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SOIL INVESTIGATION AT BEAUHARNOIS ON THE ST. LAWRENCE SEAWAY

by

J. J. Hamilton

ANALYZED

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OTTAWA

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PREFACE

The design and construction of the St. Lawrence Seaway was one of the greatest civil engineering projects to be carried out in Canada during recent years. The Division of Building Research of the National Research Council was, therefore, privileged to arrange, in 1956, for a member of its Soil Mechanics Section (the author of this report) to work with the Soil Mechanics Section of the St. Lawrence Seaway Authority on some special soil problems.

This report describes the results of a study to evaluate the shear strength of soft sensitive clay by field and laboratory testing. Information thus obtained was put to immediate use by the Seaway Authority. It has added considerably to the gradual accumulation by the Division of geotechnical information on the Leda Clay, of such wide importance in the Ottawa and St. Lawrence valleys.

This report has now been compiled for record purposes. Significant test results were included in the paper by the author of this report and his colleague Mr. W. J. Eden presented it to the American Society for Testing Materials in 1957 (4).

It is a pleasure to record the Division's indebtedness to the General Manager of the St. Lawrence Seaway Authority, Mr. A. Gordon Murphy, for his willing agreement to and interest in the cooperative work herein reported, and for the facilities he kindly authorized and made available through Mr. F. L. Peckover, head of the Authority's Soil Mechanics Section.

Ottawa
October 1960

R. F. Legget
Director

SOIL INVESTIGATION AT BEAUHARNOIS ON THE ST. LAWRENCE SEAWAY

by

J. J. Hamilton

The St. Lawrence Seaway navigation channel bypasses the Hydro-Quebec Power Dam at Beauharnois, P.Q., through two locks with a combined lift of 82 feet. The two locks, each 869 feet long and 80 feet wide are connected by a 3800-foot long regulated channel. The Lower Lock, its approach walls and the channel were constructed in the dry through and on Potsdam Sandstone, which was originally covered by little or no overburden (1, 2).

In the Upper Lock area, however, the Potsdam Sandstone, on which the lock was to be founded was overlain by a deposit of soft sensitive marine clay known as Leda clay, which increased in depth in the upstream direction, reaching a maximum depth of 44 feet in one extremity of the construction area. The construction of permenent dykes on top of the deposit was necessary, in addition to deep excavations in this material. The relatively cramped area in which the lock had to be constructed (its alignment being almost parallel to the Beauharnois Power Canal), the possible seepage problems created by the proximity of the Power Canal and the possibility of flooding the Navigation Canal and the Lower Lock construction area if a slope failure occurred, were a few of the many factors which made the design of the construction slopes a matter for critical consideration.

A total stress analysis calculation procedure (after Janbu (3)) was employed in determining the factors of safety for various slopes in the area. The "end of construction" case was considered to be critical and undrained shear strengths by laboratory and in situ tests were required for the analysis. The limited time available and the need for a comprehensive coverage of a relatively large area increased the desirability of using an in situ test method for checking the soil strength measurements made on undisturbed soil samples in the laboratory.

RESEARCH PROGRAM

As a part of the cooperative research program between the Soil Mechanics Section of the St. Lawrence Seaway Authority and DBR, a special study of field vane testing was made. The purpose of the study was to evaluate the reliability of field vane apparatus for in situ testing of sensitive and extra-sensitive clays. The field program included several field vane and undisturbed sample borings in a test plot in the

Upper Lock area (see Fig. 1). Tests were made using different vanes and different test procedures and high quality undisturbed soil samples were obtained for laboratory study. The laboratory program involved various testing techniques designed to determine the shear strength parameters of the undisturbed soil samples. Other properties of the soils were measured as part of an over-all study of the geotechnical properties of Leda clay.

SOIL CONDITIONS

Generally, the soil profile in the Upper Lock area may be described as follows:

The clay shows weathering to a maximum depth of 15 feet and this layer is drier, stronger, and of a browner colour than the underlying unweathered clay. The next stratum is also approximately 15 feet in thickness. It is a soft grey clay, exceptionally weak in shear, sensitive to remoulding, of high natural water content and of high compressibility. Below this stratum, deposits of irregularly varved silty clay of lower compressibility, moisture content and sensitivity and slightly higher shearing strength are encountered, irregularly scattered over the Upper Lock area. Above the sandstone bedrock, a layer of well graded, pervious water bearing glacial till is found in thicknesses of from 1 to 6 feet. Irregular faulting as well as fragmentation of the upper few feet of the sandstone bedrock due to weathering results in a widely varying porosity throughout this material.

LABORATORY TESTING

In Table I, geotechnical properties of the clay such as Atterberg limits, grain size, specific gravity, salt concentration and liquidity index are listed. In Tables II, III and IV, the results of all the laboratory compression tests are listed (including undrained triaxial and consolidated undrained triaxial compression tests with pore pressure measurements). Table V presents a summary of laboratory vane shearing strengths and soil sensitivities. Table VI summarizes consolidation test results on undisturbed samples.

The shearing strength records obtained from unconsolidated, undrained laboratory compression tests, laboratory vane tests, and the best straight lines for undisturbed and remoulded Seaway vane tests plotted against depth in Fig. 7. In each of the compression tests, the shearing strength of the soil was assumed to be equal to one half the compressive strength, regardless of the amount of strain occurring in the sample at peak stress conditions.

Usually, high strains at failure and curved stress-strain diagrams were indications of disturbance of the samples. The higher shearing strengths measured were accompanied by low strains-at-failure. For the soft grey sensitive clay, strain-at-failure in excess of four per cent indicated sample disturbance while in the varved clay an indication of sample disturbance was given by strain at failure greater than 6 to 10 per cent (depending on the proportion of silt varves in the sample).

The unconfined compression tests were made by the Seaway Authority on samples taken from preliminary investigation borings. Generally, these samples underwent large strains before maximum stress was developed and it is believed that the samples were not of high quality.

The samples used in the triaxial compression testing by DBR were obtained by the use of a fixed-piston type of sampler and by techniques aimed at yielding the best relatively undisturbed samples possible. The critical dimensions of the sampler used were: outside diameter of sample tube 3.000 inches, inside diameter of sample tube 2.803 inches, inside diameter of cutting edge 2.770 inches, area ratio 17.3 per cent and inside clearance ratio 1.2 per cent. The net length of the sample tube was 24 inches. Usually three samples were tested from each tube for unconsolidated-undrained triaxial compression strength, one under each of the following confining pressures: 15, 30 and 45 psi.

Nine consolidated-undrained triaxial tests (with pore pressure measurements) were run on samples obtained by the piston sampler. In an attempt to duplicate field conditions, the samples were consolidated under various pressures which corresponded to effective stresses existing in the ground. Several samples were consolidated under pressures equal to the calculated effective overburden pressure at the depth at which they were obtained. During the consolidation period drainage was allowed from the sides through filter paper drains and from both the upper and lower ends through porous stones. The consolidation period was either 24 hours or until no further deflection was measured on the strain dial. Drainage was then stopped, the cell pressure increased to some convenient value and the deviator stress increased by a small increment dead load system until failure occurred. Pore pressure measurements were made following each loading increment.

In a few tests, membrane leaks caused the effective consolidation pressure to be very much lower than the calculated value and incomplete consolidation occurred. In tests in which the effective consolidation pressure was zero, the shearing strength was the same as that determined by the unconsolidated undrained triaxial tests.

LABORATORY VANE TESTS

Undisturbed shearing strength was determined by inserting a laboratory vane into an undisturbed sample confined by a wax and aluminum foil covering and measuring the torque and rotational strain during shear. The procedure of the "partially remoulded vane test" was designed to simulate conditions existing in the field vane remoulded test, i.e. without completely remoulding the entire sample, the vane was rotated four complete revolutions in the sample in the same position as the undisturbed test and the torque required to turn the vane after it had been allowed to stand for one minute after remoulding was a measure of the "partially remoulded" shearing strength. A "completely remoulded" test was performed on a container full of the same soil in a completely remoulded state at the same moisture content. The ratios of undisturbed shearing strength to "partially remoulded" shearing strength and undisturbed shearing strength to "completely remoulded" shearing strength were defined as the "partially remoulded" sensitivity and the "completely remoulded" sensitivity respectively.

FIELD TESTING

a) Apparatus

Field vane apparatus of two different designs were used in the field program for purposes of correlation and calibration. The equipment supplied by the St. Lawrence Seaway Authority was a modification of a commercial model manufactured by the Acker Drill Company. Designed as a portable, hand-operated device, it included a four-bladed vane with conical ends. The dimensions of the vane are: blade length 3.57 inches outside (blade length increased towards the centre due to the 45° conical tips on both ends); radius of vane (one half diameter of cylinder sheared) 0.991 inch and blade thickness 0.0547 inch. The operating equipment consisted of half-inch diameter vane rods, $1\frac{1}{2}$ inch O.D. casing, a torque wrench, a jar staff and drive weight for advancing the vane through stiff materials and a hand jack for pulling the vane and casing after a test.

According to the original design, the vane was to be advanced in a retracted position with respect to the casing (i. e. although the vane itself was not protected by the casing, it was held immediately at the end of the casing, with all of the vane rod retracted inside the casing). Usually the weight of two men applied to the casing was sufficient to advance the vane and casing through the soft clay. Also included in the original equipment were a jar staff and drive weight which could be used to drive the retracted vane and casing through more resistant materials. The casing was advanced only through soil which had already been tested and thus the disturbing effect of advancing the casing was minimized at the depth of the next test. After advancing the casing, the required distance (set by the specified spacing of vane tests), the vane was advanced $1\frac{1}{2}$ feet beyond the bottom of the casing with a smooth continuous thrust. A modification of the apparatus which provided a protective housing for the vane during the advancing thrust proved to be a great improvement in reducing the damage to the vane when resistant material was encountered.

A torque wrench was used to measure the torque applied through the vane rods to the vane. The rate of strain was controlled roughly by turning the torque wrench at a rate of one degree per second. The Seaway vane torque wrench was read to the nearest 5 inch/pound increment of torque and the calculated shearing strength was therefore within a range of accuracy of $\pm$ 10 psf. Small changes in the rate of angular rotation of the torque wrench had a significant effect on the scatter of shearing strength due to the rate of stress build-up in the vane rods at torques near the shearing torque and due to some uncertainty in observing the absolute maximum reading on the torque wrench.

The NRC vane apparatus was designed specifically for use in the soft marine clays of Eastern Canada and for use with a soil sampling drill rig. The vane is attached to uncased standard drill rods and torque applied through a gear drive mechanism. The applied torque is measured by the reaction of the gear drive on a dynamometer. A rate of strain of six degrees per minute is maintained by turning a hand crank at a constant rate. A more complete description of the NRC vane has been given by Eden and Hamilton (4).

