THE SELECTION OF SOIL STRENGTH FOR A STABILITY ANALYSIS

The Fifth Spencer J. Buchanan Lecture

by

T. William Lambe

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Lecture Room A
Clayton Williams, Jr. Alumni Center
George Bush Drive and Houston Street
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College Station, Texas

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for a

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INTRODUCTION

With pleasure I give the 1997 Buchanan Lecture to honor Spencer J. Buchanan.

In the first Spencer J. Buchanan Lecture (1993), Dr. Ralph Peck related the start of soil mechanics. Professor Peck dates the birth of soil mechanics to March 1919 when Dr. Karl Terzaghi obtained his first understanding of strength, compression and permeability of clay - especially the role of pore water pressure.

Terzaghi came to MIT in 1925 and returned to Europe in 1928. Dr. Glennon Gilboy replaced the departing Terzaghi. Gilboy supervised the thesis completed by Spencer J. Buchanan in 1931.

I first came to MIT in 1943 and joined the staff in 1945. From 1945 to 1981 I taught, did research and consulted - closely integrating the three activities. I did research on field projects and used the results in my classes. Since 1981 I have continued consulting, now working somewhat less than half time. I came upon the MIT scene after the people noted by Dr. Peck departed. I did, however, know personally all of the names mentioned by Dr. Peck.

In 1948 a group of U.S. geotechnical engineers went to the second International Conference held in Rotterdam. The group included Mr. and Mrs. Buchanan and my wife and me. We crossed the Atlantic ocean in the Marine Falcon a converted troop ship. During this voyage the Lambes got well acquainted with the Buchanans.

Nearly all of the field projects I have worked on involved an earth structure malfunction and usually some dispute among the concerned parties. In all cases but one the designer of the troubled project consisted of some engineer other than me - i.e. I came into the project after trouble developed.

I created one of the projects that experienced difficulty. In the mid 50's I had charge of the design, construction supervision and surveillance of the Siburua Dam in foothills of the Andes in Venezuela. We topped out the dam in August 1957. A slide occurred in the south abutment on 28 October 1959 and a slide occurred in the north abutment on 15 July 1964. I , aided by associates, (including Dr. L.A. Wolfskill a graduate of Texas A. & M.) investigated the slides and documented the investigations in the literature.

Both slides resulted from an increase in pore water pressure. I have never forgotten this most important fact--pore water pressure increase can cause instability in an earth structure!

During the discussion following a lecture by Dr. Terzaghi someone in the audience said, “You now recommend the opposite of what you recommended two years ago”. Dr. Terzaghi replied, “Don’t you think I learned something in two years?”

Over the years I, aided by associates, have investigated 60 slope failures. In my investigations I have receive great assistance from Dr. W.A. Marr and Dr. Francisco Silva. Dr. Marr ran the stress path tests reported herein. Dr. Silva helped at all stages of most of the failure investigations. Three quarters of the slope failures I have investigated have resulted from: PORE WATER PRESSURE INCREASE.

In examining slope failures I have reviewed stability analyses by engineers from many firms. Nearly all of these stability analyses contained large fundamental errors.

My Buchanan Lecture rests on geotechnical fundamentals and lessons learned from my investigations of slope failures.

STRENGTH DEPENDS ON PATH TO FAILURE

The practicing geotechnical engineer must frequently determine the level of safety of an existing or planned earth structure. He usually makes this determination using average shear stress and average shear strength - in particular, he calculates the factor of safety as equal to average strength over average stress. The determination of average stress for a given geometry presents no difficulty. The determination of appropriate strength can prove difficult.

Figure 1 portrays the results of three direct shear tests on dry sand. As Figure 1 shows the strength depends upon the effective stress path to failure. In general, the strength from a loading (F_L) exceeds the strength from an unloading (F_U).

Test 3 comes from keeping the effective normal stress during shear constant, i.e. a vertical effective stress path to failure, $\Delta\bar{\sigma}_n = 0$. The conventional direct shear test involves keeping the total normal stress constant. The conventional stability analysis assumes a vertical effective stress path to failure. This assumption of vertical effective stress path makes stability analysis convenient however in an actual earth structure the average effective normal stress almost never remains constant during shear.

Figure 2 presents the effective stress paths from three undrained tests all starting at the same initial stresses. In the upper left hand corner of Figure 2 we define the slope of the total stress path as omega. The results in Figure 2 show that strength depended very significantly on path to failure.

