SLOPE FAILURE IN EXPANSIVE MARLS

Rupture de talus dans des marnes expansives
Böschungsbruch in quellfähigen Mergeln

GONZALEZ DE VALLEJO*, L., CAMPOS DE ORELLANA*, J. A.
BERZAL*, J. L. and LINARES-RIVAS*, A.

* Empresa Nacional Adaro de Investigaciones Mineras, S.A.
Serrano, 116 - Madrid, Spain

A B S T R A C T

A case of slope failure in marls of expansive nature with clayey gouge materials along discontinuities is presented. Due to the failure of a plane and a wedge during construction, the stability conditions anticipated during design were compared with those prevailing during excavation. It was proved that the main factors affecting the instability were the seepage forces of rainfall water flowing into the excavation area and the conventional and overloaded blasts. The need of technical assistance during construction is emphasized as an important part of the design.

R E S U M E

On étudie un cas de glissement d'un talus en marnes expansives avec des discontinuités argileuses. On a pu montrer que les causes principales agissant sur l'instabilité étaient les filtrations de l'eau provenant de la pluie et l'action des explosifs. On en déduit l'intérêt de l'assistance technique pendant l'exécution des travaux.

Z U S A M M E N F A S S U N G

Es wird ein Fall von Böschungsbruch in schwellfähigen Mergeln mit tonigen Kluftbestegen vorgestellt. Aufgrund eines ebenen und Keilbruches während der Bauarbeiten wurden die Standsicherheitsbedingungen, die während der Projektierung erwartet wurden, mit den bei der Ausführung herrschenden verglichen. Es wurde nachgewiesen, dass die für die Unsicherheit hauptverantwortlichen Faktoren die Strömungskräfte des Regenwassers im Aushubsbereich sowie die konventionellen, zu stark geladenen Sprengungen waren. Auf die Notwendigkeit einer technischen Assistenz während des Baus als einen wichtigen Bestandteil des Projektes wird hingewiesen.

INTRODUCTION

The design of an excavation of 25 x 20 m length and 25 m depth was required to set up a crushing plant of marls for a cement manufacturing company in Olazagutia (Navarra, North East Spain). The site was located on a hill side area formed by stratified marls dipping from 30 to 40 degrees southward. Site investigations consisted on structural analysis of discontinuities, borehole drilling, RQD determination, weathering grades, point load tests (PLT), and permeability assesstment and sampling. Because of the proximity to a quarry composed by similar materials it was possible to carry out a back analysis of the stability of fallen rocks. Laboratory tests mainly consisted on shear box tests along discontinuities from borehole core samples and undisturbed block samples. Compresive strength, slate durability tests (SDT) and other index tests were also performed, besides X-Ray mineralogy determination. The rock material consisted of softs calcareous marls, grey coloured of very fine grained, with 70% of CO_3 Ca, 14% of montmorillonite, 8% of illite, 5% of quartz and 2% of clorite. The major discontinuities were the stratification planes, shear joints and tension joints. Discontinuity roughness varies from smooth to undulating and some of them, particulary the stratification planes, showed clayey gouge material of 1 - 2 cm thick. SDT results carried out according to the suggested methods of ISRM indicated a high durability index. However, it was observed that longer periods of time and increasing number of cycles than those recommended by ISRM lead to very low durability index. This low index indicate much better the swelling mineral composition of the marls. Shear strength parameters are illustrated in figure 1 together with shear strength back analysis and shear box tests results. Acceptable agreement between both results were found. It is remarkable the low values of the shear strength along discontinuities filled with clay material, even lower than those obtained with the same clay but in remoulded state. For further discussion of the geotechnical properties of this material it is referred to Gonzalez de Vallejo and Berzal (1976).

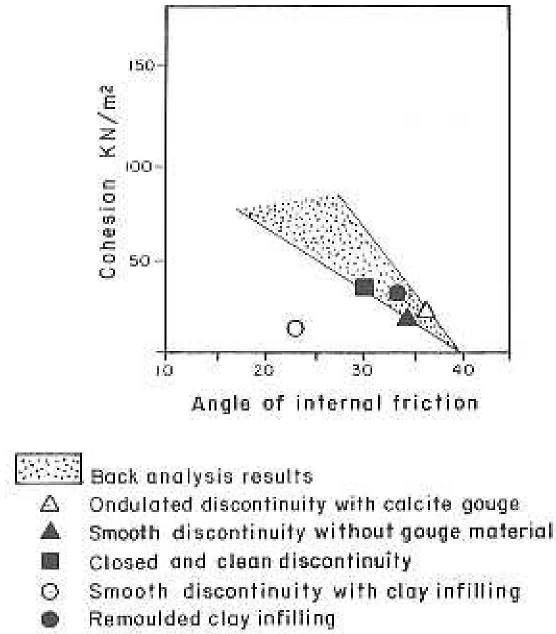


Fig. 1.— Results of shear stresses (residuals) from laboratory shear box tests and back analysis of failed slopes.

