The First Terzaghi Oration

The international community of geotechnicians, and their official organization, the International Society for Soil Mechanics and Foundation Engineering, have felt the pervading influence of Terzaghi's teachings and personality so deeply and closely, that imperceptibly they allowed time to go by without instituting a minimum adequate tribute to him and his work. To those of us who had the priviledge of first-hand contacts with him, it would seem partly as if it were unnecessary, impossible, and presumptuous to single out one gesture of homage for so vast a debt and deep a sentiment.

However, at this very special occasion, when we rejoice and pride ourselves on establishing the important milestone of the Golden Jubilee Conference, we cannot avoid recognizing the responsibility of the past and present, to the future. Ever-growing numbers of junior colleagues will be grateful for our instituting a continually renewing perpetual tribute that ennobles our beginnings, small, but sparked by the genius and hard work of Terzaghi. The Officers of the Society thus decided to create the Terzaghi Oration, to be delivered from now on at the opening ceremonies of each International Conference, as a living tribute to Terzaghi. The nomination of the Terzaghi Orator is intended to be the highest professional recognition of our International Society.

It is by an auspicious coincidence that the nomination of the First Terzaghi Orator fell upon Professor T. W. Lambe, eminent professor and consulting engineer, whose contributions to the world of geotechnicians emanated from M.I.T., the same Institution that became the center for the development of Soil Mechanics when in 1925 its President invited the Austrian engineer Terzaghi to lecture, conduct research, and develop the new field through the dynamic interaction between teaching, professional consulting work, research, and publication. "Mens et Manus".

Both the Institution and the First Terzaghi Orator stand out as select reminders of what we cherish and strive to emulate, from the spirit of the founders of our Society: internationalism, dynamic creativity, persistence of effort, service, self-criticism more severe than any that may be meted out by others, zest for life, and the joy at each error discovered as but another challenge for further quest and development.

VICTOR F. B. DE MELLO
President ISSMFE

* 4 *

Amuay landslides

T. WILLIAM LAMBE, Turner Professor of Civil Engineering (Emeritus),
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SYNOPSIS This Terzaghi Oration traces improvements made during 1955-1985 in the capability to predict landslides along the coastline of Amuay, Venezuela. Analyses, laboratory tests and field measurements (and observations) have resulted in advances in three aspects of the procedure of predicting landslides, namely: shear strength, pore pressure and expression of safety level. Advances in the profession's knowledge also contributed.

INTRODUCTION

I feel greatly honored that the International Society for Soil Mechanics and Foundation Engineering selected me to deliver the first Terzaghi Oration. This document presents a written version of the Oration.

Surely all geotechnical engineers know of the enormous contributions Terzaghi made to our profession. Bjerrum, et al (1960), give an excellent account. I consider Terzaghi's most important contribution the development of the Effective Stress Principle. The Effective Stress Principle defines effective stress as total stress minus pore water pressure and states that soil behavior relates to effective stress.

I came into soil mechanics in 1940, four years after the first international conference held in Cambridge, Massachusetts. As a student at North Carolina State University I first heard of Terzaghi. While at the Massachusetts Institute of Technology, starting in 1943, I learned more of Terzaghi's work, initially through Donald W. Taylor. Later I went to Harvard to attend Dr. Terzaghi's lectures on engineering geology and on applied soil mechanics. On some half a dozen projects I assisted Dr. Terzaghi in a minor way by carrying out special assignments, such as running laboratory tests on grout mixtures in connection with the Aswan Dam.

During each of four academic years (starting in 1956) Dr. Terzaghi spent two weeks full time at MIT, one week in the fall and one week in the spring. He worked out of my office. In addition to giving lectures, Dr. Terzaghi reviewed in detail geotechnical research at MIT, especially work I performed or directed. Terzaghi's critical review of my work proved beneficial to me in later life - although at the time I found his reviews trying.

Through the Terzaghi Oration the International Society pays homage to Dr. Karl Terzaghi. In naming me the first First Terzaghi Orator, President de Mello charged me to:

> Present a case history from consulting which emphasizes a systematic attack of a complex problem using geology, theoretical principles, laboratory tests and field measurements.

My Terzaghi Oration focuses on the landslide problem at Amuay, Venezuela.

CLIFFSIDE INSTABILITY

Since 1954 my associates (mostly from MIT) and I have worked on geotechnical problems at Amuay, Venezuela for LAGOVEN S.A., formerly the Creole Petroleum Corporation. The most important and difficult geotechnical problems at Amuay result from instability along the 12 kilometers of cliffside.

Figure 1 shows a typical section of the Amuay cliffside which varies in height from 16 to 26 meters. It also shows a typical landslide which has a wedge shape, with the failure surface going through the brown fat clay near elevation +10. Industrial activity on top of the cliff adds water to the ground. The brown fat clay traps the water causing a rise in pore pressure - thus a drop in effective stress - and a small increase in shear stress. The increase in perched water therefore decreases the stability of the cliffside. Since 1957 fourteen landslides have occurred and large cracks - indicative of impending slides - have occurred at twenty locations.

During the past thirty years we have developed and executed a Geotechnical Safety Program (Lambe, Silva and Marr, 1981) which satisfactorily controls the landslide problem. This problem includes the following:

1. Establish performance criteria;
2. Assess safety by comparing predicted level of stability with performance criteria;

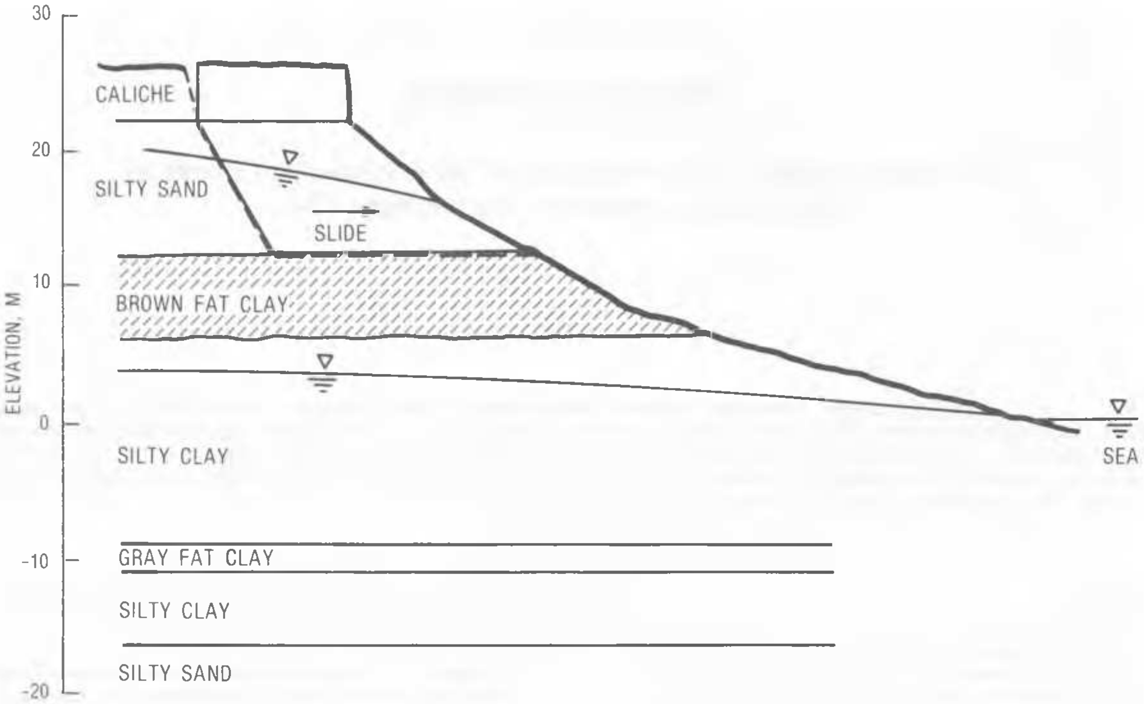


FIGURE 1. AMUAY CLIFF

3. Execute preventive measures where needed and justified.

To-date we have executed preventive and/or remedial measures at sixteen locations along the cliff.

Using field measurements and quarterly field inspections we choose locations and critical sections for study. For each chosen section, we prepare a "safety assessment", which depends on a "stability prediction".

To predict stability we use limiting equilibrium analyses. In developing and applying our stability prediction procedure for Amuay we have found three factors most important and troublesome. My Terzaghi Oration considers in turn these most important factors in the prediction technique, namely: clay strength, pore water pressure and expression of level of stability.

STRENGTH OF AMUAY CLAY

Skempton's work

Since the early 1940's Professor Skempton, aided by his students and associates at Imperial College, has studied the question: which strength acts in cuttings made in stiff fissured clay? Skempton (1977) summarized aspects of his past work and gave his current interpretations and recommendations.

Figure 2 from Skempton (1970) defines the strength parameters he recommends for analyzing cuttings in clay. Table I summarizes my interpretation of Skempton's recommendations. Figure 3 from Skempton (1977) presents the results of strength tests on brown London clay. These results support Skempton's recommendations.

In his 1977 paper Skempton states:

1. "We therefore return to the conclusion.... that the most appropriate laboratory parameters are those for the 'fully softened' or 'critical state' condition, which can be determined by measuring the strengths of remoulded, normally consolidated clay".

2. "....first time slides in London clay.... generally occur many years after a cutting has been excavated. The principal reason for this delay is the very slow rate of pore pressure equilibration; a process which in typical cuttings is not completed, for practical purposes, until 40 or 50 years after excavation".

Initial Approach

In 1957 a major landslide occurred in the Mene Grande property, 5 kilometers south of the LAGOVEN property. The first instability problems in LAGOVEN's refinery area occurred in the early 1960's. We used the available principles of soil mechanics, including the work of Skempton, to analyze Amuay landslides. I find it convenient to divide our studies into two stages, that before 1973 (Initial Approach) and that after 1973 (Stress Path Method). Note that our studies occurred concurrently with some of Skempton's. Thus,

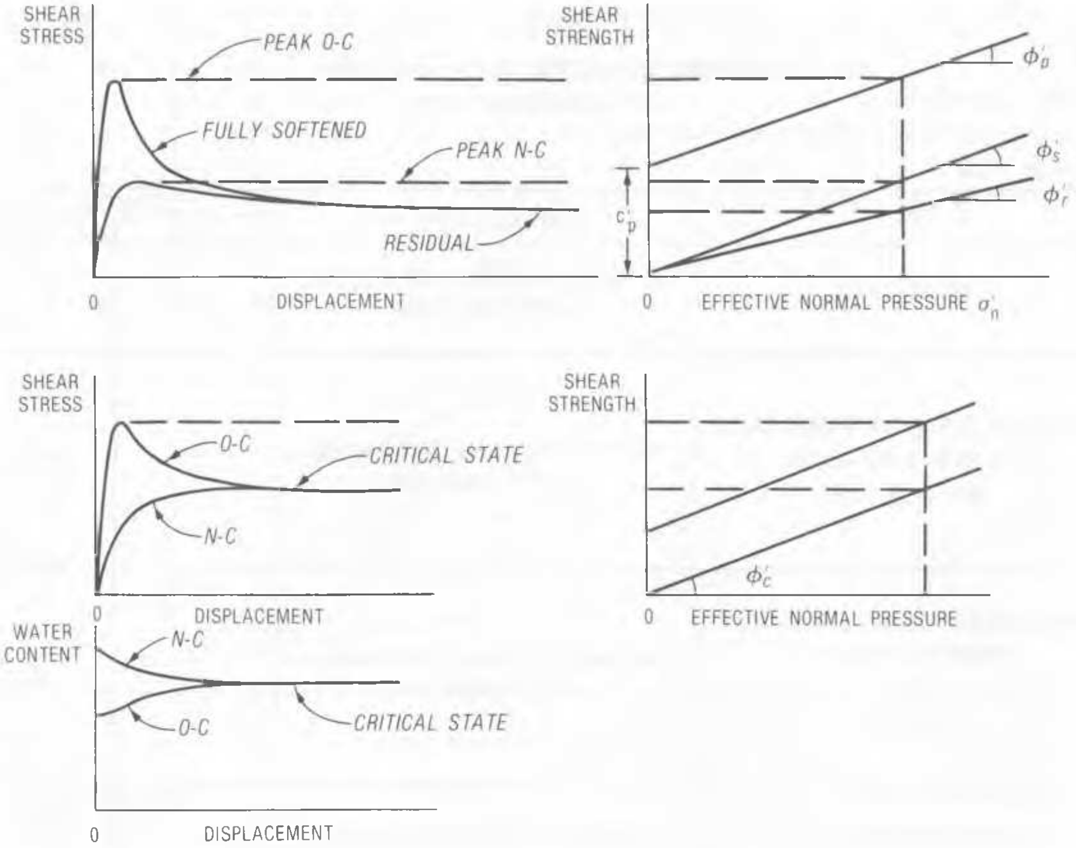


FIGURE 2. SHEAR CHARACTERISTICS (SKEMPTON, 1970)