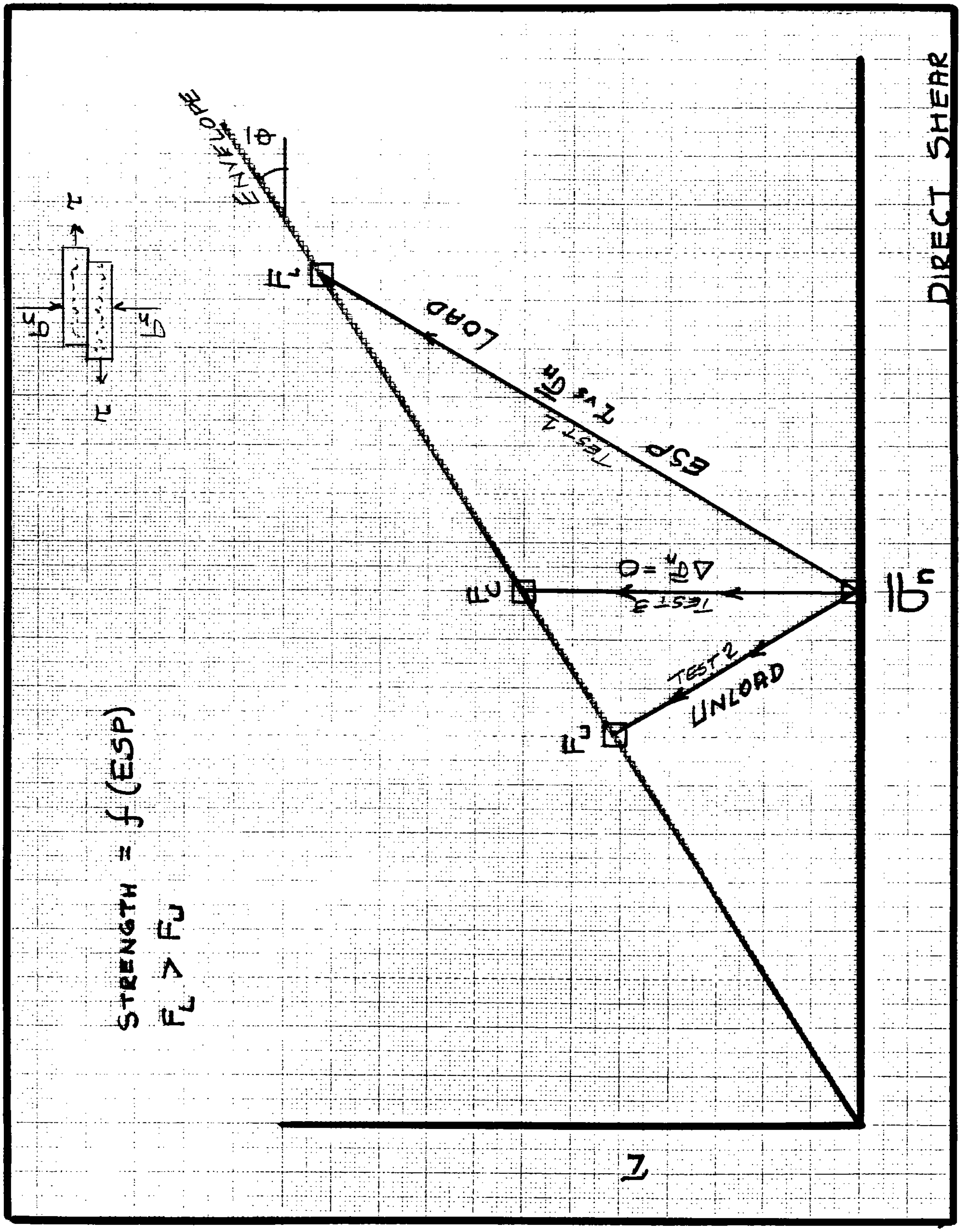


FIGURE 1

2/17/97

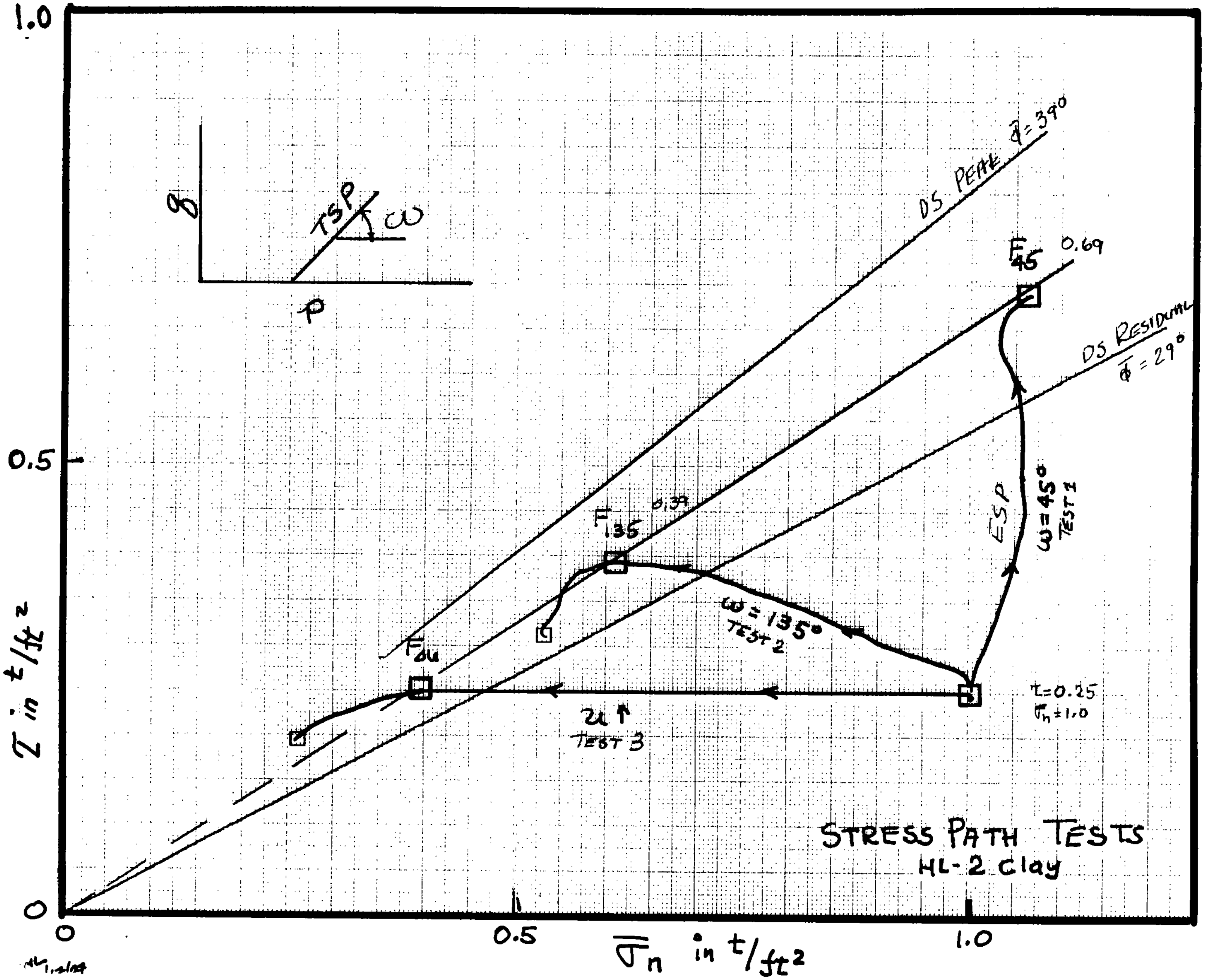

1.0
0.5
0
τ in t/ft²
0
0.5
1.0
$\bar{\sigma}_n$ in t/ft²
q
p
TSP
ω
DS PEAK $\bar{\phi}$ = 39°
F_{45}
0.69
DS RESIDUAL
$\bar{\phi}$ = 29°
F_{135}
0.39
F_{du}
ESP
ω = 45°
TEST 1
ω = 135°
TEST 2
u ↑
TEST 3
τ = 0.25
σ_n = 1.0
STRESS PATH TESTS
HL-2 Clay

FIGURE 2

In connection with actual earth structures we have run many stress path tests. Figure 3 portrays the results of these tests in the form of strength at angle omega divided by the strength at $\omega = 45^\circ$ versus omega. The results in Figure 3 show that strength varied enormously with omega. Pioneers in soil mechanics recognized the great importance stress could have on soil strength. Figure 4 presents quotations from Terzaghi's book in 1943 and from Taylor's book in 1948.

Figure 5a shows a demonstration developed decades ago at MIT. Figure 5b shows the stress paths for the demonstration. Increasing the weight gives the loading effective stress path with a strength F_L. Building up the pore pressure results in failure