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Prediction and performance for the foundation of a 40-storied tower in Tenerife Island

Prédiction et mesure des tassements pour la fondation d'une tour de 40 étages à l'île de Tenerife

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ABSTRACT
A tower, 114.2 m high, has been successfully ended at Tenerife Island. The ground is mainly made up of fractured and weathered lava flows, alternating with scoria, which present cavities of variable size. The installation of rod extensometers at different depths below the foundation slab and clinometers has permitted a comparison between measured and calculated displacements, and the estimate of in situ moduli. A first class prediction (before construction) has allowed establishment of an upper and a lower limit for the settlements of the tower slab. The measured settlement is near the geometric mean of these values.

RÉSUMÉ
Une tour, 114.2 m haut, a été finie avec succès à l'île de Tenerife. Le terrain est composé principalement de couches de lave fracturés et météorisés, alternant avec scories, qui ont des cavités de taille variable. L'installation d' extensomètres de tige à différentes profondeurs au-dessous de la dalle de fondation et des clinomètres a permis une comparaison entre les déplacements mesurés et calculés, et l'évaluation des modules in situ. Une prévision de classe 1 (avant de la construction) a permis l'établissement des limites supérieure et inférieure pour les tassements de la tour. Le tassement mesuré est près du moyen géométrique de ces valeurs.

1 INTRODUCTION

Twin towers are integrated in a privileged expansion zone of Santa Cruz de Tenerife, near outstanding structures such as the Auditorium and the Congress Palace both designed by the well-known Spanish Architect Santiago Calatrava. The first tower, with a height of 132.7 m above foundation, has been recently completed (Fig. 1). The building, with 35 storeys above ground level and 5 basements, is the highest apartment building in Spain. It was decided to found the tower on a 2 m thick slab followed by 10 cm of plain concrete. This paper deals with the foundation of this tower.

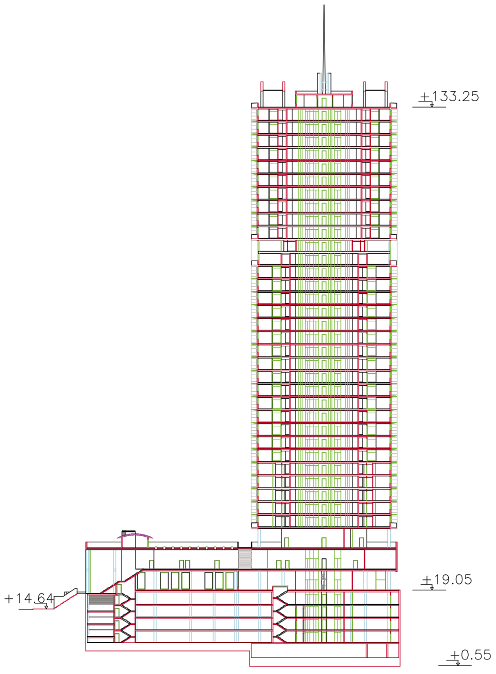

Figure 1. Section of tower and attached building

The paper describes the geological and geotechnical conditions at the site, the settlement calculations and the installation of instrumentation. All this has permitted a comparison between measured and calculated settlements and the estimate of in situ modulus.

2 GENERAL GEOLOGY

The ground is mostly formed by quaternary basaltic blue lava flows, alternating with levels of scoria, interbedded with red volcanic tuff and whitish thin pumice layers that have been swept up to this place by the wind (Fig. 2). The stratification is sub horizontal. Both the basalt and the levels of scoria present cavities of variable size. The sea level controls the water level. The volcanic material behaves as a very pervious mass that cannot take any pressure during Lugeon tests.

3 STRATIGRAPHY

Figure 2 shows the front of the 17.46 m deep excavation. Starting at level +19.05, the following layers appear from top to bottom:

a) 3.75 m of fill and volcanic tuff.
b) 6.8 m of upper blue jointed basaltic lava flow with small cavities.
c) 5 m of red volcanic tuff and whitish pumice
d) Below +1.8 m jointed, lower weathered blue basalt, with small cavities. In this stratum, mostly placed below the tower foundation there are levels of scoria and thick levels of pyroclastic breccias.