DESIGN CRITERIA

It was requested to undertake the design of the excavation not only under economic consi derations but under safety conditions, therefo re a conservative criteria was adopted in the selection of the shear strength parameters for design purposes. An angle of internal friction of 30 degrees was taken for discontinuities free of clayey gouge materials and 25 degrees for those with clay fillings. In both cases a cohe sion or 20 KN/m^2 was chosen. No water table we re intercepted during the site investigations neither regional hidrogeological evidences. No ne of both detected the presence of a high wa ter table in the area; therefore it was suposed a drained slope condition.

A short term condition was considered be cause, once the excavation were finished, the room between the slopes and the walls of the plant (Fig. 6) had to be filled with rock mate- rial, therefore a factor of safety 1.2 was taken for design. The stability analysis was carried out using John (1971) and Hoek and Bray (1974)- approaches to the problem.

Based on economic and construction reasons, two solutions were considered. The first one consisted to excavate the North, South and East slopes to an angle of 80 degrees and the West slope to 70 degrees. In this case, the North slope should require reinforcement to set back the factor of safety of plane P_1, which was 0.80 (Fig. 2 and table 1), to the factor of sa fety stablished by the design. It was esti mated that a force of 2,200 Mg would be appro piate and obtained by means of postensioned anchors. Additional reinforcing bolts could be neccesary to stabilised potential wedges (w8 and w9) in those places where the safety factor were 0.90. Similary, the South slope would require bolting for the unstable wedge w7 (F. of. S. 0.80). West slope would also need an additional force of 1,900 Mg to resti tute the safety factor of plane P4 (0.90) by means of anchoring.

The second solution consisted to excavate the South and East slopes to 80 degrees, North slope to 60 degrees and West slope to 55 degrees, requiring in all cases some boltings. But in both solutions it was recommended to carry out the recommendations listed in Table 2 (left column). Independently, it was suggested to choose the first solution because it could be implemented in a shorter period of time and would avoid the need of an extraexcavation of 3,500 m^3 and the filling of this room after - construction. The cost of both solution appear to be similar, although the second one was - slightly cheaper. Solution two will not need es pecial geotechnical works, e.g. anchoring.

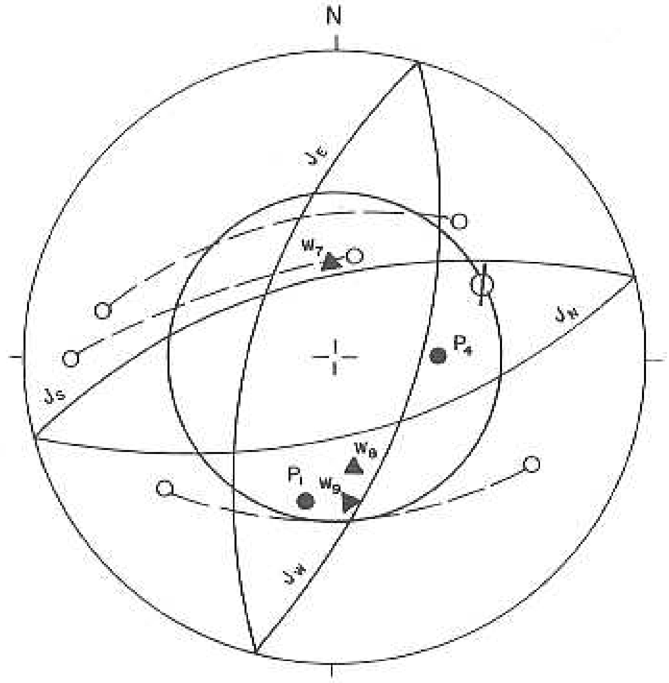
TABLA 1.- COMPARAISON OF THE STABILITY CONDITIONS ANTICIPATED BY THE DESIGN AND THOSE PRESENT DURING EXCAVATION