along the horizontal stress path to failure $F_{\Delta u}$. In the demonstration the instructor would ask students to cause failure by pushing on the weight. They could not cause failure. He then caused failure by building up the pore pressure. Figure 5b also shows a vertical effective stress path to failure.

STABILITY OF A SLOPE

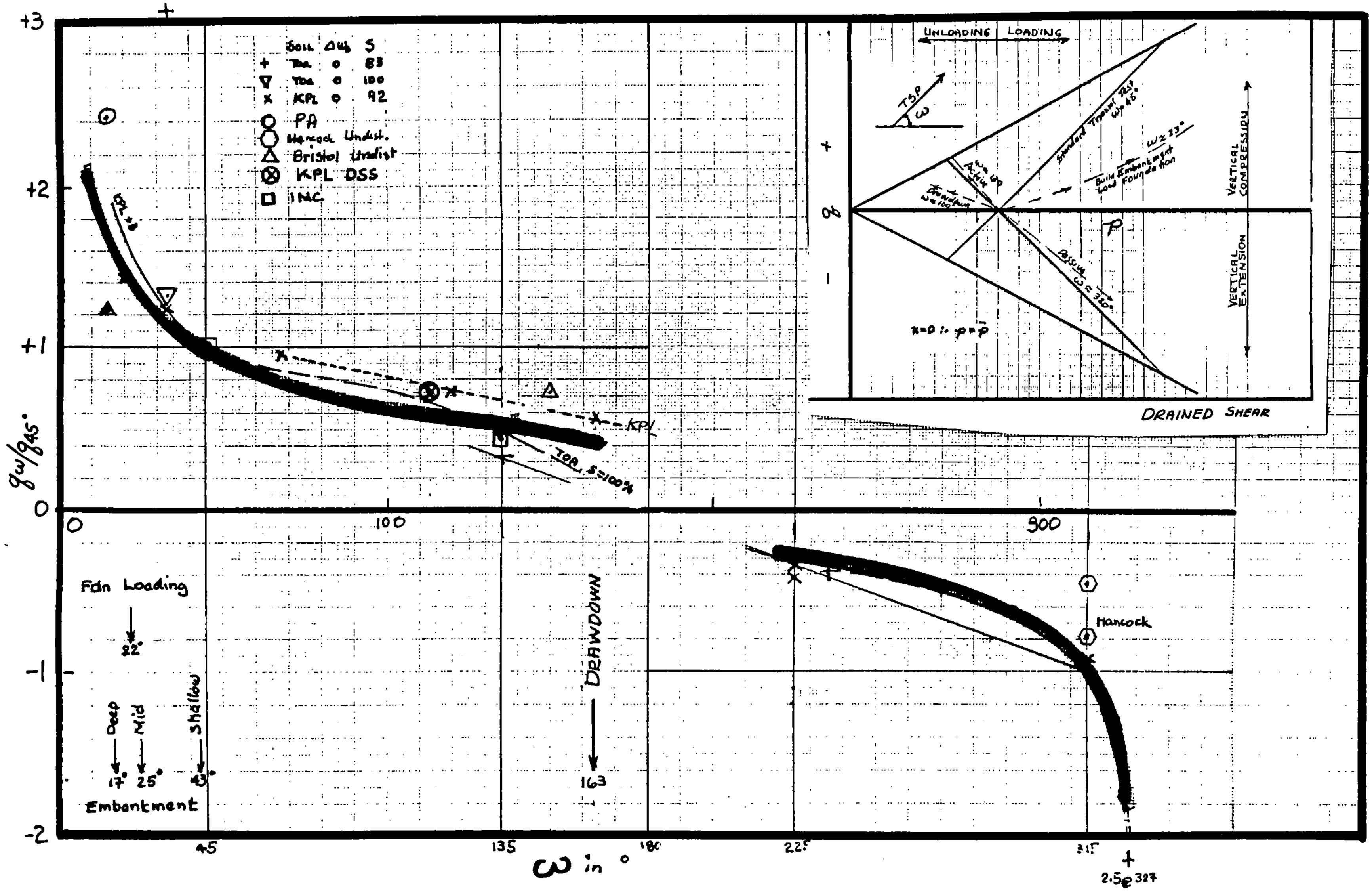
Let us now determine the factor of safety for the earth slope shown in Figure 6. We will determine the factor of safety for three events - i.e three different paths to failure. First we will determine the factor of safety for the earth slope for raising the crest until failure occurs. In the laboratory we subject a sample to the total stresses shown by point I and effective stresses at $\bar{I}$. These stresses approximate average stresses for the slope in the field. To model the affect of raising the slope we run a test along the total stress path shown (measuring pore pressures) and carry the test to failure at strength 17.0. We calculate the factor of safety for increasing the height of the slope as 2.1.

