DETERMINATION OF IN SITU STRESS IN SOIL BY HYDRAULIC FRACTURING

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ABSTRACT

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Submitted to the Department of Civil Engineering on February 28, 1974 in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering.

The purpose of this thesis is to measure the in situ horizontal stress of a soft clay using the hydraulic fracturing method. An instrument for this purpose was designed and constructed. The horizontal stresses measured by the hydraulic fracturing technique were compared with the horizontal stresses determined from other methods to evaluate the feasibility of the hydraulic fracturing method.

The hydraulic fracturing tests were performed on the Boston Blue Clay at 14 different locations using existing piezometers. The results from hydraulic fracturing tests generally agree with the results obtained from K_o-oedometer laboratory tests and empirical correlations. Further improvement of the device used is required for an accurate measurement of water outflow at low flow rates and for soils which are more permeable than clays.

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LIST OF SYMBOLS

H	= head
K	= coefficient of lateral stress
K_o	= coefficient of lateral stress at rest
k	= coefficient of permeability
KSC	= kilograms per square centimeter
OCR	= overconsolidated ratio
P.I.	= plasticity index
s_u	= undrained shear strength
$s_u/\bar{\sigma}_{vc}$	= normalized undrained shear strength
t	= time
Δu	= excess pore water pressure
u_c	= "close up" pressure
u_o	= pore water pressure of the ground
$\bar{\sigma}_3$	= effective minor principal stress
$\bar{\sigma}_h$	= effective horizontal stress
$\bar{\sigma}_v$	= effective vertical stress
$\bar{\sigma}_{vc}$	= effective consolidation stress
$\bar{\sigma}_{vm}$	= maximum effective stress
$\bar{\phi}$	= friction angle

CHAPTER I

INTRODUCTION

One of the most important jobs of the Civil Engineer is to make predictions. A poor prediction can result in loss of life or property. In order to make a good prediction an engineer needs reliable parameters for the analytical models used to represent the actual field conditions. Even the most suitable and refined analytical technique is useless without reasonably accurate parameters.

In situ testing for determining reliable engineering properties of soils has received considerable attention in recent geotechnical research. Due to unavoidable sample disturbance that occurs when a sample is removed from the ground, careful tests in the laboratory are unlikely to give reliable values for either the stress-strain parameters or for the in situ state of stress.

In 1972, Bjerrum and Anderson of the Norwegian Geotechnical Institute suggested a simple method for the in situ measurement of the lateral earth pressure in normally consolidated and lightly overconsolidated clays. The method is based on the principle of hydraulic fracturing, a technique previously used to create cracks in soil and rock masses. This thesis describes the use of hydraulic fracturing to measure in situ lateral stresses in Boston Blue Clay.

1.1 OBJECTIVE

This thesis has two primary objectives:

1) Measurement of the in situ lateral stresses in a soft clay, known as Boston Blue Clay, using the hydraulic fracturing method,

2) Evaluation of the method by comparing the results from hydraulic fracturing with the results from several other methods of estimating lateral stresses from laboratory tests.

The coefficient of lateral stress at rest ($K_o = \bar{\sigma}_{ho}/\bar{\sigma}_{vo}$) is a basic parameter needed for many approaches to problems in soil mechanics. Some of the prediction techniques for which K_o is needed are described below:

1) The stress path method, in which one subjects samples in the laboratory to the stress changes that will occur in the field during construction and measures the resulting deformations, requires a knowledge of the initial in situ stresses (Lambe, 1967).

2) The theory of elasticity is often used in soil mechanics to estimate stability and deformations, and it needs modulus and Poisson's ratio which in some situations can be related to K_o.

3) Undrained shear strength of a soil is related to the initial consolidation stresses with $s_u/\bar{\sigma}_{vc}$ varying as K_o varies.

4) Retaining walls and supported cuts are sometimes designed for K_o lateral stresses.

