Foundation of a 200 m high chimney on collapsible weak rock

Fondation d'une cheminée de 200 m sur un sol susceptible d'effondrement

S. URIEL ROMERO, Professor of Soil Mechanics and Foundation, Laboratorio de Carreteras y Geotecnia "José Luis Escario", Madrid, Spain
F. AZCUE, Dr. Eng., INITEC, Madrid, Spain
J. M. VALENTI, Geologist, INITEC, Madrid, Spain

SYNOPSIS The 200 m high chimney of the Almeria thermal power plant is founded on calcareous sandstone of very low density (about 1,55 T/m^3). The shear strength, collapsibility, and deformability were studied by laboratory and "in situ" tests. The maximum allowable pressure was stablished in 10 daN/cm^2 to have a sufficient safety factor against failure due to shear stress or collapse. The measured settlements closely agree with the results of plate bearing test in horizontal direction.

INTRODUCTION

The thermal power plant of Almeria owned by "Empresa Nacional de Electrictidad (ENDESA)" is located near the small village of Carbonera, at 200 m from the Mediterranean sea, in the southest of Spain. This region can be submmited to earthquakes of grade VIII, and to winds of more than 100 Km/H, after the seismological and meterological institutes of Spain.

To avoid the land contamination, the smoke is discharged to the atmosphere through a chimney of 200 m high. The cross section of the chimney is circular, with an exterior diameter of 19,50 m at the base and 12 m at the crown (Fig. 1). For the above mentioned nature actions the forces transmitted to the ground are the following:

Own weight of the chimney................	10.500 T
Earthquake action	
Moment - 54.000 mT;	Shear - 818 T
Wind action	
Moment - 33.000 mT;	Shear - 322 T

Having into account the special characteristics of the foundation soil, it was designed an annular footing, of 5 m width and 24 m exterior diameter. The justification of this figures and the behaviour of the foundation soil is the main objetive of this report.

Fig. 1.- View of the 200 m high chimney

THE FOUNDATION SOIL

The soil on which is founded the plant belongs to the upper Miocene, and it is about 6 million year old. The main formations are represented on the site by calcareous sandstones in the surface, white or yellowish, laying on thick strata of marl and limestone. Near the sea the Miocene is convered by quaternary marine sediments, composed by quartz sand and gravel.

The chimney is located on the Miocene, near the contact with the quaternary soils. The thickness of the calcareous sandstone is about 23 m (Fig. 2). The ground water level is situated 6 m under the general level of the power plant and it coincides with the sea level near the site. Practically the only formation which is affected by the stress transmitted by the chimney is the sandstone. We shall refer in the following paragraphs to this rock.

The petrographic studies carried out on samples have revealed that it is a bioclastic limestone, composed by small fragments of limestone cristals and shells, welded together by a cement of chemical precipitation. The most outstanding characteristic of the sandstone is the very low dry specific weigth, about 1,55 T/m^3. This soil can be classified as a "rigid soil" or a very "soft rock", and by this reason it is probably susceptible of tensional collapse, that is a sudden destruction of its natural vacuolar structure, when the stress field attains a certain level, followed by great deformations. Generally the sandstone is massive, with very few joints. In any case, the most part of the joints are now welded in a similar way to the rock itself. This low density determines its easy rippability, with very modest mechanical machinery, or with hand outils.

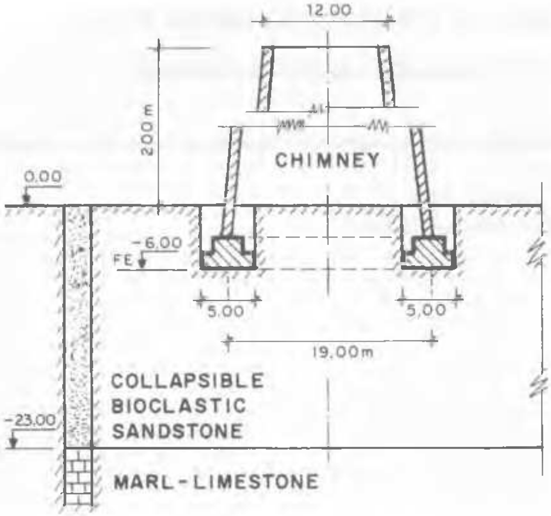

Fig. 2.- Schematic cross section of the chimney and the foundation ground

The shovel spoon "scrapes" easely the rock, and the teeth of spoon remain clearly marked in the slope excavation (Fig. 3). Nevertheless, as we shall coment afterwards, is sufficiently high to allow the excavation of 40 m high slopes of 60° with the horizontal. (See the slopes in Fig. 1, behind the chimney).

