until refusal in the deeper dense sands was obtained.

A relieving platform type bulkhead along the shore property line permitted the placement of fill and roadway and act^ed as a final contribution to the stabilization of the plant area.

The plant has now been in full operation for seven years, during which time careful level and transit recirds have shown that there have been no further movements of soil or structures. A small maintenance crew is charged with the responsibility of keeping all openings in pavements and all joints between pavements and structures, caulked or cement grouted, so as to assure that no storm water will enter the soil in the paved area on the river side of the cut-off drainage trench. The same crew regularly cleans out the drainage pipe and the catch basins provided in the man holes in the drainage line. As noted above, the drainage pipe has been steadily discharging a stream of clear water at each of its river outlets.

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IV c 10

SLOPE STABILITY STUDIES FOR THE DELTA-MENDOTA INTAKE CANAL

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SUMMARY

Because of the complexity of the field conditions, the critical character of the materials involved and the unusual depth of the cut, the slope stability analysis for the Delta-Mendota Intake Canal was performed on the basis of data obtained by a detailed field investigation and laboratory testing program. The stability analysis was based upon the Swedish theory (slip circle) as developed by Petterson, Hultin, Fellenius, and others. Before applying the Swedish method to the study of the proposed slopes, the unit weight, cohesion, and internal friction values were determined on undisturbed soil samples in the Earth Materials Laboratory.

The purpose of this paper is to present the general procedures followed for the systematic use of field investigations, sampling, and laboratory testing with a well-known slope analysis method in obtaining a rational design of earth slopes.

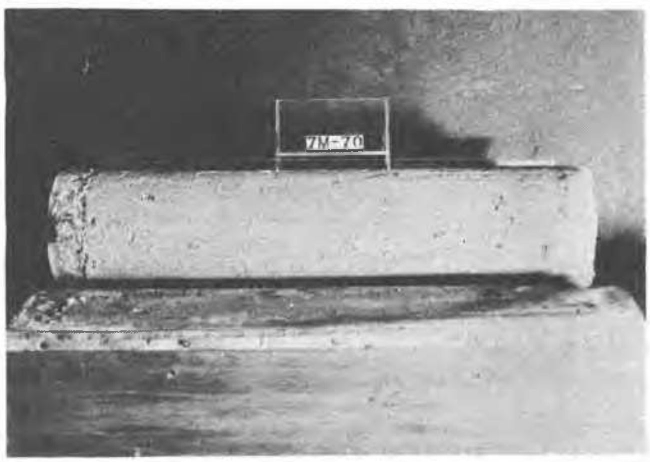
INTRODUCTION.

The intake canal for the Delta-Mendota Pumping Plant is located at the foot of the low-lying hills to the west of Tracy,California. Up to a cut depth of 80 feet at the pumping plant, preliminary estimates indicated that it was more economical to extend the intake canal rather than pressure pipes on the discharge side of the pumping plant. The logs of test holes and samples of the soil indicated a saturated cohesive soil (sandy clay,clayey sand, sand, and clay). For cuts deeper than 80 feet in this type of material, the required slopes would be flattened to such an extent that the cost of excavation plus the cost of additional right-of-way would be higher than the cost of a discharge pipeline. Due to the unusual depth of cut and the type of soil, it was deemed advisable to incorporate field experience and a mathematical analysis supported by laboratory test data in arriving at the final canal sections. The mathematical analyses which were performed on four sections (cuts 50, 65, 80, and 100 feet in depth) were based on the Swedish theory as developed by Petterson, Hultin, Fellenius, and others.

FIELD INVESTIGATIONS. SAMPLING. AND LABORATORY TESTING.

The investigation program consisted of drill-hole exploration, sampling, and laboratory testing to determine the characteristics of the natural materials in the cut area.

The soils throughout the cut area are sedimentary, lenticular clayey sands and sandy clays with some pockets of sand and clay.These sediments are compact and cohesive except for a few sandy beds, up to 7 feet thick, that are moderately friable. The clays are stiff and compact. The groundwater table lies at a depth



Dennison sample after casing was removed.

