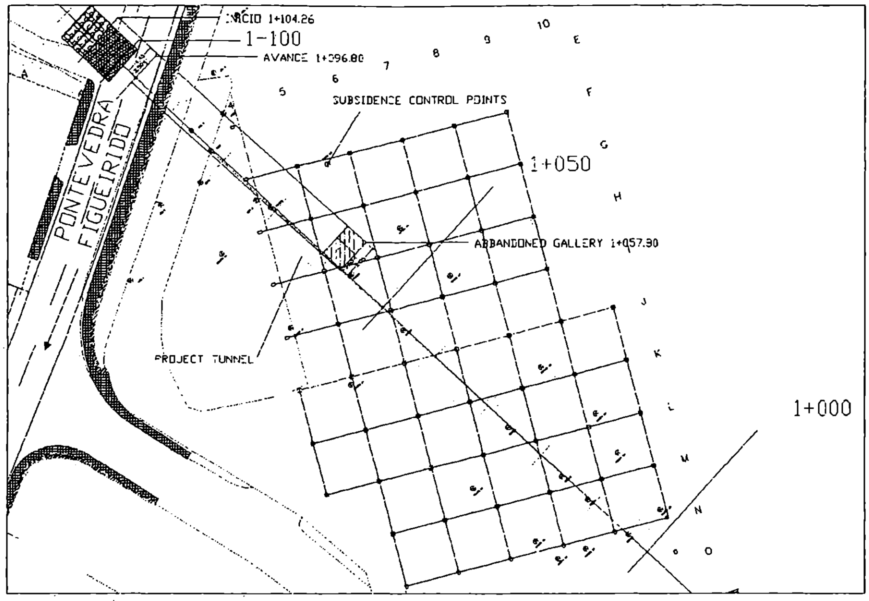

Fig. 1 Schematic view of the project

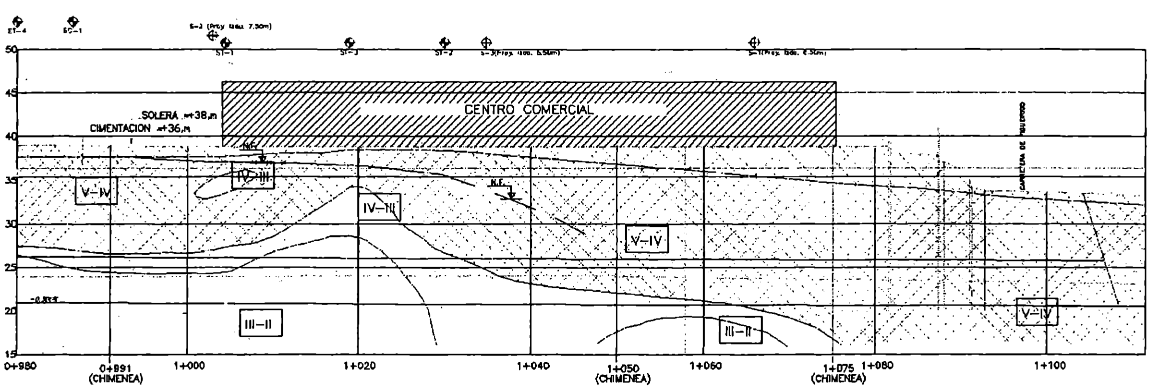

Fig.3 Longitudinal geotechnical profile

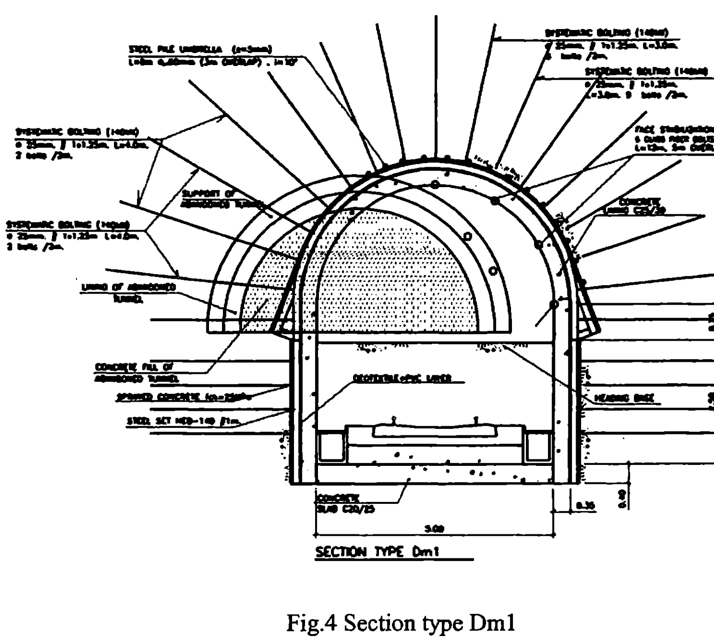

Fig.4 Section type Dm1

the boreholes at a depth of 2m-3m below the surface. During excavation it drained out rapidly and posed no problem.

3 SUPPORT SECTION TYPES

Two section types were defined to excavate below the commercial centre.

Section type Dm2, to be excavated in class IV-V with mechanical means (roadheader), in two stages (heading and bench). Section type is as follows:

Table 1. Section type Dm2.

Face & head protection	8m long steel tubes with 3m overlap (Fe510, Øp800mm, Øext600mm, Øint600mm). Eventual fibre-glass dowels at the face when needed.
Excavation support	Steel arches every 1.00m (CA37b HEB-140)