The NRC vanes are four-bladed with 45° conical ends and are mounted on tapered shafts threaded to fit "A" drill rod. Two NRC vanes having the same over-all dimensions, outside blade length 5.60 inches and diameter 2.80 inches, but with blade thicknesses of 1/8 inch and 5/64 inch were used to investigate the effects of area-ratio on measured shearing strength. The area ratios of these vanes are 25 per cent and 10 per cent respectively.

The Seaway vane was used extensively in the Upper Lock area, following the calibration and correlation program. Tests were carried out throughout the winter months (1956-57) by the use of a heated portable drill shack. A crew of three was successful in completing a boring to approximately 40-foot depth (vane tests every foot) on the average each working day.

b) Vane Test Results

Figure 2 is a plot of all undisturbed and remoulded shearing strengths obtained with the Seaway vane in four borings in the same area. The best straight lines describing the average undisturbed shear strengths have been plotted for each of the three main soil strata encountered. The best straight line for all remoulded tests has also been plotted. The range of maximum and minimum values is indicated. In the soft grey sensitive clay, the maximum range of undisturbed shearing strength at any particular depth from one boring to the next was approximately 150 psf.

In the reworked banded zone and the grey irregularly varved clay stratum, the range of variation in shearing strengths at any particular depth increased considerably and the best straight line calculated for these results did not fall within the maximum and minimum limits as well as it did in the more sensitive clay above. The greater scatter of shearing strength results might be explained by the irregular nature of these deposits and the random occurrence of small stones. The best straight lines for these two lower strata showed a considerably higher rate of increase of shearing strength with depth than was shown for the sensitive grey clay above.

Shear strengths (undisturbed and remoulded) measured with the NRC 25 per cent area ratio vane are shown in Fig. 4. Values obtained with the 10 per cent area ratio vane are shown in Fig. 5.

c) Rod Friction

Past experience in testing sensitive Leda clay in the Ottawa area had indicated that in normally consolidated deposits at least, the effect of rod friction (the adhesion of soil to the uncased vane rod) on the total torque required to shear the undisturbed soil cylinder was insignificant and could be neglected. At Beauharnois, however, in the two adjacent borings in which the 25 per cent and the 10 per cent area ratio vanes were being tested, an interesting variation in remoulded shearing strength was observed. In Fig. 4, the uncorrected remoulded shearing strength appeared to increase almost linearly with depth from the test at 19 feet to the test at 31 feet; then after a reduction in magnitude, increased with depth from $32\frac{1}{2}$ feet to 43 feet. In Fig. 5, the uncorrected remoulded shearing strength increased with depth from 19 feet to 22 feet; from 25 to 28 feet, from 31 to

34 feet and from 37 to 40 feet, i.e., in three foot increments, with reductions in remoulded shearing strength between these increments.

This pattern of increasing remoulded shearing strength with depth coincided with the depth of vane below the bottom of the washed borehole. In the boring represented by Fig. 4, the hole was washed and cased to the 10-foot depth; then tests were run at $1\frac{1}{2}$ foot intervals to the 19-foot depth. The borehole was washed to the 17-foot depth; the vane tests were run in $1\frac{1}{2}$-foot intervals to the 31-foot depth. The borehole was washed to the 30-foot depth and the remaining vane tests were run at $1\frac{1}{2}$-foot intervals to the 43-foot depth.

In the boring represented by Fig. 5, the borehole was washed and cased to the 10-foot depth, rod friction tests (as described later) were carried out at $1\frac{1}{2}$-foot intervals to the $16\frac{1}{2}$-foot depth. The borehole was washed to the 17-foot depth, then tests were run with the small area ratio vane at 19-, $20\frac{1}{2}$-, and 22-foot depths. A rod friction test was carried out with the bottom of the rods at the $23\frac{1}{2}$-foot depth. The borehole was washed to $23\frac{1}{2}$-foot depth. Vane tests were run at the 25-, $26\frac{1}{2}$- and 28-foot depths. A rod friction test was carried out with the bottom of the rods at $29\frac{1}{2}$-foot depth. The borehole was washed to the $29\frac{1}{2}$-foot depth, vane tests were run at 31-, $32\frac{1}{2}$- and 34-foot depths. A rod friction test was run at $35\frac{1}{2}$-foot depth, the borehole was washed to $35\frac{1}{2}$-foot depth and vane tests were run at 37-, $38\frac{1}{2}$- and 40-foot depths.

It is apparent from these observations that rod friction has a considerable effect on the test result.

d) "Rod Friction" Correction Method

The rod friction test mentioned above involved inserting a string of standard "A" drill rods (fitted with the small conical point to prevent plugging and to improve driving qualities) into undisturbed soil a certain distance and measuring the amount of torque required to rotate the rods at the rate of 6 degrees a minute. The results of these tests are plotted (Fig. 3) in terms of dynamometer force per foot length of rod in contact with the soil versus depth. The curve thus obtained was used to calculate the correction that had to be subtracted for any particular length of vane rod in contact with the soil at any depth. The results of subtracting this correction from each of the undisturbed and remoulded strength values are shown in Figs. 4 and 5.

e) "Remoulded Strength" Correction Method

In this correction technique, the "minimum" field vane remoulded strength is used as a basis for correcting for rod friction in all other tests. Laboratory testing in this clay has revealed that the remoulded shearing strength is lower than that found by field vane methods. Reference to Table V and Fig. 2 reveals that the "partially remoulded" laboratory vane shearing strength averages about 80 psf lower than the average field vane remoulded strength. The "completely remoulded" laboratory vane shearing strength may be as much as 200 psf lower than average field vane remoulded strength. The average rate of increase of remoulded strength with depth is the same for laboratory and field determinations.

It is reasoned that most of the increase in measured remoulded strength between borehole washing is due to increased length of rod in contact with the soil. A straight line was therefore drawn through the minimum remoulded strength values shown on Fig. 5. This line is considered to represent the true remoulded strength value. All undisturbed strength values (shown in Figs. 4 and 5) were then corrected by subtracting from them an amount equal to the difference between the actual measured remoulded strength and this assumed true remoulded shearing strength. As can be seen in Figs. 4 and 5, the two correction methods give comparable results.

DISCUSSION

Results obtained by the three vanes (the NRC 25 per cent and 10 per cent area ratio vanes and the Seaway vane) are compared in Fig. 6. It appears that the average shearing strength as determined by the Seaway vane is higher than a similar average which could be calculated for the two NRC vanes. It is also observed that the remoulded shearing strength by the Seaway vane is higher than the corrected remoulded strength from the NRC vane tests. By referring to Table V, it can be seen that the laboratory determined partially remoulded shearing strength lies between that determined by the NRC and Seaway vanes. It might be concluded therefore that a correction should be applied to the Seaway vane results (undisturbed and remoulded) to bring the remoulded strength in line with that measured in the laboratory. This would tend to bring the average undisturbed strengths into line with those obtained by the NRC vanes. Since the Seaway vane rod was cased, the nature of the correction which might be applied to the indicated shearing strengths would be somewhat different from that applied to the NRC vanes tests.

It is suspected that friction between the vane rod and the couplings of the casing which acted as loose-fitting bearings in the Seaway equipment might be the source of an error requiring correction. The friction between the vane rod and casing would be greatly affected by the alignment of the boring and by the eccentricity of load application through the torque wrench. The use of a two handled torque wrench would greatly reduce the latter; the former could only be completely eliminated by the use of a very rigid casing. No satisfactory arrangement of calibration for these unknowns was devised and no corrections have been made to the results plotted. A possible correction method would be to follow a technique similar to the "remoulded strength" correction method used with the NRC vane by subtracting a correction factor based on the difference between the laboratory determined remoulded strength and the Seaway vane remoulded strength at any depth, from the undisturbed field vane shearing strength determined for that depth.

For comparison purposes all the laboratory determined shearing strength results are plotted on Fig. 7 along with the best straight lines for the undisturbed and remoulded Seaway vane tests. The shearing strengths obtained from the laboratory testing are scattered over a wide band ranging from values near the field vane remoulded shearing strength up to values slightly lower than the average field vane undisturbed shearing strength. In general, the shearing strengths (one half the maximum compression stress) are slightly higher than those obtained by the Seaway Authority unconfined compression tests. The strains-at-failure are also somewhat lower. This might be an indication of slightly less disturbed samples obtained by the improved sampling techniques.

The majority of shearing strengths thus obtained fall between 1/3 and 2/3 the values obtained by field vane testing with only a very small number of tests giving equal values. The closest agreement between field vane and unconsolidated undrained triaxials was found in the clays of lowest plasticity and liquidity index while the greatest disparity occurred in the clays of highest plasticity and liquidity index. It is believed that sample disturbance due to change of stress conditions is of greatest significance in clays of high plasticity and liquidity indices. There is considerable evidence to support the theory that the sensitivity of marine clays is related to liquidity index (4, 5) with sensitivity increasing logarithmically with increase in liquidity index.

Shearing strengths obtained by the laboratory vane apparatus agree quite favourably with those of the field vane as shown in Fig. 7. In the laboratory vane test, the sample undergoes a minimum amount of trimming disturbance and the stress conditions in the centre of the sample, where the vane test is carried out, are probably fairly close to field in situ conditions. The "partially remoulded" shearing strengths agree fairly well with minimum field vane remoulded strengths. The ratio of "completely remoulded" to "partially remoulded" shearing strengths varies from 1/5 for the soft grey sensitive clay to ½ in the grey irregularly varved clay to one in the weathered clay and silty till. In stiff clay the completely remoulded shearing strength is sometimes higher than the partially remoulded, indicating that once the cylinder has been completely sheared by the rotation of the vane, there is little resistance of the sheared cylinder with respect to the soil mass around it.

In Fig. 8, the results of consolidation testing on these samples are plotted along with effective stress curves for conditions at the time of sampling and for conditions prior to the construction of the Beauharnois Power Canal in 1930 (6). Prior to the construction of the Canal the groundwater table probably dropped seasonally to the 14-foot depth which is the maximum depth of oxidation of this soil. This groundwater lowering has slightly overconsolidated the soil below. With the construction of the Power Canal a higher stable groundwater elevation was established and the effective stresses thereafter were reduced to the conditions existing at the time of sampling.

Figure 9 shows the Mohr's circles and envelope in terms of effective stresses for the consolidated-undrained triaxial tests (with pore pressure measurements), on the soft grey sensitive clay. Figure 10 presents the same information for tests on the reworked banded clay. The effective stress parameters for the two soil strata investigated were: for the soft grey sensitive clay: $c^1 = 1.9$ psi; $\phi^1 = 21°$ and for the reworked banded clay, $c^1 = 1.7$ psi; $\phi^1 = 31°$.

Figure 11 is a plot of undrained shearing strength versus pressure for shearing strengths obtained by various testing techniques. The average field vane shearing strength when plotted against a maximum preconsolidation pressure from consolidation results gives the best relationship with

$$\frac{c}{p} = 0.38.$$

The laboratory vane shearing strength when plotted against the maximum preconsolidation pressure from consolidation tests results gives some results which compare favourably on this plot with field vane results while other tests seem to have suffered from the effects of stress reduction.

