Stabilization of a natural landslide by deep drainage and ground treatments

Stabilisation d'un versant naturel par voie de drainage profond et de techniques supplémentaires de soutènement du terrain

Luis Sopeña Mañas, José Estaire Gepp & José Luis Salinas – *Laboratorio de Geotecnia, Cedex*

ABSTRACT: Benamejí slope, 950 m long and 170 m of difference in level between crest and toe, has periodically suffered instability movements. The hidrogeological and geotechnical models, based on field data, were used to design the stabilization measures, designed to tackle the double origin of the problems: the poor geotechnical ground characteristics and the high general piezometric level. The stabilization was based on drainage and reinforcement measures: a wall of deep draining wells (1,5 m diameter and 50 m deep) and two rows of dowels (drilled shafts and piles), acting as ground retaining elements and protection of buildings affected.

RESUME: Le versant de Benameji, de 950 de longueur et 170 m de hauteur a subi des glissements périodiques. Des modèles hydrogéologiques et géotechniques, basés sur les données obtenues en chantier, furent utilisés pour optimiser les mesures de stabilisation, établies pour affronter l'origine double des problèmes: les conditions géotechniques assez pauvres du terrain et le niveau piézométrique général assez haut. La stabilisation fut basée sur mesures de drainage et de soutènement: une ligne de puits chainants, de 1.5 m de diamètre et 50 m de profondeur, et deux lignes d'étriers (sous forme de barrettes et de pieux) comme éléments d'appui et de protection des immeubles affectés.

1 INTRODUCTION

Benamejí slope, originated by the erosive action of Genil river, has periodically suffered instability movements associated to deep landslides. Figure 1 shows a morphological interpretation of such instability process. The slope has a length of some 950 m from the town to the river, with a difference of level of 170 m. The slope can be considered divided in two parts by two clayey hills. In the upper part, the slope opens sideways, in a shell shape with a perimeter of some 300 m, inside which rotational type instabilities, that affect the buildings of the village, occurr. In the lower part, an extensive clayey flow, 650 m long and up to 20 m thick, is generated by the material coming from the upper part, including calcareous blocks.

From a historic point of view, a great slide that provoked a backward movement of its upper crest and the destruction of more than 100 buildings occurred in 1963. Since that date, the upper part of the slope was used as a dump for demolition materials, forming a great horizontal surface. In 1989, after a period of heavy rains, all the dumped material suffered a new slide that also affected some buildings. In the autumn of 1997, the Laboratorio de Geotecnia CEDEX began a campaign of geotechnical surveillance to determine the origin of the problem; this included 18 mechanical boreholes with depths between 30 and 80 m. The solutions designed were collected in a working project and carried out from September 1998 to April 1999.

2 GEOTECHNICAL AND HYDROGEOLOGICAL MODEL OF THE SLOPE

2.1 *Introduction*

A model of the slope ground was made from the lithological and piezometric data obtained by the geotechnical research and in accordance with lithological, structural and hydrogeological criteria based on field observations, as it can be seen in Figure 2.

2.2 *Geotechnical model*

The geotechnical units found in the slope were the following:

a.- Anthropic fills: material formed basically by rubble, that covers the natural soil with variable thickness up to 10 m.

b.- "Caliche": it is a sandy-silty soil, formed by a mixture of fine and gross sizes, encrusted by carbonation, between 2 and 7 m thick, positioned as a plataforme that crowns the slope and where the buildings of the village are founded.

c.- Clay (Upper Geotechnical Unit): sedimentary formation constituted by clays with occasional decimetric layers of calcarenites. From a geotechnical point of view, the soil can be classified as a high plasticity clay (CH) with a liquid limit of 70 and plasticity index of 35. Its mechanical behaviour is conditioned by water coming through the layers of calcarenites, which degrades its resistance and produces great deformability. The results obtained in shear box tests indicate a shear resistance of c=8 kPa and ϕ= 19,5° for peak states and c=5 kPa; ϕ= 11,5° in residual conditions of great deformations.