We next carry the same slope to failure by excavating at the toe. In the laboratory we subject a sample to the stresses I, $\bar{I}$ and run the test along the total stress path of $\omega \simeq$ approximately 135°. The strength of this unloading equals 9.5, giving a factor of safety of 1.2.

The third event consists of building up the pore pressure. We carry the sample in the laboratory to failure by holding the total shear stress and total normal stress constant while building up the pore pressure thus giving a horizontal stress path to failure.

This example illustrates three important points, namely:

1. The factor of safety for the existing slope depends very significantly on the path to failure;
2. The most dangerous situation consists of building up pore pressure and for this situation one cannot use the usual procedure of dividing shear strength by shear stress;



STRENGTH vs ω

FIGURE 3

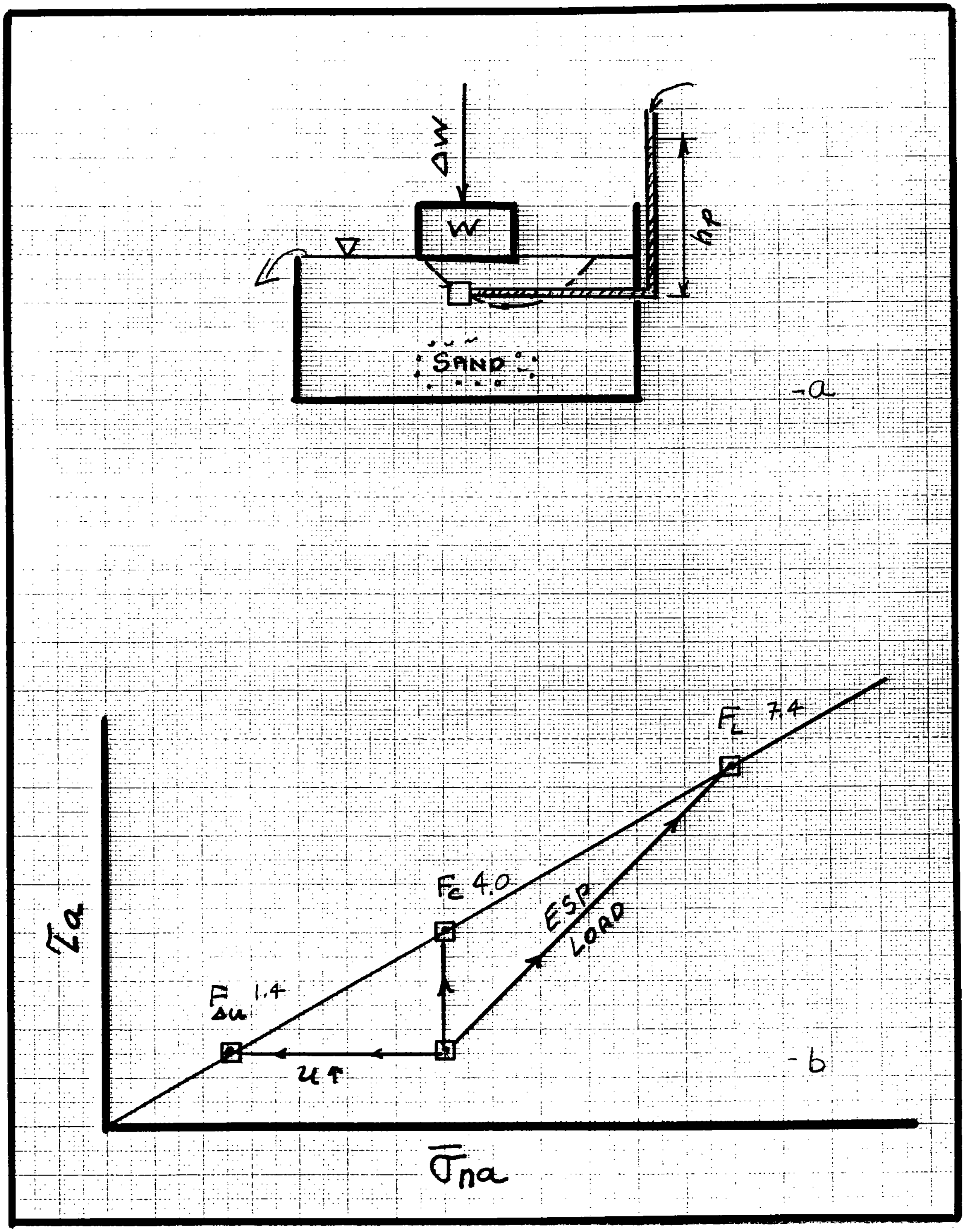
We test the clay in the laboratory under conditions of pressure and drainage similar to those under which the shear failure is likely to occur in the field and we introduce the values c and ϕ thus obtained into our equations. It is obvious that the success of this procedure depends chiefly on the degree to which the experimenter has succeeded in imitating the field conditions.

p 15 Theoretical Soil Mechanics 1943
KARL TERZAGHI

Of course, the test conditions should be chosen to reproduce natural conditions as closely as possible.