The effects of stress reduction following sampling are best demonstrated by the plots of the shearing strength determined by unconsolidated-undrained compression tests against maximum preconsolidation pressure from the consolidation tests and against calculated effective overburden pressure. These shearing strength values seem to be a function of effective overburden pressure (and therefore the pore water pressure) at the time of sampling (independent of the confining pressure to which they have been subjected during the laboratory tests) rather than a function of the maximum load to which the soil has been subjected in the past (preconsolidation pressure). The hypothesis, that in undrained tests on laterally unconfined samples pore water tension created by meniscus effects at the surfaces holds the sample from rebounding, cannot hold true in soils containing pore water with dissolved gases or gas bubbles which come out of solution or grow larger with the increased temperatures and reduced pore water pressures of laboratory testing conditions. Unconsolidated undrained compression tests on samples which have been overconsolidated to any extent in their past history are of questionable value, in this material at least.

Figure 12 is a plot of $\frac{c}{p}$ versus plasticity index (after Skempton 1953 (7)). There appears to be no significant relationship between these two parameters for the testing techniques and soils studied.

CONCLUSIONS

1. In the soft grey sensitive clay, the Seaway vane and the NRC vanes give reproducible results from one boring to the next at comparable depths. The reproducibility in the lower strata is not as good, probably reflecting the lack of homogeneity in these materials.

2. The effect of the size of the area ratio of vane has not been conclusively shown. There is, however, some evidence that vanes of small area ratios (approximately 10 per cent) may give slightly higher shear strength values than will larger area ratio vanes. There is also some evidence that the over-all dimensions of the vane (i. e. the size of cylinder sheared) may have some influence on the shearing strengths measured. Small vanes may show very high shearing strengths when erratic stones, etc., are encountered. Larger vanes tend to average out the effects of small irregularities in the soil.

3. Rod friction is considered to be a significant factor at least in the soil deposit investigated at Beauharnois. Two correction methods have been suggested; the author prefers the method based on field determined rod friction values. This study points to the value of a completely cased vane rod as found in the equipment developed by the Acker Company and the Norwegian Geotechnical Institute.

4. It would appear that the laboratory vane test gives a reasonably good correlation with field vane results. Other testing techniques only approach the higher values obtained by field or laboratory vane tests with their maximum values, with many of the results lying closer to the remoulded

vane strengths. A good correlation between increase in strength with depth and ϕ^1 from the effective stress test has been shown.

5. The field vane provided a rapid and economical extension of the information required for the design of construction and permanent slopes in the Upper Lock area.

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SOIL TEST SUMMARY RECORD — CLASSIFICATION

PROJECT: P154

REMARKS: Co-op. Research St. Lawrence Seaway Auth. — N.R.C.

LOCATION: Upper Lock Beauharnois

HOLE No. 623 A Located 4' South of S.L.S.A. Hole #623

Tube SAMPLE Nº	SOIL DESCRIPTION	DEPTH (FEET)	GEODETIC ELEV. (FEET)	NATURAL W/C (%)	L.L. (%)	P.L. (%)	P.I. (%)	GRAIN SIZE PERCENTAGES				G	Salt Conc. gm/litre	Liquidity Index	REMARKS
								GRAVEL	SAND	SILT	CLAY				
	Ground Surface Elevation 146.5' (Geodetic)														
76-1	Mottled Grey Brown Silty Clay	7'-11" 9'-5"	138'7" 137'-3"	34.9	61.6	27.6	34.0	—	—	45	55	2.80		0.215	
76-2	Grey Sensitive "Leda" Clay	16'-6" 18'-6"	130'0" 128.0'	65.7	51.1	22.3	28.8	-		41.5	58.5	2.78	0.51	1.509	
76-3	" "	18'-6" 20'-6"	128'0" 126'0"	76.5	57.5	23.4	34.1	-	-	35.0	65.0	2.80		1.557	
76-4	" "	20'-6" 22'-6"	126'0" 124'0"	77.8	65.7	24.8	40.9	—	-	33.0	67.0	2.80	0.10	1.29	
76-5	" "	22'-6" 24'-6"	124'0" 122'0"	81.5	61.7	25.5	36.2	-	-	32.0	68.0	2.82	0.51	1.55	
76-6	" "	24'-6" 26'-6"	122'0" 120'0"	77.8	68.5	25.1	43.4	-	—	29.5	70.5	2.80	0.43	1.21	
76-7	" "	26'-6" 28'-6"	120'0" 118'0"	79.6	68.5	26.4	42.1	-	-	28.0	72.0	2.80	0.48	1.26	
76-8	Red Banding Material	28'-6"	118'-0"	53.3	55.0	24.6	30.4			38.0	62.0	2.79	0.20	0.94	Grey Clay with Red Bands up to 2½" Thick (some shells)
76-8	Grey Banding Material	30'-6"	116'-0"	73.2	62.0	25.7	36.3	—	—	38.0	62.0	2.75	0.64	1.35	
76-9	Grey Sensitive Clay	30'-6" 32'-6"	116'-0" 114'-0"	53.3	57.5	31.6	25.9	—	—	37.0	63.0	2.80		0.84	Sample contained ½" angular rock
76-10	Grey Clay with some red banding	32'-6" 34'-6"	114'-0" 112'-0"	49.1	44.2	21.8	22.4								
76-11		34'-6" 36'-6"	112'-0" 110'-0"	64.2	58.0	24.8	33.2					2.81	0.50	1.19	
76-12		38'-6" 40'-6"	108'-0" 106'-0"	48.5	51.2	26.4	24.8							0.89	
76-13		40'-6" 42'-6"	106'-0" 104'-0"	33.0	34.1	19.4	14.7					2.79		0.92	

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TABLE I

SOIL TEST SUMMARY RECORD - STRENGTH

PROJECT: P154

REMARKS: Co-op Research St. Lawrence Seaway Authority - N.R.C.

LOCATION: Upper Lock Beauharnois

Hole #623A Located 4' South of S.L.S.A. Hole #623

SAMPLE Nº	SOIL DESCRIPTION	DEPTH (FEET)	GEODETIC ELEV. (FEET)	NATURAL W/C (%)	NATURAL DENSITY (P.C.F.)	LATERAL PRESS (P.S.I.)	COMP. STRENGTH (P.S.I.)	FAILURE STRAIN (%)	Φ	C Kg/cm²	TYPE OF FAILURE (SEE BELOW)	∠ of Failure Plane	Pore Pressure p.s.i.	REMARKS:
	Ground Surface Elevation 146.5' (Geodetic)													
76-1-1		8'-0" 8'-3"	138'-6" 138'-3"	34.0	123.9	30	28.2	12			1	48°		mottled brown stiff
76-1-3		8'-5" 8'-8"	138'-1" 137'-10"	35.9	122.4	45	22.5	5.7			1	60°		" " "
76-2-1		16'-8" 16'-11"	129'-10" 129'-7"	66.5	96.5	30	2.7	—			1	52°		no definite failure plane of weakness before testing
76-2-2		17'-0" 17'-3"	129'-6" 129'-3"	68.0	96.6	15	5.6	3.0			1	43°		
76-2-4		17'-5" 17'-8"	129'-1" 128'-10"	62.7	99.3	45	4.9	3.5			1	45°		
76-3-1		20'-0 20'-4"	126'-6" 126'-2"	76.5	93.6	30	1.7	—			3	—		Sample badly disturbed with great deal of free water in tube
*76-4-2		21'-0" 21'-4"	125'-6" 125'-2"	74.7	97.8	17	7.87	3.7			1	52°	3.9	*Consolidated Quick Triaxial Consolidated to estimated
*76-4-4		21'-5" 21'-8"	125'-1" 124'-10"	74.2	98.3	27	8.42	3.2			1	56°	4.1	preconsolidation load: 7.07 psi then confining pressure increased
*76-4-5		21'-8" 21'-11"	124'-10" 124'-7"	79.8	96.5	37	7.59	4.2			1	58°	37	consolidation stopped & sample tested
76-5-1		22'-10 23'-2"	123'-8" 123'-4"	91.6	93.3	30	2.4	2.5			1	54°		
76-5-2		23'-2" 23'-5"	123'-4" 123'-1"	74.3	94.1	15	3.7	2.5			1	57°		Sheared before test
76-5-4		23'-6" 23'-10"	128'-0" 122'-8"	78.6	93.0	45	4.9	4.5			1	65°		

TYPES OF FAILURE: (1) SHEAR (2) BULGING WITH SHEAR (3) BULGING

P.C.F.: POUNDS PER CUBIC FOOT
P.S.I.: POUNDS PER SQUARE INCH

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TABLE II

SOIL TEST SUMMARY RECORD – STRENGTH

PROJECT: P 154

REMARKS: Co-Op Research St. Lawrence Seaway Authority - N.R.C.

LOCATION: Upper Lock Beauharnois

Hole #623 A Located 4' South of S.L.S.A. Hole #623

SAMPLE Nº	SOIL DESCRIPTION	DEPTH (FEET)	GEODETIC ELEV. (FEET)	NATURAL W/C (%)	NATURAL DENSITY (P.C.F.)	LATERAL PRESS (P.S.I.)	COMP. STRENGTH (P.S.I.)	FAILURE STRAIN (%)	Φ	C Kg/cm^{-2}	TYPE OF FAILURE (SEE BELOW)	∠ of Failure Plane	Pore Pressure p.s.i.	REMARKS:
*76-6-2		24'-6" 24'-9"	122'-0" 121'-9"	75.4	96.6	28.2	7.15	0.7			1	55°	3.3	* Consolidated Quick Triaxial Consolidated to estimated
*76-6-4	Test defective	25'-5" 25'-8"	121'-1" 121'-10"	78.1	104.6	28.0	4.0	5.0			1	56°	Negative	preconsolidation load = 8.2 psi. then confining pressure increased
*76-6-6		26'-0" 26'-3"	120'-6" 120'-3"	69.7	93.7	35.2	12.42	3.5			1	55°	8.1	consolidation stopped, & sample tested
76-7-1		26'-7" 26'-10"	119'-11" 119'-8"	77.8	93.0	30	4.9	5.0			2	–		
76-7-2		26'-10" 27'-1"	119'-8" 119'-5"	81.1	92.3	15	4.7	3.5			1	52°		
76-7-4		27'-3" 27'-6"	119'-3" 119'-0"	79.8	92.3	45	4.6	2.5			1	52°		
76-8-1	Red & Grey Banded Clay	28'-7" 28'-10"	117'11" 117'-8"	72.0	91.6	30	4.1	4.5			2	–		
76-8-2	" " "	28'-11" 29'-2"	117'-7" 117'-4"	61.5	105.0	15	6.8	2.0			1	52°		sample contained an angular sand layer ⅓" thick
76-8-4		29'-3" 29'-6"	117'-3" 117'-0"	74.3	95.8	45	5.5	2.5			1	52°		
76-9-2		30'-11" 31'-2"	115'-7" 115'-4"	56.2	106.3	20.1	16.0	3.5			1		6.2	Consol. Quick Triaxial Consolidated to estimated
76-9-4		31'-5" 31'-8"	115'-1" 114'-10"	53.1	107.5	30.2	11.87	7.8			1		5.5	preconsolidation load = 10.8 psi. then confining pressure increased,
76-9-5		31'-8" 31'-11"	114'-10" 114'-7"	51.5	108.7	40.8	13.0	4.0			1		3.0	consolidation stopped, & sample tested.