	19cm of steel fibre-reinforced sprayed concrete (fck_{28days}=25MPa, fck_{1day}=10MPa)
	8-9 Fe510 Ø25mm bolts L=2.5m every 1.00m in advance, plus 4 additional bolts for bench support.
Final lining	35cm of cast-in C25/30 concrete, and 40cm of cast-in C20/25 concrete at the bench.

Where the abbandoned tunnel ran parallel to the new one, bolting length was increased to 4m-5m to prevent the falling of blocks coming from the lining or the concrete filling. The so modified section was called "section type Dm1" (see Fig.4).

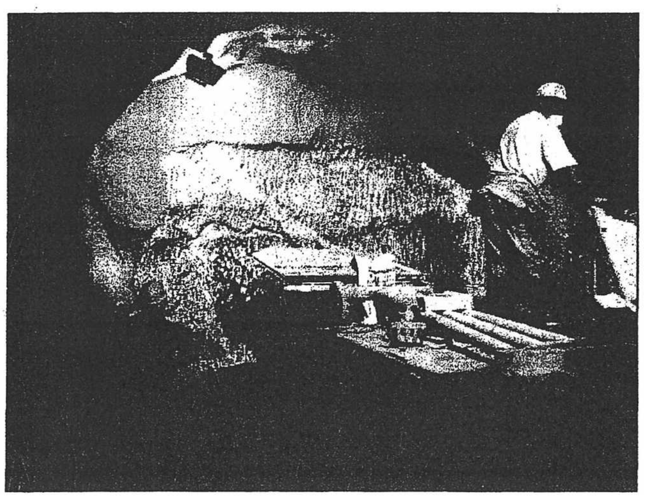

Fig.5 Roadheader at work.

The initial geotechnical study did not foresee the presence of gneiss III-II at the bench. After some studies, it was decided to excavate the bench with explosives, limiting the maximum load per hole, in order to keep vibration felt in the commercial centre below the group II vibration threshold imposed by UNE-22-381-93.