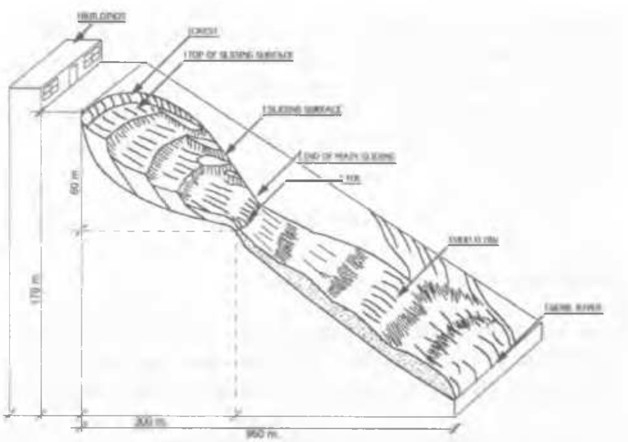

Figure 1. Morphological Interpretation of the Slope Instability

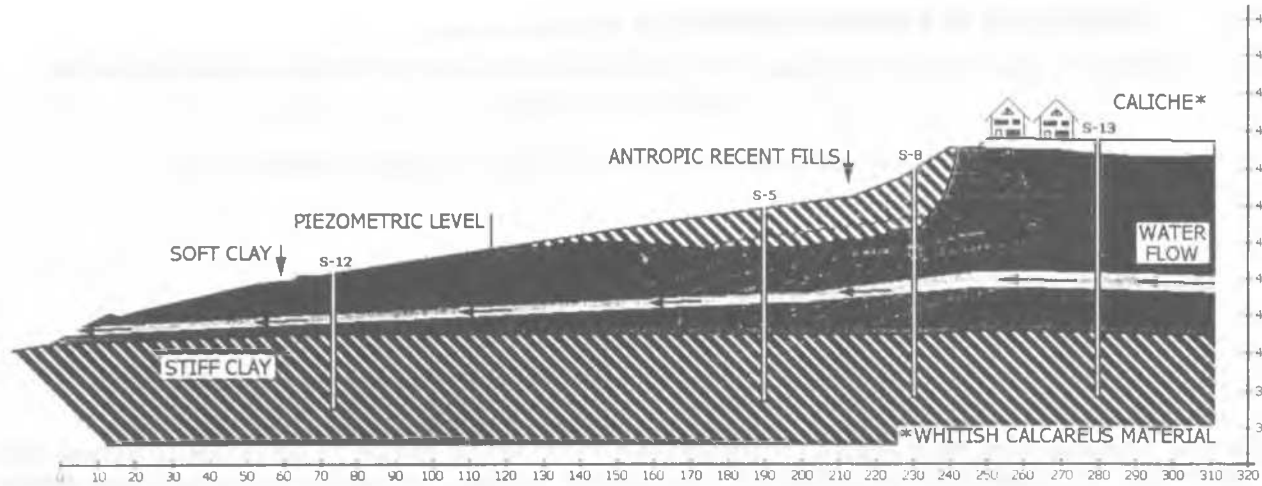

Figure 2. Hidrogeological model of the slope

d.- Lutites (Lower Geotechnical Unit): consolidated and stiff clays, with very low contents in calcarenites and a mean simple compression resistance around 600 kPa.

From a geometric point of view, the top of lutites is almost horizontal and it can be considered as the base of the potential sliding mass. The diference in level between this bottom layer and the crest of the slope, where there are buildings, is about 60 m.

2.3 Hydrological model

During the field research, two aquifer levels were detected. The upper one, whose thickness varies between 10 and 15 m, is formed by the layer of caliche and, in some zones, the calcarenites that underlie it. This aquifer is of free recharge as it contains water from relatively recent rainfalls.

The deep aquifer is formed by a more sandy permeable layer, 4 to 5 m thick, located immediately above the more clayed materials in the top of the Lower Unit; it is confined, contains old water and discharges in the lower central zone of the slope.

Although there is a rather impermeable soil stratum between both aquifers, they are intercommunicated, from an hydraulic point of view, through a network of fissures and discontinuities present in the calcarenites levels, producing an unique piezometric level in the boreholes.