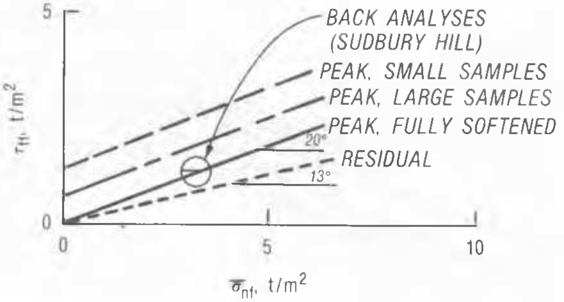


FIGURE 3. STRENGTH OF BROWN LONDON CLAY (SKEMPTON, 1977)

we did not have available Skempton's latest work when we developed our initial approach to strength selection.

Figure 4 presents the results of laboratory tests on Amuay brown fat clay and shows strength envelopes corresponding to those of Skempton given in Figure 3. Crespo (1973) ran strength tests on samples taken on and immediately next to the failure surface of an Amuay landslide. Figure 5 presents the results of his tests.

Figure 6 presents the results of back analyses of five Amuay landslides. We attempted to relate the back-figured shear strength with the age of the slope.

During the Initial Approach we used Figures 4 6 and attempted to select the strength for analyzing a slope on the basis of its "age". We used the concept of "residual factor", R, to express where the appropriate strength lay between the peak and residual values.

$$R = \frac{S_p - S_a}{S_p - S_r}$$

S_p = Peak Strength

S_a = Available Strength

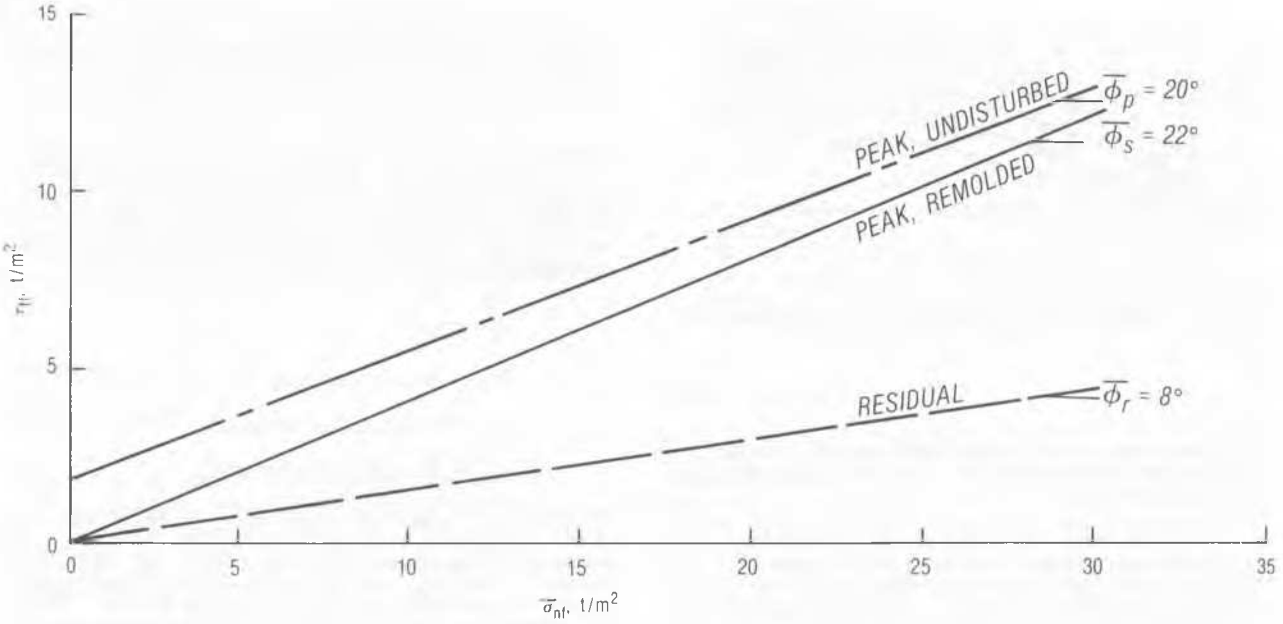
S_r = Residual Strength

We found our Initial Approach to selecting shear strength unsatisfactory for several reasons. How does one determine the age of a slope? Further, we do not believe that time constitutes the primary variable in the loss of shear strength. If one believes Figure 6, he should design all slopes using $\bar{\phi} = 8^{\circ}$. Nonsense!

TABLE I

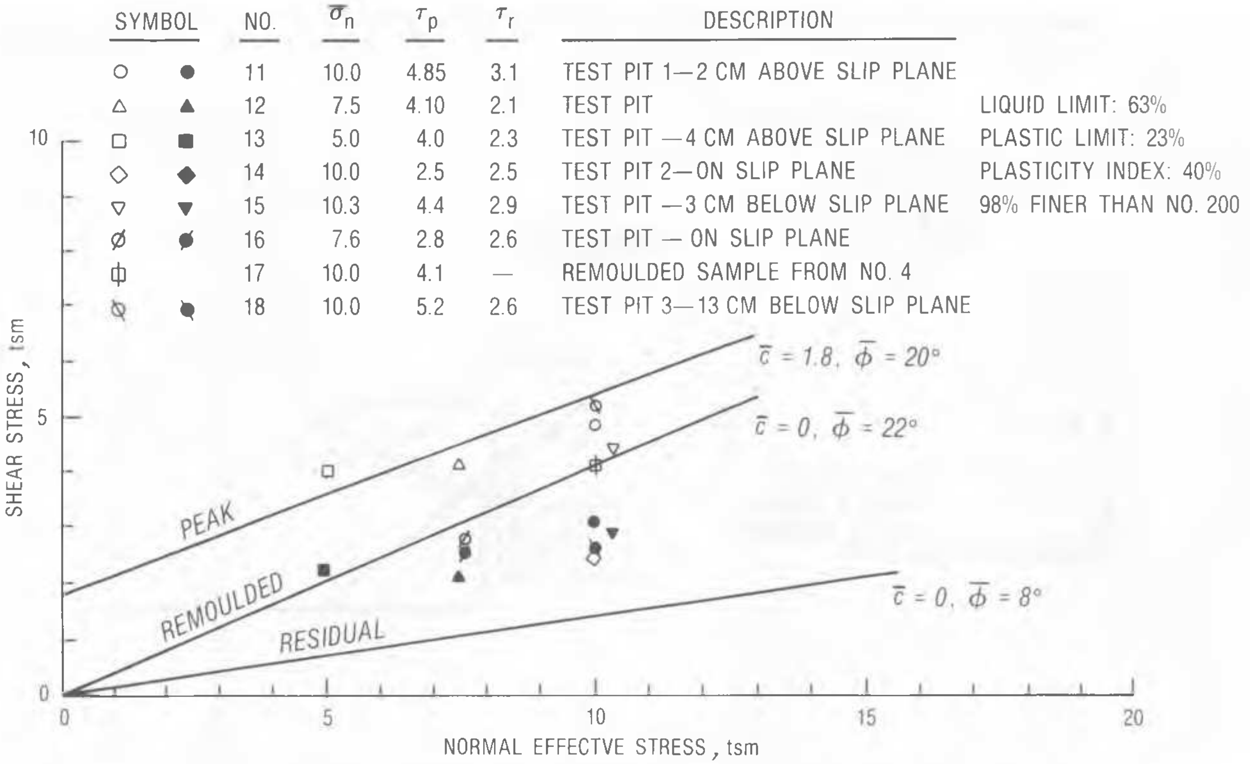
SKEMPTON'S RECOMMENDED STRENGTH FOR ANALYZING CUTS

SOIL	DEFORMATION HISTORY	STRENGTH	
		c'	ϕ'
INTACT CLAY	NO PREVIOUS LARGE DEFORMATION (FIRST TIME SLIDE)	c'_p	ϕ'_p
OVERCONSOLIDATED FISSURED CLAY NOT HIGHLY EXPANSIVE NOT HIGHLY OC	NO PREVIOUS LARGE DEFORMATION (FIRST TIME SLIDE)	c'_s	ϕ'_s
OVERCONSOLIDATED FISSURED CLAY HIGHLY EXPANSIVE OR HIGHLY OC	NO PREVIOUS LARGE DEFORMATION (FIRST TIME SLIDE)	0	ϕ'_r
OVERCONSOLIDATED CLAY	PREVIOUS LARGE DEFORMATION	0	ϕ'_r



BASED ON TESTS RUN PRIOR TO 1973

FIGURE 4. STRENGTH OF FAT CLAY



NOTE: ENVELOPES COME
FROM FIG 4

(FROM CRESPO, 1973)

FIGURE 5. STRENGTH OF AMUAY CLAY

Stress Path Method

As noted, the Initial Approach for selecting strength in predicting instability of the cliffside did not prove satisfactory. Thus, during the period 1974 through 1980 we conducted a second and much more detailed investigation into the strength of the Amuay fat clay.

Laboratory tests (Figure 7) confirmed Skempton's picture (Figure 2) of strength of clay. As seen in Figure 7, at very large deformation, fat clay in an undisturbed state and fat clay in a remolded state reach the same ultimate strength, i.e. residual strength.

Tests on samples of brown fat clay obtained at many locations at Amuay revealed a wide variation in plasticity index, PI. The PI varied from 20% to 50%. Data shown in Figures 8 and 9 indicate that the residual friction angle varied with both limits and normal effective stress. We thus replaced our single straight line envelope for the residual strength with a curved band.

Our Amuay experience led us to believe that the effective stress strength of the Amuay fat clay depended, not on <u>time</u> per se, but rather on the <u>stress-strain history</u> of the clay. We thus used the Stress Path Method to express numerically the geotechnical history of the Amuay clay. Figure 10 shows the results of this work. Figures 11 and 12 portray in terms of total stress path and effective stress path the history of Element B, close to the average element for a landslide.

We attempted (unsuccessfully) to carry a sample of clay slurry through the sequence of events shown in Figure 10. We had hoped to obtain strength, in terms of effective stress, for the stages from E through R. We had difficulty in duplicating in the laboratory the effects of the early events, especially "Desiccation".

At the same time we embarked on a program that proved very successful. We identified within the refinery area where the fat clay existed in the following stages: D, C, P_i, P_r and R. We obtained undisturbed samples from these locations and conducted laboratory strength tests. Figure 13 shows strength data from these tests and envelopes based on the test results.

Strength Selection

The procedure we now use to select strength for a stability evaluation differs significantly from that we used initially.