SLOPES FACES	ANTICIPATED BY DESIGN		PRESENT DURING EXCAVATION		BEHAVIOUR DURING EXCAVATION	
	Unstable discon. according with Fig. 2	F. of S. unstable discon. of Fig. 2 for design condit. (1)	Unstable discon. according with Fig. 3	F. of S. unstable discon. of Fig. 3 for design condit. (1)	Half of excavation ≈ 10 m deep	End of Excavation ≈ 20 m deep
NORTH	Plane P_1 Wedges W_8+W_9	F. of S. > 1,2 See recommendations during construction specified by design. Table 2	Planes P_1+P_2 Wedges W_1+W_2+W_3+ W_4	F. of S > 1,2	No major stability problems. Partial failure of blocks less than 3 m³.	Plane P_2 failed (approx. Vo. 400 m³) See text for discussion
SOUTH	Wedge W_7		Plane P_3 Wedges W_5+ W_6		Wedge W_1 failed (approx. vol. 15 m³). See text for discussion	Minor stability problems. Partial failure of blocks less than 3 m³
WEST	Plane P_4		Wedges W_1+ W_2+ W_3 + W_4			
EAST	None	-	none	-	hauledge ramp	hauledge ramp

(1) For Short term stability Conditions, F. of S. $\geq$ 1,2
angles of slope faces 60°, $\emptyset$ = 30° or 25°
C = 20 KN/m^2, and no water pressures.

TABLE 2.- COMPARAISON OF THE RECOMMENDATIONS GIVEN BY THE DESIGN AND THOSE FOLLOWED DURING EXCAVATION

DESIGN RECOMMENDATIONS	CONDITIONS DURING EXCAVATION
a) Necesary to excavate under geotechnical supervision	No geotechnical supervision
b) Diversion of surface waters out side area of excavation and to proceed it during the dry season	Surface waters were not diverted. Excavation was carried out during wet season
c) Smooth blasting and presplitting techniques	Conventional and some uncontroled blasting.
d) Gunite to be applied to all slopes just after cutting. Additional reinforcement e.g. bolting and mesh, to be decided by the technical supervision	Some gunite was applied to limited areas and at the end of the excavation



J_E, J_W, J_S and J_N : Slope faces of the cutting

○ Poles of discontinuities

● P_1 Dip direction of discontinuities

▲ w_7 Wedge intersection

ϕ Circle of internal friction

Fig. 2. – Stereoplot (Schmidt) of potential unstable planes and wedges anticipated by the design

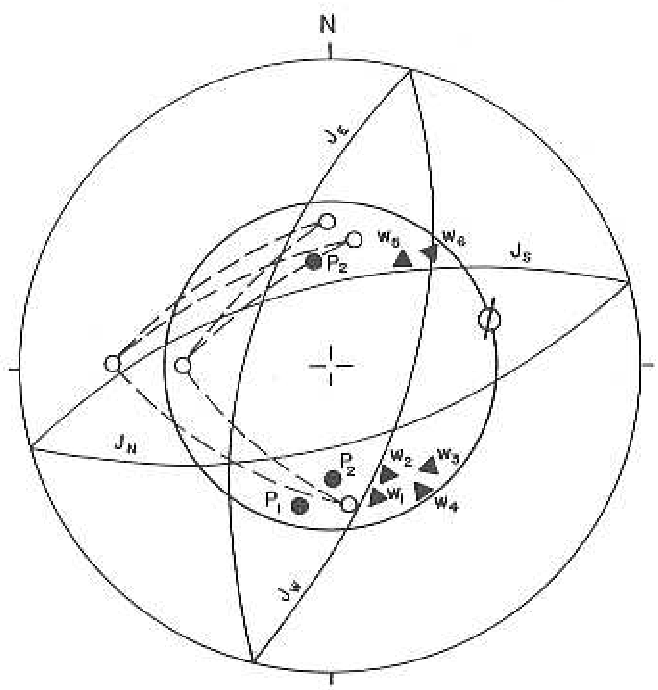


Fig. 3. – Stereoplot of planes and wedges which presented inestability during the excavation. Note that only plane P_2 and wedge w_1 shown major failure problems (see legend from fig. 2)

BEHAVIOUR DURING EXCAVATION

The contractor decided to excavate the pit walls to 60 degrees in all slopes and under the conditions listed in Table 2 (right column). When it was reached 10 m depth (half of the excavation) wedge W_1 failed on the West slope. Neverthelesses the excavation continued under the same conditions and when it was 20 m deep (almost end of the excavation) plane P_2 failed on the North slope. Both failures allowed us to check the stability conditions during construction and to compare it with those predicted during the stage of design.