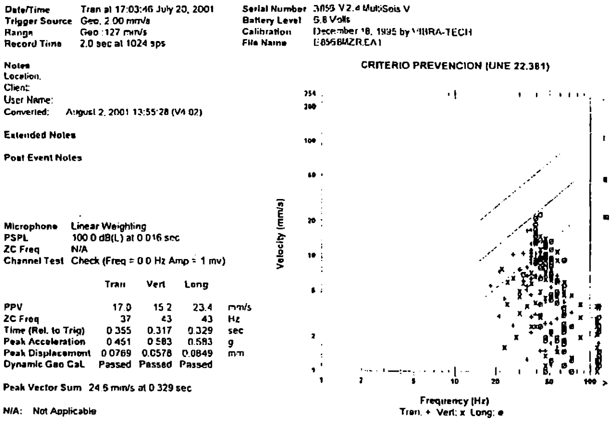

Fig.6 UNE 22-381-93. Damage criteria prevention against vibration induced by blasting.

4 GROUND MOVEMENT CONTROL

In order to measure movements induced by the excavation of the tunnel, a settlement control system was implemented. A permanently based-on-site topographic team was in charge of monitoring movements at the pillars' base. Settlement troughs at specific cross-sections were also monitored during construction. Both activities were performed on a daily basis. Tunnel convergences were also monitored.

The analysis of monitored movements against admissible values allowed the implementation of several corrective measures (gravity injections of mortar when cavities were suspected), tunnel support section type reinforcement, TAM *"tubes-à-manchettes"* installation for eventual water-cement-sand injections under pressure, and for the assessment of stability.

According to Bjerrum (1963), angular distortion is the governing variable in the assessment of permissible deformations (Table 2).

Table 2. Damage criteria based on angular distortion

Angular distortion	Damage assessment
1/100	Limit where structural damage is to be feared. Safe limit for flexible brick walls with h/L<0.25. Considerable cracking in panel walls and brick walls.
1/250	Limit where tilting of high rigid buildings may become visible.
1/300	Limit where difficulties with overhead cranes can be expected.
1/500	Safe limit for buildings where cracking is not permissible.
1/600	Danger limit for frames with diagonals.
1/750	Lower limit for sensitive machinery.

At the design level, though, in order to take into account a factor of safety, criteria on additional allowable movements adapted from those used for Madrid's Metro Extension (1995-1999). These criteria assign maximum additional allowable movements (settlement, horizontal displacement & angular distorsion) according to the conservation and characteristics of the element to be protected against subsidence induced by the excavation of a shallow tunnel.

Table 3. Additional allowable movements for building and structures showing a high isostaticity degree

Settlement [mm]	GREEN	<15
	YELLOW	15 to 25
	RED	>25
Angular distorion [m/m]	GREEN	<1/1000
	YELLOW	1/1000 to 1/500
	RED	>1/500
Horizontal deformation [%]	GREEN	<0.15
	YELLOW	0.15 to 0.20
	RED	>0.20

The aim of the above table is to set a first control level during monitoring of subsidence. It must be reminded that speed of variation of a given variable should be interpreted as an indicator of achieved stabilization.

The following variables were monitored: vertical settlement of the surface, movement at the pillars (base and middle-height) and tunnel convergence

During excavation of the tunnel, all pillars and pins standing within an area 20m long and 60m wide strip ahead of the face were monitored. Displacements were read twice a day.

A decision procedure was set as follows, on the basis of the value actually monitored.

Table 4. Actions to be taken according to the control level.

LEVEL	ACTION
GREEN	Keep on monitoring indicated variables. Keep on with construction as planned.
YELLOW	Increment frequency of monitoring, and a visual inspection of the affected elements. Keep on with construction as planned.
RED	Study the case in detail. Install complementary instrumentation if necessary. Revise the construction procedure, and modify it if necessary. Evaluate the needs for the implementation of corrective measures or structural reinforcement.