Numerous efforts have been directed toward measuring lateral stress directly or indirectly, and this study is one approach toward obtaining better results.

The hydraulic fracturing technique for measuring in situ lateral stress of soils has been introduced to Geotechnical Engineering recently and has potential of wide applications if its reliability can be established. This technique is a very economical method, particularly, if existing piezometers are available for testing. Since pore pressure is one of the most important parameters required for predicting soil behavior, in most cases piezometers are readily available. Also, the principles involved in this technique are very simple, and one can easily design and construct a device which is suitable for a particular project.

Thus far the technique has received limited use (Bjerrum and Anderson, 1972) although several researchers are trying it on various projects. Little is known about the reliability of the method or the influence of the type and shape of the piezometer on the results. This thesis deals with measuring in situ minor principal stress in a deposit of Boston Blue Clay, the properties of which are fairly well known. Results were obtained using two types of piezometers in locations with a variable stress history. These results were compared with values obtained from other methods of estimating lateral stress.

1.2 PREVIOUS ATTEMPTS TO MEASURE HORIZONTAL STRESS

1.2.1 IN SITU MEASUREMENT

Several techniques have been developed in the past to measure horizontal stress in situ.

Menard (1957) devised a pressure-meter which requires preboring a hole before placing an expandable probe in a soil deposit. The pressure required to expand the probe to the diameter of the prebored hole is taken as a measure of the in situ lateral stress. The soil around the probe is disturbed before the probe is ever inserted by the hole making technique and the stress relief occurs in the adjacent soil. This procedure has had some success in stiffer soils and weak rocks.

Wroth and Hughes (1973) and Baguelin (1972) independently developed devices which do not require preboring to try to minimize disturbance of the ground around the probe. These devices consist of a thick walled cylindrical probe which is slowly pushed into the ground while the soil is excavated by a cutter rotating inside a cutting head attached to the tip of the cylinder. Water circulates through the core of the probe and washes the soil cuttings to the ground surface. Disturbance from drilling and stress release are greatly reduced. These devices can directly measure not only horizontal stress but also the stress-strain properties of the soil. Both the Wroth-Hughes and Baguelin devices have a rubber membrane around the probe which is expanded by fluid pressure. The Wroth-Hughes device uses gas to inflate the membrane while the Baguelin device uses water. The volume change within the membrane is measured to compute the horizontal strain to obtain the stress-strain relation. Both devices have pore-pressure measuring equipment attached to the probes. Although these devices have a promising future they are still in the developing stage. Preliminary results indicate reasonably good measurements of lateral stress can be obtained if stress measurements are allowed to come to equilibrium (which may take several days).

Kenney (1967) devised an apparatus for the field measurement of K_0 which consists of a steel-pile section with earth-pressure and pore-pressure cells attached to it. This approach is fairly complex and expensive, having thus far been used only on a limited research basis.

Attempts to measure in situ earth pressures using total pressure cells have been made for many years and various types of pressure cells are described by Hanna, 1973. Generally, total pressure cells have been used to measure changes in stress that result from construction at distances immediately adjacent to the construction, such as the pressure beneath a slab or the pressure on a retaining wall. Cunningham (1968) describes an attempt to measure in situ horizontal stress several feet below a building foundation by pushing an earth pressure cell into the soil and allowing it to come to equilibrium. Large disturbance of soil may occur, in this case, when the fairly thick cell is forced into position.

1.2.2 INDIRECT MEASUREMENT

The expression suggested by Jaky (1944) has proved to give a reliable representation of K_0 values measured in laboratory tests. He gives K_0 values for normally consolidated soils as a function of the friction angle ($\bar{\phi}$):

$$K_0 = 1 - \text{Sin}\ \bar{\phi}.$$

This expression has since been evaluated using laboratory tests by many people (Bishop, 1958, Simons, 1958, Brooker and Ireland, 1965, and

Henkel and Wade, 1966) and was shown to be basically valid, although they have suggested a slight modification of the equation for different types of soil. For example, Brooker and Ireland recommended $K_0 = 0.95 - \sin\bar{\phi}$ for normally consolidated clays and $K_0 = 1 - \sin\bar{\phi}$ for cohesionless soil. Lambe and Whitman (1969) show a plot of the laboratory test results run by Hendron on sand which agrees with Jaky's expression.