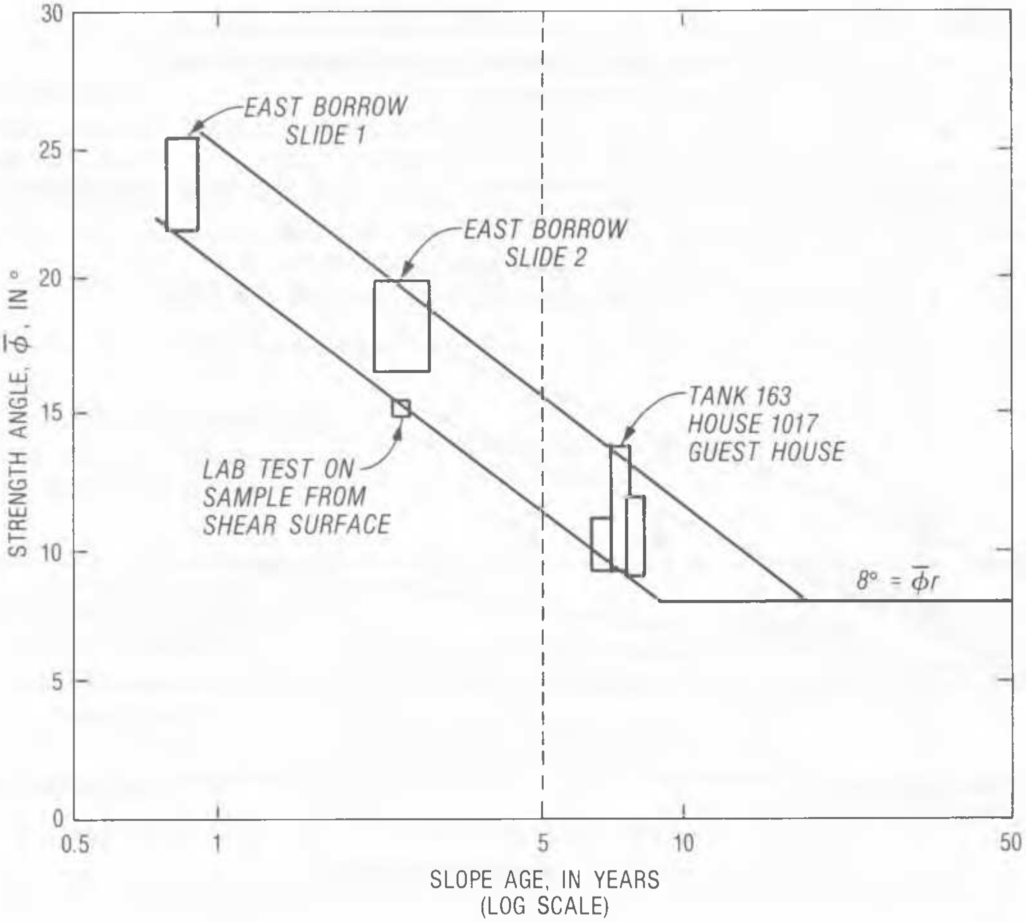


FIGURE 6. STRENGTH FROM BACK-ANALYSES

We now use the strength envelopes in Figure 13 along with our predicted normal effective stress for the time and condition appropriate for the evaluation. A later portion of this paper considers the selection of appropriate normal effective stress. This portion of the paper considers the strength parameter portion of the strength.

Figure 14 consists of a superposition of the initially determined strength envelopes (Figure 4) and the presently used strength envelopes (Figure 13). From Figure 14, one can detect the following significant differences between the initial envelopes and the present envelopes based on the Stress Path Method. These include:

1. The residual strength envelopes fall in a band, depending on the plasticity index of the clay.
2. The present strength envelopes curve.
3. The present strength envelopes reflect the stress-strain history - i.e. stage - of the clay.

Table II uses Figure 14 to tabulate strength for an average element in a landslide for three different stages. For the element selected and for the assumption of constant normal stress, we see from Table II that:

> The strength selected by our present procedure equals almost 2½ times that selected by our initial procedure. Not only do we feel that the present procedure gives a more nearly correct value but represents an enormous financial saving in the design of preventive measures.

One encounters difficulty in applying Skempton's recommendations (Table I) to select strength from Figure 14. At Stage C the Amuay clay has fissures, an OCR>5 and a PI≈40%. It appears as a "stiff fissured clay". As the clay goes to Stages P_i and P_c it softens and loses the effects of overconsolidation. One has difficulty in deciding which strength to select from Table I. We conclude that Skempton's recommendations guide one in planning tests to determine strength for a stability evaluation.

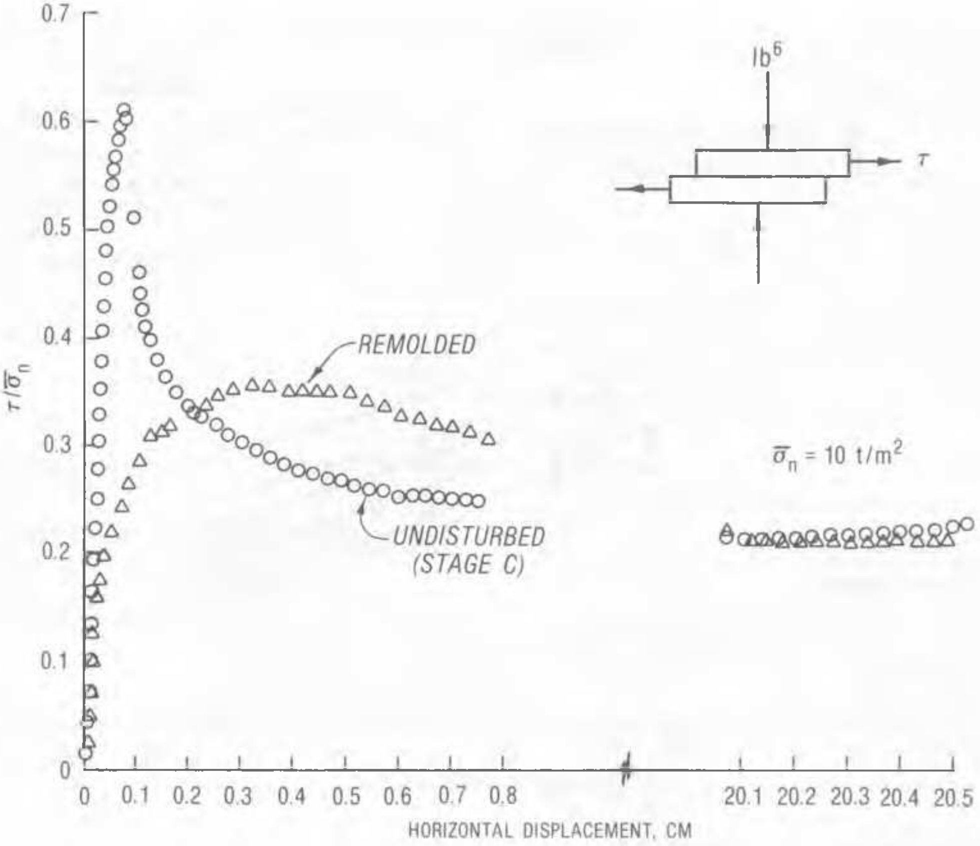


FIGURE 7. DIRECT SHEAR TEST

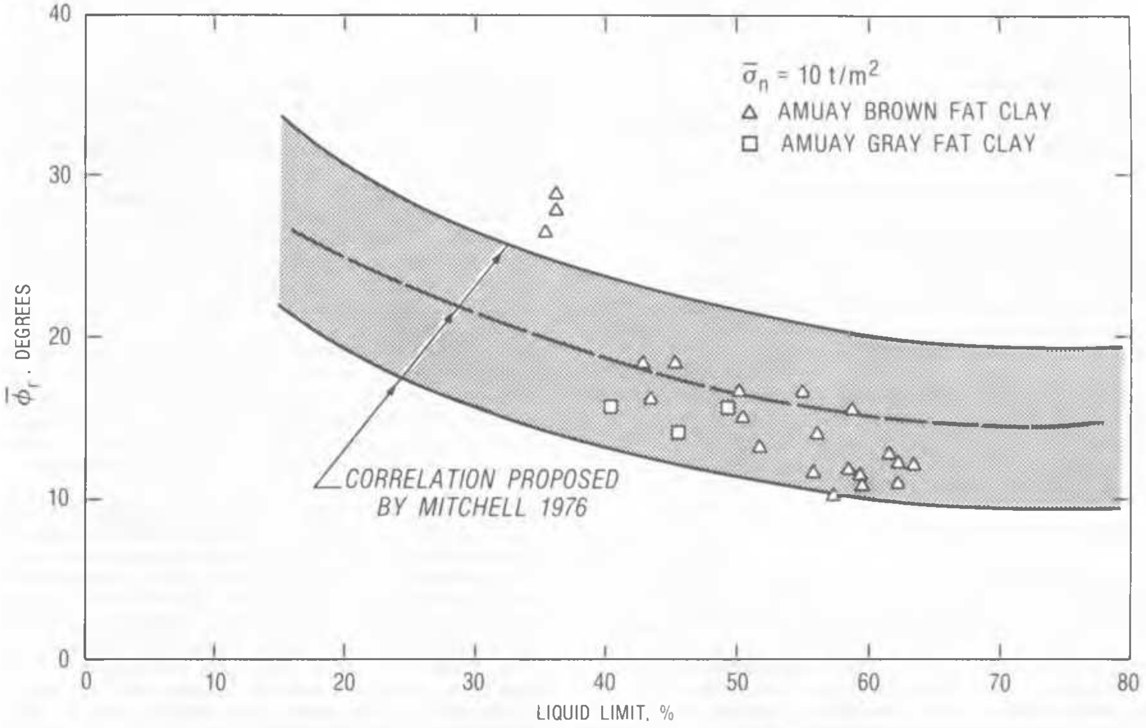
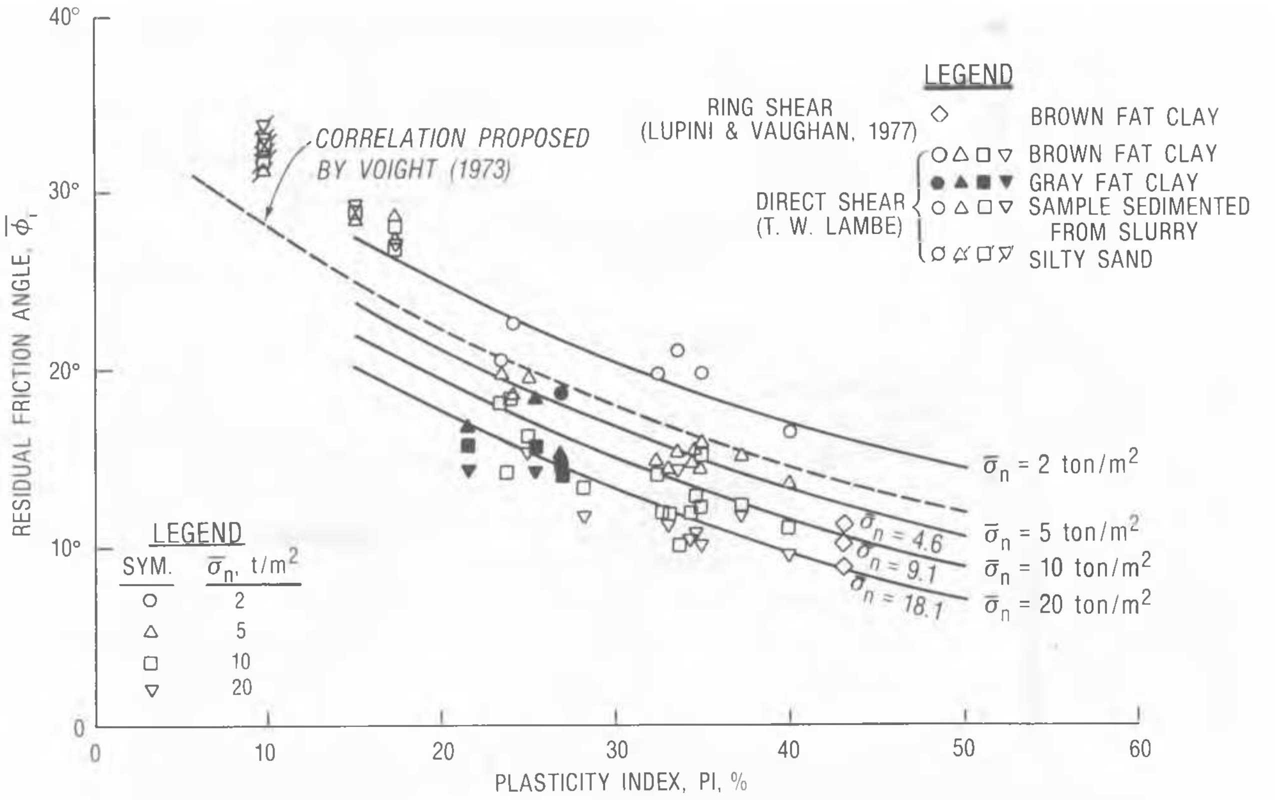


FIGURE 8. LIQUID LIMIT VS RESIDUAL $\overline{\phi}$



*SAMPLES OF BROWN FAT CLAY FROM STAGE D

FIGURE 9. RESIDUAL STRENGTH OF AMUAY SOILS

PORE WATER PRESSURE

For a soil (having no cohesion intercept) the shear strength, s, equals:

$$s = (\sigma - u) \tan \bar{\phi}$$

The total stress (σ) and pore pressure (u) equal those values acting on the failure surface at time of failure. The preceding section of this paper considered the choice of the effective stress strength angle, $\bar{\phi}$. This section looks at the pore pressure.