p 362 Fundamentals of Soil Mechanics 1948
DONALD TAYLOR

FIGURE 4



PATH TO FAILURE

FIGURE 5

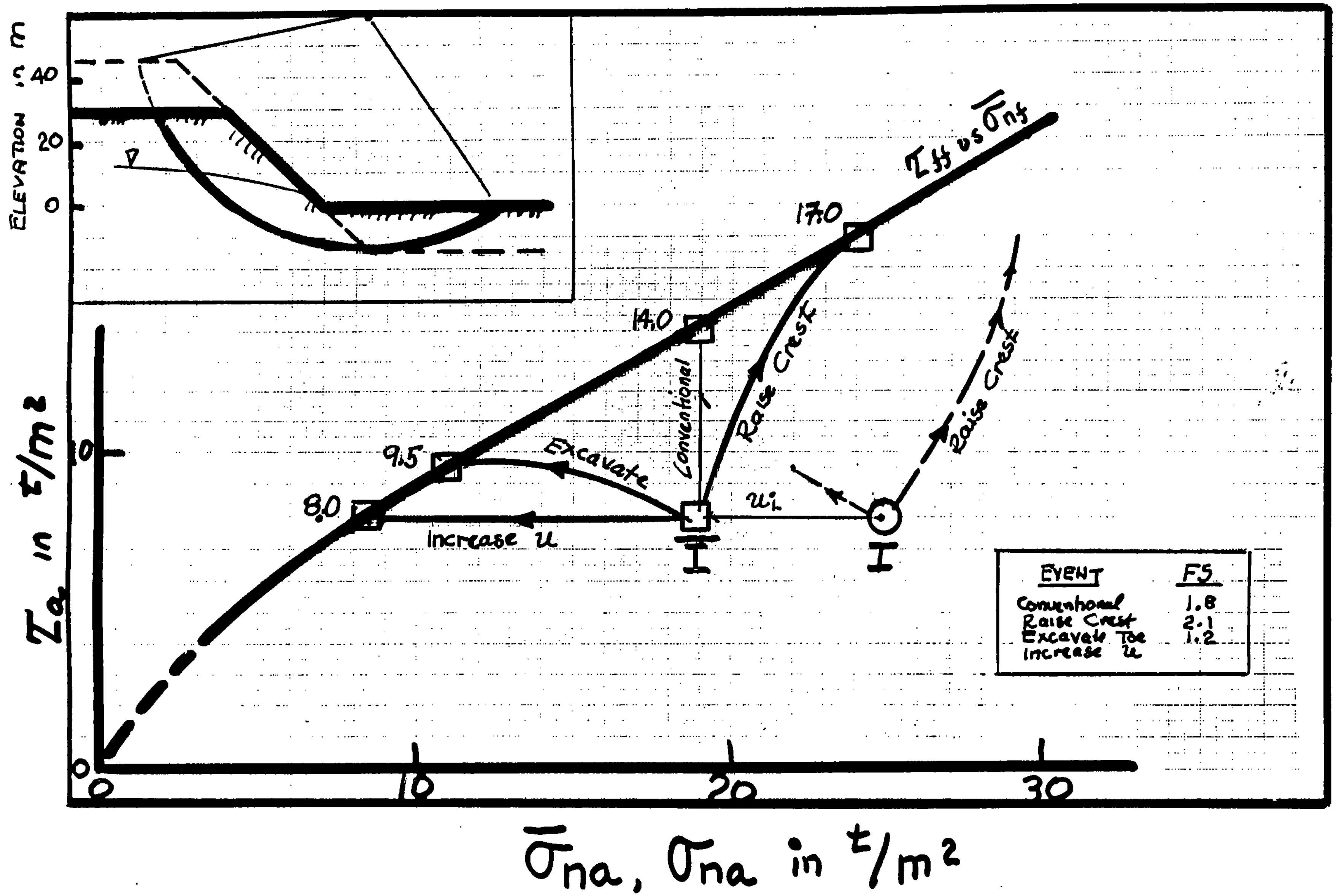
ELEVATION in m
40
20
0
τ_{ff} vs $\bar{\sigma}_{nf}$
17.0
14.0
9.5
8.0
Raise Crest
Raise Crest
Conventional
Excavate
Increase u
u_i
I
II
EVENT
FS
Conventional
1.8
Raise Crest
2.1
Excavate Toe
1.2
Increase u
τ_a in t/m²
0
10
20
30
$\bar{\sigma}_{na}$, σ_{na} in t/m²

FIGURE 6

3. We see that the conventional procedure of assuming a vertical stress path to failure makes no sense.

STABILTY OF AN EARTH DAM

Let us now determine the factor of safety for the dam shown in Figure 7a. Figure 7b shows the calculated total stress path for building a dam to elevation 140 and then raising the reservoir to elevation 135. Pore pressure equals the average pore pressure along the study section based on a flow net for steady state seepage.

Figure 8 shows the calculation for the factor of safety for overbuilding the dam. We ran a stress path laboratory test with the results shown in Figure 8.

Figure 9 shows the effective stress path for failure from an increase in pore water pressure. Such a buildup in pore pressure would have to come from some exterior source such as a leak in a buried pipe or, as in the case of the Siburua Dam, from pore pressure from a nearby hill.

Figure 10 shows stress paths for a rapid lowering of the reservoir - i.e sudden drawdown. We ran a laboratory test starting at O,$\bar{O}$ and proceeding along the total stress path as shown. Figure 10 also shows an approximate calculation of factor of safety based on strength from Figure 3. The strength for a total stress path equals approximately half the strength for $\omega = 45^\circ$ i.e the standard $\overline{\text{CAU}}$ test.

CONCLUSIONS

1. Soil strength depends significantly on the effective stress path to failure.

2. To obtain the appropriate strength for a stability analysis the engineer should run a test along the field effective stress path.

3. The conventional methods of stability analysis based on an assumption of a vertical effective stress path to failure make no sense.