FIG.2

TABLE I
SUMMARY OF LABORATORY TEST DATA

Sample Identification				Mechanical Analysis Test Data						Triaxial Shear Test Data									
											Initial Specimen Data			Test Values at Failure				Shear Values	
Drill-hole No.	Proposed Cut (feet)	Sample Depth (feet)	Laboratory Sample No.	% Clay -.005 mm.	% Silt .005 to .05 mm.	% Sand .05 to 2.0 mm.	% Gravel + 2.0 mm.	Soil Classification	Specific Gravity	Specimen No.	Dry Density (pcf)	Moisture Content (%)	Degree of Saturation (%)	Applied Lateral Pressure (psi)	Pore Pressure (psi)	Effective Lateral Pressure (psi)	Deviator Stress (psi)	Tan φ	Cohesion (psi)
6A	100	100-102	7M-59	49	34	17	0	Lean clay	2.74	4	111.4	18.1	92.5	6.2	-1.8	8.0	57.1		
										1	107.6	20.0	93.0	12.5	0.3	12.2	53.8		
										2	108.5	19.5	92.5	25.0	7.3	17.7	60.8		
										3	109.6	19.1	93.4	50.0	19.8	30.2	96.9		
										6	111.3	17.4	89.1	75.0	31.1	43.9	145.6		
										5	112.9	16.8	89.4	100.0	32.0	68.0	151.0		
										Avg.	110.2	18.5	91.6					.53	11.0
		104-106	7M-60	32	31	35	2	Lean clay	2.73	1	102.3	23.0	94.8	12.5	4.5	8.0	23.5		
										3	100.0	24.1	93.9	50.0	33.6	16.4	40.2		
										4	100.0	23.7	92.2	100.0	62.9	37.1	75.5		
										Avg.	100.8	23.6	93.6					.53	3.0
		122-124	7M-61	35	36	14	15	Lean clay	2.74										
7	100	100-102	7M-62	23	23	47	7	Sandy clay loam											
38A	80	34-36	7M-63	32	42	21	5	Lean clay											
		56-58	7M-64	44	38	18	0	Lean clay											
		65-67	7M-65	37	24	38	1	Lean clay	2.69	5	115.4	16.1	95.1	12.5	5.1	7.4	90.6		
										6	116.1	15.7	94.9	25.0	10.4	14.6	126.0		
										7	115.6	15.8	93.8	50.0	7.9	42.1	147.8		
										8	115.3	15.9	93.7	100.0	35.3	64.7	188.8		
										Avg.	115.6	15.9	94.4					.48	28.3
		79-81	7M-66	22	26	46	6	Sandy clay loam	2.68										
		95-97	7M-67	13	39	39	9	Loam											
39	80	28-30	7M-68	45	38	17	0	Lean clay	2.73	4	100.5	24.4	95.9	6.2	4.3	1.9	20.9		
										1	100.7	24.1	95.2	12.5	9.6	2.9	22.6		
										2	101.8	23.4	95.1	25.0	21.0	4.0	25.0		
										3	98.6	25.6	96.4	50.0	42.3	7.7	29.6		
										Avg.	100.4	24.1	95.6					.47	5.8
		42-44	7M-69	24	22	53	1	Sandy clay loam	2.70	4	114.1	16.2	91.6	12.5	0.8	11.7	58.6		
										1	114.3	15.6	89.2	25.0	1.3	23.7	74.1		
										2	113.9	15.7	88.7	50.0	9.4	40.6	97.7		
										3	112.4	17.5	94.7	775.0	25.5	49.5	114.4		
										Avg.	113.7	16.2	91.0					.47	13.0
		62-64	7M-70	19	27	50	4	Sandy loam											
		77-79	7M-71	26	16	56	2	Sandy clay loam	2.69	1	110.0	17.7	90.8	12.5	3.3	9.2	31.6		
										2	112.3	17.0	92.5	25.0	9.4	15.6	52.5		
										3	111.8	17.0	91.1	50.0	27.7	22.3	55.7		
										4	115.0	16.5	96.6	100.0	68.4	31.6	82.4		
										Avg.	112.3	17.0	92.7					.60	4.0
		85-87	7M-72	40	39	17	4	Lean clay	2.71	1	112.2	17.7	94.6	12.5	8.0	4.5	84.8		
										5	114.2	17.0	95.8	25.0	9.2	15.8	116.2		
										3	112.3	17.8	95.7	50.0	24.1	25.9	117.9		
										6	114.9	17.0	97.7	75.0	23.2	51.8	151.9		
										4	112.8	18.0	97.6	100.0	82.7	17.3	88.9		
										Avg.	113.5	17.5	96.3					.45	25.7
40	65	12-14	7M-73	25	42	33	0	Clay loam	2.75	4	109.9	18.8	92.0	6.2	2.3	3.9	18.1		
										2	108.6	19.3	91.2	25.0	13.6	11.4	27.2		
										6	108.6	19.5	92.9	37.5	24.9	12.6	29.6		
										3	107.7	19.8	91.6	50.0	38.7	11.3	27.0		
										5	109.6	19.3	93.7	75.0	59.1	15.9	35.3		
										Avg.	108.9	19.3	92.3					.45	3.9
		32-34	7M-74	27	51	21	1	Silty clay loam	2.71										
		52-54	7M-75	5	5	89	1	Sand											
		59-61	7M-76	40	39	19	2	Lean clay	2.73										
		73-75	7M-77	49	34	17	0	Lean clay											
41A	50	24-26	7M-78	49	29	21	1	Lean clay											
		34-36	7M-79	57	32	11	0	Medium clay											
		44-46	7M-80	48	28	23	1	Lean clay											
		49-51	7M-81	12	6	82	0	Sand	2.68	1	112.0	17.4	94.4	12.5	-2.1	14.6	58.2		
										4	112.8	16.7	93.1	25.0	0.7	24.3	66.0		
										2	111.8	17.1	92.8	50.0	-1.6	51.6	178.3		
										3	112.1	17.0	92.5	100.0	-0.9	100.9	320.1		
										Avg.	112.2	17.0	93.2					.73	4.0
		63-65	7M-82	31	33	36	0	Lean clay											