TYPES OF FAILURE: (1) SHEAR (2) BULGING WITH SHEAR (3) BULGING

P.C.F.: POUNDS PER CUBIC FOOT
P.S.I.: POUNDS PER SQUARE INCH

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TABLE III

SOIL TEST SUMMARY RECORD — STRENGTH

PROJECT: P 154

REMARKS: Co-op Research St. Lawrence Seaway Authority – N.R.C.

LOCATION: Upper Lock Beauharnois

Hole #623A Located 4' South of S.L.S.A. Hole #623

SAMPLE Nº	SOIL DESCRIPTION	DEPTH (FEET)	GEODETIC ELEV. (FEET)	NATURAL W/C (%)	NATURAL DENSITY (P.C.F.)	LATERAL PRESS (P.S.I.)	COMP. STRENGTH (kg/cm^2)	FAILURE STRAIN (%)	ϕ	C kg/cm^{-2}	TYPE OF FAILURE (SEE BELOW)	∠ of Failure Plane		REMARKS:
76-10-1		32'-8" 32'-11"	113'-10" 113'-7"	46.2	105.6	30	6.3	7.5			2	52°		
76-10-2		33'-0" 33'-3"	113'-8" 113'-5"	41.6	108.0	15	10.4	6.0			2	—		Bulges at top & bottom of sample in silty layers 3/4" thick
76-10-3		33'-3" 33'-6"	113'-5" 113'-2"	48.0	104.7	45	11.8	5.5			2	—		" " " " " "
76-10-4		33'-7" 33'-10"	113'-1" 112'-10"	60.6	100.9	0	7.6	3.4			1	45°		Unconfined Comp.
76-11-1		34'-8" 34'-11"	111'-10" 111'-7"	66.9	95.5	30	8.0	6.0			1	52°		Sample contained few small stones < 1/8" ϕ
76-11-2		35'-0" 35'-3"	111'-6" 111'-3"	64.0	97.4	15	7.9	5.0			1	52°		small silt pocket in sample
76-11-3		35'-3" 35'-6"	111'-3" 111'-0"	61.6	99.6	46	8.2	4.5			1	52°		contained sharpley angular igneous stone 1/2" largest dimension
76-12-2		39'-0" 39'-3"	107'-6" 107'-3"	40.8	109.1	30	10.6	9.5			2	49° & 58°		Sample contained sloping stratifications
76-12-4		39'-5" 39'-8"	107'-1" 106'-10"	56.6	109.2	45	9.7	5.0			2	56°		Barrelling shear in a 3/4" layer of silt
76-12-6		40'-0" 40'-3"	106'-6" 106'-3"	48.1	104.8	15	10.0	3.5			2	58°		alternating 3/4" clay & silt layers possibly varving
76-13-2		41'-0" 41'-3"	105'-6" 105'-3"	37.8	119.3	30	10.9	9.5			2	—		" " " "
76-13-4		41'-5" 41'-8"	105'-2" 104'-11"	28.1	123.3	45	10.5	10.0+			2			" " " "

TYPES OF FAILURE: (1) SHEAR (2) BULGING WITH SHEAR (3) BULGING

P.C.F.: POUNDS PER CUBIC FOOT
P.S.I.: POUNDS PER SQUARE INCH

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TABLE IV

Table V Summary of Laboratory Vane Shear Strengths and Soil Sensitivities

Sample No.	Depth (ft.)	Laboratory Vane Shearing Strength (psf)			Soil Sensitivity	
		Undisturbed (1)	Partially Remoulded (2)	Completely Remoulded (3)	Partially Remoulded (1)/(2)	Completely Remoulded (1)/(3)
76- 1-4	9.0	3140	752	2450	4.2	1.3
76- 2-5	18.0	510	87	54	5.9	9.4
76- 4-6	22.2	725	104	60	7.0	12.1
76- 5-5	24.0	725	114	54	6.4	13.4
76- 6-5	25.8	634	135	28	4.7	22.7
76- 7-6	28.0	914	147	59	6.2	15.5
76- 8-5	29.8	557	137	77	4.1	7.2
76- 9-1	30.8	849	218	115	3.9	7.4
76-10-5	34.0	1097	148	104	7.4	10.5
76-11-5	35.8	646	174	59	3.7	10.9
76-12-5	39.9	967	202	98	4.8	9.9
76-13-5	41.9	549	98	104	5.6	5.3

SOIL TEST SUMMARY RECORD - CONSOLIDATION

PROJECT: D154

REMARKS: Co-op Research National Research – St. Lawrence Seaway Auth.

LOCATION: Upper Lock, Beauharnois

Hole #623 A Located 4' South of S.L.S.A. Hole #623

SAMPLE Nº	SOIL DESCRIPTION	DEPTH (FEET)	GEODETIC ELEV. (FEET)	NATURAL W/C (%)	SPECIMEN SIZE HEIGHT (INS.)	SPECIMEN SIZE AREA CM²	PRECONSOLIDATION (kg/cm²) MIN.	PRECONSOLIDATION (kg/cm²) PROB.	PRECONSOLIDATION (kg/cm²) MAX.	INITIAL VOID RATIO	C_c			REMARKS
	Ground Surface Elev.				146.5'		Geodetic							
76-1-2	Mottled grey brown silty clay	8'-4"	138'-2"	35.5	0.750	31.17 cm²	1.53	1.79	1.94	1.061	0.198			weathered
76-2-3	Grey Sensitive Clay (mottled)	17'-4"	129'-2"	70.9	0.623	17.98	0.90	0.96	0.99	2.008	2.00			not weathered ↓
76-4-3	" "	21'-4"	125'-2"	68.6	0.750	31.17	0.92	0.97	0.99	1.941	2.58			
76-5-3	" "	23'-6"	123'-0"	74.2	0.750	31.17	1.08	1.14	1.16	2.136	2.36			
76-6-3	" "	25'-4"	121'-2"	74.3	1.001	31.10	0.98	1.03	1.04	2.119	2.80			
76-7-3	" "	27'-2"	119'-4"	81.8	1.001	31.10	0.96	1.01	1.03	2.336	1.81			
76-8-3	Grey Clay with Red Bands up to 2½" thick	29'-2"	117'-4"	77.7	0.756	31.18	0.68	0.80	0.85	2.178	1.14			
76-11-4	Stratified Silt & Clay	35'-7"	110'-11"	61.7	1.006	31.10	1.06	1.22	1.25	1.756	1.06			
76-12-3	Stratified Silt & Clay	39'-4"	107'-2"	57.2	0.623	17.98	1.16	1.46	1.51	1.640	0.85			
76-13-3	Grey Silt	41'-4"	105'-2"	22.5	0.756	31.18	No definite break in press-void ratio curve			0.643	0.03			