5 MONITORING RESULTS

Initially, a complete underpinning of the structure was designed, in order to transfer building loads below the tunnel's level. However, during drilling, the abbandoned gallery was intercepted, and it was discovered that it was slightly eccentric with respect to the new tunnel. After filling the abbandoned gallery with low quality concrete, it was decided not to continue with the underpinning. Instead, in order to be able to respond to excessive subsidence, a series of TAM (*tubes à manchettes*) was installed at the base of the pillars (4 per pillar) located above the tunnel, and a water-sand-cement mixture was injected to fill eventual remnant voids.

Vertical settlement of each pillar was plotted versus distance to the face. As a rule of thumb, it can be confirmed that within normal conditions, settlement as the face passes through the plane normal to the axis that contains the pillar represents around 20% to 30% of final settlement (Fig.7).

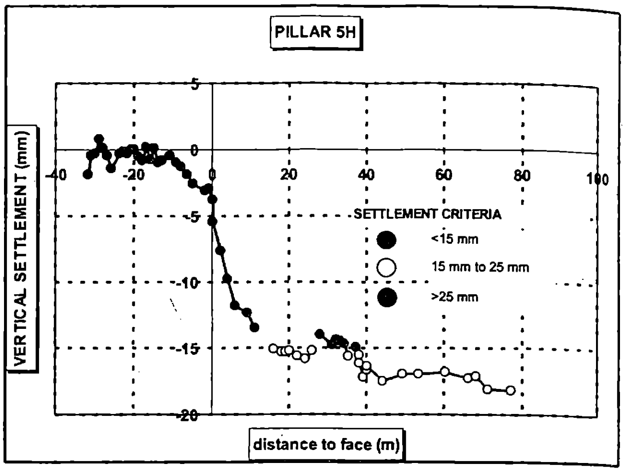

Fig.7 Settlement at the base of pillar 5H vs. distance to the face.

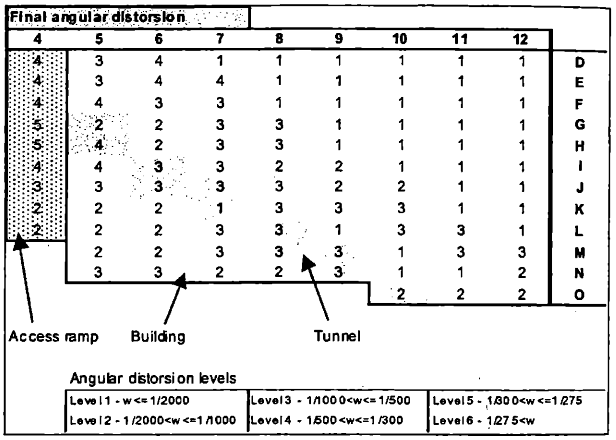

Fig.8 Final angular distortion at the end of excavation of the tunnel.

Levels of final angular distortion (w) are shown on Fig.8. The level assigned to each pillar corresponds to the worst possible value of distortion calculated for adjacent pillars. Values corresponding to level 5 were only detected in the access ramp, where there was no element sensible to cracking.

Some damage was observed at the floor of the toilets (M7, M8), which could be easily repaired, and some cracks appeared at an internal brick wall of the cafeteria's kitchen (next to the toilets).

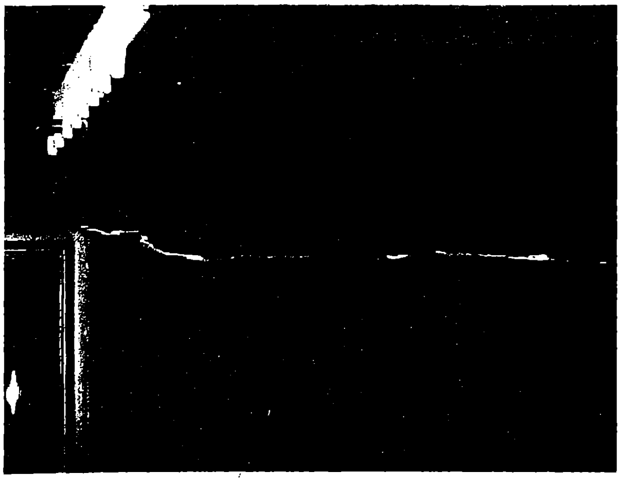

Fig.9 Damage observed in the wall of the cafeteria's kitchen (M7, M8).