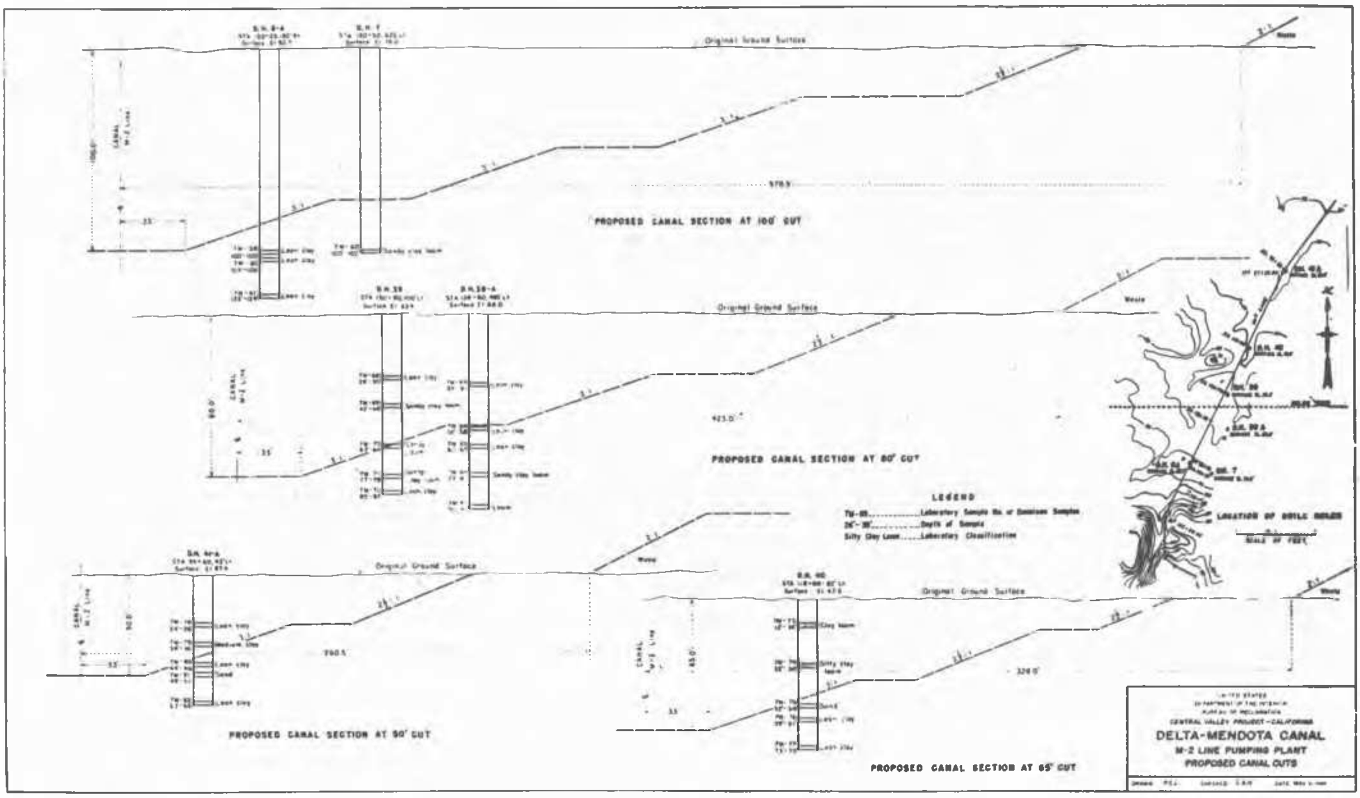
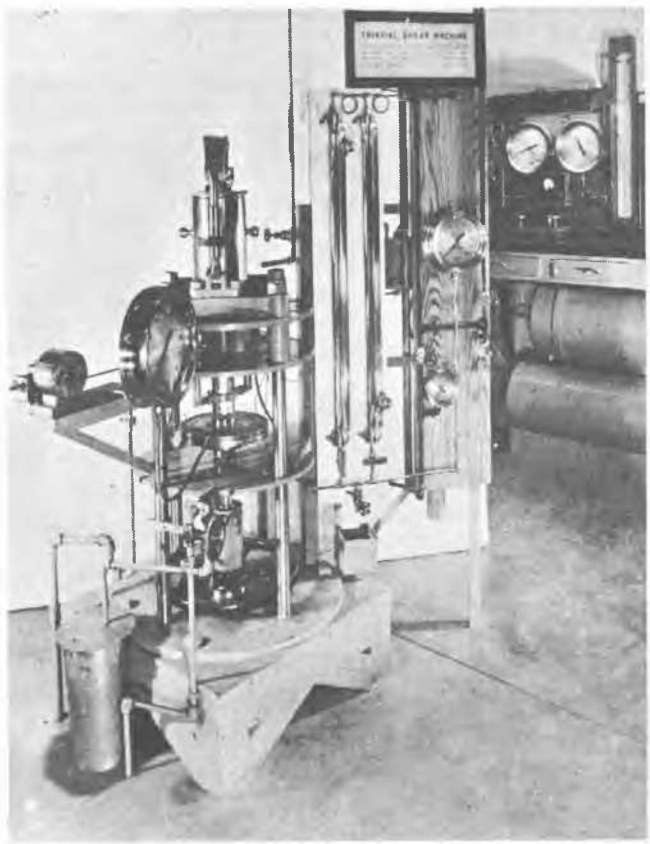