TABLE VI

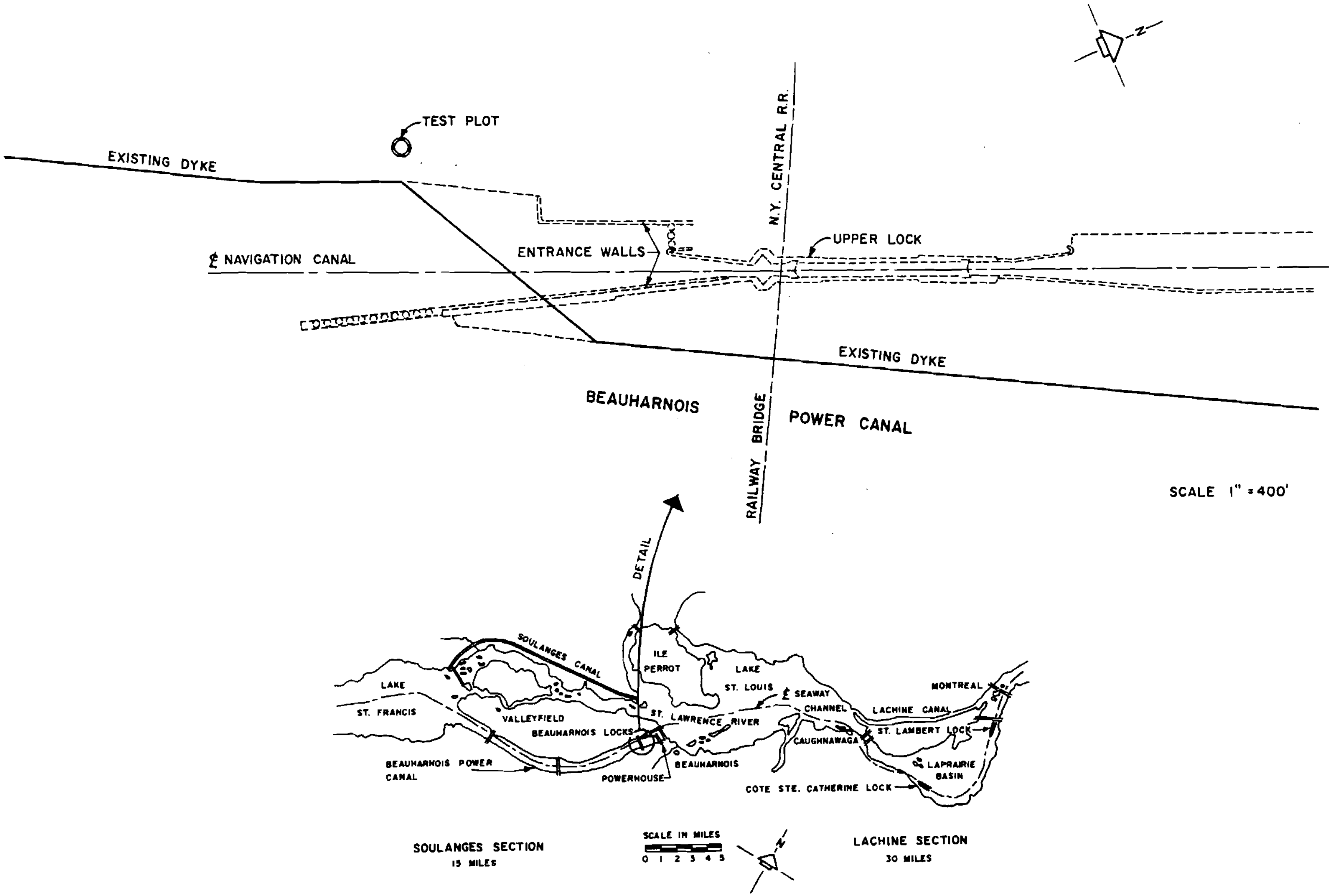

FIGURE 1
LOCATION OF TEST PLOT IN UPPER LOCK AREA, BEAUHARNOIS

BR 2321-1

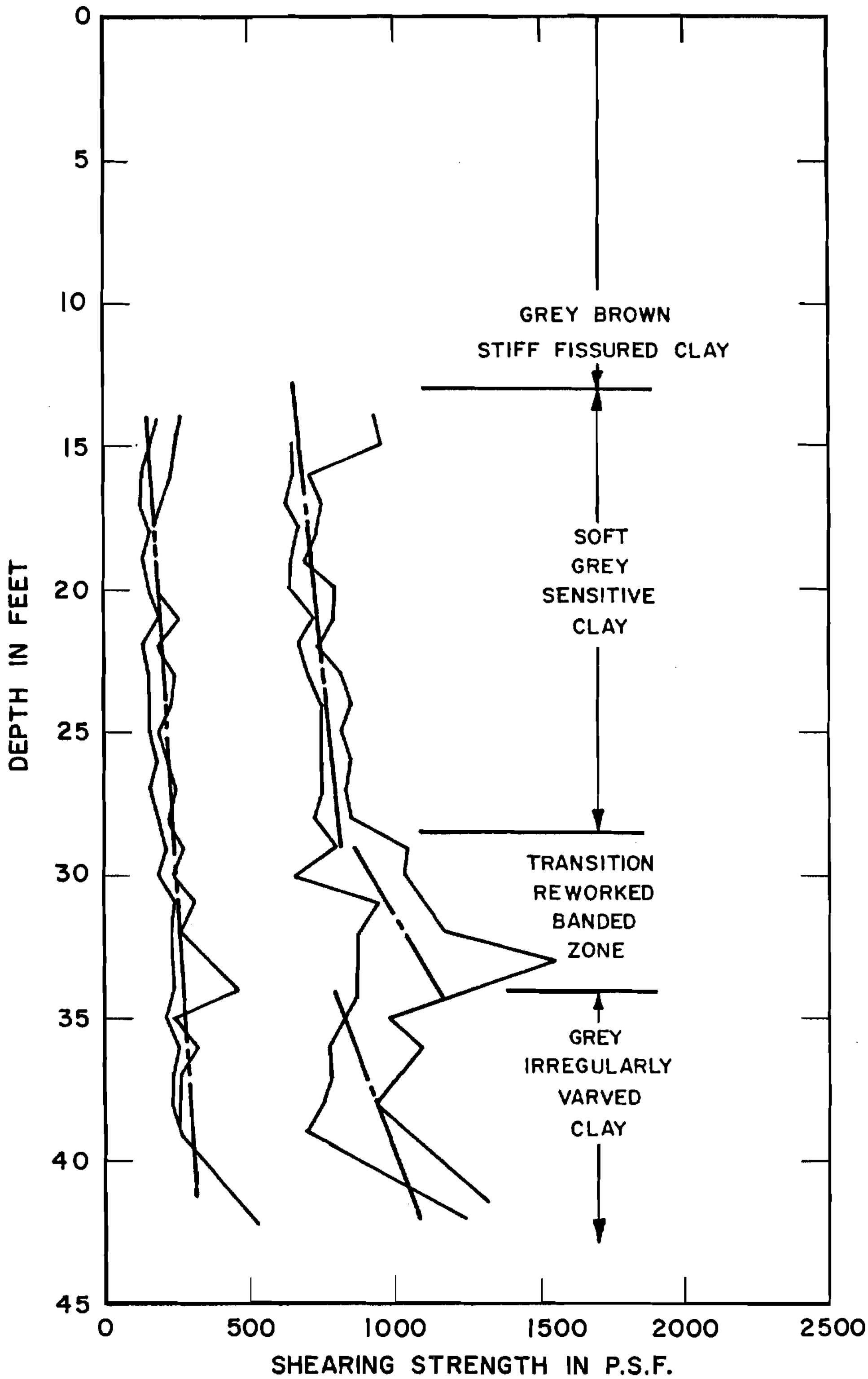

FIGURE 2

AVERAGE, MAXIMUM AND MINIMUM SHEARING STRENGTHS FOR 4 BORINGS WITH THE SEAWAY VANE EQUIPMENT

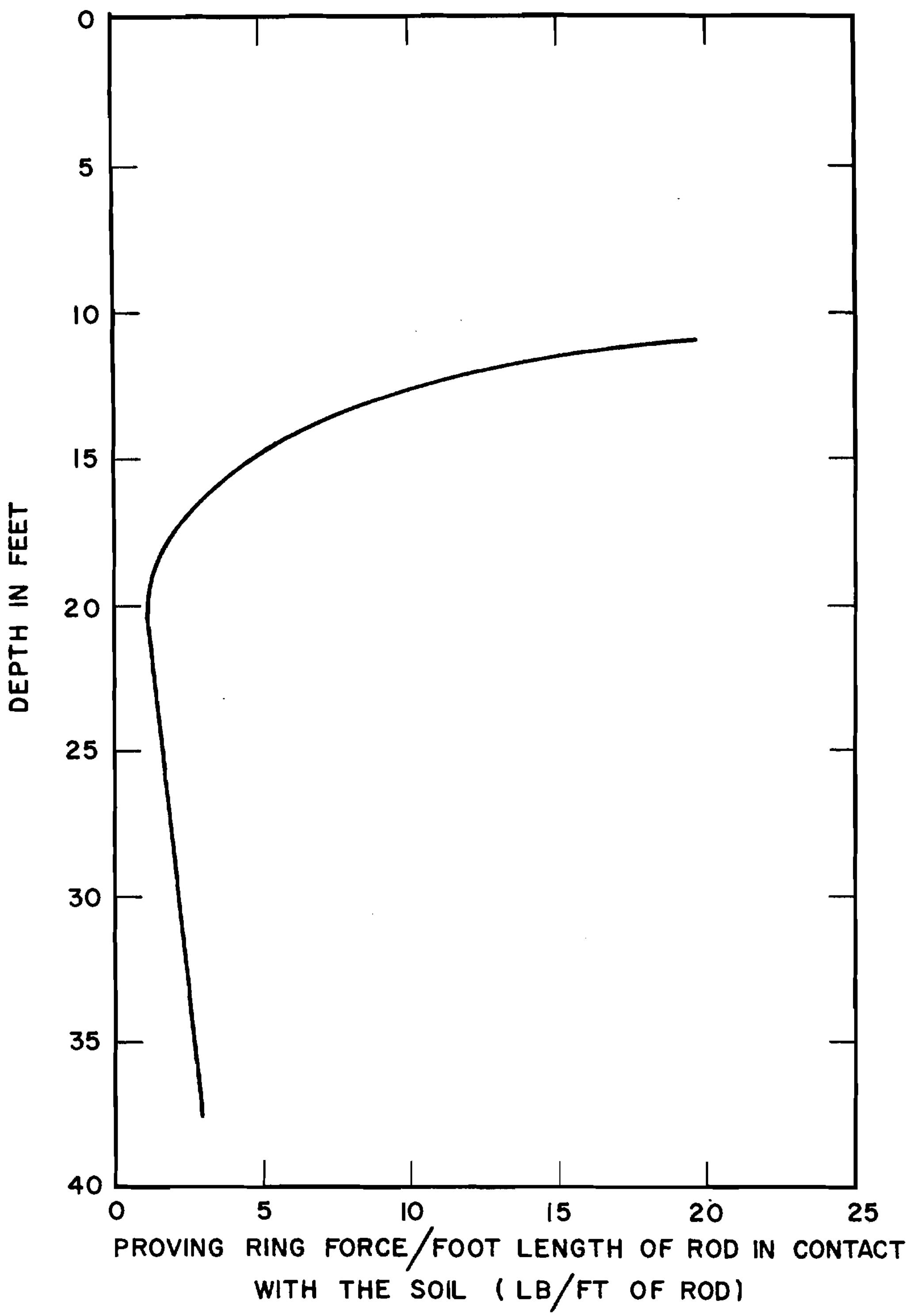

FIGURE 3

RESULTS OF "ROD FRICTION" TESTS