4 SITE INVESTIGATION AND LABORATORY TESTS

Ten boreholes were drilled up to a depth of 30 m below the bottom of the excavation . Unconfined compression tests were carried out at the cores. In some tests in basalt, the major and minor principal strains, measured with strain gauges (Fig. 3), allowed finding the deformation modulus and Poisson's ratio.

Figure 2. Front of the 17.5 m deep excavation

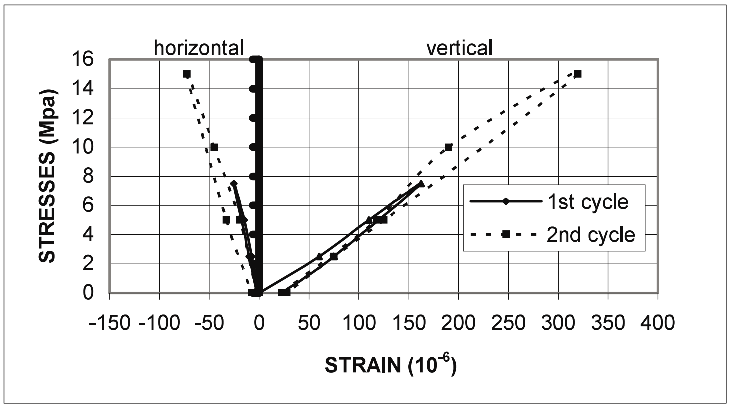

Figure 3. Uniaxial compression test on massive basalt.

The behaviour of the samples was not far from elastic, with small plastic strains. The modulus is little dependent upon basalt type or stress level (for stresses up to 15 MPa). Median moduli of 64 GPa for loading and 80 GPa for reloading were obtained. The corresponding Poisson's ratio values were 0.34 and 0.37 respectively. The median E_c/q_u ratio was 880. Six pressuremeter tests were carried out in the boreholes. The pressuremeter moduli ranged from 800 to 2600 MPa, with an average value of 1700 MPa. In the pyroclastic breccia a modulus of 400 MPa was found.

Beniawsky's (1979) Geomechanics Classification was carefully applied to the basalt, scoria and pyroclastic materials. It is based upon six parameters: the uniaxial compressive strength of the rock material, drill core quality *RQD*, spacing, orientation and condition of discontinuities and ground water conditions. Importance ratings were allocated to each parameter and total rock mass rating (*RMR*) was determined. The mean uniaxial compressive strength ranged from 0.8 MPa in the compact pumice to 83 MPa in the lower massive basalt. The mean *RQD* ranged from 10 in the weathered or fractured basalt to 80 when the basalt was massive. Finally, the *RMR* index varied between 42 in the scoria and 69 in the lower massive basalt. The in situ modulus of deformation was determined from the rock mass rating according to the equation suggested by Beniaswsky:

$$E\ (\text{GPa}) = 2\ RMR - 100 \qquad (1)$$

The moduli obtained in the basalt and pyroclastic breccias at the site ranged from 22000 to 38000 Mpa.

5 DESIGN PARAMETERS

The rock is strongly jointed and it would not be reasonable to obtain the modulus from cores (v. McMahon and McMahon, 1980). The pressuremeter modulus were elected as the lower limit and the upper limit could correspond to those obtained from Beniawsky's *RMR* index. Doing this the limits of table 1 were established for the finite elements (FE) calculation. A Poisson'ratio value of 0.35 was assumed.

Table 1: Design moduli.