2.4 Analysis of the model

The analysis of the geological-geotechnical models previously developed made possible to establish the actual causes of the instability problems present in the slope: the existance of a very high general piezometric level and the poor geomechanical quality of the materials existant in great zones of the slope.

3 DESCRIPTION OF THE STABILIZATION MEASURES

3.1 Introduction

The stabilization of Benamejí slope and the attached urban area required the execution of the following measures, designed to tackle the double origin of the problems, as it can be seen in figure 3:

a.- a deep stabilization of the global sliding mass, whose stable root is situated in the top of the so called Lower Unit;

b.- a local stabilization of the present slope crest edge and the functional recuperation of the attached urban area;

c.- a slope surface stabilization, with special reference to the anthropic fills.

3.2 Deep stabilization

The deep stabilization was based on drainage and reinforcement measures. As drainage measure a wall of deep drainage wells was made to intercept the existing preferential deep flows in the slope and to achieve a lowering effect on the general piezometric level. The wall is formed by 22 wells of 1500 mm diameter and 50 m depth that converge in a collecting well. The connections between wells, separated between 9 and 15 m, towards the collecting well were made in 45 m deep points. The water is discharged by means of a pump device placed in the bottom of the collecting well.

The reinforcement measures consisted of two rows of large vertical structural elements, normally called dowels, that lock together the sliding mass with the stabl substrate when the sliding ground tends to move.

The upper structural line is formed by 37 drilled shafts ("barretes") of two different sizes (4x1 m and 5,5x1,2 m), with depths between 30 and 50 m and variable separations between 6,5 and 7,5 m. This line has the mission to safeguard the upper zone of the slope and to serve as support to the reinforcement measures of the slope crest, later described.

The other row, situated in the lower third of the slide, is formed by 22 piles of 1500 mm diameter. Its main mission is to avoid a ground movement in that zone which could jeopardize the stability of the upper line of dowels.

At 50 m distance of the row of piles, a rouble mound was also built to serve as a frontal closure of the zone susceptible to be mobilized by slides and as drainge of the lower part of the slope.

3.3 Upper crest stabilization

The stabilization of the upper crest of the slope was solved by means of:

a.- a soil nailing wall;

b.- earth movement of the existing soil to form a soft slope between the head of the upper dowel row and the foot of the soil nailing wall;

c.- the placement of a rouble wall to give stability in the toe.