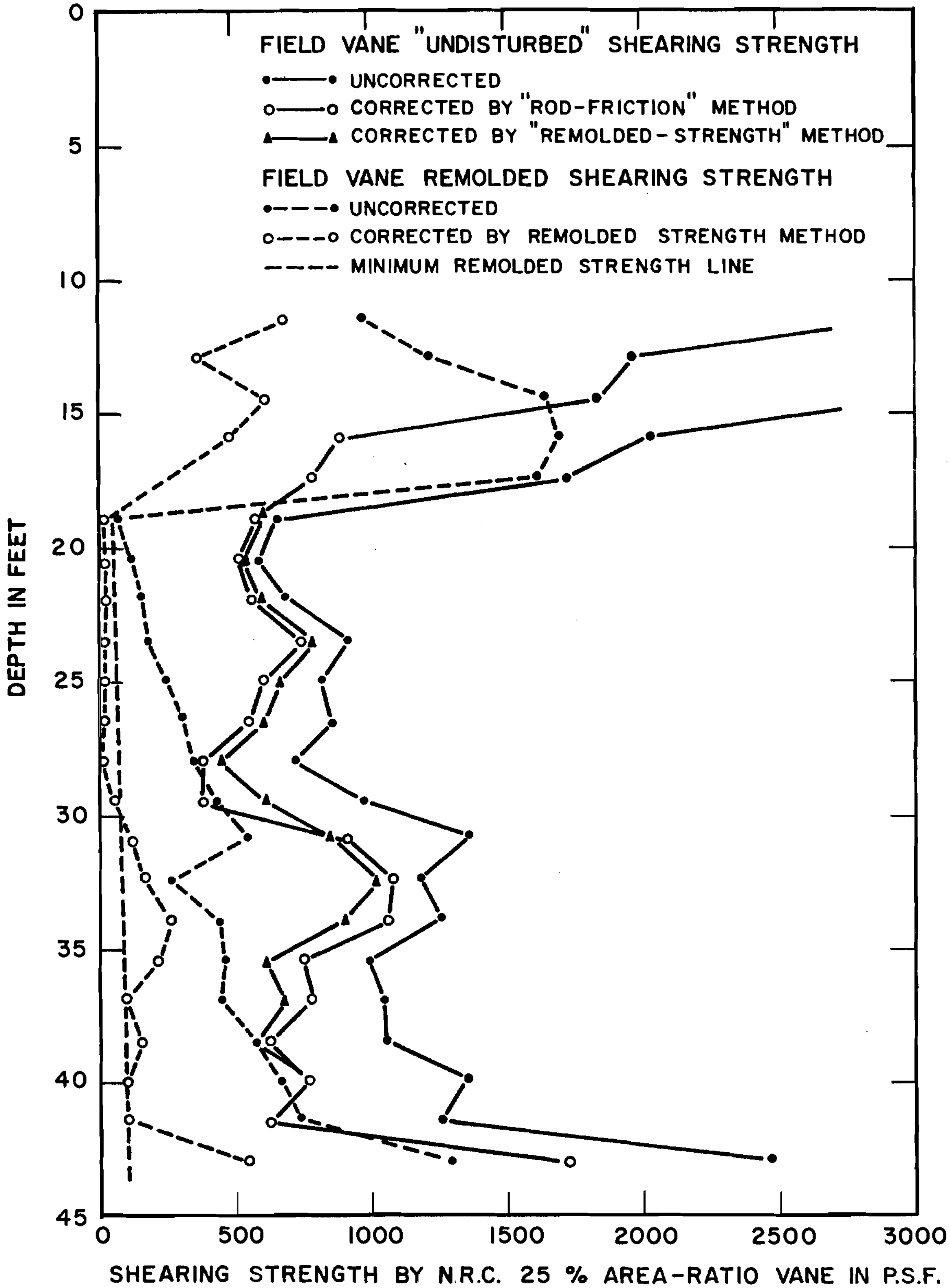

FIGURE 4

SHEARING STRENGTHS BY N.R.C. 25 % AREA-RATIO VANE

BR 2321-4

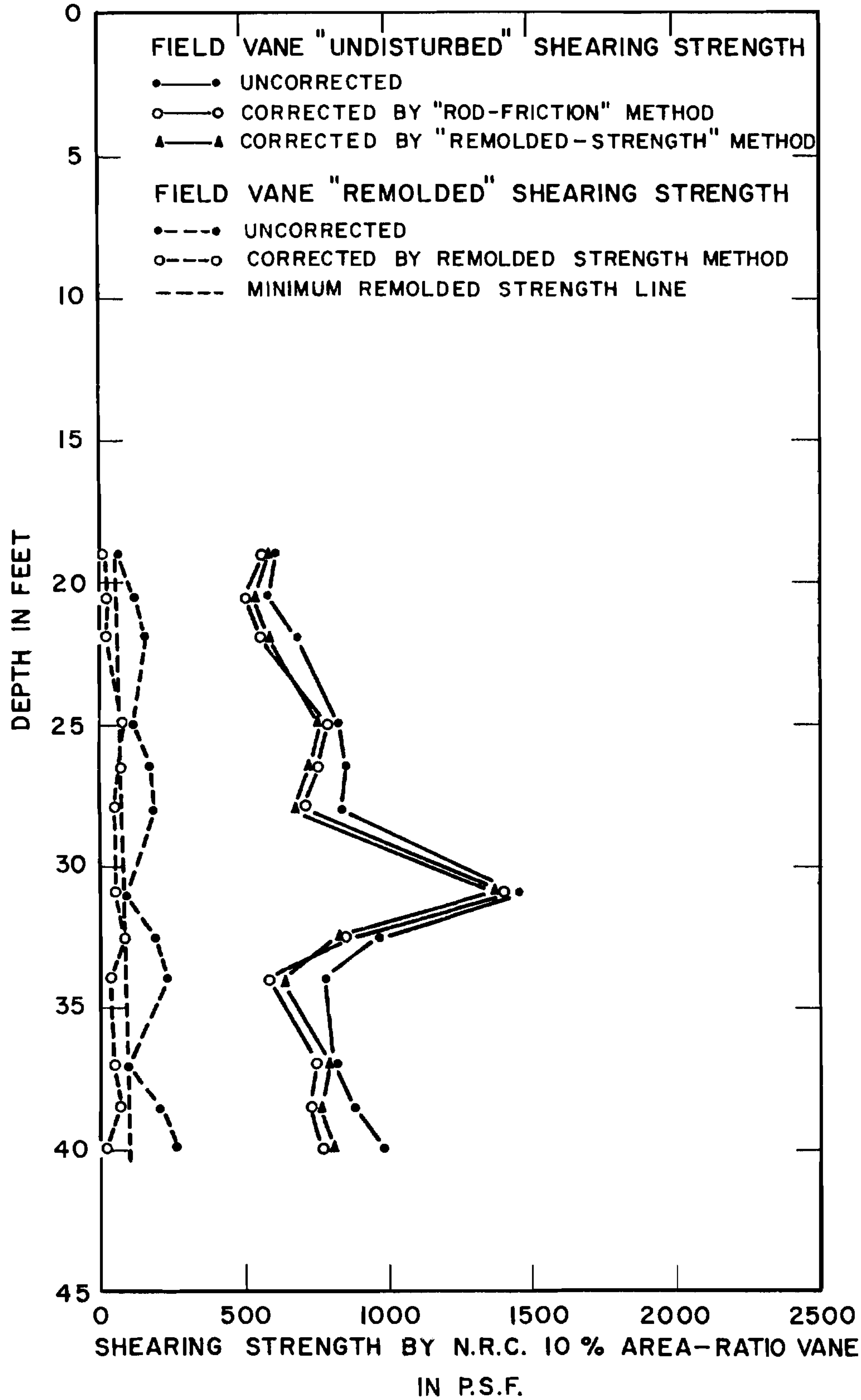

FIGURE 5
SHEARING STRENGTHS BY N.R.C. 10 % AREA-RATIO VANE

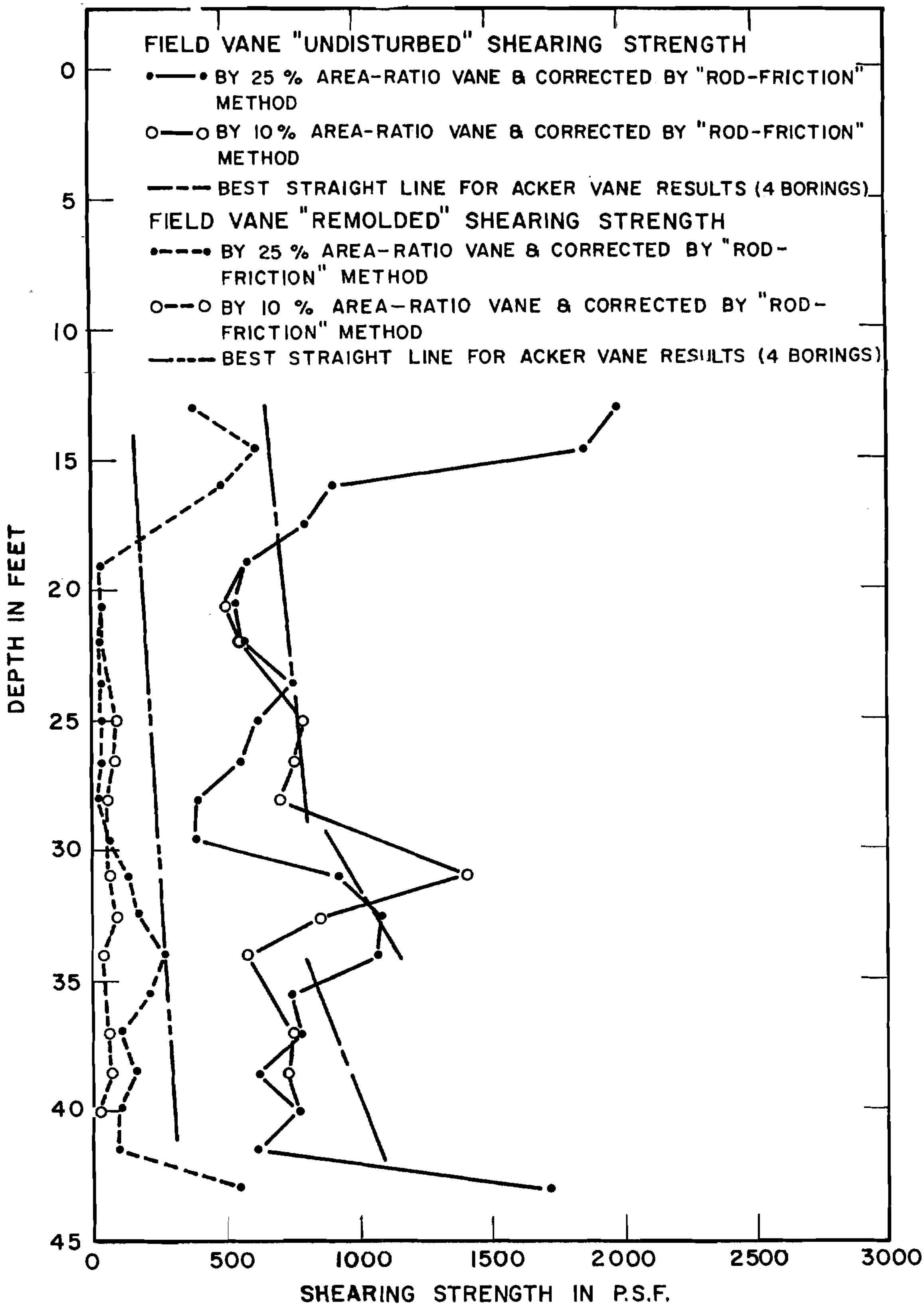

FIGURE 6

COMPARSION OF SHEARING STRENGTH BY ALL VANES

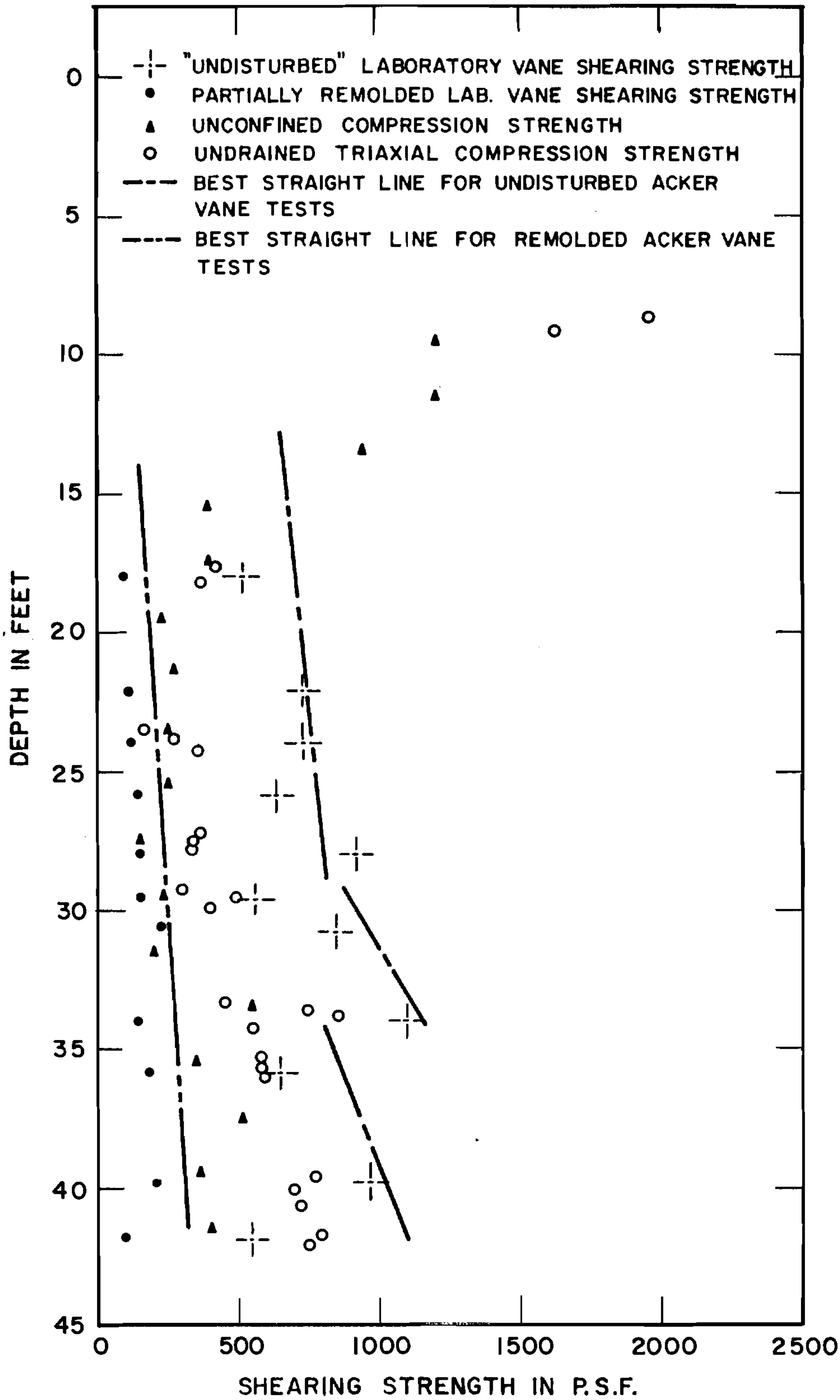

FIGURE 7