Fig. 3 and Table 1 show stability analysis of the discontinuities existing during excavation. In all cases the factor of safety - could be higher than 1.2 under design conditions (Table 2). However, wedge W_1 and Plane P_2 failed.

Fig. 5 shows the rock slide of wedge W_1. This wedge failure gave place to approximately 15 m^3 of rubble rock on the West face of the cut. Fig. 3 and 4, and Table 3 present a synthesis of the stabily analysis. Water pressures distributions are indicated in Fig. 4 for (C' + Ø') condition. In this case an acceleractiong "a" due to blasting expansion wave was - also considered.

From these results it seems that overloaded blasting has been the main factor contributing to failure of wedge W_1. If it is assumed an acceleration of 0.20 g as usual for this situation, just a lower value, e.g. 0.18 g but with water forces acting on the slope can lead to a factor of safety of 1.0. However without water pressures it should be required an acceleration higher than 0.20, to get a factor of safety of 1.0 (Table 3).

Fig. 6 shows the excavation after the failure of plane P2 located on the North slope -

J_w – Plane of the slope face
J_1 and J_2 – Planes of wedge discontinuities
J_T – Plane of the tension crack
P_1 and P_2 – Poles of J_1 and J_2
$N_{1,2}$ – Resultant of normal components of weight to J_1 and J_2
R – Resultant of weight and hydraulic force on J_T
RG – Horizontal component of blasting force
FS_1 – Factor of safety only with ϕ
FS_2 – F. of s. with ϕ + C
FS_3 – F. of s. with ϕ' + C'

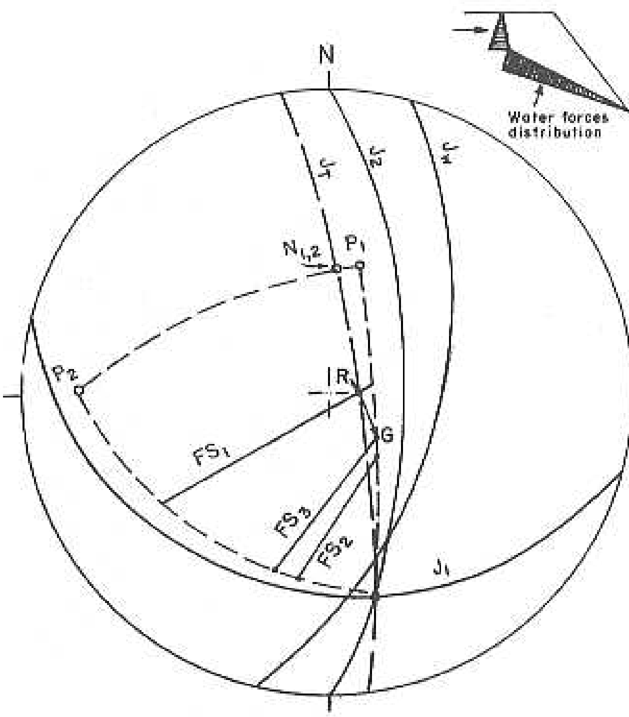


Fig. 4.– Graphical analysis of the stability of wedge w_1 acting internal friction, cohesion, water pressure and acceleration due to blasting