FIG.1

Triaxial shear specimen after failure.

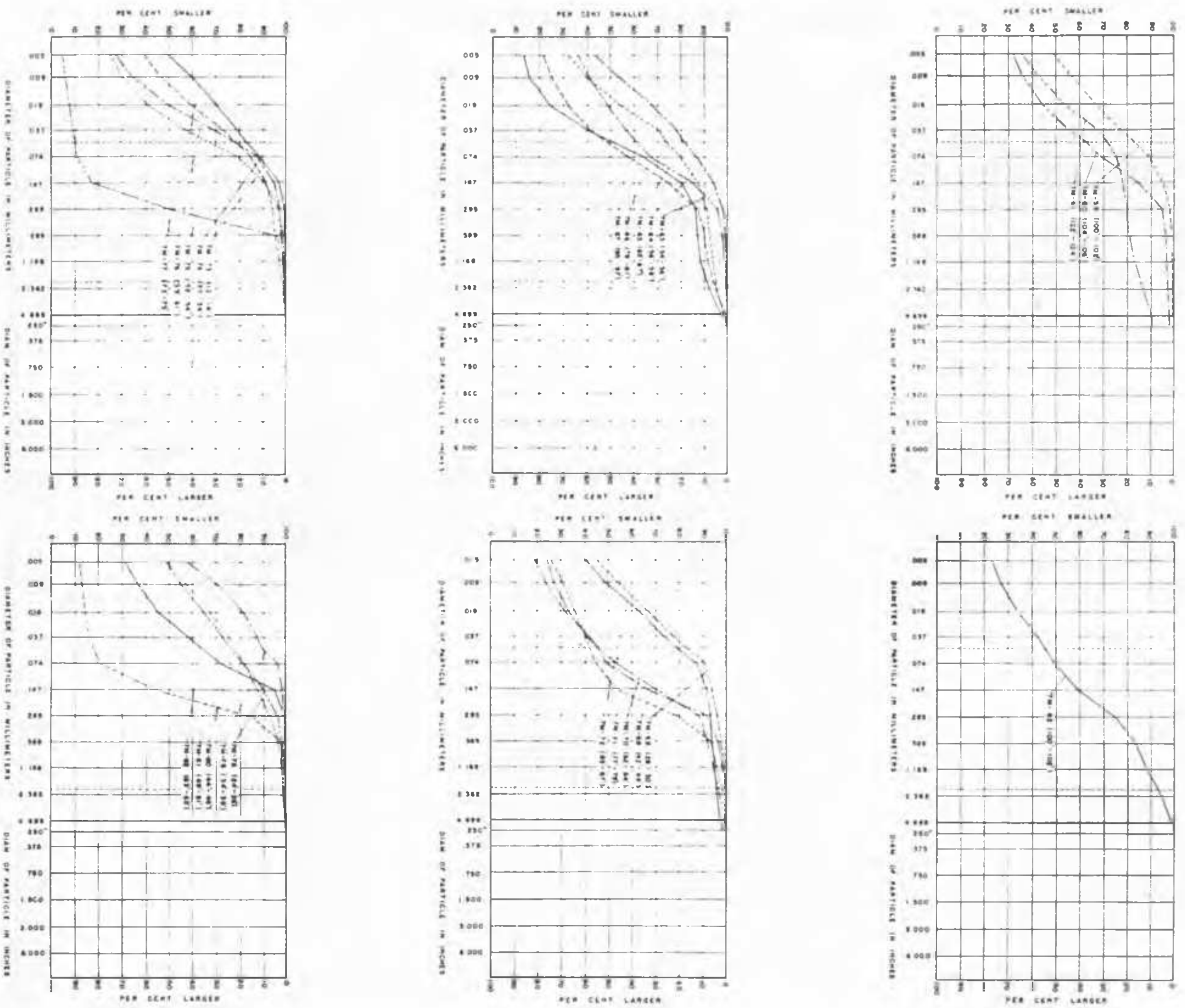
FIG.3



Triaxial shear machine.