LABORATORY UNDRAINED SHEARING STRENGTHS

BR 2321-7

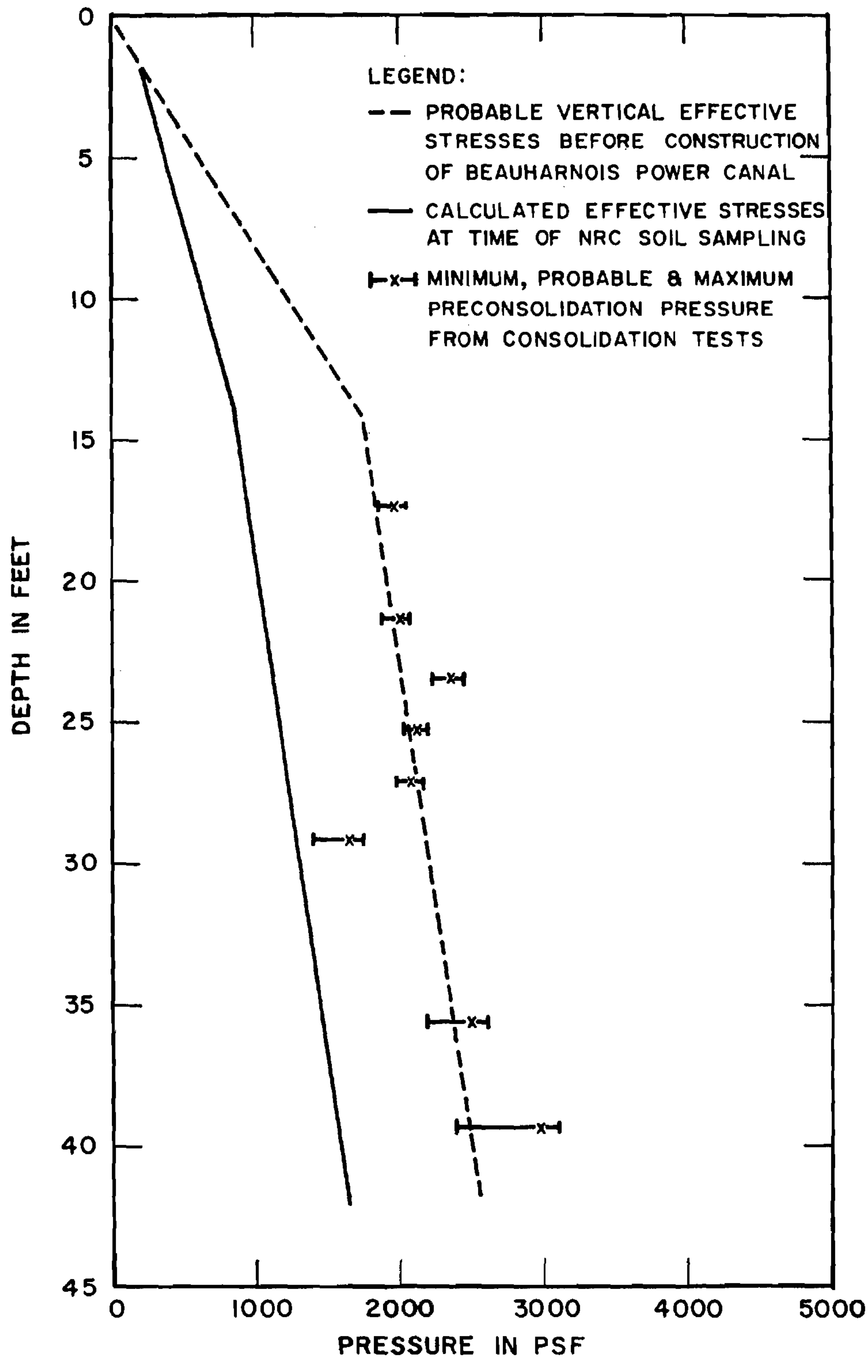

FIGURE 8
CALCULATED EFFECTIVE STRESSES AND PRECONSOLIDATION PRESSURE VS DEPTH

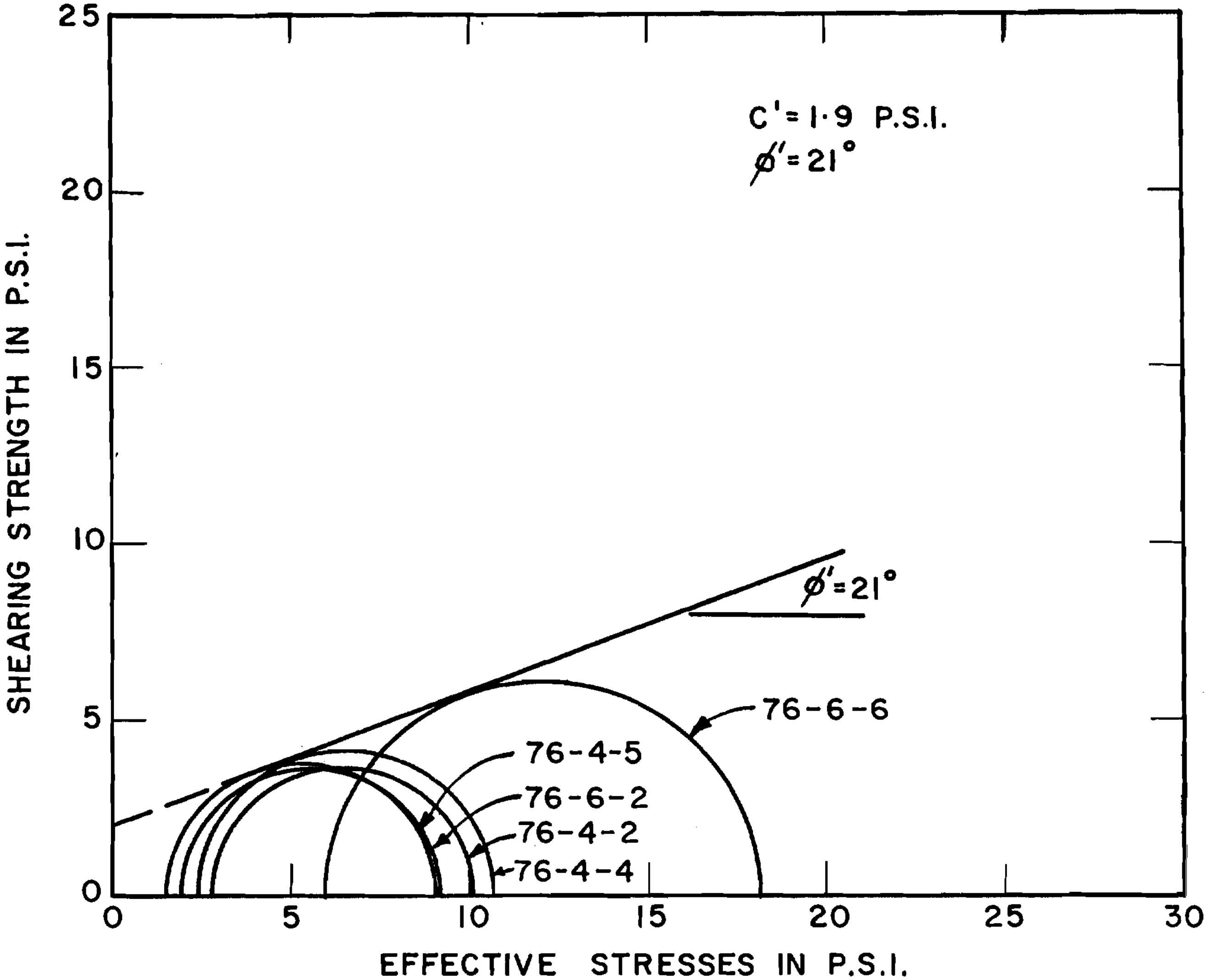

FIGURE 9
MOHR'S CIRCLES AND ENVELOPE IN TERMS OF EFFECTIVE STRESSES FOR CONSOLIDATED–UNDRAINED TESTS ON THE SOFT GREY SENITIVE CLAY BETWEEN 13' AND 28 1/2' DEPTHS