Fig. 3.- Excavation of the annular trench for the chimney foundation on calcereous sandstone

GEOTECHNICAL PROPERTIES OF THE SANDSTONE

The shear strenght, deformability, and specially the collapse laws, are the main factors determining the morphology and the dimensions of the foundation structure. Many samples were obtained from the borings, and several blocks were trimmed from the walls of the excavations in order to carry out laboratory test. In the same way several in situ tests were performed to know the deformability of the rock. In the Table 1 are summarized some of the mean results of the laboratory test.

Table 1

Dry specific weight	1,55 T/m^3
Porosity	41%
Moisture content	
Below water	24,7%
Above water table	13%
Carbonate content	75,2%
Unconfined compression strength	
Below water	14,0 - 23,0 daN/cm^2
Above water	18,5 - 26,0 daN/cm^2

The shear strength, and the collapse laws were investigated by means of undrained triaxial test, on cilindrical samples of 10 cm diameter, with three lateral pressure on the triaxial cellule (σ_3 = 0; 15 and 30 daN/cm^2). These tests were completed with isotropic collapse tests, in which the sample is summited to an incresing homogeneous pressure ($\sigma_1 = \sigma_3$), to obtain the stress under which a marked change in the stress-strain curve, is produced, that is the isotropic stress which destroys the natural structure of the soil.

In the unconfined compression test (σ_3=0), in all samples the failure is of brittle type, as it is usual for soft rocks. The pic strenght coincides with a sudden deformation of the sample. One or several failure planes are observed, and the specimen is divided into several fragments, each of them mantaining its previous natural structure. The average unconfined strenght obtained were 20 daN/cm^2.

In the isotropic test, ($\sigma_3 = \sigma_1$) the collapse is clearly marked by an elbow in the stress-strain relation. Seven samples were tested, and the average value of the isotropic collapse varied between 48 and 61 daN/cm^2. With an average value of 57 daN/cm^2 (see curve 4. Fig. 4 and 5).

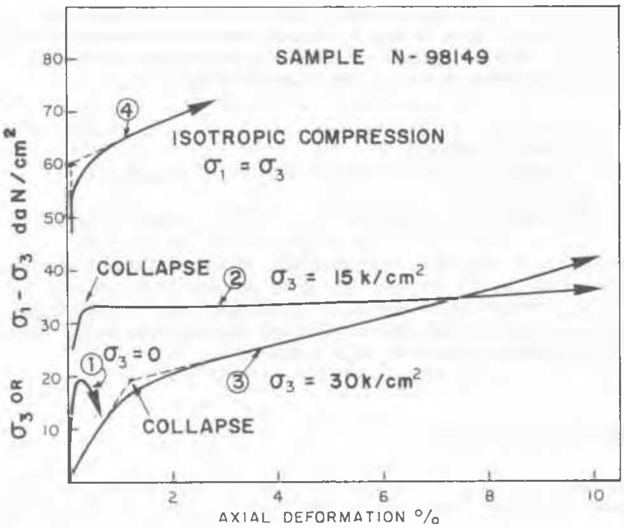

Fig. 4.- Stress-strain curve. Triaxial test

When σ_3 = 30 daN/cm^2, the collapse of the natural structure is clearly noted, for a deviatoric stress ($\sigma_1 - \sigma_3$) rather moderate, 21 daN/cm^2 as a mean (see curve 3. Fig. 4-5).

For σ_3 = 15 daN/cm^2 the failure is a mixed one. Firstly, for a deviatoric stress of about 31 daN/cm^2, the collapse begins, but finally the samples show a clear rupture plane, similar to a failure of brittle type (see curve 2, Fig. 4-5).