Brooker and Ireland (1965) tested five different types of clays with varying plasticity index (P.I.) at different overconsolidation ratios, and gave a correlation among K_0 value, OCR and P.I. This correlation is shown in Figure III-24.

Alpan (1967) suggested an expression which gives K_0 values as function of P.I. for normally consolidated clays as:

$$K_0 = 0.19 + 0.233 \text{ Log}_{10}(\text{P.I. in } \%).$$

The above expression was based on Kenney's (1959) empirical correlation between $\sin\bar{\phi}$ and Log P.I. assuming $K_0 = 1 - \sin\bar{\phi}$.

For the laboratory determination of the coefficient of lateral stress for one-dimensional strain (K_0) several devices and techniques have been used. Oedometers that permit measurement of lateral stress or lateral strain and the triaxial cell are the most widely used to determine the laboratory K_0 values. Several techniques can be used in testing with these two devices, which include:

A) K_0-Oedometer

1) A fairly rigid confining ring restrains lateral straining of

a sample as the axial load is applied, and a strain gage instrumented to the ring measures a small lateral strain in order to determine the lateral force.

2) A fairly rigid confining ring equipped with a strain gage and a device which can apply external pressure laterally senses lateral straining of a sample as the axial load is applied, and the K_0 can be determined by measuring a lateral pressure which maintains zero lateral strain.

3) A teflon walled ring equipped with a cell filled with relatively incompressive oil prevents a sample from lateral straining as the sample is loaded axially, and a low displacement pressure transducer connected to the cell measures the lateral stress (Wissa, 1973). This apparatus as shown in Figure III-22 was used to determine the laboratory K_0 of undisturbed Boston Blue Clay for comparison.

B) Triaxial Consolidation

1) A lateral strain indicator placed in position at the mid-height of a sample detects lateral straining as the axial load is applied, and the K_0 value is determined by measuring the cell pressure which maintains zero lateral strain (Bishop and Henkel, 1969).

2) A cell pressure which maintains equal area of a sample, as load is applied axially, can be measured to estimate K_0 value.

> This technique consists of measuring axial straining and volume change of the sample, and the cell pressure is adjusted to maintain the volume change of the sample, the same as the volume change due to axial straining of the sample (Bishop, 1950 and Bishop and Eldin, 1953).

These laboratory tests give consistent and reproducible results generally confirming empirical correlations, however, no one knows yet how these results compare with field values.

1.3 HISTORY OF HYDRAULIC FRACTURING

In 1948, the oil industry developed a process called "Hydrafrac" in order to increase the productivity of an oil well (Clark, 1948). In the process an oil producing formation is fractured with a high hydraulic pressure which increases the mass permeability of the formation. The hydraulic fluid used for fracture often carries in suspension a granular material, such as sand, to keep the fracture from closing off after release of pressure. This technique was progressively refined, and by the end of 1957 significant progress has been made in formulating techniques for fracturing rocks. Hubbert and Willis (1957) considered the stress redistribution resulting from drilling a borehole and increasing the internal fluid pressure. They were able to show that a crack or fracture should develop at pressures below the overburden stress. They related the cracking pressure to the minor principal stress and suggested the cracking plane should be in the plane on which the minor principal stress was acting. They presented results of a few simple tests with gelatin to confirm their conclusions.

In rock mechanics, Scheidegger (1962) has the credit for the first introduction of hydraulic fracturing method for measuring horizontal stress. He made the suggestion based on the principles established by Clark (1948) and Hubbert and Willis (1957). The idea has received considerable attention as a simple method of determining the "approximate" minor principal stress in a rock mass (Haimson and Fairhurst, 1968).