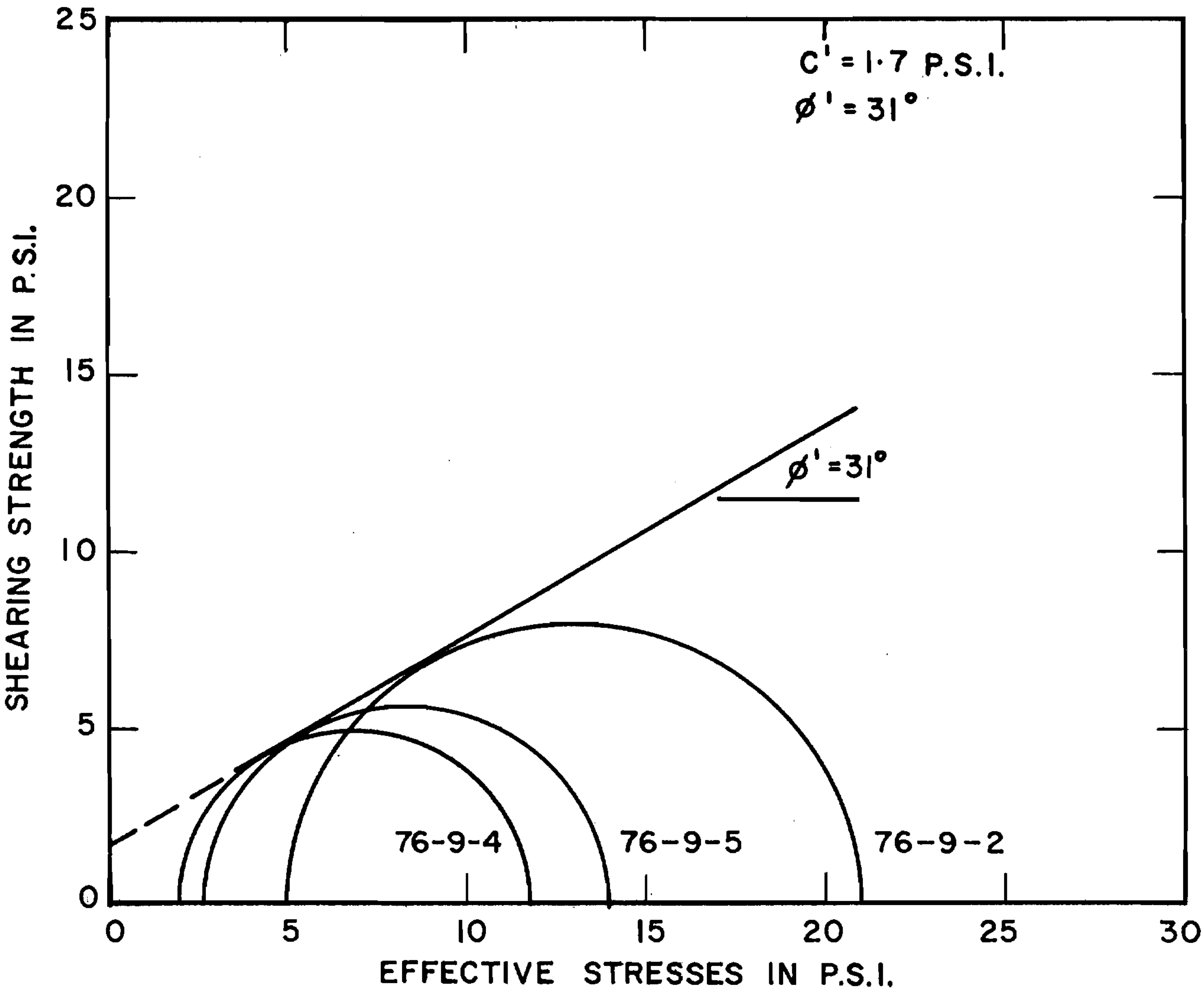

FIGURE 10

MOHR'S CIRCLES AND ENVELOPE IN TERMS OF EFFECTIVE STRESSES FOR CONSOLIDATED-UNDRAINED TESTS ON THE TRANSITION REWORKED BANDED CLAY 28 ½' TO 34' DEPTH

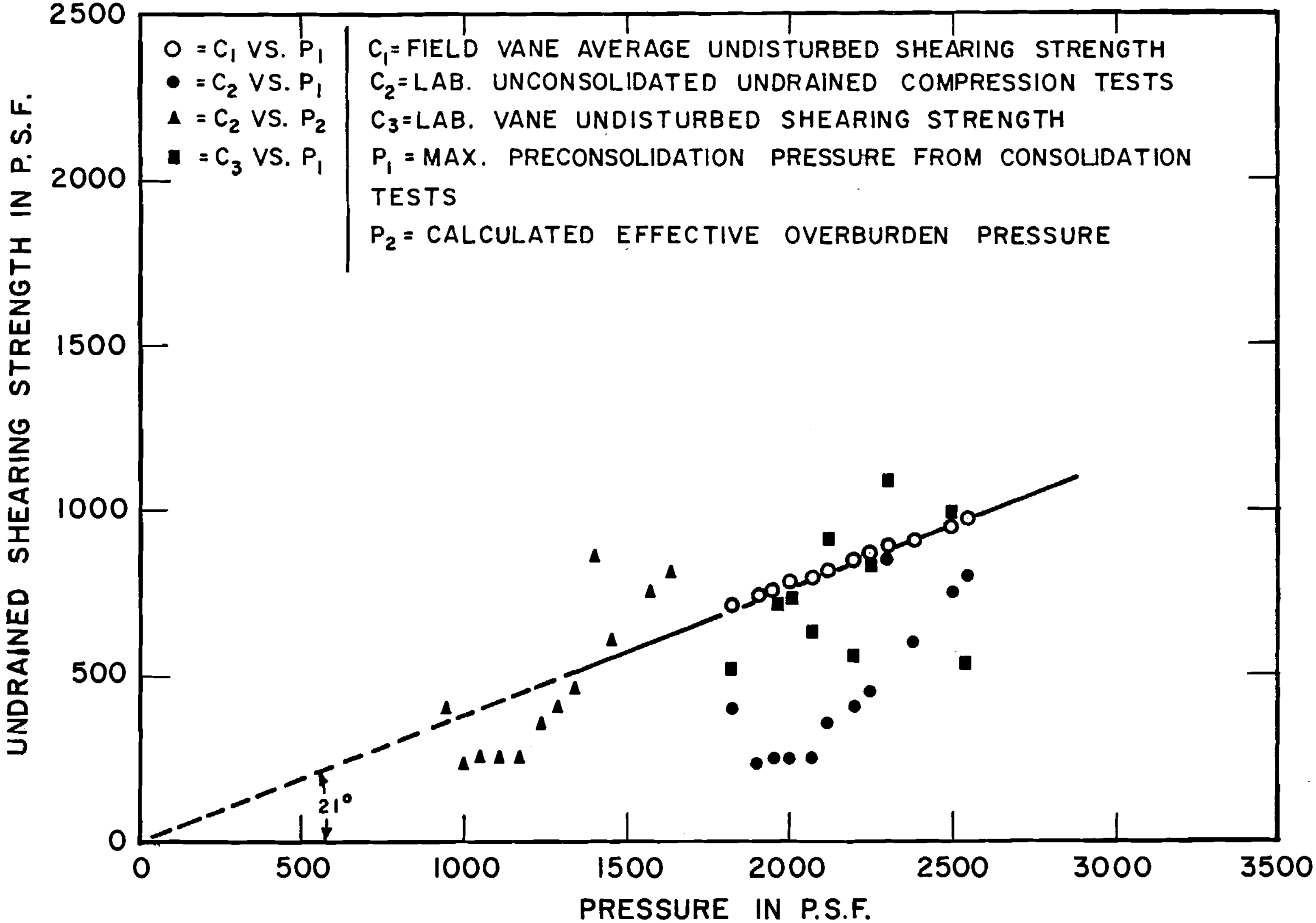

FIGURE 11

C/P RELATIONSHIP FOR ALL TESTS

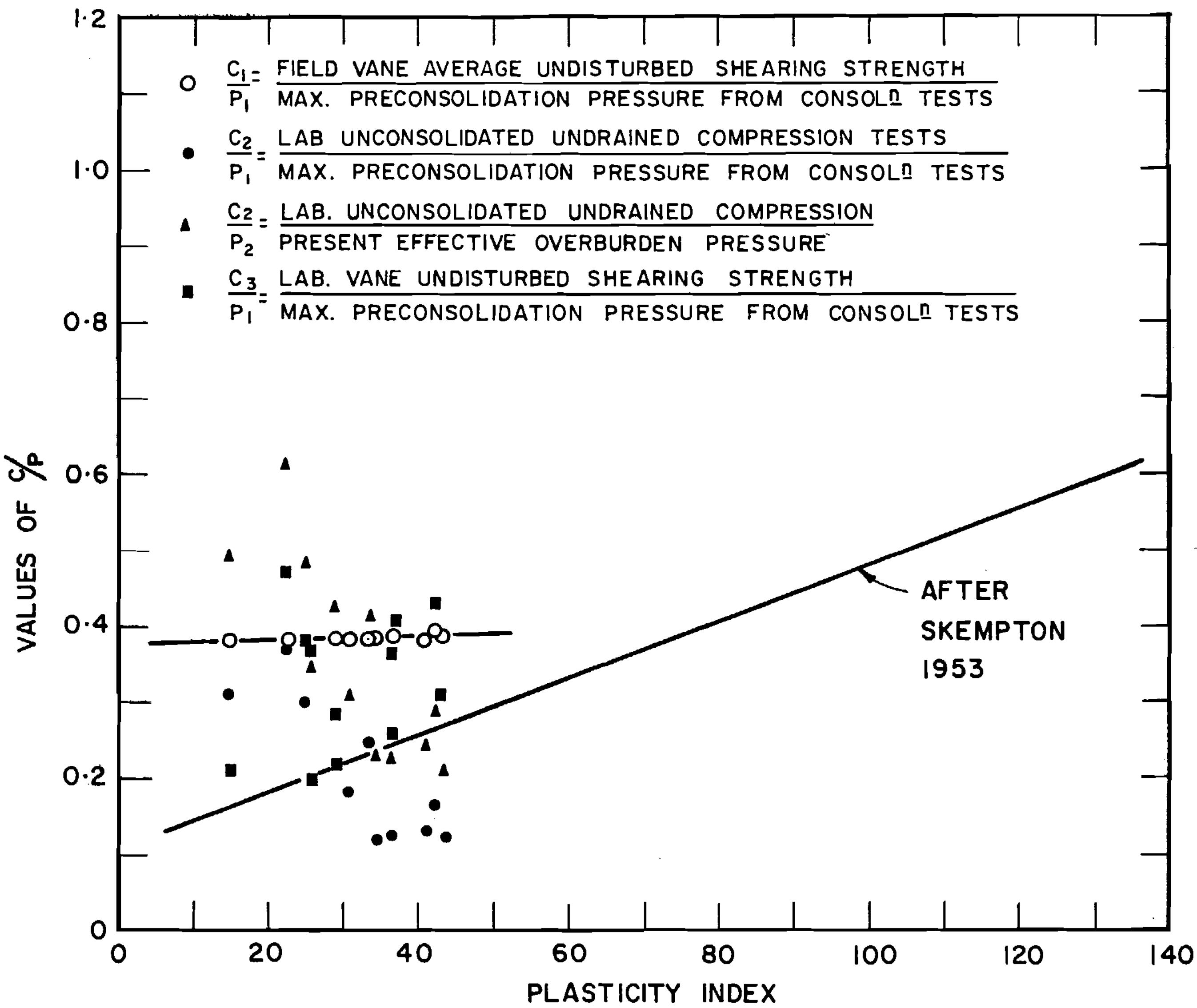

FIGURE 12

C/P VS. PLASTICITY INDEX FOR ALL TESTS