Instability of natural slopes in Puerto Rico
L'instabilité des pentes naturelles à Porto-Rico

P.C.LAMBE, Associate Professor, North Carolina State University, Raleigh, North Carolina, USA
F.SILVA, Consulting Geotechnical Engineer, Lexington, Massachusetts, USA
W.A.MARR, President, Geocomp Corp., Concord, Massachusetts, USA
T.W.LAMBE, Consulting Geotechnical Engineer, Longboat Key, Florida, USA

SYNOPSIS Six natural slopes in residual soil derived from limstone failed one to nineteen years after the excavation that formed the slopes. The paper reviews the failure geometry, soil shear strength, and pore pressure based upon investigations made during a geotechnical safety program. Sliding occurred along stiff fissured clay layers dipping downslope at 7°-18°. Back analyses indicated that at failure the fully softened shear strength was mobilized in the clay and the pore pressures were greater than zero.

INTRODUCTION

In 1954 the Commonwealth Oil Refining Company (CORCO) began constructing an oil refinery on Puerto Rico's southern coast. Excavation of the limestone residual soil to make flat patios for oil storage tanks steepened existing natural slopes. Figure 1 shows a N-S profile through the refinery. Beginning in 1958 natural slopes started failing and by 1977 ten landslides had occurred. Since these landslides threatened pipelines, power lines, roadways, and storage tanks CORCO had to prevent further failures.

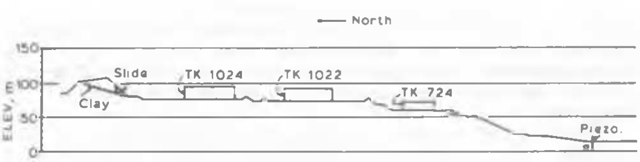

Fig. 1 North-South Section through CORCO Refinery

The ten failures fall into two categories, namely: Type 1, 4 slides in homogeneous soil and Type 2, 6 slides along a stiff fissured clay layer. This paper describes certain aspects of Type 2 failures: Failure geometry (determined from surveys and exploration of failures), shear strength and pore pressures (determined from lab and field measurements and inferred from back analyses of the failures). Figure 1 shows the location of Type 2 failures located south of Tank (Tk.) 724 and north of Tk. 1024. The slope south of Tk. 724 failed 10 years after excavation while the slope north of Tk. 1024 failed one year after excavation.

FAILURE GEOMETRY

In January, 1975 a nineteen year old slope north of Tk. 973 failed. Figure 2 shows sections before and after the slide. The Tk. 973 slope had an average surface angle of 47°, a height of 9.8 meters, a tension crack height of 2.4 meters, and slid along a clay layer dipping 7° downslope. A refinery worker observed that it "took a few hours for the slide mass to move several feet". Movement continued for several days after the initial slide. The failure geometry consists of three soil wedges sliding along a planar clay layer.

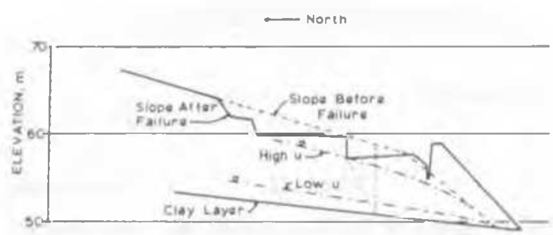

Fig. 2 Tk. 973 Slide

Figure 3 shows the section for a slope failure north of Tk. 1024. The Tk. 1024 slope failed in November, 1975 - only one year after excavation. The failed slope had an average slope angle of 46°, a height of 18 meters, a 12.2 meter high tension crack and a clay layer dipping 18° downslope. At Tk. 1024, a single block slid along the clay layer.