The student in soil mechanics learns to estimate pore pressure for a variety of situations including:

1. Static ground water;

2. Steady-state seepage;

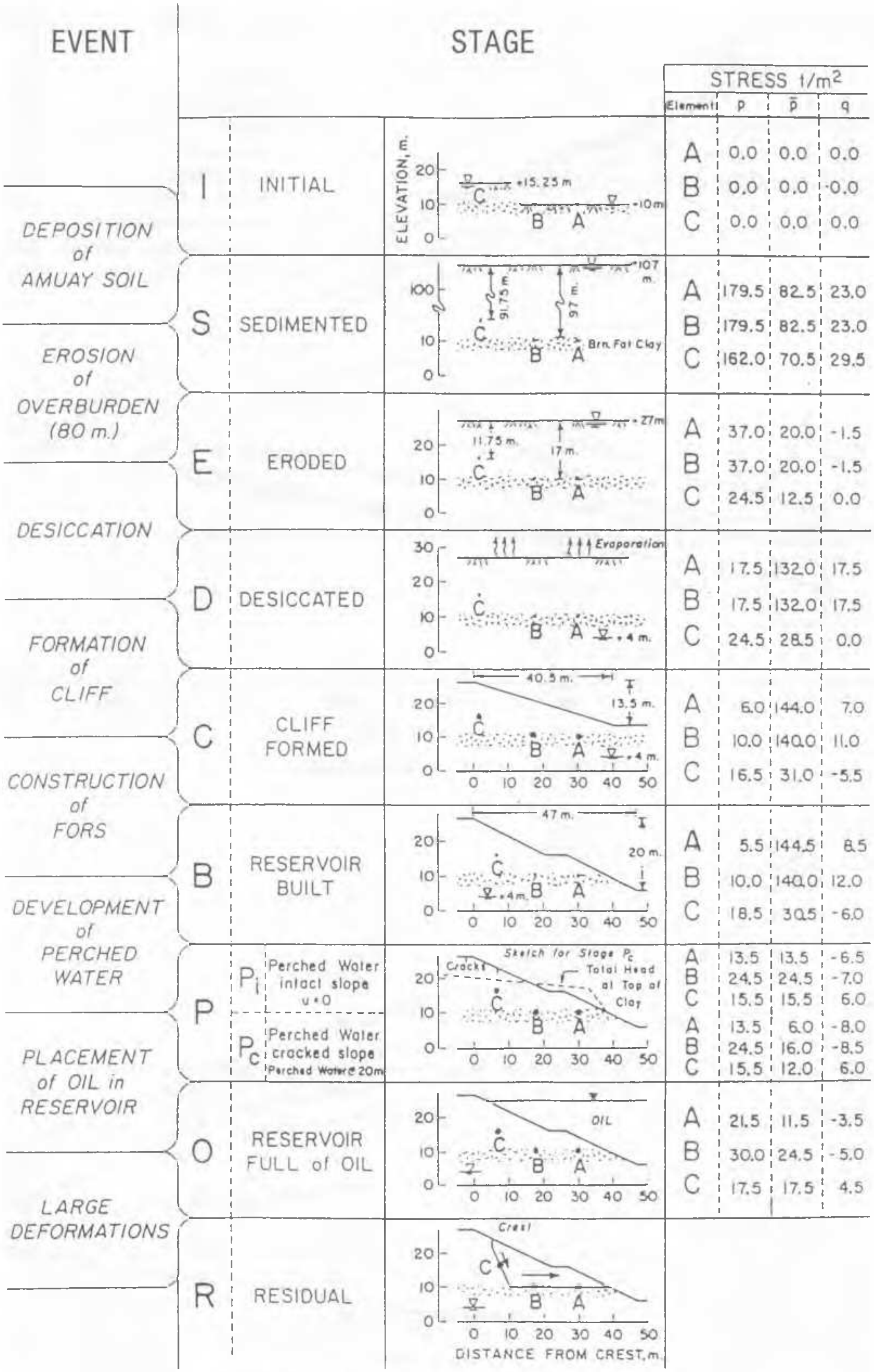
3. Consolidation;

4. Undrained or partially drained shear.

In solving problems involving the stability of natural slopes, the geotechnical engineer commonly estimates pore pressure assuming a static or steady-state seepage case. In other words, he neglects excess pore pressure resulting from shear. At Amuay we estimate pore pressure for stability analyses employing steady-state seepage analyses and field measured pore pressures. We reason that we should base our stability predictions on conditions at the start of a landslide, feeling at that time significant excess pore pressure from shear has not developed. We have only once measured any Δu_{shear} just prior to a slide. We have in effect "calibrated" our stability prediction method for $\Delta u_{shear} = 0$.

We feel uneasy about neglecting excess pore pressure from shear since laboratory tests indicate excess pore pressure for both undrained and partly drained conditions, and field measurements near the failure zone in an actual landslide indicate excess pore pressure.

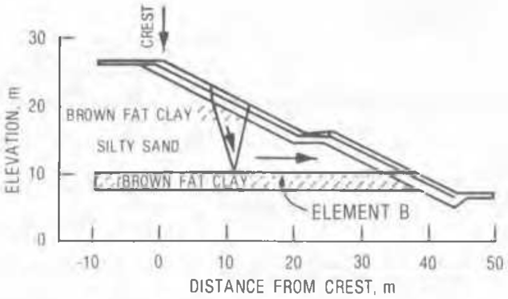
Excavating in clay can cause negative excess pore water pressure. Considerable time may elapse before this negative pore pressure dissipates. As noted earlier, Skempton explains the fact that first time slides in London clay resulting from excavations occurring as long as 40 to 50 years after the excavation on the long time required for the dissipation of negative pore pressure generated by the excavation. We need not worry about this phenomenon on the Amuay cliffside. Formation of the Amuay cliffside occurred many years ago



Stage		Element	p	$\bar{p}$	q
I	INITIAL	A	0.0	0.0	0.0
		B	0.0	0.0	0.0
		C	0.0	0.0	0.0
S	SEDIMENTED	A	179.5	82.5	23.0
		B	179.5	82.5	23.0
		C	162.0	70.5	29.5
E	ERODED	A	37.0	20.0	-1.5
		B	37.0	20.0	-1.5
		C	24.5	12.5	0.0
D	DESICCATED	A	17.5	132.0	17.5
		B	17.5	132.0	17.5
		C	24.5	28.5	0.0
C	CLIFF FORMED	A	6.0	144.0	7.0
		B	10.0	140.0	11.0
		C	16.5	31.0	-5.5
B	RESERVOIR BUILT	A	5.5	144.5	8.5
		B	10.0	140.0	12.0
		C	18.5	30.5	-6.0
P P_i	Perched Water intact slope u = 0	A	13.5	13.5	-6.5
		B	24.5	24.5	-7.0
		C	15.5	15.5	6.0
P P_c	Perched Water cracked slope Perched Water @ 20m	A	13.5	6.0	-8.0
		B	24.5	16.0	-8.5
		C	15.5	12.0	6.0
O	RESERVOIR FULL of OIL	A	21.5	11.5	-3.5
		B	30.0	24.5	-5.0
		C	17.5	17.5	4.5
R	RESIDUAL				

STRESS t/m²

FIGURE 10. GEOTECHNICAL HISTORY OF AMUAY CLIFF



	STAGES	STRESS POINTS		
		p	$\overline{p}$	q
I	INITIAL	0	0	0
S	SEDIMENTED	179.5	82.5	23.0
E	ERODED	37.0	20.0	-1.5
D	DESICCATED	17.5	132.0	17.5
C	CLIFF FORMED	10.0	140.0	11.0
B	RESERVOIR BUILT	10.0	140.0	12.0
P_I	PERCHED WATER (INTACT)	24.5	24.5	-7.0
P_c	PERCHED WATER (CRACKED)	24.5	16.0	-8.5
0	RESERVOIR FULL OF OIL	30.0	24.5	-5.0