In 1968, geotechnical engineers first recognized the significance of hydraulic fracturing of soils when a rock fill dam at Hyttejuvet in Norway developed severe leakage during the first filling of the reservoir. A subsequent study of this incident concluded that the increased flow probably resulted from fracturing caused by high water pressures. In situ permeability tests carried out on borings made in the core supported this conclusion, as they showed a small outflow of water under a gradually increasing head as the casing was filled with water, but at a given level the water disappeared suddenly, indicating that a fracture had developed (Bjerrum, et. al., 1971).

The possibility of hydraulically fracturing a soil to determine the minor principal stress has since been explored. Bjerrum and Anderson (1972) undertook a research program on the technique starting from 1968. Test results from six different sites indicated that K_0 values obtained were approximately constant with depth for normally consolidated clays. Vaughan (1970 and 1972) conducted a series of hydrauric fracturing tests in the core of a dam on Casagrande type piezometers to investigate the hypothesis that hydraulic fracturing of soil due to increased pore pressure could lead to increasing seepage as the reservoir was filled. Subsequently he concluded that the determination of minor principal stress using hydraulic fracturing is feasible.

1.4 PRINCIPLES OF HYDRAULIC FRACTURING METHOD

When a water pressure is created in a borehole or cavity higher than a certain critical value during an in situ permeability test, the rate of water outflow will increase abruptly because cracking occurs in the soil around the piezometer tip. As the pore water pressure is gradually increased, the soil particles around a piezometer tip will experience an outward force in the radial direction and the effective circumferential stress in the soil will be reduced. If the effective tensile strength of the soil is negligible and K_0 is less than one, fracturing will take place when the effective circumferential compressive stress of soil around the piezometer reduces to zero and becomes tensile stress. In this case a vertical fracture will appear. If K_0 is greater than one a horizontal crack will start before a radial cracking is initiated. The "fracturing pressure", which is the excess pore water pressure required to produce a fracture, is related to the in situ state of stress. The results from laboratory tests verify the above hypothesis (Bjerrum, et. al., 1972).

The magnitude of the fracturing pressure depends on the effective tensile strength of soil, and the in situ stresses of the ground next to the piezometer which may have been altered during installation of piezometer.

Once the crack is initiated, the additional increase of pore pressure will extend the rupture along the path of least resistance, i.e., perpendicular to the minor principal stress. At this point the pressure at the edge of the piezometer is the sum of total minor principal stress and head loss due to flow of water through the crack. When the pressure is allowed to decrease below the minor principal stress, the crack will close.

There are two possibilities of determining the horizontal stress by the hydraulic fracturing test.

1) Measure the "fracturing" pressure.

2) Measure the "close up" pressure.

Previous researchers discovered that the "fracturing" pressure was always higher than the "close up" pressure (Vaughan, 1972 and Bjerrum and Anderson, 1972).

In order to avoid the uncertainties involved in the "fracturing" pressure, Bjerrum and Anderson (1972) suggested that measuring the "close up" pressure to determine the horizontal stress is more reliable.

CHAPTER II

HYDRAULIC FRACTURING TEST

2.1 DEVICE

Two types of test equipment, a constant head and a variable head field permeability test device, can be used for the hydraulic fracturing test. Vaughan (1972) used a constant head test device which used gas pressure to obtain high fracturing pressures. Bjerrum and Anderson (1972) used a variable head test device which has a mercury manometer and a water pump to apply high pressure to the piezometer.

Since one can obtain the "close up" pressure conveniently with a variable head permeability test, the Bjerrum and Anderson (1972) approach was chosen for this study. Modifications to their recommended procedure were necessary because most of the piezometers to be tested were much deeper and had larger collection zones than ones they had used. A device capable of producing pressures up to 50 TSM and a flow capacity of a few liters was necessary to successfully run tests on Casagrande type piezometers.