6 SUBSIDENCE ANALYSIS

The following Möhr-Coulomb ground parameters were adopted based on triaxial laboratory tests on recompacted disturbed samples.

Parameters were adopted so that the analysis could represent an unfavourable scenario (grade V-IV rock for the entire excavation). The presence of the commercial centre was introduced applying an equivalent vertical pressure.

Table 5. Ground parameters for subsidence estimation.

Ground modulus [MPa]	Poisson's coefficient [-]	effective cohesion [MPa]	effective friction angle [°]	peak shear deformation [%]	natural density [kN/m3]	natural wetness [%]
20+z	0.35	0.03	35	2.0	21	17

Subsidence was analysed with both numerical (FLAC2D v3.40) and empirical methods.

6.1 Empirical estimation of subsidence envelope

According to Peck (1969), the settlement trough can be adjusted to the following gaussian curve:

$$\delta_x = \delta_{\text{máx}} \cdot e^{\left(\frac{-x^2}{2 \cdot i^2}\right)} \tag{1}$$

where the maximum settlement δ_{max} is related to volume loss Vs and the position of the inflexion point through the following expression:

$$\delta_{\text{máx}} = \frac{V_s}{2.5 \cdot i} \tag{2}$$

Oteo & Sagaseta (1982) suggest the following expression for the ratio i/R:

$$\frac{i}{R} = \eta \cdot \left(1.05 \cdot \frac{H}{D} - 0.42\right) \tag{3}$$

where η is a function of the constructive procedure and in-situ ground conditions, and may vary between 0.70 and 1.30; R is the excavation radius; H the overburden at axis level and D the excavation diameter.

The elastic volume loss may be calculated (Deere et al, 1969) with the following expression:

$$\Delta V_e = (1 + Ko) \cdot \sigma' \cdot \frac{(1+\nu)}{E} \cdot V_o \tag{4}$$

where E is the elastic modulus of the ground, Ko the pressure coefficient at rest, σ' the effective stress at the tunnel's axis level, ν Poisson's coefficient and Vo the excavated volume.

The amount of volume loss at tunnel level is usually less than actual volume loss at ground level, so that volume loss that may cause subsidence can be roughly estimated as:

$$V_s = 0.70 \cdot \Delta V_e \tag{5}$$

6.2 Numerical estimation of subsidence envelope

It is a well-known fact that the excavation of a tunnel induces a modification of the ground stress-state and the development of movements ahead of the face. The process is 3D, but it is possible to model it with a 2D code by introducing a fictitious pressure around the excavation perimeter in order to take into account the support effect of the face.

A relaxation coefficient of 72% was adopted at the advance face, (i.e. fictitious pressures at the front equal 72% of initial field pressure), and an additional 10% of relaxation was allowed before support installation, to simulate the 1.0m excavation pass. Steel sets and sprayed concrete were modelled with equivalent beam elements. Since deformation is a phenomenon that takes place during the first stages of excavation, a low constant Young modulus of 10GPa was considered for fresh sprayed concrete.

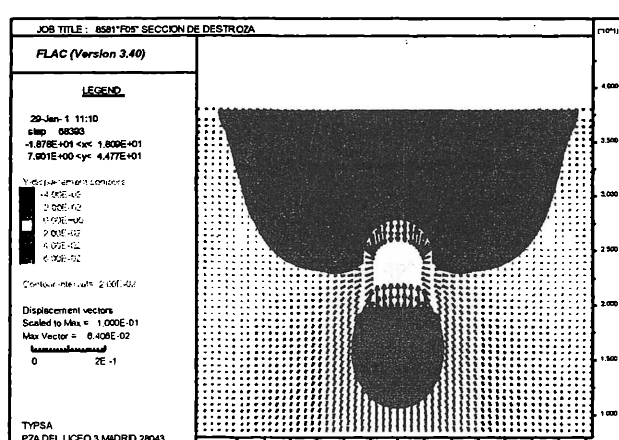

Fig.10 Vertical calculated displacement [m] at the end of tunnel construction.