Ground	Zone	Level (m)	*E*(MPa)	
			Min.	Max.
Upper basalt	All	All	670	22,000
Volcanic tuff	All	All	32	73
Lower basalt	Attached	>-7.5	250	4,900
	Building	<-7.5	1400	32,000
	Tower	>-11	670	13,000
	Rear zone	<-11	1,400	32,000
	Tower	>-11	920	19,000
	Medium	-11 to -29	910	32,000
		<-29	1,400	32,000
	Tower	>-11	1,200	25,000
	Front zone	-11 to -29	420	32,000
		<-29	1,400	32,000

6 FINITE ELEMENT CALCULATION

A linear elastic FE calculation was performed including the foundation of both the tower and the attached building. The following elements were modelled: the two slabs, the cellar walls, the structural floors of both buildings, the ground surrounding the building up to a depth of 50 m and distance of 30 m around the buildings. Shell elements modelled the slabs, walls and structural floors, and solid elements the ground. The model had 39,998 elements and 42.810 nodes. The dilation joint between both buildings was simulated. Figure 4 shows the discretized ground volume.

Figure 4. Ground discretized volume

Both short-term calculations, with a concrete modulus of 30.000 MPa, and long-term calculations with a concrete modulus of 12.800 MPa were carried out. Table 2 shows a summary of the calculations under self-weight, surcharge and wind.

Table 2: Finite element results obtained at the tower for the maximum load combination.

Calculation	Moduli	Settlement (mm)		Slab moments (mkN)	
		Min.	Max.	Min.	Max.
Short term	Minima	8 (6)	17 (13)	-7,600	5,200
Long term	Minima	8 (6)	18 (14)	-5,900	5,500
Short term	Maxima	0.3 (0.2)	1 (0.8)	-4,800	4,700

Negative bending moments at slab correspond to tension at the bottom. The settlements at the end of construction are shown inside brackets.

7 MEASUREMENTS

Rod extensometers were placed at different depths below the tower slab and biaxial clinometers were installed in one cellar (Fig. 5). Figure 6 shows the settlements (automatic measurement) of the extensometers up to the end of construction and Figure 7 displays the rotations of the clinometers.

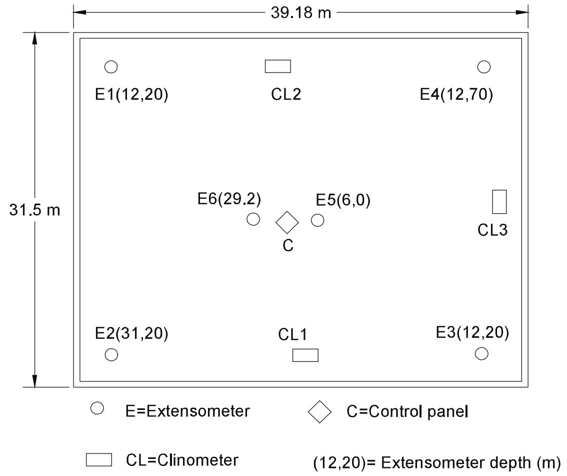

Figure 5. Position of extensometers and clinometers at basement level. Inside brackets depth below the slab base of the tower

8 DISCUSSION

The pressure of the excavated soil was 417 kPa. The average pressure at the bottom of the plain concrete at the end of construction was nearly equal (427 kPa). So up to that moment the foundation was floating. With the surcharge, the average pressure will increase to 558 kPa.

Table 3 shows a comparison between the settlements measured at the extensometers and those calculated by Steinbrenner (1934) method.

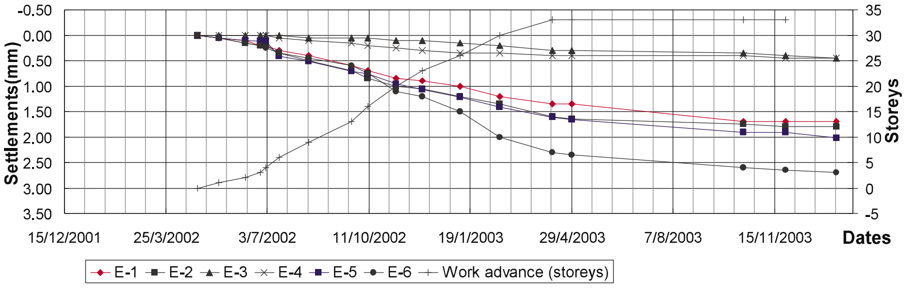

Figure 6. Work advance and settlements at extensometers.