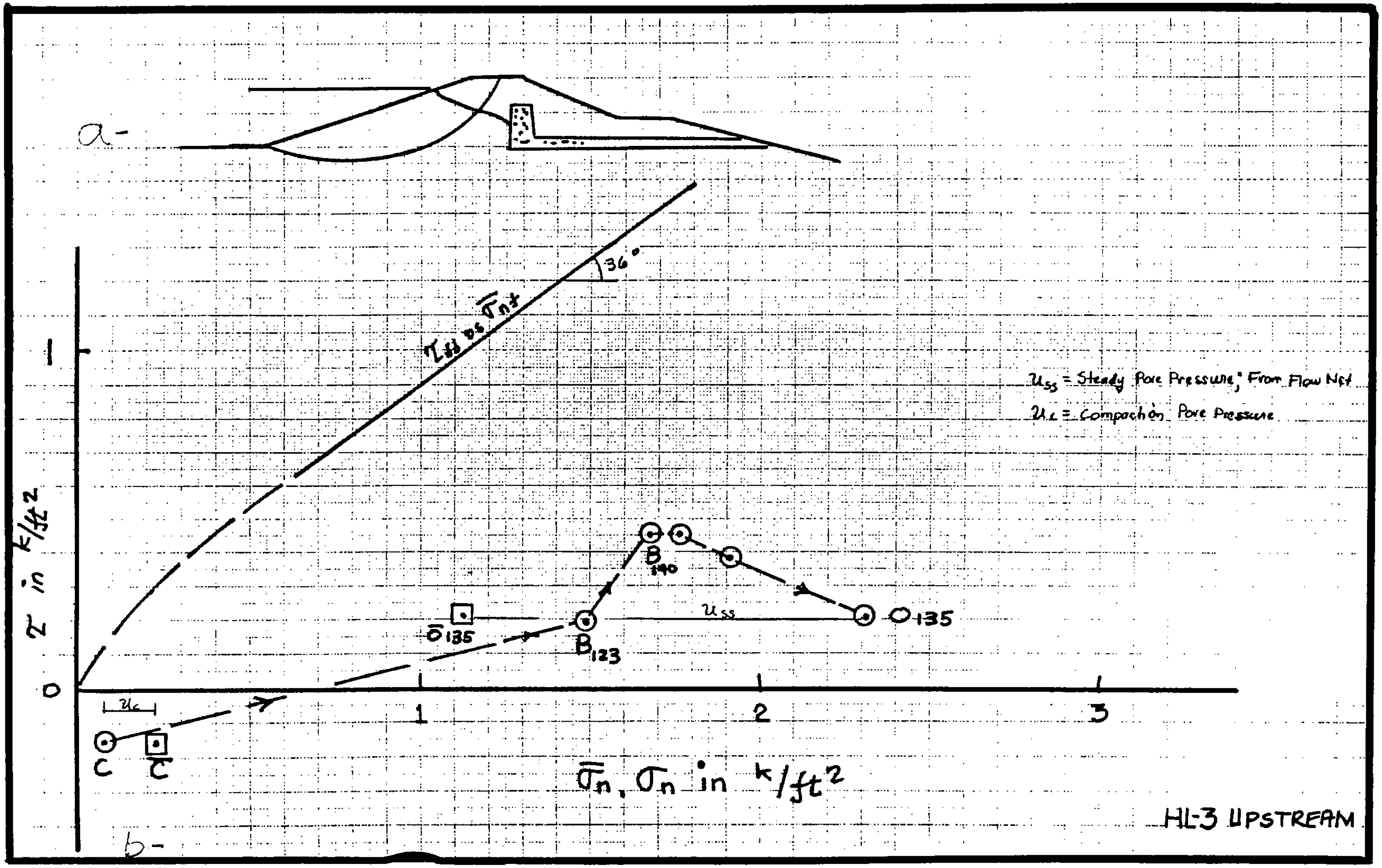

a-
b-
τ_{ff} vs $\bar{\sigma}_{nf}$
36°
u_{ss} = Steady Pore Pressure; From Flow Net
u_c = Compaction Pore Pressure
τ in k/ft²
0
1
2
3
u_c
C
$\bar{\tau}$
$\bar{\sigma}_{135}$
B_{123}
B_{140}
u_{ss}
σ_{135}
$\bar{\sigma}_n$, $\dot{\sigma}_n$ in k/ft²
HL-3 UPSTREAM