The soil nailing wall is 300 m long and 5,5 m high, and it was

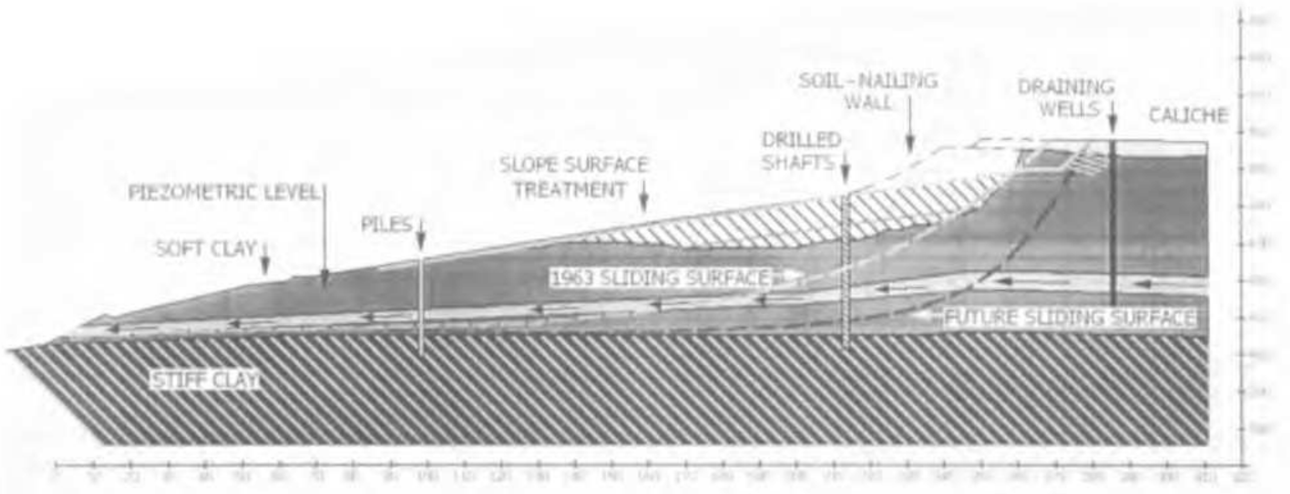

Figure 3. Stabilization measures

built with 2H:3V inclination. The reinforcement consisted of four rows of 7 m long bolts, separated 2 m and a layer of proyected gunite of 9 cm thick. To disminish the visual impact of the wall in that semiurban area, the more external gunite layer was coloured in soft brown.

3.4 Slope surface stabilization

The stabilization of the slope surface consisted of:
a.- extending of an impermeable clay layer to avoid water infiltration;
b.- construction of a drainage system to collect rainfall water;
c.- plantation of trees and bushes to avoid surface erosion.

4 ANALYSIS OF THE PROBLEM

4.1 Stability calculations

The first step was to make a stability back-analysis using the 1963's sliding surface. This let know that the ground resistance parameters, that generate a safety factor equal to 1, are equivalent to a cohesion of 10 kPa and a friction angle of 11,5°, which are quite similar to those ones obtained in the shear box tests, previously mentioned.

The next step was to determine the most probable future sliding surface. This surface had a safety factor of about 1,05-1,1. Since these values represented a safety deficit compared to those desired for a permanent situation, to carry out stabilization actions was considered necessary.

Finally, stability analysis were performed to design and optimize the stabilization measures: a lowering of the piezometric level due to a deep drainage and an improvement of ground resistance characteristic due to the stabilization dowels.

The average increase of the safety factor due to drainage measures was 0,15, assuming a only 50% global efficiency of the drainage system.

The reinforcement measures were taken into account in the calculations by means of external horizontal forces that would represent the action of the two rows of the structural elements. The calculation results indicate that to obtain an increase of the safety factor of about 0,1 would be necessary a total force of some 150 t/ml. This force was divided in one third (50 t/ml) in the lower row and two thirds (100 t/ml) in the upper row. The increase of safety factor obtained in the calculations for the reinforcement measures is in accordance with the usual practice. (Sommer,1979).

Therefore, the global increase of the safety factor would be of about 0,25-0,3 when the two kinds of the above measures act simultaneously.

4.2 Dimensioning of dowels

The dimensioning of dowels was done following the next steps:
a.- Distribution of the stabilization force

The determination of the distribution of the stabilization force in each dowel was done using the theory developed in Design Manual 7.01. According to that theory, the pressure law is parabolic, althoug in practice it can be considered as triangular. The law to be used in the calculations is homothetic to the theoretical one, but having into acount that the resultant of the calculation law must be equal to the stabilization force previously determined by stability analysis.

b.- Determination of bending moment and shear force laws

The next step is the definition of the calculation squeme that lets calculate the bending moment and shear force laws acting in dowels. Such squeme, in figure 4, is based on that the two only contributions of the ground between the stable layer and the slope surface are the pressure law, acting on dowel, defined in above "point a" and a vertical surchage equivalent to its weight applied in the top of the stable layer. In the stable layer, the ground develops the corresponding active or pasive pressures to maintain the equilibrium in the dowel embedded on that layer.

With this calculation squeme, the bending moment and shear force laws can be calculated using wall classical theories or any informatic program based on Winkler's model.

c.- Structural dimensioning

Given the bending moment and shear force laws, the necessary steel reinforcemet can be calculated, considering the dowel as a usual concrete piece.

d.- Separation between dowels

Known the maximum bending moment and shear force that a dowel can support, with its corresponding steel reinforcement, the separation between dowels is obtained dividing those maxima between the corresponding values per unit of length, obtained in "point b".

The only point to have into account is the separation of dowels must not be very small, as in that case it could act as an quasi-impermeable wall for the water flows, currently involved in this kind of problems.