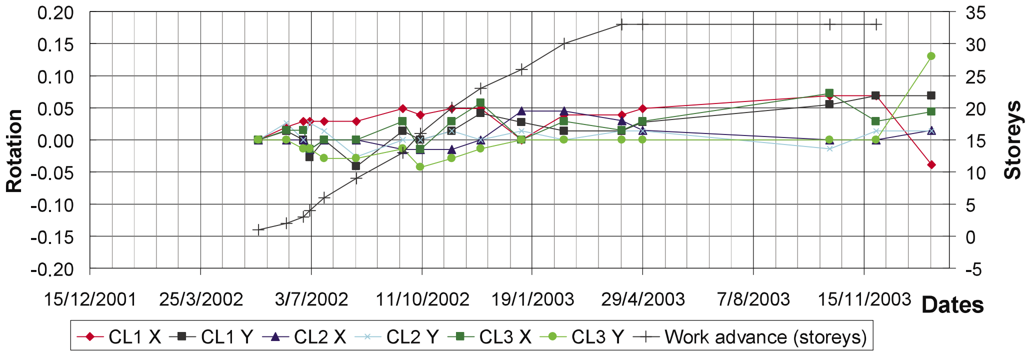

Figure 7. Work advance and rotations at clinometers.

Tabla 3: Comparison between the settlements measured at extensometers and those calculated by Steinbrenner method

Extensometer	Depth (m)	Settlement at extensometer (mm)		Deformation modulus		Calculated settlement at surface (mm)
		Measured	Calculated	Measured	*E* (MPa)	
1	0-12.25	1.66	0.54	0-6	800	2.02
2	0-31.20	1.75	0.98	6-12,25	6000	2.02
3 & 4	0-12.25	0.45	0.45	12,25-30	3500	1.72
5	0-6.00	2.00	1.72	>30	6000	4.43
6	0-29.20	2.69	3.22		6000	4.43

The rotations of the clinometers, are generally less than 0.07º, that is within the precision of the instruments. The rotation calculated from the extensometer measurements is 0.002º towards the attached building.

The horizontal distinction in plan between the moduli indicated in Table 1, corresponding to pressuremeter results and *RMR* values was not reflected into the extensometer measurements. This suggested that the rock profile should be simplified. Only four layers: 0-6 m, 6-12,25 m, 12,25-30 m and > 30 m from the bottom of the slab, as indicated in Table 3, have been considered in these preliminary calculations. These depths coincide approximately with the positions of the extensometers. The moduli indicated in the Table are those that best fit the measured settlements at the extensometers using Steinbrenner approximate calculation method. In a forthcoming paper the measurements will be interpreted using the FE method. At the end of construction, the calculated settlement below the centre of the slab using these moduli (4.4 mm) nearly coincides with the geometric mean (4.1 mm) of the predicted values corresponding to the maxima (17 mm) and minima moduli (1 mm) in Table 2. This was a first class prediction, as they were predicted before construction (Justo et al. 2001).

9 CONCLUSIONS

The construction of the tower has successfully ended with negligible settlements and rotations. The estimate of the moduli in jointed and vesicular basalt is a difficult task. A first class prediction has allowed establishing upper and lower limits of the in situ moduli. The installation of rod extensometers at different depth and clinometers at the first cellar has allowed a preliminary estimate of the in situ moduli and of the slab settlements. Beniawsky's correlations with *RMR* index give moduli that are too high. The pressuremeter modulus is too low. The geometric mean of both estimates nearly coincides with the measured value.

ACKNOWLEDGEMENTS

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