A device, as shown in Figure II-1, was designed and constructed for the hydraulic fracturing test which allows one to accomodate a wide range of fracturing pressure and flow rate. The device was intended to function as a combination of a variable head and a constant head permeability test device. The main features of the device are:

a) A mercury manometer for a falling head permeability test,

b) An air pressure supply system to obtain high pressure and to run a constant head permeability test,

c) A water reservoir to supply water into the mercury manometer,

d) A safety pot to collect mercury in case mercury is blown out of the manometer.

Appendix A describes the materials used, dimensions and the recommended test procedures in detail.

2.2 DETERMINATION OF THE HORIZONTAL STRESS

The technique adopted for this study is to measure the "close up" (u_c) pressure to determine horizontal stress by running a variable head test. Water is forced into the ground with a high pressure to form a crack. Afterwards the water pressure is allowed to decrease as the mercury in the manometer drops.

In this study the "close up" (u_c) pressure was defined by determining the head, after fracture, at which the flowrate of water reduces back to the value before the crack was formed. In order to obtain the unfractured flowrate, the pressure was increased by steps and at each step a falling head test was run. After a crack was formed the head drop with time was monitored until the flowrate reduced to near the value before the soil was fractured.

The "close up" pressure thus defined was taken as an approximate total stress across the crack at crack closure. The value of K then can be obtained, if the vertical total stress (σ_v) and the initial pore water pressure of the ground (u_0) are known, as:

$$u_c - u_o = \Delta u = \bar{\sigma}_h$$

$$K = \frac{\bar{\sigma}_h}{\sigma_u - u_o}$$

in which $\bar{\sigma}_h$ is the horizontal effective stress and Δu is the excess pore water pressure at crack closure. If the stress conditions are one-dimensional then $K = K_o$.

This technique is limited at present by the permeability of the piezometer which does not allow it to be used for sands with most common piezometers. This method is also limited as a technique for measuring horizontal stress to cases where K is less than one. When K exceeds 1 then the "close up" pressure should be the overburden stress.

In order to determine the "close up" pressure the results from a test are plotted as a constant, C, in logarithm scale versus the excess water pressure (Δu) at time t_1. The constant, C, is defined as:

$$C = \frac{\ell n \frac{H_1}{H_2}}{t_2 - t_1}$$

$$H_n = \frac{u_n - u_o}{\gamma w}$$

in which t_1, t_2 = time, u_n = total pressure applied at time t_1 and u_o = pore water pressure of the ground. Appendix B shows an example of this computation.

The coefficient of permeability of the ground can be obtained by multiplying the C value with a constant which is a function of the geometry of the collection zone (Lambe and Whitman, 1969). In order to avoid uncertainties involved in the geometry of the collection zones for the Casagrande type piezometers, the C value was chosen to present the results instead of the coefficient of permeability.

In computing the C values the head loss due to water flow through tubings and fittings was neglected. The effect of change in volume of tubing under high pressure was also neglected in the computation. Appendix B shows that the head loss through the tubing and the magnitude of volume change of tubing under high pressure are negligible for the flow rate that occurs at the "close up" pressure.

In this study the soil was fractured twice in order to compare the two "close up" pressures resulting from the two tests with each other.

CHAPTER III

IN SITU TESTING PROGRAM

3.1 INTERSTATE ROUTE 95

In 1967 and 1968, a 2.4 mile long embankment through the Revere-Saugus tidal marsh, near Boston, Massachusetts was constructed for Interstate Route 95. The subsoil profile of the area consists of a peat layer underlain by a thin layer of fine silty sand. The sand layer is underlain by a deposit of a soft clay known as Boston Blue Clay which varies in depth from 40 to 160 feet. A plan location of the embankment is shown in Figure III-1.

In order to monitor construction and to obtain information on the reliability of techniques of predicting stability and deformation of such embankment, a large instrumentation program was undertaken to measure the performance and to compare the actual movement of the embankment with predictions. The instrumentation included a total of 95 hydraulic piezometers installed in the clay at varying depths as well as instrumentation for measuring deformations (Wolfskill and Soydemir, 1971).