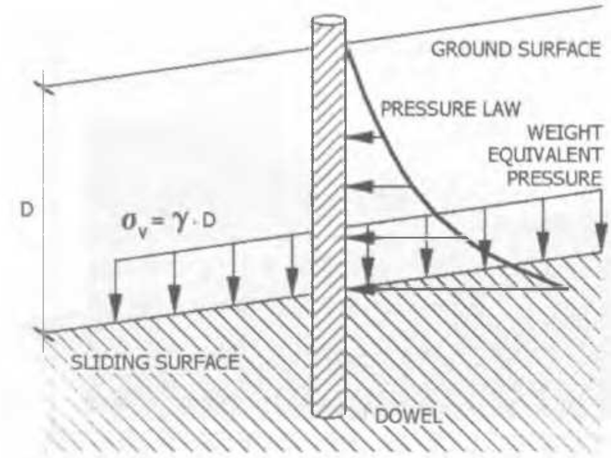

Figure 4. Squeme for the calculation of bending moment and shear force laws

5 INSTRUMENTATION INSTALLED

5.1 Introduction

A number of auscultation apparatus (piezometers, extensometers and inclinometers) were installed in different places of the slope and the structural elements to control and to test the efficiency of the stabilization measures carried out.

5.2 Piezometers

A total of 8 piezometers are active three years after the finish of the works. These piezometers indicate an important lowering of the piezometric level in the upper part of the slope. In the rest of the slope, the decrease in such level can be quantified in about 25%. These figures show the hypothesis made in the stability calculations was quite right and the draining measures are efficient in the most important part of the slope, the upper one where buildings are founded.

5.3 Extensometers

There are 29 vibrating cord extensometers installed in four different dowels of the upper row and other 9 ones installed in the piles of the lower row. The pressures measured are represented in figure 5. The analysis of the values indicate that these ones are mainly above 20% of reinforcement steel elastic limit, although some values are near 50%. This fact implies a great margin of safety from a structural point of view.

5.4 Inclinometers

There are three inclinometers installed in some dowels and one in a pile. The inclinometers located in the dowels have hardly suffered movements, indicating a situation of great stability. However, the inclinometer situated in the pile shows a general bending movement, with a head displacement of about 30 mm in both directions, as it can be seen in figure 6. This fact implies that the pile row is active and working to support ground pressure of the lower part of the slope.

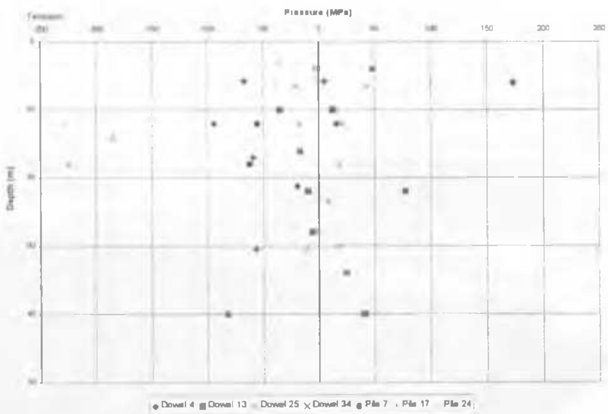

Figure 5. Pressures measured with extensometers

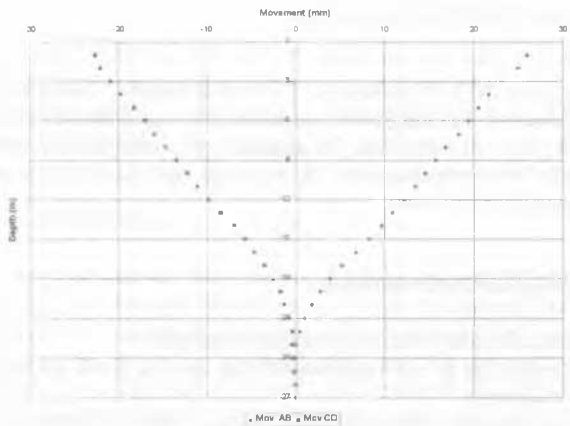

Figure 6 Movements of pile n° 24 measured by inclinometer

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