FIG.4

of approximately 12 feet in the test holes. Three 2-inch diameter drill-holes were put down along the proposed inlet canal line, and the drill-holes were logged. Using the drill-core data as a basis for location, six Denison holes were drilled and 6-inch diameter soil cores were secured. These undisturbed cores were shipped to the Denver laboratories for testing. Extreme care was taken in drilling, handling, shipping, and opening the samples so that disturbance to the samples was minimized. The lo-



DELTA-MENDOTA INTAKE CANAL
FOUNDATION MATERIALS
GRADATION CURVES

FIG.5

cations of the exploration holes and the cross-sections of the proposed canal cuts, with the depths and classifications of the samples, are shown on Figure 1. Figure 2 is a picture of a typical Denison sample after the casing was removed.

The samples were opened in the laboratory and were inspected by members of the design and laboratory staffs, and the holes were logged. Gradation and visual classification tests were performed on each sample. These data are included in Table 1, and the gradation curves of the materials are shown on Figure 5. The soil cores were grouped as to soil types, and "representative" samples were selected for triaxial shear and consolidation tests. The consolidation tests were performed to secure information regarding the pumping plant structure foundation. As the foundation information is not pertinent to this paper, no further discussion of the consolidation tests will be made.

The triaxial shear tests were performed primarily to determine the internal friction and cohesion characteristics of the soil. Other data, including unit weight (dry and saturated), natural moisture, degree of saturation in place, and the pore pressure characteristics, were also determined. The triaxial shear tests were conducted on three or more small companion specimens from each "representative" sample. The small specimens, 1-3/8 inches in diameter by 2-3/8 inches long, were used in this testing program so that all of the companion specimens could be cut from the same horizon of the sample, thus insuring greater uniformity. At least two similar specimens, tested to failure under different lateral pressure conditions, are required to determine the internal friction and cohesion values. Since undisturbed specimens are rarely identical as to density, moisture, and composition, three or more specimens were tested in order that the results might serve as a mutual check.

The procedure for the triaxial shear tests performed in this program is given briefly as follows: The small cylindrical specimens were cut from a block of the Denison sample by means of knives and a trimming tool. Each specimen was then placed in a thin rubber sleeve, and the sleeve was clamped to perforated end-plates. The initial specimen volume was determined by weighting the specimen and container in air and water, the volume of the container being deducted from the total volume thus obtained. A specimen was then placed in the triaxial shear machine and the top perforated end-plate was connected to a "no-flow" pressure cell for pore pressure measurement. The pressure chamber around the specimen was filled with water and the desired constant lateral pressure was applied. Constant applied lateral pressures of 6.2 to 100 psi were used. After an initial con-