FIGURE 7

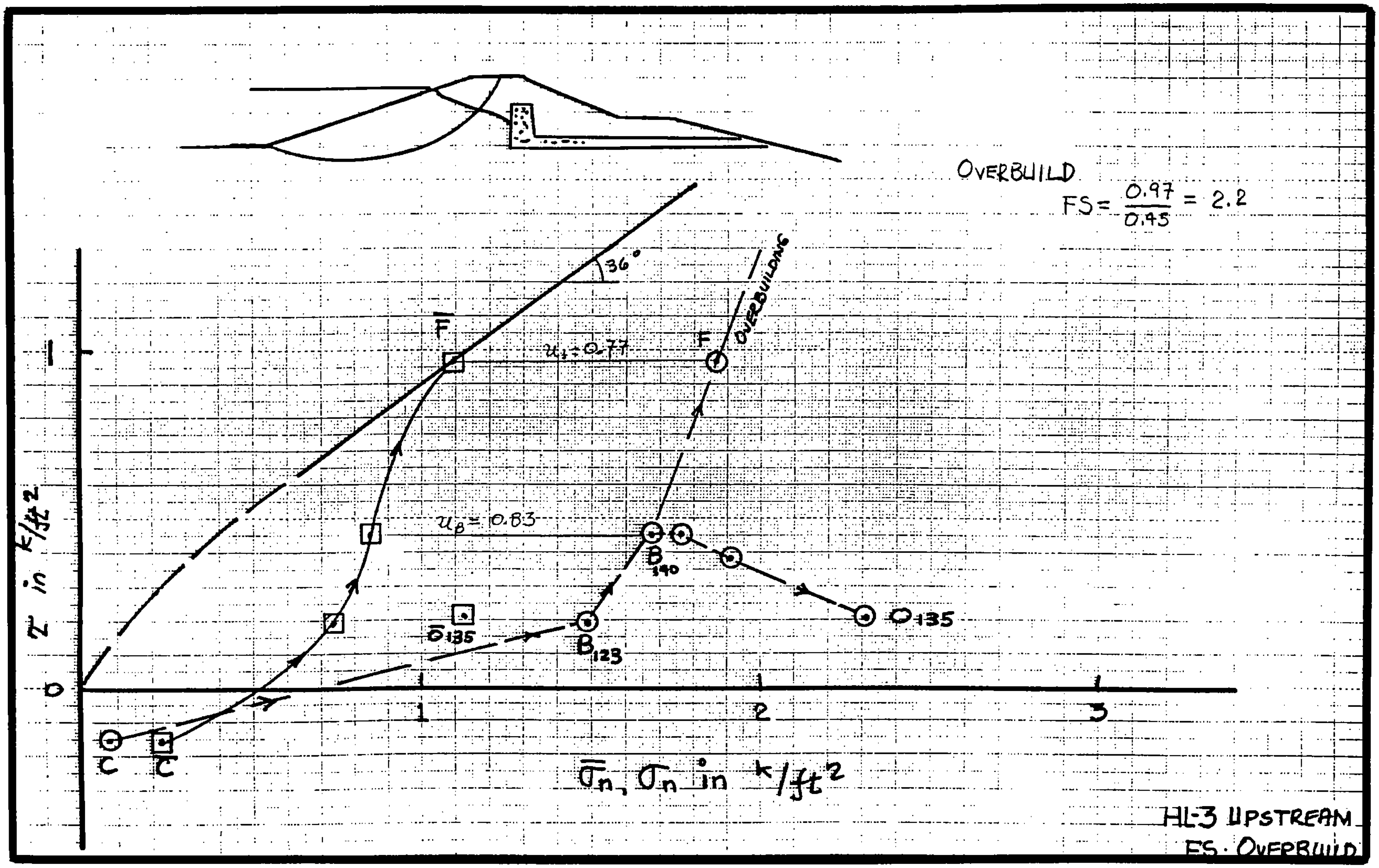
 OVERBUILD
FS = 0.97/0.45 = 2.2
36°
OVERBUILDING
F̄
F
$u_f = 0.77$
$u_B = 0.83$
B_{140}
B_{123}
$\bar{O}_{135}$
O_{135}
C
C̄
0
1
2
3
τ in k/ft²
σ̄n, σn in k/ft²
HL-3 UPSTREAM
FS. OVERBUILD