There are several theories for explaining the collapse of porous weak rocks and cemented soils (see references)

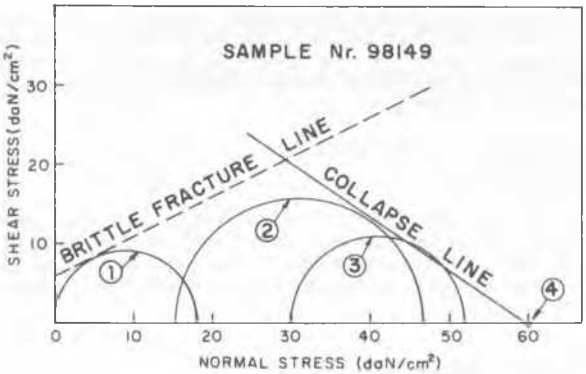

Fig. 5.- Mohr envelope for brittle fracture and collapse

and the differences with a conventional brittle failure. It has been demostrated experimentally and theoretically that the collapse is produced when the bonds between soil or rock particles are broken by compression. In the normal brittle failure the bonds are broken by shear. The collapse begins when the Mohr circle of stress is tangent to a line, which passes through the point, that we have called isotropic collapse, and a negative slope, with a similar value to the conventional friction angle for the brittle failure. Fig. 4-5 shows the stress-strain reactions and Mohr circles for the four cases above expained. The following average results were obtained.

For the brittle failure
- Cohesion .... 6,3 daN/cm^2
- Angle of friction .... 22,5°

For the collapse
- Isotropic collapse .... 60 daN/cm^2
- Negative angle of friction .... 25,5°

For determining the deformability of the sandstone, several 60 cm, circular plate test were performed, in horizontal direction, on the walls of trenches opened near the location of the chimney (Fig. 6). The load on the plates were increased by steps of 2 daN/cm^2, till a maximum of 16 daN/cm^2. After an unloading cycle, nine additional cycles between 0 and 16 daN/cm^2 were applied. A typical diagram load-deformation is presented in the Fig. 7.

From these tests, the following conclusions were establish:

a) For the first load cycle, the curve stress-deformation presents a small curvature from the origin, but till the maximum pressure applied, the modulus of deformation is high.

b) From the cycle Nr 7 on, the behaviour of the soil is elastic, and no permanent additional settlement is observed for the following steps.

c) Under water, sistematically, the modulus of deformation is greater than above the ground water level. The average relation between this moduli is about 0,6. Under water the soil is saturated and the test are, therefore, partially drained.

From these tests, the following moduli of deformation were considered, for the design.

For permanent vertical loads
- Between 0-4 daN/cm^2 .... 40.000 daN/cm^2
- Between 0-14 daN/cm^2 .... 16.700 daN/cm^2