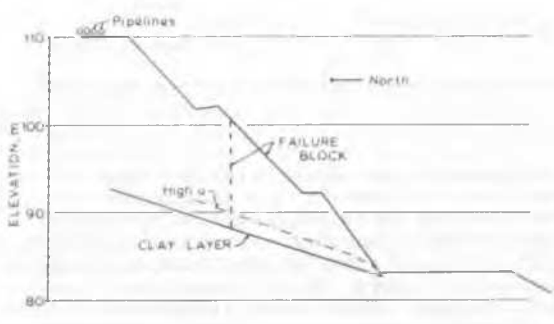

Fig. 3 Tk. 1024 Slide

These two slope failures intensified a geotechnical safety program initiated in 1974. Surveys of the Tk. 973 and Tk. 1024 slides provided the failure geometry while data for the other four Type 2 failures provide a less complete description of the failure geometry. At Tk. 707 the clay dipped at about 12° and at Tk. 724 the clay dipped at 8°-12°. The measured clay layer dips at other locations in the refinery ranged from 7°-18°.

Figure 4 shows the two-block failure geometry used for analyzing the slides. The surface profile data can define H_C, H, and θ for all the failures. We determined the sliding surface lengths after measuring or estimating the angles β and η. In the Tk. 1024 failure we measured an η of 90° and we used an estimated η of 62.5° for the other five failures. Table 1 summarizes the geometric values used in analyzing each of the six failed slopes.

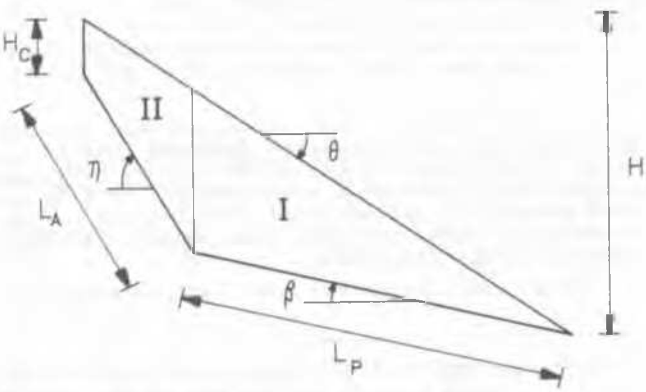

Fig. 4 Two-Wedge Failure Geometry

TABLE I

Summary of Age and Failure Geometry for CORCO Landslides

	701	707	724	973	981	1024
Age, yrs	9	10	10	19	4	1
θ, deg.	38.5	24	33	47	39	46
H, m	9.8	9.8	8.7	9.8	9.8	18.0
β, deg.	12.5	12.5	12.5	7.0	12.5	18.0
L_P, m	8.4	28.2	11.9	16.8	10.1	18.0
L_A, m	9.0	2.3	3.8	7.0	7.3	0.0
H_C, m	0.0	1.5	2.7	2.4	2.1	12.2
Area I m^2	19.4	75.7	28.6	100.2	28.2	117.6
Area II m^2	9.8	2.8	6.4	17.4	13.3	0.0

STRENGTH

The soils forming the CORCO hillside have weathered from the lower Ponce limestone formation. During deposition of the 900 meter thick lower Ponce limestone, rivers brought in sand, silt and clay. Following climatic changes, 400 meters of the upper Ponce limestone deposited over the lower Ponce limestone. Crustal movements then raised these sediments causing subsequent erosion that removed all of the upper Ponce limestone from the CORCO site. Tectonic action also caused folding along an E-W axis with limestone layers now dipping south at 5°-20°.

Borings made for construction on the hillside show two soil types:

(1) Stiff-Fissured Clay - A yellow brown, very plastic clay which breaks into small blocks when handled.

(2) Sandy Silt - A yellow, brown and tan clayey sandy silt with rock fragments, occasional clay pockets, and boulders.

Figure 1 shows the location of stiff fissured clay in south-dipping layers which vary from 0.3 to 4.6 meters thick. The stiff fissured clay, a CH material according to the USCS, has liquid limits of 65 to 105 and plasticity index values of 40 to 80. X-ray diffraction tests indicate that clay minerals make up to 60% of the soil, with three times more montmorillinite than kaolinite. Consolidation tests measured maximum past pressures of 280 kPa for a sample from Tk. 1024 with a natural water content of 47.5% and 430 kPa for a sample from Tk. 973 with a natural water content of 31%. These maximum past pressures suggest erosion of only 15 m to 23 m of overburden instead of 400 m as expected from the geologic history. We could not determine if the clay accumulated along bedding planes as a result of weathering or deposited in layers and pockets during the formation of the lower Ponce limestone.