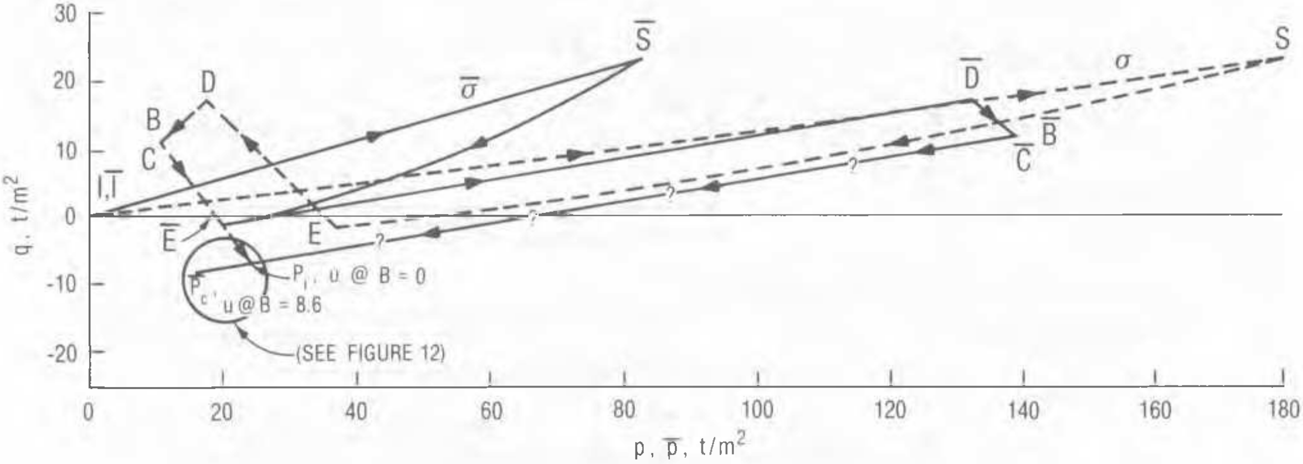


FIGURE 11. STRESS PATHS FOR ELEMENT B

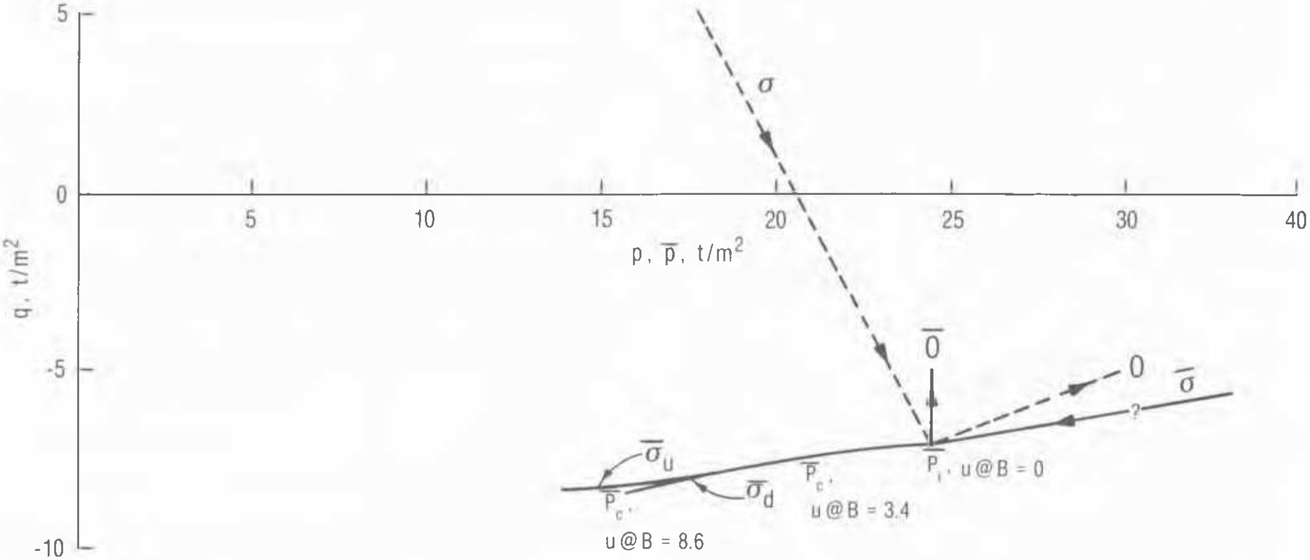


FIGURE 12. STRESS PATHS FOR ELEMENT B

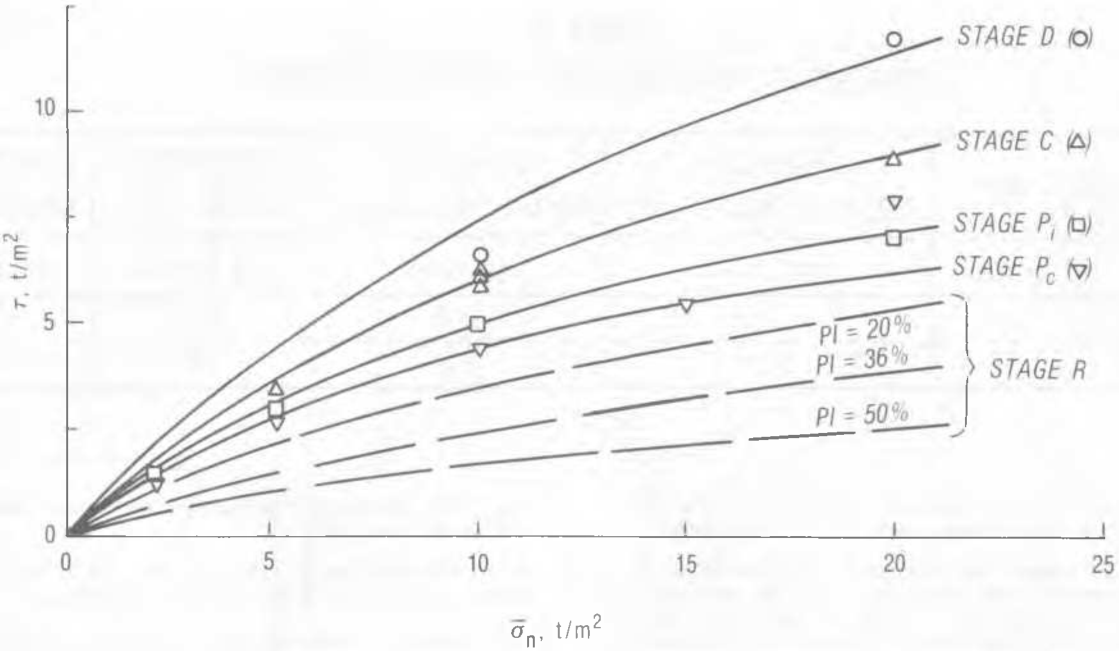


FIGURE 13. STRENGTH OF FAT CLAY

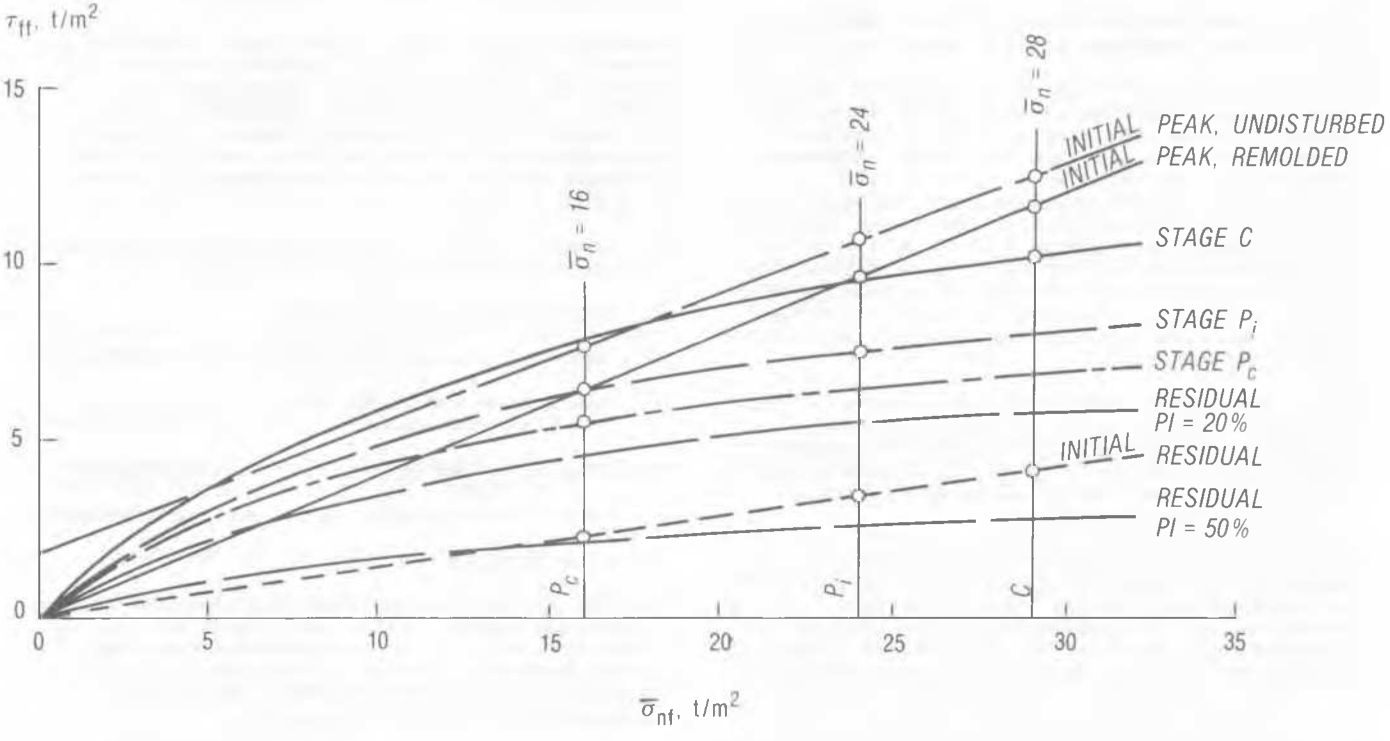


FIGURE 14. STRENGTH OF CLAY

TABLE II
STRENGTH FOR CONSTANT NORMAL STRESS

STAGE	$\overline{\sigma_n}$	STRENGTH PRESENT PROCEDURE	STRENGTH INITIAL PROCEDURE	STRENGTH INITIAL PEAK	STRENGTH REMOLDED
C	28 t/m^2	10.3 t/m^2	4.2 t/m^2	12.7 t/m^2	11.8 t/m^2
P_i	24	7.6	3.4	10.8	9.7
P_c	16	5.5	2.3	7.8	6.4

and excess pore pressures resulting from the cliff formation have long since dissipated.

Figure 15 shows the typical soil profile for the cliffside and portrays head in the ground water vs. elevation. When we made our first subsoil investigations at Amuay in 1955, we expected to find the phreatic surface a meter or so above sea level. We installed wells near the site for FORS-1 and did indeed measure the phreatic surface at elevations +1 to +3.

The data show either:

1. In 1955 there existed no perched water at Amuay; or
2. In 1955 perched water existed, but the wells did not indicate its presence.

In the early 1960's we detected water breaking out on the cliffside at elevations above the layer of brown fat clay. We then inserted piezometers and measured the heads shown in Figure 15. As shown in Figure 15, the development of the perched water raises the pore pressure in the top part of the brown fat clay from a possible negative value* of 8 t/m^2 to a positive value of 10 t/m^2 - i.e. increasing the pore pressure by 18 t/m^2. Increasing the pore pressure on the top of the fat clay has had devastating effects in creating instability along the cliffside.

Our Geotechnical Safety Program (Lambe, Silva and Marr, 1981) rests on measuring pore pressure, especially near the key facilities along the cliffside. By mid 1984 LAGOVEN had installed a total 251 piezometers of which 192 now operate properly.

Figure 16 shows contours of total head measured in January 1977 on top of the fat clay. A computer drew the contours using the measured data listed in Figure 16. Figure 17 presents similar contours for October 1984 and Figure 18 shows several total head contours for 1984 superimposed upon the total head contours for 1977.

From Figures 16, 17 and 18 we can draw two very important conclusions: namely,

1. Lateral flow occurs in all directions from a zone east of FORS-1;
2. Pore pressures have increased, in some locations as much as 1 meter per year.

As dictated by the safety assessment in our Geotechnical Safety Program, we installed drainage devices near key facilities along the cliffside. These devices have lowered pore pressure.

Looking at contours of total head (such as those in Figure 16), one logically expects a source of perched water to exist east of FORS-1. Field inspections have not located any major source of perched water. Various activities within the refinery result in the introduction of water to the subsoil. These include:

1. Leaks in pipes, both above-ground pipes and buried pipes;
2. Spray from cooling towers.
3. Leaks of cooling water from the power plant;
4. Leaks from the reservoirs storing water for fire protection;
5. Dumping of water used for testing tanks;
6. Water from training exercises for firemen;
7. Flow from septic tanks.

LAGOVEN has not had success reducing the inflow of perched water. Since Amuay gets only a few inches of rain a year, evaporation from the ground exceeds rainwater. Because of low soil permeability relatively little water can increase perched water pressure.

Our measurements and analyses to-date leave us with three conclusions, namely:

1. The perched water comes from an assortment of activities within the refinery;

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*If the clay exists at a low degree of saturation, the pore pressure could equal a value lower than -8 t/m^2.