FIGURE 8

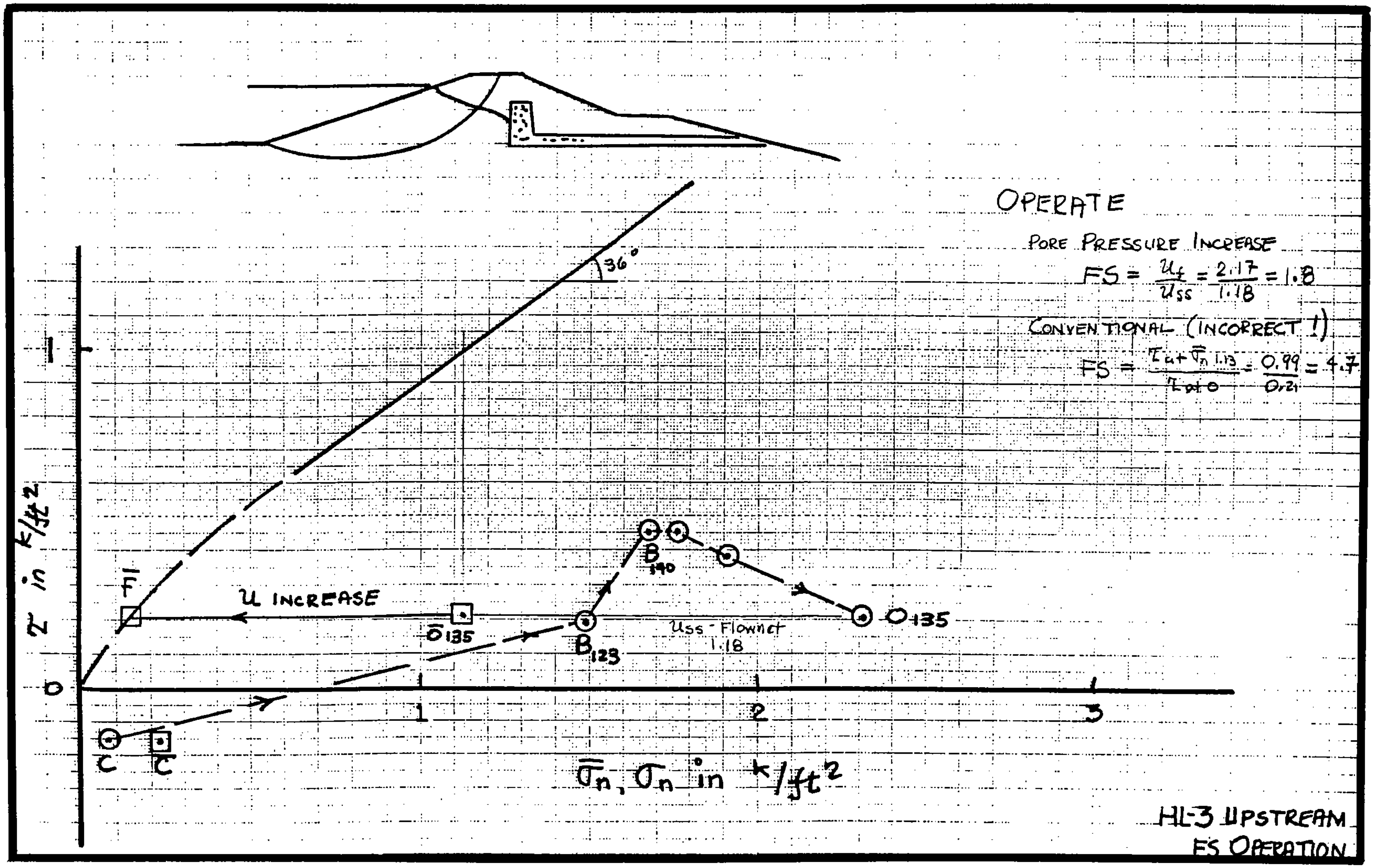
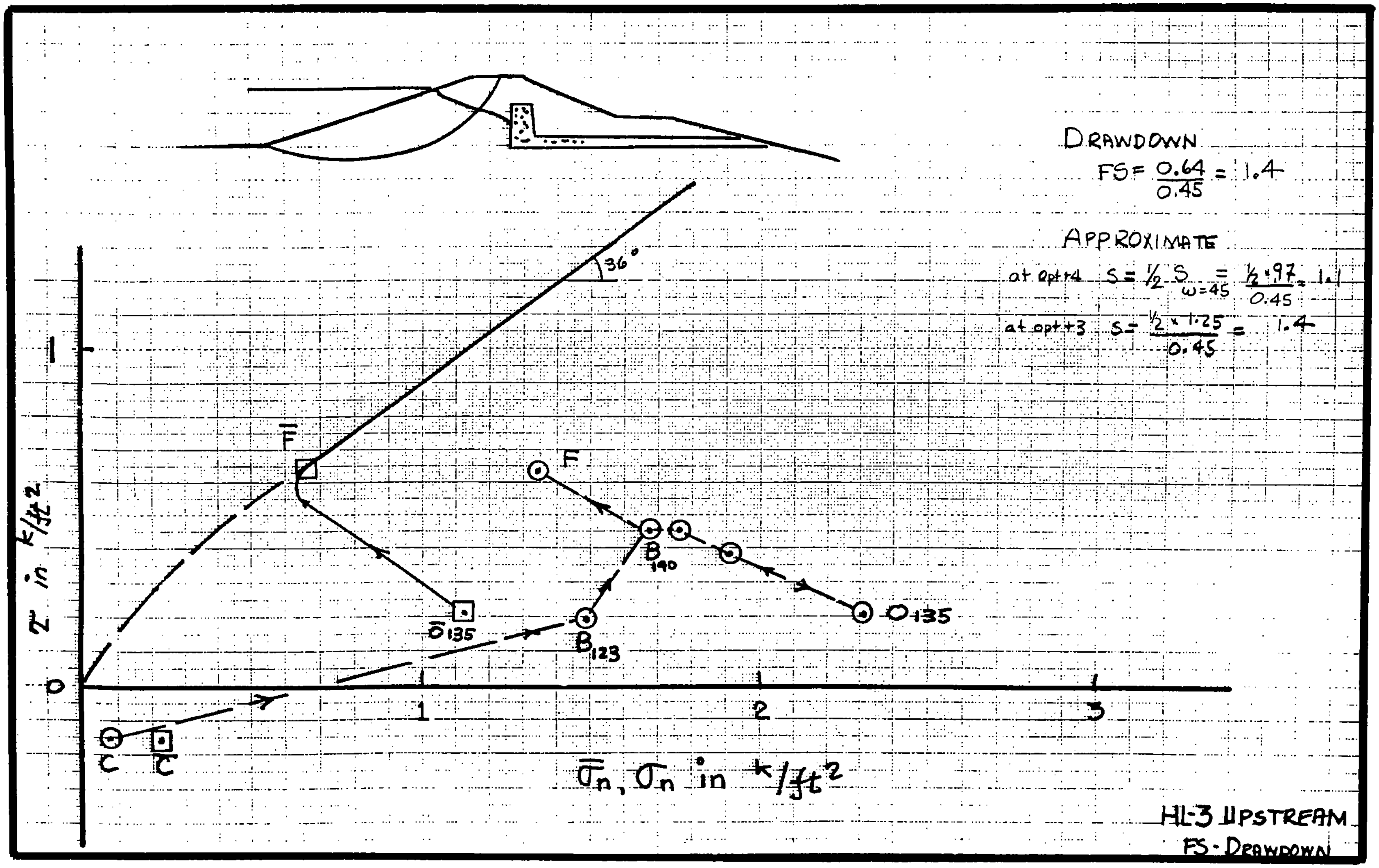

OPERATE
PORE PRESSURE INCREASE
$FS = \frac{u_f}{u_{ss}} = \frac{2.17}{1.18} = 1.8$
CONVENTIONAL (INCORRECT !)
$FS = \frac{\tau_{u+\bar{\sigma}_n 1.13}}{\tau_{at\,0}} = \frac{0.99}{0.21} = 4.7$
36°
1
0
τ in k/ft^2
F
u INCREASE
$\bar{O}$ 135
B 123
B 190
O 135
u_{ss} - Flownet 1.18
C
$\bar{C}$
1
2
3
$\bar{\sigma}_n, \sigma_n$ in k/ft^2
HL-3 UPSTREAM
FS OPERATION

FIGURE 9



DRAWDOWN

$$FS = \frac{0.64}{0.45} = 1.4$$

APPROXIMATE

at opt+4 $S = \frac{1}{2} S_{\omega=45} = \frac{\frac{1}{2} \cdot .97}{0.45} = 1.1$

at opt+3 $S = \frac{\frac{1}{2} \times 1.25}{0.45} = 1.4$

FIGURE 10