For earthquake action
- Between 0-10 daN/cm^2 .... 19.800 daN/cm^2

For wind action
- Between 0-10 daN/cm^2 .... 15.000 daN/cm^2

Fig. 6.- Horizontal plate deformability test Ø 60 cm

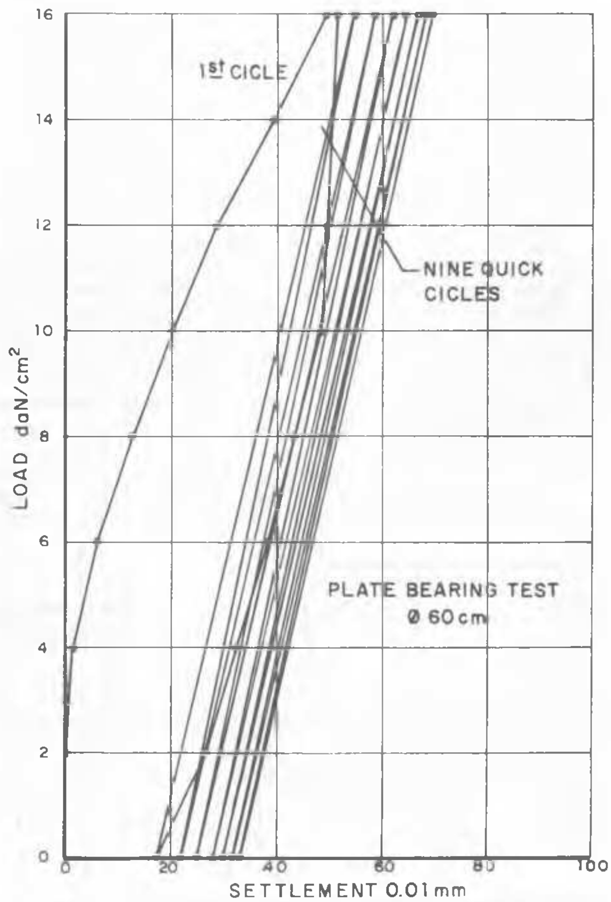

Fig. 7.- Typical diagram of a plate deformability test

ALLOWABLE BEARING PRESSURE

Two are the points which must be examined for the design: the safety factor against the failure, of brittle or collapse type, and the expected movements under normal or extraordinary circunstances.

In relation with a brittle failure, the parameters which define the shear strength are very high. The unconfined compression strength (20 daN/cm^2), or the triaxial strength (c = 6 daN/cm^2; φ = 22,5°), allows pressures on the ground greater than 20 daN/cm^2, with a safety factor equal or greater than 3.

Nevertheless, the failure by collapse is more restrictive. There are no rules in relation with this possibility. In Spain were many works are founded on collapsible volcanic rocks in the Canary Islands, is a current practice to adopt, for important projects an allowable pressure with a safety factor of 5, that is the maximum major principal stress must be 1/5 of the isotropic collapse stress. This figure can be considered as pesimistic, but it must be taken into account several circunstances, which points toward a prudent safety factor. Among them the most important is the possibility of stress concentration at the edges of the rigid foundation structure.

In contact with a more deformable soil, the stress in the limits of the footing can be several times the mean pressure on the ground. In this way the collapse can be attained in some zones, probably of small extension, which produce a redistribution of the stress, with an increase towards the center of the foundation. With that criterium the maximum allowable pressure was fixed in about 11 daN/cm^2. A ring of 5 m midth and 24 m exteriour diameter was designed for the chimney. With this dimensions the mean stress under the weight of the chimney were 3,55 daN/cm^2. For seismic and wind action, the maximum pressure would be 10,2 daN/cm^2, and the tilting of the chimney no more than 3 x 10^{-4}.

THE OBSERVED SETTLEMENT AFTER CONSTRUCTION

The observed settlement during the construction can be in Fig. 8. The movements were measured for the first time when the chimney attained 70 m high, and a pressure on the foundation of about 1,5 daN/cm^2. The settlement between 0 and 1,5 daN/cm^2 was extrapolated. Between 1,5 and 3,5 daN/cm^2 the measured settlement was 6 mm with an error of ± 1 mm. The total settlement was estimated in 11 mm. The calculated value, for a modulus of deformation of 40,000 daN/cm^2 was 10,1 mm.

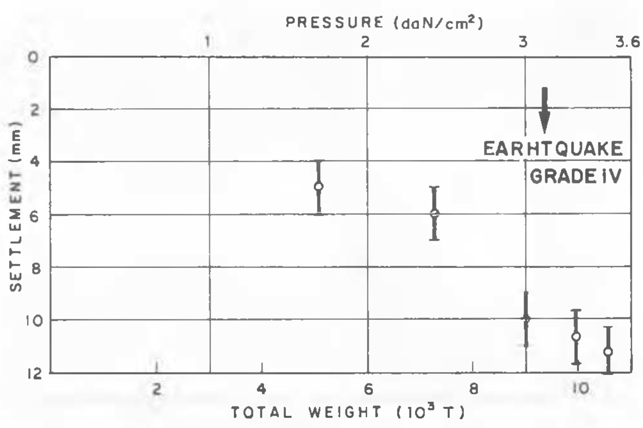

Fig. 8.- Measured settlement of the chimney

It must be emphasized that when the chimney was almost completed (See Fig. 8). The site was attained by an small earthquake, grade IV. The following measurement revealed an elastic behaviour of the soil, and no additional settlement was registered.

CONCLUSION

The exposed case is a noticeable record of a high chimney founded on a collapsible soil, with a low density of 1,55 T/m^3. The deformability of the soil is in accordance with the results of plate test performed in horizontal direction.

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