We tested both intact and remolded soil sampled from the Tk. 707, Tk. 973, and Tk. 1024 slopes in direct shear. To reach large horizontal displacements the direction of shear box travel was repeatedly reversed while only counting travel in one direction. The samples were sheared at a displacement rate of .0012 cm/min for early tests and slowed to 0.000049 cm/min for later tests. Tests at both displacement rates gave similar results. The intact samples exhibited strain softening behavior.

Based upon over 20 years of experience on slopes in London Clay Skempton (1977) has developed a series of guidelines for selecting the governing shear strength. Using the results from reversible direct shear tests he recommends three straight line strength envelopes representing three different stress-strain stages for the overconsolidated clay. He calls these envelopes the peak, the fully softened and the residual strengths. Figure 5 summarizes the results of direct shear tests on intact and remolded samples of the CORCO stiff fissured clay with three straight lines representing the peak, fully softened, and residual envelopes.

Triangles represent two direct shear tests performed on samples including the field failure plane. The sample trimmed from the Tk. 707 sliding plane exhibited an initial peak strength higher than the eventual residual strength while the sample trimmed from the Tk. 973 sliding plane exhibited its residual strength during the first shear. Skempton (1984) says direct shear test results on samples trimmed from field shear planes provide a good measure for the residual friction angle.

The remainder of the hillside soils consist of the sandy silt which generally has a cemented cap and properties that vary in both the vertical and horizontal directions. The consistency of the sandy silt varies from granular to lightly cemented. The large cemented chunks made laboratory testing difficult. We combined direct shear test results on laboratory compacted samples of sandy silt with the cohesion inferred from tension crack heights observed at the six failed slopes. The direct shear tests measured friction angles of 33° to 36.5°. For analyzing the failure we used ϕ' of 35° and $c' = \gamma_T H_C / (2\tan(45+\phi'/2))$. With a total unit weight of 19.52 kN/m^3 (measured in the intact material near Tk. 1024) and ϕ' of 35°, c' equals $5.1 H_C$ kPa.

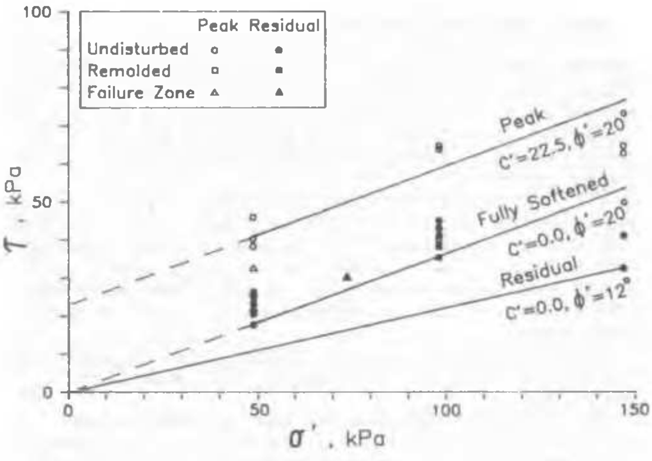

Fig. 5 Direct Shear Test Results

PORE PRESSURES

At the start of the failure investigation we installed Casagrande-type standpipe piezometers at several hillside locations. Only those instruments installed at elevation zero recorded a positive pressure head. Figure 1 shows the location of a piezometer which measured a total head equal to elevation zero. We attempted to measure the pore pressure in the clay layer using piezometer sensors constructed of a high air entry ceramic. The piezometers cavitated 2 to 14 days after installation and measured negative pore pressures of -41 kPa to -48 kPa before cavitation. During periods of dry weather the clay experiences negative pore pressure.