DELTA-MENDOTA INTAKE CANAL
FOUNDATION MATERIALS
MOHR'S ENVELOPES OF LIMITING SHEAR RESISTANCE

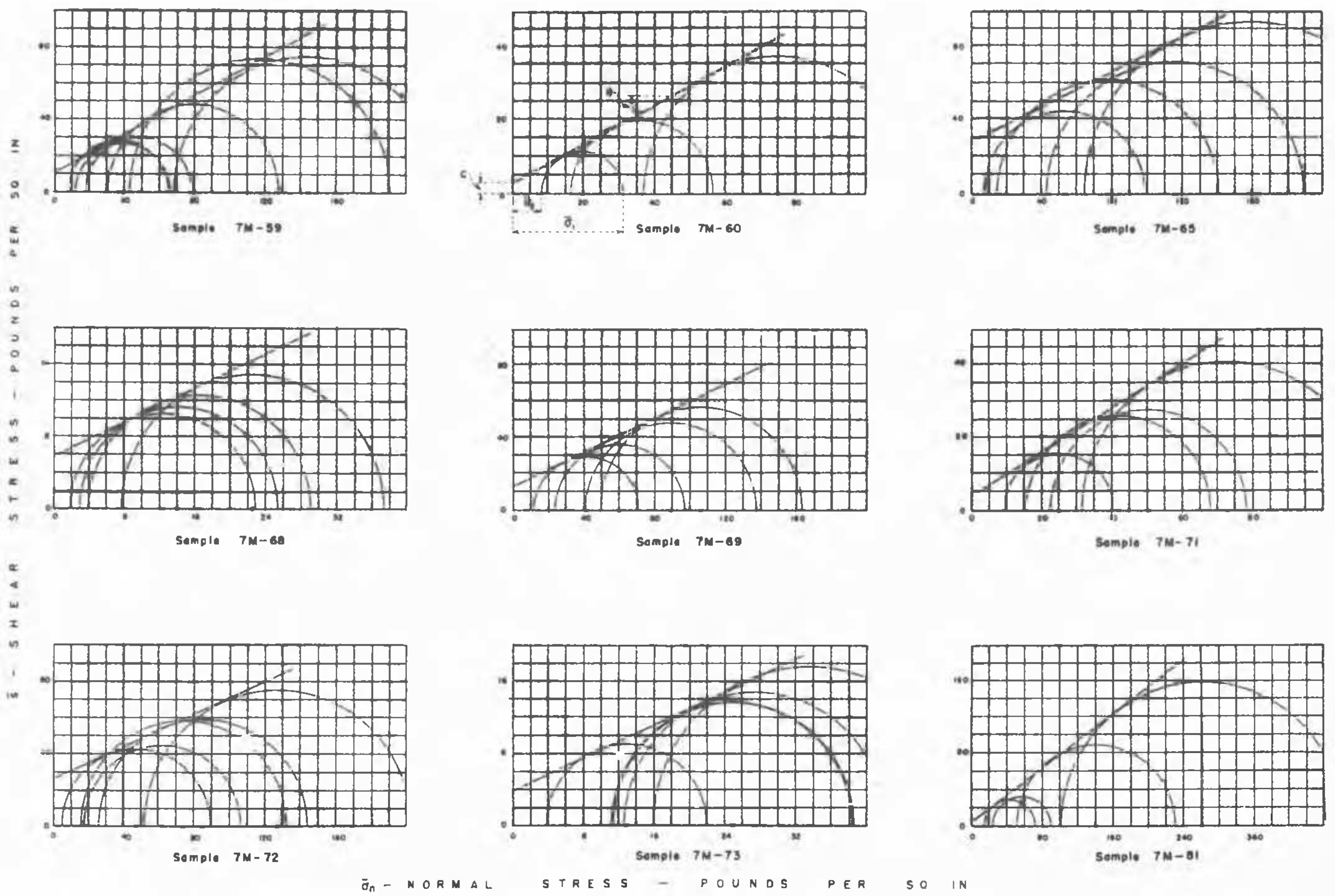


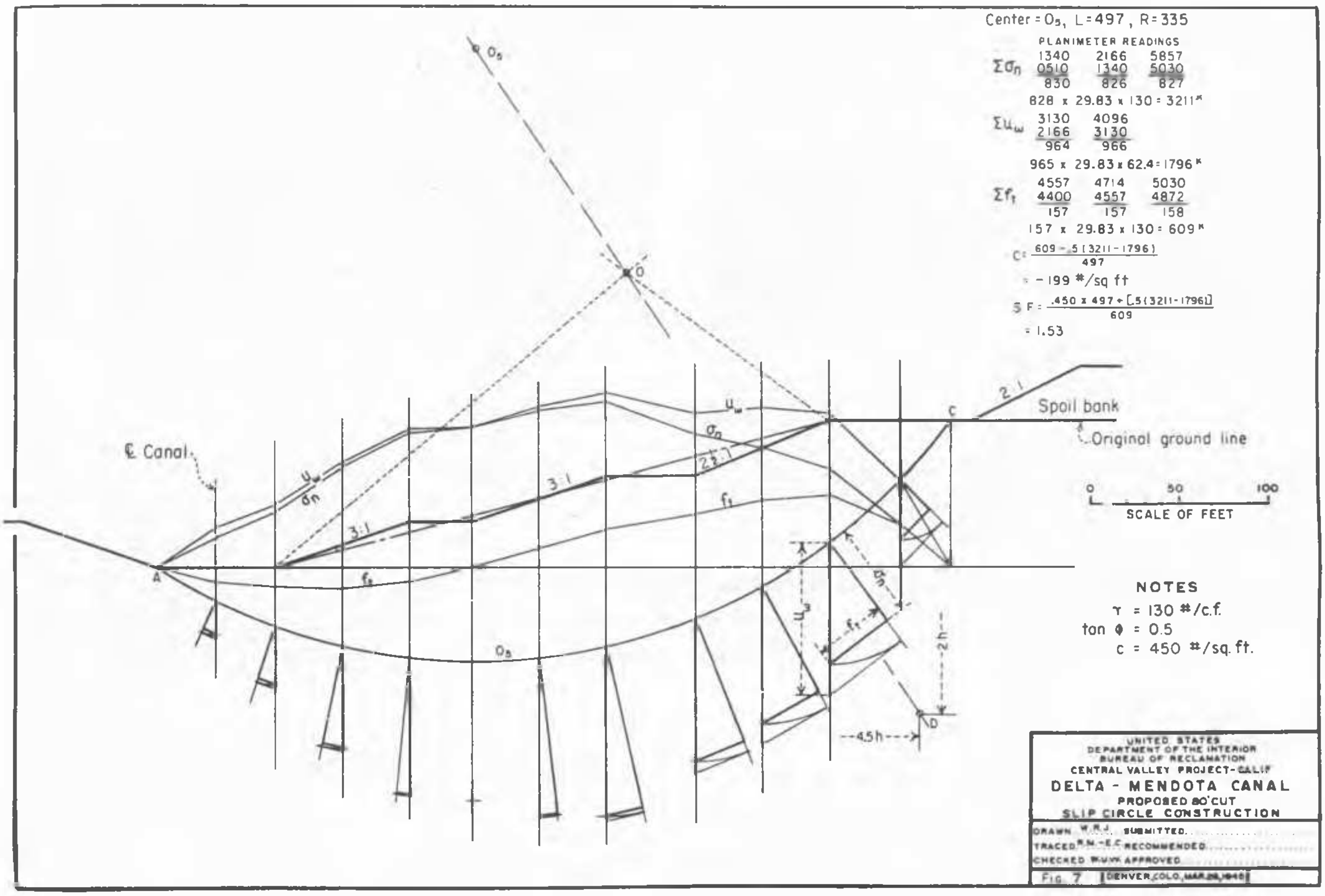
FIG.6

solidation period under the lateral pressure, the axial load was applied at a rate of 0.01 inch of axial strain per minute until failure occurred. Axial load, axial strain, volume change, and pore pressure measurements were taken at 2-minute intervals during the testing period. This procedure was repeated using other constant applied lateral pressures (from 6.2 to 100 psi) for each companion specimen of any one sample, and a relationship was thus established between the lateral pressure and axial pressures at failure. Figure 3 shows a typical triaxial shear specimen after failure, and Figure 4 shows the triaxial shear machine. The test data were analyzed graphically by the use of Mohr stress diagrams, and the internal friction and cohesion values were determined from the envelope of limiting shear resistance (Figure 6). A least squares method was used to determine the most probable tangent to the stress circles. For the construction of the Mohr diagrams, the effective lateral pressure was considered as the minor principal stress and the effective axial pressure to be the major principal stress, where the effective pressures equal the applied pressures minus the measured pore pressures. It is assumed that this correction for pore pressure allows the determination of the shearing resistance which exists in the absence of pore pressure.

Inasmuch as loading conditions imposed in the laboratory are different from those imposed upon a soil mass in a structure, and as drainage restrictions may also be different, the reduction of the sealed triaxial test data to zero pore pressure conditions offers a method of analysis that is more easily interpreted and applied. End pore pressure measurements were made, instead of insert measurements, because the end method is more applicable to standard test procedures. Previous laboratory studies have shown that accurate pore pressure measurements can be obtained at the ends of the specimens, provided rigid test procedures and adequate testing equipment are used.

<u>ANALYSIS OF CANAL SLOPES.</u>

The Swedish theory or "slip circle" as developed by Petterson is based chiefly upon two assumptions: (1) that, when failure occurs, the sliding surface is an arc of a circle, and (2) Coulomb's theory that the effective unit shearing resistance $\bar{s}$ of soils can be expressed as $\bar{s} = c + \bar{\sigma}_n \tan\varphi$ where c is the unit cohesion, $\bar{\sigma}_n$ is the effective unit normal pressure, and Ø the angle of internal friction. The cylindrical surface having the least stability is found by comparing the shearing force with the shearing resistance of the material along several possible surfaces of failure. The least stable circle is called the critical circle.



Proposed 80' cut. Slip circle construction.

FIG.7