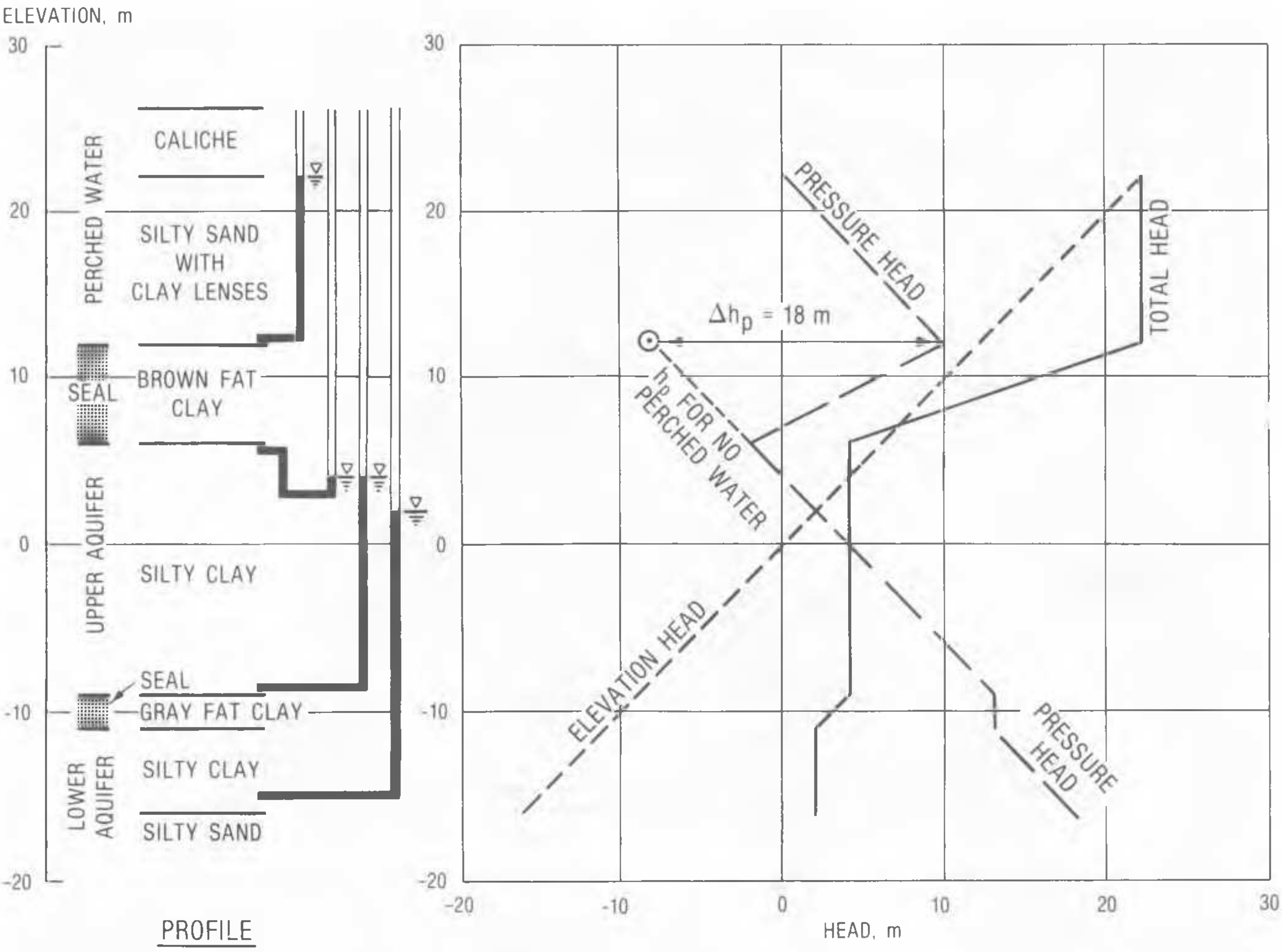


FIGURE 15. AMUAY GROUNDWATER

2. The perched water level continues to rise with time;

3. Drainage can effectively reduce the perched water in a local zone.

LEVEL OF STABILITY

STABLE means "fixed", "steadfast". In stability problems the engineer needs a means of expressing,

Closeness to failure,

Closeness to instability,

Margin of safety,

Nearness to gross movement,

Level of stability.

The geotechnical engineer most commonly uses "factor of safety" to express level of stability, wherein, factor of safety (FS) equals average shear strength (S_a) divided by average shear stress (τ_a);, i.e.

$$FS = \frac{S_a}{\tau_a}$$

A factor of safety less than 1 indicates an unstable slope and a factor of safety greater than 1 indicates a stable slope.

A useful approach consists of expressing level of stability in terms of "probability of failure" and "risk", where risk equals the product of annual probability of failure and the consequences of failure. One should not consider probability of failure (or risk) as an alternative to factor of safety, rather as a supplementary indicator. The computation for probability of failure does not avoid the uncertainties and problems faced with computing factor of safety. As a matter of fact, the engineer calculating probability of failure should first determine factor of safety. At the end of this section, I suggest

Instrument	Total Head
CT-P3	17.63
CT-P5	19.09
CT-P6	18.37
JP4-P3	14.56
JP4-P4	12.22
JP4-P6	14.94
JP4-P7	12.54
PB-PI	12.15
PB-P5	12.90
PB-P6	15.04
PB-P7	16.40
SW-P4	19.92
T118-PI	16.25
T156-PI	16.31
T156-P3	16.98
T156-P4	15.42
T156-P5	12.48
T187-PI	21.87
T201-PI	10.00*
PI-1	14.00*
P3-16	20.40*
P3-22	22.52
P3-23	21.69
P3-24	20.35
EF-P2	14.88*
N176500 E170[illegible]	8.00*

FIGURE 16. PERCHED WATER TOTAL HEAD CONTOURS JAN. 1977

FIGURE 17. PERCHED WATER OCT 1984

Instrument	Total Head
CT-P3	17.69
CT-P5	19.89
CT-P6	18.37
JP4-P2	14.56
JP4-P4	12.22
JP4-P6	14.94
JP4-P7	12.54
PB-P1	12.15
PB-P5	12.90
PB-P6	15.04
PU-P7	16.40
SW-P4	19.92
T118-P1	16.25
T156-P1	16.31
T156-P3	16.98
T156-P4	15.42
T156-P5	12.48
T187-P1	21.87
T201-P1	10.00*
P1-1	14.00*
P3-16	20.40*
P3-22	22.52
P3-23	21.69
P3-24	20.35
EF-P2	14.80*
N176500 E17[illegible]	8.00*

FIGURE 18. PERCHED WATER TOTAL HEAD CONTOURS 1984 SUPERIMPOSED ON TOTAL HEAD CONTOURS JAN 1977

a semi-empirical relationship between factor of safety and probability of failure.

The heart of our Amuay Geotechnical Safety Program consists of a periodic "safety assessment" of each key section of the cliffside. We make a safety assessment by comparing performance criteria with stability prediction. In selecting performance criteria for the Amuay slopes we consider:

1. Consequences of failure;
2. Nature of nearby facilities;
3. Uncertainties in the situation;
4. Most probable failure mode;
5. Quality and thoroughness of engineering construction.

In the Amuay Geotechnical Safety Program we make an annual safety assessment of each key slope. Since experience at Amuay shows that one needs two years to design and implement a preventive measure, the stability prediction for an existing slope considers "existing" as two years from the time of the assessment. For any slope not meeting performance criteria or for any failed slope we design measures using a design life of 25 years. (Sometimes we design for a shorter life, planning to implement preventive measures later.) We thus make a stability prediction for an existing or future slope for conditions 25 years in the future. Figure 19 helps one understand the initiation of a landslide at Amuay. After discussing landslide initiation, I will illustrate stability prediction for several situations at Amuay.

We start at Stress Point C - the cliff exists and the average element shown in the key (Figure 19) has a negative pore pressure of -5 t/m^2. The development of the perched water increases pore pressure moving the Stress Point along the effective stress path to P_i for 0 pore pressure. Further build-up of perched water raises the pore pressure to 8.5 t/m^2. Tension cracks develop behind the crest of the slope. The Stress Point now reaches the failure line for the P_c condition - average shear stress equals average shear strength and failure occurs.

The rising perched water carries the Stress Point along the effective stress path from C to P_c. During this process the following three things occur:

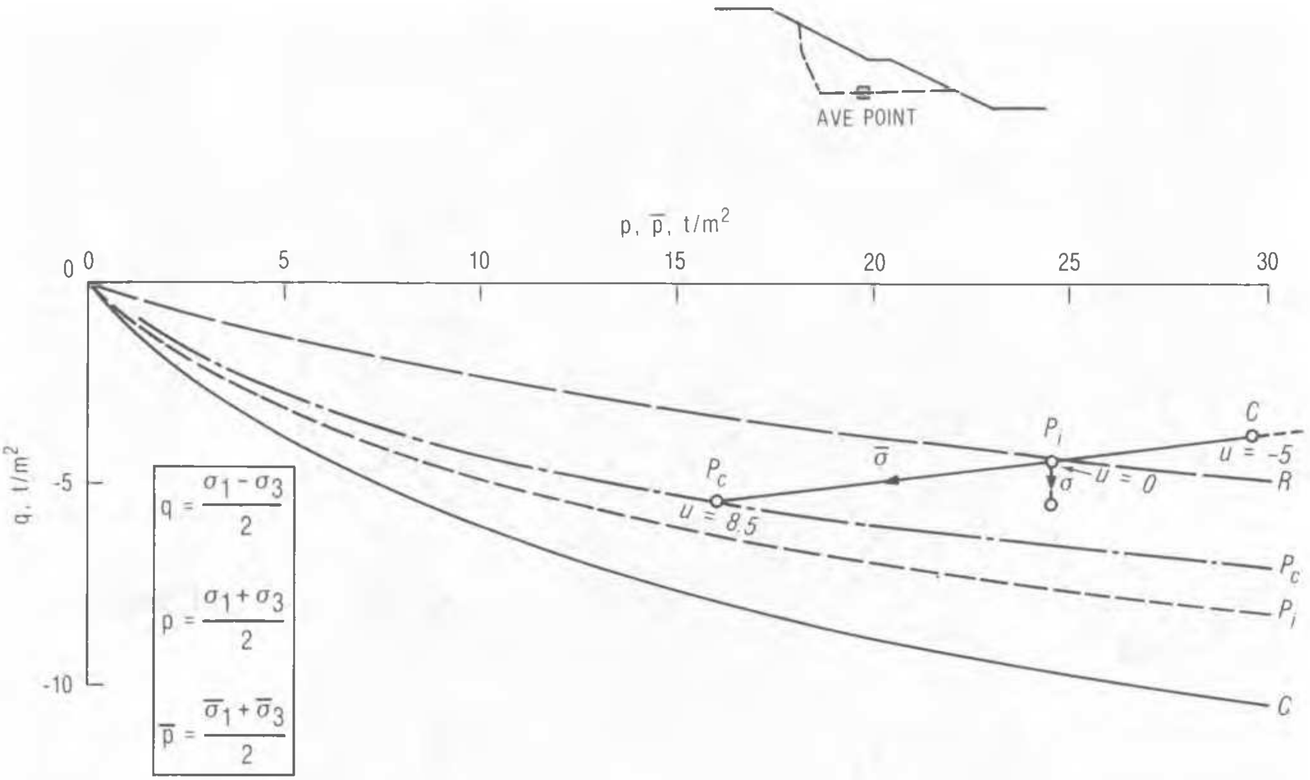


FIGURE 19. STRESS PATHS FOR AVERAGE POINT

1. The average shear stress increases;
2. The average effective stress decreases;
3. The failure line decreases.

One has considerable difficulty in correctly expressing level of stability for various locations on the stress path between C and P_c.

For conditions at Stress Point C, $S_a/\tau_a = 2.7$. At point P_i the ratio equals 1.7. For a stress point where the pore pressure equals +7.5. t/m^2, i.e. just before Point P_c, the factor of safety equals 1.1. The calculation of each of these safety factors employs the shear stress existing at the stress point and the shear strength corresponding to the existing effective stress. We have found stress paths, like those shown in Figure 19, essential to evaluating stability at Amuay. For illustration, consider the following for the slope in Figure 19. At Stage C, the FS = 2.7. Yet a build-up of pore pressure equal to 13.5 t/m^2 carries the slope to failure! Note, Figure 15 shows that the development of perched water to +22 resulted in an increase of pore pressure in the fat clay equal to 18 t/m^2. Studying Figures 15 and 19 leads one to expect stability problems when the perched water rises to elevation +18.5 near mid height of the cliff.