3.2 PIEZOMETERS

Fifty-five Casagrande type (See Figure III-2) piezometers were installed at the center of the embankment at different locations and varying depths along the embankment. The embankment has undergone large deformations (up to 3 feet) over time and the leads of some of the piezometers have been pinched or snapped. Of the 55 piezometers installed

20 still seem to give good readings of pore pressure. The author has run hydraulic fracturing tests on those 20 piezometers but has obtained reliable results from only 7 piezometers. The others were either partially or completely plugged.

In July 1968, the installation of thirty-three hydraulic piezometers at the MIT-MDPW Test Section, located at Station 246+00 was completed. The piezometers were manufactured by Geomeasurement, Inc., and consist of an 18 inch long porous plastic sensor with two plastic riser tubes, a 3/8 in. reading lead and a 1/4 in. flushing lead. All leads were brought into an instrument tunnel or four manholes and connected to weatherproof pressure gages (Wolfskill and Soydemir, 1971). The author has attempted to run hydraulic fracturing tests on all 23 piezometers inside the tunnel but was unable to obtain any reliable results because of partial plugging or pinching of the leads due to the large settlements. Although some of the piezometers were still good for long term pore pressure readings, they were not sensitive enough to run hydraulic fracturing tests. Tests on the piezometers located in manholes were not undertaken due to freezing weather.

In the summer of 1973, seven Geonor Type M-206 piezometers (See Figure III-3) were installed at Station 263+00 to monitor the performance of the embankment under an additional surcharge. Figure III-4 shows the locations of these piezometers. Hydraulic fracturing tests were successfully completed on all of these.

Table III-1 lists piezometers on which successful hydraulic fracturing tests have been completed.

3.3 TEST RESULTS

This section describes the results from the hydraulic fracturing tests and from other methods of determining horizontal stress. The next section discusses the comparisons between the results from hydraulic fracturing and the results from the other methods.

The data from hydraulic fracturing tests are plotted as a constant, C, in log scale versus excess pore pressure (Δu). The approximation of the horizontal effective stress is determined by defining an excess pore pressure (Δu) at which the C value returns back to the values before the soil was fractured. Figures III-5 through III-11 show the results from the Casagrande type piezometers and Figures III-12 through III-18 from the Geonor Type M-206 piezometers. For comparison the clay was fractured twice as mentioned in Section 2.2. The solid lines in Figures III-4 through III-18 indicate the first fracture and the dotted lines indicate the second fracture. The in situ horizontal stress was obtained from the first fracturing test, and the arrow marks in the figures indicate these points. Table III-2 lists the horizontal effective stresses determined from the hydraulic fracturing tests.

In both the first and second fracturing the in situ coefficient of permeability (k) returns back to approximately the values before the soil was fractured. For example, in case of P-5 (Figure III-16) the in situ k value before the soil was fractured was 5.7×10^{-6} cm/sec (at $\Delta u = 1.5$ KSC), and the k value returned to 5.9×10^{-6} cm/sec (at $\Delta u = 1.5$ KSC) after the first fracture and 8.3×10^{-6} cm/sec (at $\Delta u = 1.6$ KSC) after the second fracture. The above k values were computed by using the following equation:

$$k = \frac{d^2 \cdot \ell n\left(\frac{2mL}{D}\right)}{8 \cdot L} \times C$$

in which d = inside diameter of manometer, D = diameter of intake, L = length of intake and $m = \sqrt{k_v/k_h}$. For the Geonor Type M-206 piezometer d = 1.905 cm, D = 3.15 cm, L = 25 cm and m was taken as 1.

In order to determine the K ($\bar{\sigma}_h/\bar{\sigma}_v$) values of clay, the vertical stress was computed by adding unit weights of overlying soils. The unit weights used are: 125 psf for the embankment, 120 psf for the sand layer, 115 psf for the normally consolidated clay and 120 psf for the overconsolidated clay.