When the slope is not a straight line or when a number of slopes must be investigated, the Swedish method using finite segments may prove lengthy and cumbersome as it involves the determination of the centers of gravity of irregularly shaped areas. The following procedure as developed by D.R. May (Transactions--Second Congress on Large Dams, Vol. IV, 1936) eliminates this feature and simplifies the solution for Σf_t and $\Sigma \sigma_n$. Values of f_t and σ_n for several points of the arc are plotted as ordinates and connected as shown in Figure 7. The areas between the respective curves and the X axis are planimetered and the total shearing force and the total normal pressure are found by multiplying the areas obtained by γ, the unit weight of soil.

The effect of groundwater is found in the same way. The total normal pressure and the total shearing force are found using the saturated weight of the material. The uplift of the hydrostatic pressure being always normal to a circular arc does not alter the total shearing force. It does, however, reduce the normal pressure available for friction between soil particles by the amount of the hydrostatic pressure that is transmitted across the arc through the water.

In solving for the safety factor, this uplift is subtracted from the total normal pressure found from the saturated weight of the material.

By denoting this uplift as Σu_w the factor of safety is given as:

$$SF = \frac{Lc + \tan\varphi(\Sigma\sigma_n - \Sigma u_w)}{\Sigma t_f}$$

where L is the length along the arc.

An example of this construction is shown in Figure 7 for the 80-foot cut.

Before applying the Swedish method to the study of the proposed slopes, the unit weight, cohesion, and internal friction values of undisturbed soil samples were determined by the Earth Materials Laboratory. The lowest cohesion value was used to minimize the effect of cohesion on the final result. The average values of the saturated weight and tan φ were used to simplify the slip circle calculations. These assumptions gave the following results:

γ = 130 lb per cu ft (saturated weight)

c = 450 lb per sq ft (cohesion)

tan $\emptyset$ = 0.50

Using these laboratory values of c, $\emptyset$, and the weight of the saturated soil, the various canal sections shown in Figure 1 were investigated by the Swedish method. As the depth of the water in the canal will be approximately 17 feet, the effect of this weight on the toe of the slip circle was neglected. Because of the saturated condition of the soil, the position of the groundwater surface and the possibility of slight pore pressure development, the total uplift of the hydrostatic pressures was assumed to be equal to the weight of a column of water based on a groundwater surface along the face of the cut. The material in the spoil bank was assumed to be without cohesion in these stability calculations. By trying several circles and noting the required cohesion values for each arc beside their respective centers, curves were drawn through these centers representing equal values of cohesion required for stability. From a study of the isostatic lines illustrated by these curves, the approximate center of the critical circle was more readily ascertained. On the basis of these analyses, the 80-foot cut was selected as the most economical section for the pumping plant.