If we have an established performance criteria of minimum factor of safety equals 1.5, we must prevent pore pressure at the average point from exceeding about 2 t/m^2. In designing remedial measures for a failed slope, we must use the residual strength in our stability conditions. Figure 20 summarizes the guidelines we use for the selection of strength to make stability predictions for Amuay slopes.

In expressing level of safety of an important portion of the cliffside to management we have made use of the principles of probability. For each of the main categories of performance (force-deformation, stability, and flow) we have developed performance criteria for the Amuay cliffside. For stability and flow we have related "factor of safety" with "annual probability of failure" as shown in Figure 21 for stability. We prepared Figure 21 as a numerical expression of our judgement - we had little data to prepare these plots.

Having probability of failure and an estimate of the consequences of that failure, one can calculate risk, the product of the two. To estimate the consequences of a landslide which takes out a facility we consider:

1. Cost of replacing the lost facility;
2. Cost to clean up and repair the slope;

Stage at Present + 2 years	Stage at end of Design Life	Applicable Strength	
		Assess-ment	Design
D	D	D	use C
	C	D	C
	P_i	D	P_i
	P_c	D	P_c
	R	D	R
C	C	C	C
	P_i	C	P_i
	P_c	C	P_c
	R	C	R
P_i	P_i	P_i	P_i
	P_c	P_i	P_c
	R	P_i	R
P_c	P_c	P_c	P_c
	R	P_c	R
R	R	R	R

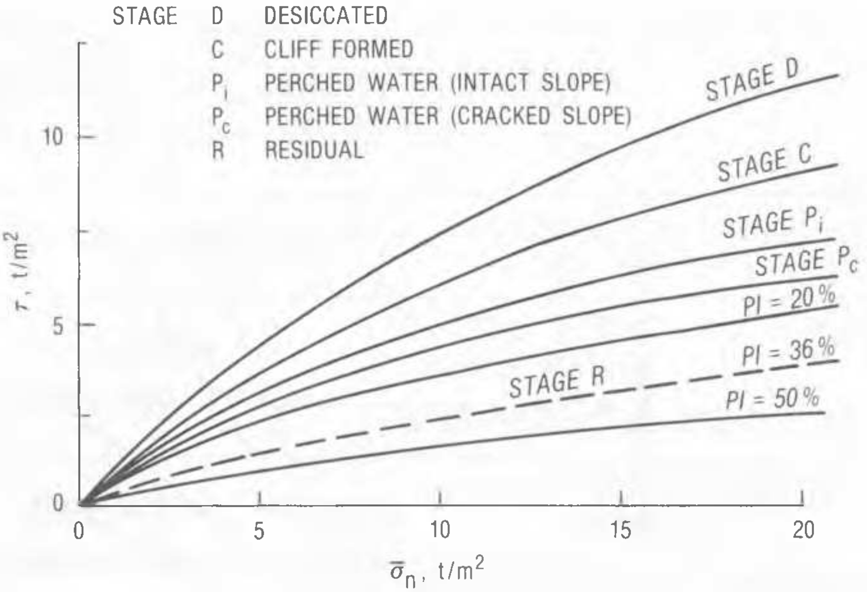


FIGURE 20. STRENGTH SELECTION

3. Cost of lost service for the facility.

To estimate the consequences of a landslide which breaches one of the three oil storage reservoirs and thereby dumping oil into the nearby sea constitutes a difficult and nebulous task. We have a datum point from the damages associated with the Amoco Cadiz which ran aground off the coast of France on the night of March 16, 1978 (Time Magazine, 3 April, 1984). The spill involved 68 million gallons of oil (1.6 million barrels). Time says the "....eventual bill could reach nearly 2 billion dollars". Each of the oil storage reservoirs at Amuay has a capacity many times 1.6 million barrels.

INSTABILITY PREDICTION AT AMUAY

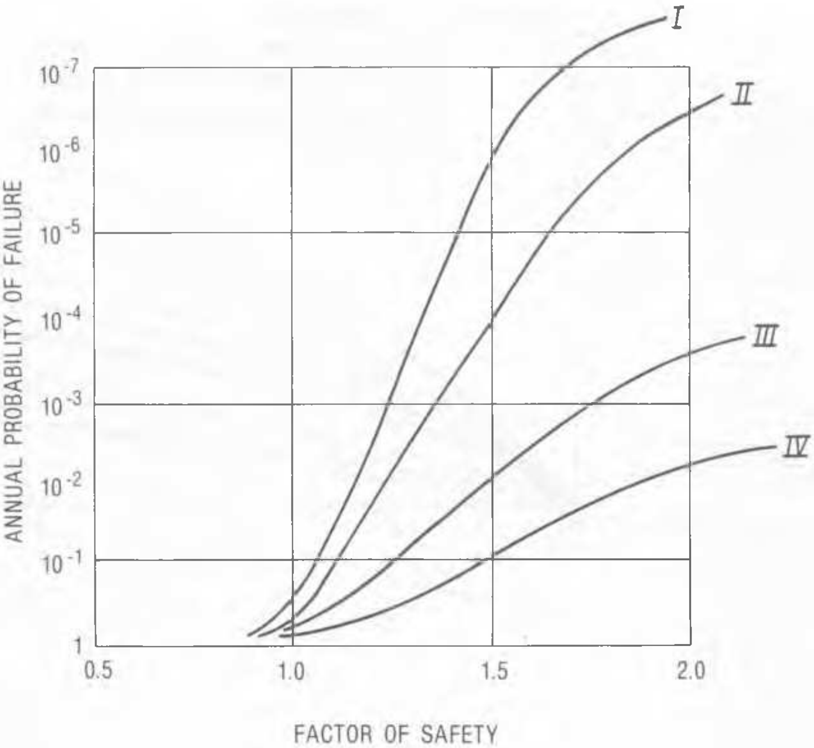
During the past thirty years we have developed a procedure for predicting instability of the Amuay cliffside. Every three months we inspect the cliffside. We study field measurements (especially pore pressures) taken monthly. On the basis of the inspections and study of field data, we predict stability at selected locations. This prediction procedure involves:

1. Selecting geometry of the critical section from visual inspection and field surveying measurements of the slope in question and failed slopes;
2. Choosing the strength line (K_f Line) based on the stress-strain history and physical appearance of the slope in question and the index properties of the clay in the slope;
3. Predicting pore pressure from measurements on nearby field piezometers and seepage analyses;
4. Using a limiting equilibrium wedge analysis to determine factor of safety.

This procedure seems to give reliable predictions of cliffside instability. Two aspects of the procedure present difficulty, namely:

1. Correctly identifying the existing stress-strain state of the clay in the slope;
2. Predicting the pore pressure in the clay, especially when the clay has or will have a negative pore pressure or an excess pore pressure.

Today our capability to predict landslides at Amuay far exceeds our prediction capability of thirty years ago. Table III lists the important improvements in our prediction capability. Table III divides the improvements



	DESIGN	CONSTRUCTION	PERFORMANCE	FS DETERMINATION
I	BY QUALIFIED ENGINEER BASED ON MEASURED DATA FOR SITE HAS KEY COMPONENTS (FILTERS, CUT-OFF, ETC.) NO ERRORS, OMISSIONS COMPLETE ASSESSMENT OF GEOLOGIC CONDITIONS 0.25	FULL-TIME SUPERVISION BY QUALIFIED ENGINEER FIELD CONTROL MEASUREMENTS NO ERRORS, OMISSIONS 0.25	COMPLETE PERFORMANCE PROGRAM NO MALFUNCTIONS (SLIDES, CRACKS, ARTESIAN HEADS...) CONTINUOUS MAINTENANCE BY TRAINED CREWS 0.25	EFFECTIVE STRESS ANALYSES BASED ON MEASURED DATA (STRENGTH, GEOMETRY, PORE PRESSURE...) FOR SITE COMPLETE FLOW NET 0.25
II	BY QUALIFIED ENGINEER HAS KEY COMPONENTS NO ERRORS, OMISSIONS GEOLOGY CONSIDERED 0.5	PART-TIME SUPERIVSION BY QUALIFIED ENGINEER NO ERRORS, OMISSIONS 0.5	PERIODIC INSPECTION BY QUALIFIED ENGINEER NO MALFUNCTIONS FEW FIELD MEASUREMENTS ROUTINE MAINTENANCE 0.5	EFFECTIVE STRESS ANALYSES 0.5
III	APPROXIMATE DESIGN USING PARAMETERS INFERRED FROM INDEX TESTS 0.75	INFORMAL SUPERVISION 0.75	ANNUAL INSPECTION BY QUALIFIED ENGINEER NO FIELD MEASUREMENTS MAINTENANCE LIMITED TO EMERGENCY REPAIRS 0.75	RATIONAL ANALYSES USING PARAMETERS INFERRED FROM INDEX TESTS 0.75
IV	NO RATIONAL DESIGN 1.0	NO SUPERVISION 1.0	OCCASIONAL INSPECTION BY NON QUALIFIED PERSON 1.0	APPROXIMATE ANALYSES USING ASSUMED PARAMETERS 1.0

FIGURE 21. SAFETY WITH RESPECT TO SHEAR

TABLE III

IMPROVEMENTS IN CAPABILITY TO PREDICT LANDSLIDES—1955 TO 1985

	ADVANCES IN PROFESSION	RESULTS FROM AMUAY. INVESTIGATIONS
VERY IMPORTANT	COMPUTER STRESS PATH METHOD	RELATING STRENGTH AND STRESS-STRAIN HISTORY MEASUREMENT OF PORE PRESSURE
IMPORTANT	FIELD MEASUREMENT SYSTEMS STABILITY ANALYSES	MEASUREMENT OF LANDSLIDE GEOMETRY BACK ANALYSES OF LANDSLIDES

into two groups, those constituting advances in the geotechnical profession and improvements of a more local nature, based on the results of the Amuay investigations.

Advances in Profession

In making geotechnical analyses in the 1950's we used desk calculators and slide rules. The development of the computer has had a great impact on our work at Amuay. We now use the computer to:

1. Reduce, store and portray field data and laboratory data;
2. Make analyses for flow, stress-deformation and stability.

With the computer we can rapidly analyze and portray large quantities of data. We have had to expend an enormous effort to develop software. We still, however, check all of our key stability predictions by making a hand solution of each.

Our present landslide prediction technique rests on the Stress Path Method. We use the Stress Path Method to express numerically the geotechnical history of the Amuay cliff. Further, stress paths help greatly in identifying the fundamentals of a landslide prediction, especially in choosing the appropriate strength and expressing the level of stability (Figure 19).

I, aided by associates, formulated the Stress Path Method during the 1960's and 1970's (Lambe, 1967; Lambe and Marr, 1979). The idea of stress paths goes back far earlier than the 1960's. Both A. Casagrande and Taylor traced stresses on a soil sample subjected to shear. In October 1940 Taylor reported values of

$$\frac{\sigma_1 + \sigma_2 + \sigma_3}{3} \quad \text{and} \quad \frac{\sigma_1 + \sigma_3}{2}$$

in his work on the shear strength of soil for the U.S. Corps of Engineers. On the Union Falls Project during 1946 through 1949, Taylor and I used stress paths to solve stability problems. Figure 22 shows stress paths to solve stability problems. Figure 22 shows stress paths drawn in 1947 for triaxial tests on undisturbed samples of clay from Union Falls, Maine. In the 1940's we plotted the stress on a given plane, usually the $\theta = 60^o$ plane. Figure 23 traces effective stress on the $\theta = 60^o$ surface and that on the $\theta = 45^o$ surface (the effective stress path) for a triaxial test on Amuay clay.

During our work at Amuay we have installed many field instruments to measure movement, stress, strain, temperature, pore pressure etc. Pore pressure has proved by far the most useful measurement. We tried vibrating wire piezometers and in recent years have gone to pneumatic piezometers. The Casagrande hydraulic piezometer dates back to the 1930's. During the Amuay investigation many improvements in field instrumentation occurred, especially in readout equipment.