Figure 6 indicates the failure dates on a rainfall histogram with intensity measured over three month periods. The failures at Tks. 981, 973 and 1024 followed wet quarters while the failures at 701, 707, and 724 followed dry quarters. Although related to rainfall the actual time of failures cannot be directly correlated with rainfall events. Brand et al (1984) showed that effective correlations between rainfall and landslides in Hong Kong residual soils require accurate observations of failure times and hourly rainfall measurements made near the failed slopes. We do not have these records for the CORCO slopes.

After heavy rainfall refinery workers observed water flowing from the slopes above and along the clay layers. Following the failure at Tk. 707 water flowed from the toe of the slope providing indirect evidence at CORCO of positive pore pressures. Field and laboratory permeability test results show that the clay acts as a low permeability layer within the residual soil. While heavy precipation causes time varying seepage through the sandy silt we used the principles of steady state flow to determine the shape of the phreatic surface. Pavlovsky (Harr, 1963) proposed an exponential equation to describe the top line of seepage for horizontal seepage along a sloping impermeable boundary. At CORCO we used Pavlovsky's equation to calculate the top line of seepage for different average positive pore pressures. Figures 2 and 3 show three of these calculated seepage lines referenced as either the high u or low u lines.

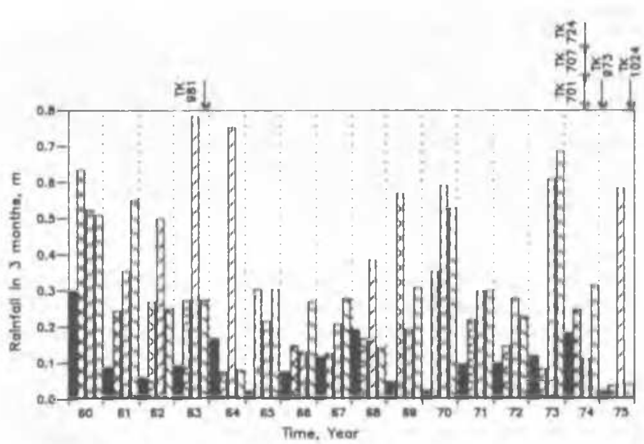

Fig. 6 Rainfall Histogram

ANALYSIS

To analyze the stability of a slope, the engineer needs to determine three fundamentals - Geometry, Strength and Pore Pressure. At CORCO we had:

(1) GEOMETRY - Well defined surface geometry before and after the landslides. Complete subsurface geometry for the Tk. 1024 slide, partial information for the other slides.
(2) STRENGTH - The strength envelopes shown in Fig. 5, based on laboratory measurements.
(3) PORE PRESSURE - Field measurements of negative pore pressures in stiff fissured clay during wet periods and evidence of zero or positive pore pressures during rainy periods.

Our analyses using these data indicate that the fully softened strength and positive pore pressusre in the stiff fissured clay represent the probable conditions at the time of failure. We can get a safety factor equal to one with these conditions.

The failure mass was divided into two wedges as shown in Figure 4 and analyzed with an inter-slice force inclination of ϕ' equal to 35°. We solved for the factor of safety using the equations of vertical and horizontal force equilibrium. For cases involving horizontal flow along the clay layer, postive pore pressures acted along all sliding surfaces located below the top seepage line. For cases involving no perched water, negative pore pressures acted only along the stiff fissured clay layer.

We analyzed the failed slopes in two different ways; using zero pore pressure acting along all sliding surfaces and calculating the average failure stresses along the clay layer; and using either the fully softened strength or residual strength acting along the clay layer and calculating the corresponding failure pore pressures.

Figure 7 portrays the average failure stresses acting along the clay layer for the six failures using zero pore pressure in the clay layer. We divide the six slides into two groups; first the slopes near Tanks 701, 981, and 1024 have stresses close to the fully softened strength envelope and second, the slopes near Tanks 724, 707, and 973 have stresses close to the residual strength envelope. Of the first group of slopes only Tk. 701 was more than than 5 years old at failure while the second group of slopes were all more than 10 years old at failure. Figure 7 suggests that analyzing slopes using the residual strength and zero pore pressure in the clay provides a lower bound for design of remedial measures.