CONCLUSIONS.

The results of the field investigations and laboratory testing indicated that although the materials were sedimentary, lenticular, clayey sands and sandy clays with some pockets of sand and clay, the friction values were fairly uniform and the tan φ values were closely grouped about 0.50. The cohesion varied considerably and a minimum cohesion value of 450 pounds per square foot was selected for the slope analysis.

The sections shown in Figure 1 were determined by the stability analysis. Under the design conditions, the slopes have a safety factor of 1.4 to 1.5. These slopes were in close agreement with those recommended by the field after they had inspected existing cuts (up to 50 feet deep) in the general vicinity. As a result of the analyses, field studies, and economical considerations, this design of the 80-foot cut was adopted and construction, which is now under way, will be completed in the spring of 1948.

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IV c 11 FAILURE OF A BIG RETAINING WALL AT " WIENFLUSS " CREEK-SHORE IN VIENNA, VI

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(Vienna, Austria)

INTRODUCTION.

A part, about 163 meters long, of the left shore retaining wall (about 10 meters high) at "Wienfluss"-creek in Vienna was displaced outward and deformed by pressure of clay-soil and fill at wall-back. Cause of that failure was erosion by ground water seapage in consequence of deteriorations of concrete lining of creek-bottom by bombholes. During repairwork slope of earthcut behind the wall slid down after a year of stability in an extent of about 50 meters in length, drawing with it two big flats at its top. That secondary earth movement occurred within the reach of influence of an artesian well. Safety against sliding had been computed following Prof. Fellenius to be at least 1,7 (-and was lateron reckoned at the base of observed rather logarithmic sliding-cylinder to be at least 1,5-), using soilmechanical characteristics found by testing "undisturbed" clay-samples drawn out of normal drillholes. So unstability of that terrain could not be recognised by soil mechanical methods adopted. Secondary sliding may have been effected by a lubricating layer between two layers of normal clay, that intermediate layer not having been detected by boring. On the other hand resistance of clay soil may have been found too great because of an eventual compression of the samples during sampling work.

The regulated bed of "Wienfluss"-creek in the inner districts of Vienna is bordered at left side by the retaining wall here in question. At right hand a lower wall is lining the creek-bed and at the same time the cut, containing municipal electric railway which runs along "Wienfluss"-creek in that part of the city. The second of that creek dealt with in this report is situated between 2 bridges and shown in figures 1 and 2, the latter photo having been taken before Vienna had been attacked by bombers. Sole of creek-bad is lined with concrete, about 0,6 m thick. Both shore walls are built up in concrete at the base and in stone masonry at the top, foundation of them not being deeper than 1 m below creek-bottom.

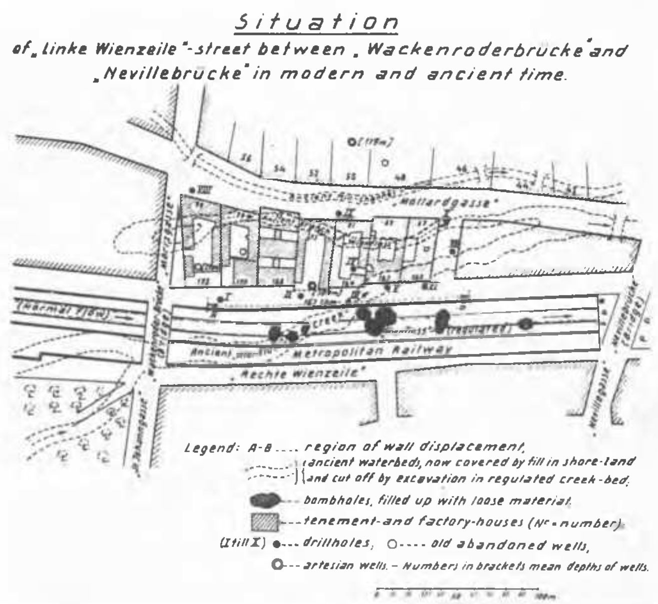

FIG.1

On April 23rd 1946 the above mentioned wall almost suddenly moved outside simultaneously tilting creek-ward. Dimensions of that retaining wall and the extent and manner of its displacement are to be seen in figure 3. The photos in figures 4 and 5 illustrate by view the immediate effects of that wall displacement. Figure 6 shows the steadily diminishing progress of wall movement after its first big step.

In order to investigate the cause of that disastrous phenomenon the topographic conditions (see figure 1) and the characteristics of the underground in situ had to be studied. 10 drillholes (I till X) and two investigationshafts were at once sunk down. The results drawn from them could be used for designing the geological profiles shown in figures 7 and 8. It may be recognised from these figures and the geological map of Vienna that the underground of the "Wien-