6.3 Summary of results

Fig. 11 shows the results of settlement estimation and the actual measured values of settlement at the end of construction.

It can be seen that the previous subsidence estimation results define an envelope of actual measured settlements, being the difference due to the fact that ground conditions were on average better than those initially considered.

7 CONCLUSIONS AND RECOMMENDATIONS

Within normal circumstances, the estimation of an envelope of subsidence can be performed with either empirical or numerical procedures. However, it is difficult to predict actual subsidence related to face failure or when the ground shows high geotechnical variability.

Settlement gives an intuitive "feeling" of the level of movement actually taking place as a consequence of tunnel excavation, but angular distortion seems to be the controlling variable when assessing eventual damage on nearby elements or structures.

At design level, maximum allowable movements should be considered so that a sufficient level of safety is taken into account in the design of the construction method. The characteristics of the element affected by subsidence may also be considered in the assessment of movement thresholds.

The degree of isostaticity plays an important role in assessing susceptibility against induced subsidence: for all remaining variables constant, the more isostatic a structure is, the higher the value of angular distortion that it will be able to tolerate.

Values suggested by Bjerrum (1963) and levels of damage associated to them have proven to be realistic.

When determining the allowable level of induced subsidence, an economic analysis should be carried out, so that a balance between the cost of repairs and the cost of avoiding damage is actually met.

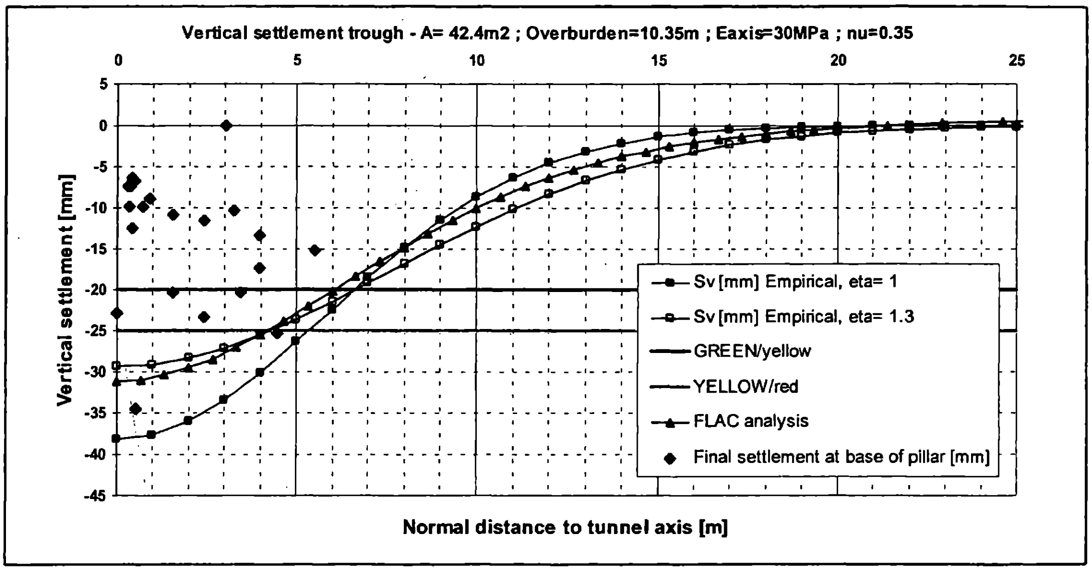

Fig.11 Estimated settlement envelope vs. measured settlements.

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