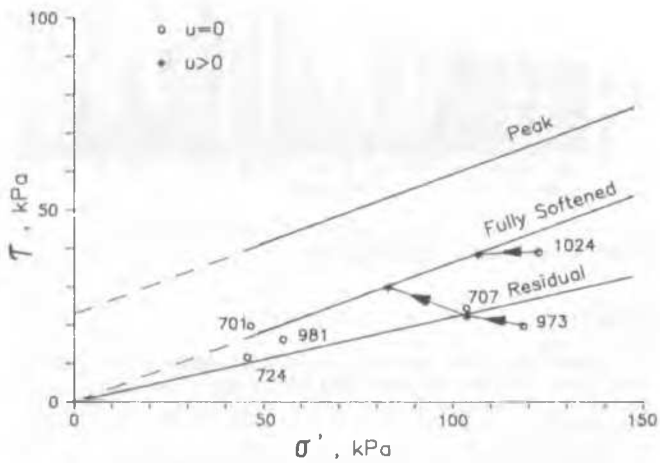

Fig. 7 Back Analysis of CORCO Landslides

If the failure pore pressures in the slope differ from zero then the average stresses in the clay will change. On Figure 7 for slopes Tk. 973 and Tk. 1024, the average stresses move from the circle representing zero pore pressure to the crosses labelled as either high u or low u corresponding to the top lines of seepage portrayed on Figures 2 and 3. The high u case represents the acting pore pressure if the fully softened strength controls while the low u case represents the acting pore pressure corresponding to the residual strength. Table 2 summarizes the calculated average failure pore pressures in kPa corresponding to both the fully softened strength and the residual strength. If the fully softened shear strength controls then average failure pore pressures vary from -5.9 kPa to 38.2 kPa. If the residual strength governs the failure pore pressures vary from -63.7 kPa to 11.8 kPa.

TABLE II

Summary of Back Analyzed Failure Pore Pressures

Slope	High u, kPa $\phi' = 20°$	Low u, kPa $\phi' = 12°$
701	-5.9	-43.1
707	15.7	-5.9
724	11.8	-7.8
973	38.2	11.8
981	4.9	-23.5
1024	15.7	-63.7

SUMMARY AND CONCLUSIONS

At the CORCO refinery six landslides in slopes constructed by excavating into a limestone residual soil hillside failed along a stiff fissured clay layer. Back analyses of the six failures have led to the following four conclusions.

(1) During dry weather the clay layer experiences negative pore pressure. Percolating rainwater dissipates the negative pore pressures and triggers landslides. Water from pipeline leaks and other industrial activities possibly contribute to the dissipation of negative pore pressures.

(2) We found no simple correlation between the quarterly rainfall intensity and landslide occurrence.

(3) Conservative design can be based upon using zero pore pressure along the clay layer and residual strength parameters for the clay while including proper internal drainage and channelling surface runoff.

(4) It seems probable that the fully softened strength parameters govern first time slides at CORCO and positive pore pressures exist in the stiff fissured clay at the onset of instability. Future work in these slopes should concentrate on describing the hydrogeology and on measuring the maximum pore pressures in the clay during heavy rainfall.

REFERENCES

Brand, E.W., Premchitt, J., Phillipson, H.B. (1984). Relationship between Rainfall and Landslides in Hong Kong. Proc. 4th Symp. Landslides, Toronto, Vol. 1, pp. 377-384.

Harr, Milton E. (1962). Groundwater and Seepage. McGraw-Hill, New York.

Skempton, A. W. (1977). Slope Stability of Cuttings in Brown London Clay. Proc. 9th Intl. Conf. Soil Mech., Tokyo, Vol. 3, pp 261-270.

Skempton, A. W. (1984). Residual Strength of Clays in Landslides, Folded Strata and the Laboratory. Geotechnique, Vol. 35, No. 1, pp. 3-38.