We have installed 39 inclinometers at Amuay. We have found measurements from inclinometers helpful in locating the shear surface in a failing slope. We have not had much success, however, in using inclinometer measurements to predict an impending landslide. The piezometer constitutes the key field measuring device for predicting instability at Amuay.

When we made our first stability analyses at Amuay in the mid-1950's, the profession understood the fundamentals of stability

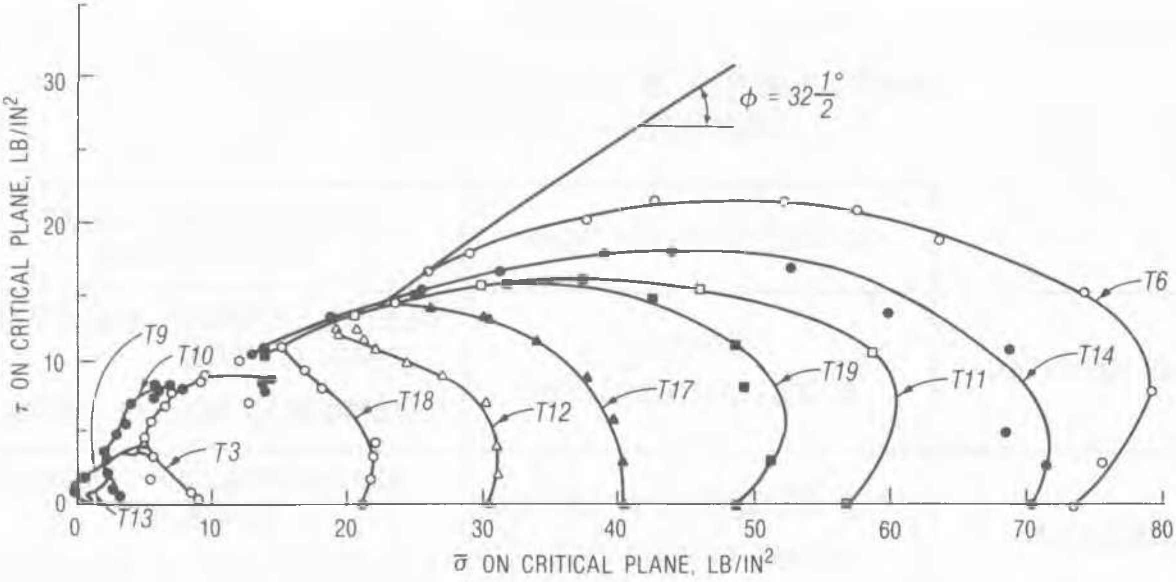


FIGURE 22. STRESS ON CRITICAL PLANE DURING SHEAR (FROM UNION FALLS REPORT JUNE 1949)

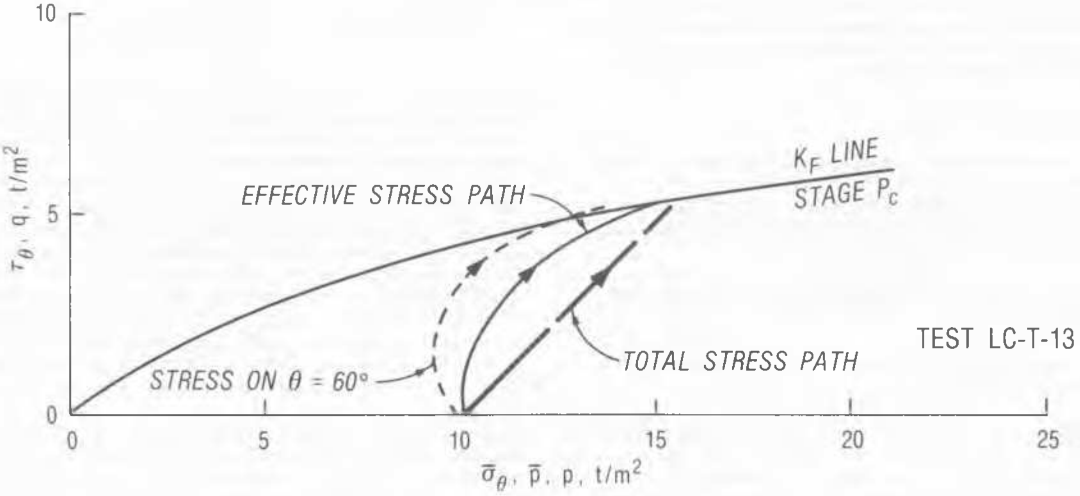


FIGURE 23. TRIAXIAL TEST ON AMUAY CLAY

analysis. Taylor's book (1948) presents these fundamentals. During the past thirty years Bishop and others developed approximate methods for accounting for stress distribution along a trial failure surface.

Results from Amuay Investigation

Our laboratory and field investigation on the shear strength of the Amuay fat clay has contributed greatly to the improvement of our capability to predict Amuay landslides. The availability of the Stress Path Method and Skempton's work on cuttings in the London clay helped considerably in our relating strength of the fat clay with its stress-strain history.

Identifying the importance of the perched water layer proved very important. Seepage analyses based on field measured pore pressures permit us to identify critical zones and to forecast pore pressures.

On eight Amuay landslides we have made surveying measurements to determine the external geometry of each slide. We have also made excavations and borings to help locate failure surfaces. These measurements have helped us identify the geometry of the critical surfaces at different locations along the cliff.

Back-analyses on eight Amuay landslides have helped us calibrate our prediction techniques. These analyses have, for example, led us to use the steady-state pore pressure prior to the landslide in our stability analysis.

CONCLUSIONS

As required by President de Mello this Terzaghi Oration presents a case study involving the solution of a geotechnical problem using geology, theoretical principles, laboratory tests and field measurements. Over a period of thirty years we have developed a procedure for predicting landslides along LAGOVEN's cliffside at Amuay, Venezuela. The landslide prediction procedure used in 1985 gives far more satisfactory results than did the procedure we used initially. Table III notes the major improvements in our capability to predict landslides at Amuay. We have made considerable use of advances in the geotechnical profession which have occurred during the last thirty years. We have also made considerable use of the results of our investigations on the Amuay landslide problem.

During the years ahead we will attempt to improve further our landslide prediction capability by concentrating on two topics. We will try to develop better means of identifying the stress-strain state of the Amuay clay at a given location. We hope to improve our procedure for estimating the pore pressure acting at the start and during a landslide. We shall attempt to develop a procedure to measure (or estimate) negative pore pressure in clay.

ACKNOWLEDGMENTS

We have received considerable help and support from LAGOVEN engineers and managers during the past thirty years. Without this support we would not have our effective control of the landslide problem at Amuay.

A large number of my students and associates at MIT contributed to the Amuay investigation. More than twenty graduate students performed thesis research on aspects of the Amuay landslide problem.

During the last ten years Dr. Francisco Silva has made a major contribution to the Amuay work. Preparing computer software and directing the clay strength investigation have proved particularly important.

REFERENCES

Bjerrum, L., Casagrande, A., Peck, R.B. and Skempton, A.W. (1960). From Theory to Practice in Soil Mechanics, Wiley, New York.

Bjerrum, L. (1967) Progressive Failure in Slopes of Overconsolidated Plastic Clay and Clay Shales. Journal Soil Mechanics and Foundations Division ASCE (93) No. SM5, pp. 1 - 49.

Chandler, J.R. and Skempton, A.W. (1974). The Design and Permanent Cutting Slopes in Stiff Fissured Clays. Geotechnique (24) No. 4. pp. 457 - 466.

Crespo, T., C.D. (1973). Strength of the Clay in a Natural Slope in Venezuela. S.M. Thesis, Department of Civil Engineering, MIT.

Grismala, R.E. (1978). Stability Analysis of Natural Slopes with Weak Strata. S.M. Thesis, Department of Civil Engineering, MIT.

Lambe, T.W. (1951). Soil Testing for Engineers. Wiley, New York.

Lambe, T.W. (1967). The Stress Path Method. Journal Soil Mechanics and Foundations Division ASCE (93) SM6, Proc. Paper 5613, pp. 309 - 331.

Lambe, T.W. and Whitman, R.V. (1969). Soil Mechanics, John Wiley and Sons, Inc., New York.

Lambe, T.W. (1973). Investigation of the Unstable Cliffside at the Amuay Refinery. Consulting Report to the Creole Petroleum Corporation.

Lambe, T.W. and Marr, W.A. (1979). Stress Path Method - Second Edition. Journal Soil Mechanics and Foundations Division ASCE, No. GT6, pp. 727 - 738.

Lambe, T.W. (1980). Properties of Amuay Soils. Consulting Report to LAGOVEN S.A.

Lambe, T.W., Marr, W.A. and Silva, F. (1981). Safety of a Constructed Facility: Geotechnical Aspects. ASCE Vol 107, No. GT3, pp. 339 - 352.

Lambe, T.W., Silva, F., and Marr, W.A. (1981). Key Features of the Geotechnical Safety Program for the Amuay Cliffside. Geotechnical Engineering Vol II, Bangkok.

Lambe, T.W. (1983). Undrained Triaxial Tests on Amuay Brown Fat Clay. Consulting Report to LAGOVEN S.A.

Lengster, M. (1977). Stability of a Natural Slope. S.M. Thesis, Department of Civil Engineering, MIT.

MIT Soil Mechanics Laboratory (June 1940 - July 1942). Shear Research for U.S. Engineer Department, Reports 1 - 8.

Mitchell, J.K. (1976). Fundamentals of Soil Behavior, John Wiley and Sons, Inc.

Morgenstern, N.W. and Price, V.W. (1965). The Analysis of the Stability of General Slip Surfaces. Geotechnique (15), No. 1, pp. 79 - 93

Platt, G.A. (1966) Investigation of a Slope Instability at Amuay, Venezuela. S.M.

Thesis, Department of Civil Engineering, MIT.

Skempton, A.W. (1964). Long-Term Stability of Clay Slopes. Geotechnique (14), pp. 77 - 102.

Skempton, A.W. and Hutchinson, J. (1969). Stability of Natural Slopes and Embankment Foundations. Proc. of the VII International Conference on Soil Mechanics and Foundation Engineering, Mexico, State-of-the-Art Volume, pp. 291 - 340.

Skempton, S.W. (1970). First-Time Slides in Over-Consolidated Clays. Geotechnique (20) No. 3, pp. 320 - 324.

Skempton, A.W. (1977). Slope Stability of Cuttings in Brown London Clay. Proc. of the IX International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Vol. 3, pp. 261 - 270.

Taylor, D.W. (1948). <u>Fundamentals of Soil Mechanics</u>, John Wiley and Sons, Inc., New York.

Taylor, D.W. (1949). Soil Investigations and Earth Dam Design, Union Falls Project, Maine. Consulting Report.

Terzaghi, K. (1936). Stability of Slopes of Natural Clay. Proc. I International Conference on Soil Mechanics, Harvard, Vol. 1, pp. 161 - 165.

Time Magazine (30 April, 1984). Big Bill for a Big Spill.

Tong, C.K. (1970). Stability Control of a Natural Slope. S.M. Thesis, Department of Civil Engineering, MIT.

Vaughan, P.R. and Walbancke, H.J. (1973). Pore Pressure Changes and Delayed Failure of Cutting Slopes in Over-Consolidated Clay. Geotechnique (23), No. 4, pp. 531 - 539.

Vaughan, P.R. (1977). Report on the Variation of the Residual Angle of Shearing Resistance for a Sedimentary Clay from Amuay, Venezuela. Unpublished Report.

Viranuvut, S. (1979). Shear Strength of Overconsolidated Clay in an Unstable Cliffside. C.E. Thesis, Department of Civil Engineering, MIT.

Voight, B. (1973). Correlation Between Atterberg Plasticity Limits and Residual Shear Strength of Natural Soils. Geotechnique (23), pp. 265 - 267.