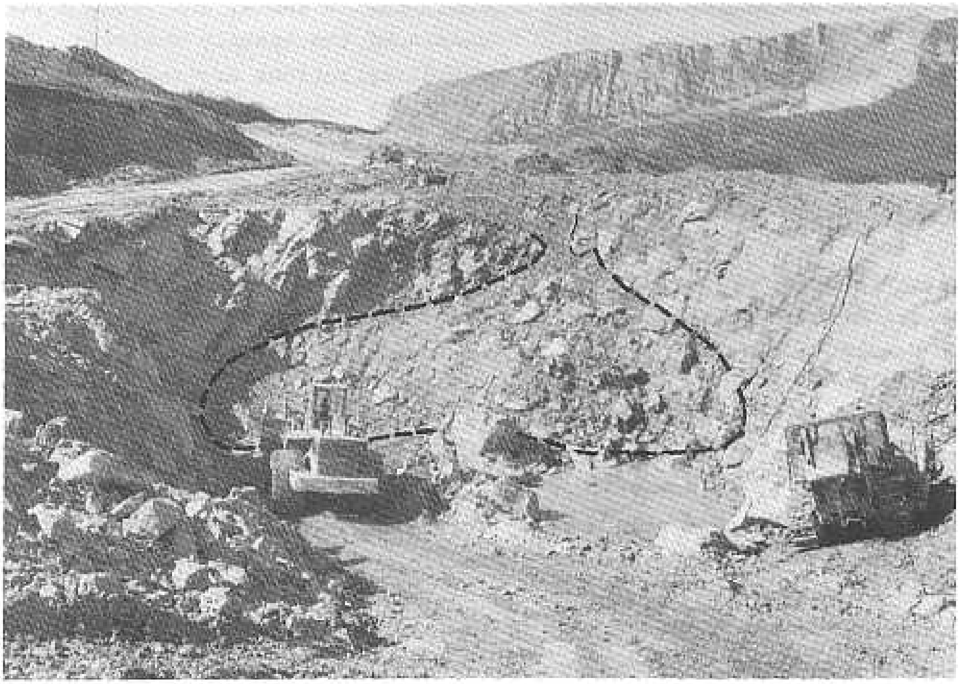


Fig. 5.- Wedge failure (w_1) on the West face of the excavation

Fig. 6.- Plane failure (P2) on the North face of the excavation (see man in the circle for scale).

when it reached a depth of 20 m. The volume of fallen rocks was about 400 m^3. Table 3 summarized the stability conditions of this plane and it is shown that for design purposes (Table 2) the factor of safety should be higher than 2.0. It was concluded that the main reasons of the failure was the presence of water in a tension crack. Other possible reasons could be the loads of the lorries along a road sited just on the top of the excavation (Fig. 6), and the loosening of the rock by conventional blasting patterns.

TABLE 3.- STABILITY CONDITIONS OF WEDGE W_1 AND PLANE P2

CONDITIONS	FACTOR OF SAFETY	
	WEDGE W_1	PLANE P2
Ø	1,11	-
C + Ø	2,04	1,21
C + Ø + tension crack on top	-	1,17
C' + Ø'	1,70	0,81
C + Ø + a (a = 0.28 g)	1,0	-
C' + Ø' + a (a = 0.18 g)	1,0	-

DISCUSSION AND CONCLUSIONS

The expansive nature of the material and clayey fillings along the discontinuities have largely controlled the behaviour of the Olazagutía marls. In spite of borehole drilling cores and excelent outcrops some of the discontinuities shown different orientations than those anticipated during site investigations. It was demostrated that if design recommendations were maintained no major failures could be occurred during construction. The main reasons of the failure of wegde W_1 was the conventional blasting without any pre-splitting or smooth firing and the overloaded charges used. It was shown that if a geotechnical assistance would be carried out during construction the blasting effects could be minimized by means of a drilling pattern, being able to accomodate blasthole downlines favoring northward plunging (Fig. 4). Water pressures along a tension crack on the top of the slope seems to be the main reason of the failure of plane P2. Other causes could be the conventional blasting and the loads of the lorries passing along a road just on top of the slope.

In this case some questions has been raised up:

- The importance of controlling the seepage of rainfall water in the excavation area.

- The need to of undertaking smooth blasting and to consider the effects of blasting in the stability analysis of rock slopes in order not only to optimize the design but to recomend the best blasting schemes, too.

- The need of geotechnical advice during construction as a part of the design in order to accomodate the conditions anticipates during design and those prevailing during construction.

ACKNOWLEDGEMENTS

The authors are grateful to Dr. A. Uriel from the Laboratorio del Transporte y Mecánica del Suelo (Spain) for his comments during the preparation of this paper, and to Dr. F. Ojeda, Head of the Geotechnical Dept. of The Empresa Nacional Adaro (Spain), for his help.

REFERENCES

- González de Vallejo, L. and Berzal, J. L. (1976).- Geotechnical properties of the Olazagutía Marls (in spanish). Nat. Symp. on Soft Rocks. Soc. Española de Mec. de Rocas Madrid, I, A-2.

- Hoek, E. and Bray, 1974. Rock Slope Engineering. Inst. Mining. and Met. London.

- John, K. W. 1971.- Three Dimensional Stability Analysis of Slopes in Jointed Rock. Int. Symp. Open Pit Mine Planning, Johanesburg.