The stress history of clay at Station 246+00 (MIT-MDPW Test Section) was used to estimate the OCR (Overconsolidation Ratio) of clays at other stations. The soil properties at the MIT-MDPW Test Section have been extensively investigated for predicting the performance of the Test Section. The index properties and stress history of the clay at this section are given in Figures III-19 and III-20. The computed OCRs at other stations are listed in Table III-2 assuming that the stress history of the clays along the entire length of the embankment is the same. The clays located under the center of the embankment was assumed to have consolidated one dimensionally because the crest and base of embankment are fairly wide (90 ft. plus for crest and 200 ft. plus for base) in comparison to the depth of clay.

Since the results for minor principal stress ($\bar{\sigma}_3$) obtained from the four piezometers (at Station 263+00) located at the toes of embankment and berms are not in the horizontal direction, the minor principal stresses ($\bar{\sigma}_3$) computed by the "FEECON" (Finite Element Analysis of Embankment Construction) program at Station 246+00 were used to make comparisons. This comparison is possible because the geometry of embankment and subsurface profiles at those two stations are similar.

The "FEECON" program computes the stresses and deformations within continuous bodies due to internal and external loads on the body. The stress-strain behavior of the soil is modelled with a non-linear hyperbolic relation which includes provisions to describe yielding. Values of total stress computed by this program are shown in Figure III-21. The minor principal effective stress ($\bar{\sigma}_3$) was obtained by subtracting the present pore pressure indicated by the piezometers. Table III-3 lists the $\bar{\sigma}_3$ obtained by hydraulic fracturing and the partially drained $\bar{\sigma}_3$ calculated using results from the "FEECON" analysis.

Values of K_o for different OCR's have been obtained on laboratory samples using a K_o-oedometer developed at MIT by Wissa. This device was described in Section 1.2.2. The horizontal stresses calculated from three different K_o-oedometer tests are available for comparison with the results from hydraulic fracturing tests. The three different K_o-oedometer tests are:

1) Undisturbed samples with silicone lubricant on the side of ring,

2) Undisturbed sample without silicone lubricant on the side of ring,

3) Remolded sample (Ladd, 1965) without silicone lubricant.

The tests on the undisturbed sample were run as follows: a) The sample was loaded to a high stress ($\bar{\sigma}_{vm}$) and unloaded in increments. b) The sample was then reloaded in steps and the horizontal stresses were recorded. Figure III-23 shows plots of K_0 versus OCR obtained from the three tests.

The horizontal stresses were calculated from the empirical correlation given by Brooker and Ireland (1965) for each of the hydraulic fracturing locations. Figure III-25 shows the K_0 versus OCR relation redrawn from Figure III-24. The plasticity index of Boston Blue Clay at I-95 was 20.1% (Recker, 1973). Tables III-4 through III-6 list the horizontal stresses computed from this relationship.

3.4 COMPARISONS AND DISCUSSIONS

The test results from the hydraulic fracturing test are separated in three groups and are compared with the results from other methods of determining minor principal stresses as follows.

Group 1: The minor principal effective stress ($\bar{\sigma}_3$) obtained from four piezometers, P-1, P-2, P-6 and P-7, located at the toes of embankment and berm are grouped together and compared with the $\bar{\sigma}_3$ calculated by the "FEECON" program. Table III-3 lists these results and Figure III-26 graphically portrays the comparison.

Group 2: Results from seven piezometers located at the center of embankment for which the top elevation of embankment is + 38.50 feet are grouped together and are compared with the results obtained from the K_0-oedometer tests and the previously mentioned empirical correlations. Tables III-4 and III-5 list the results and Figure III-27 shows the graphical comparison.

Group 3: Results from three piezometers located at the center of embankment for which the top elevation of embankmenc varies are grouped and compared with the results determined from the K_0-oedometer test and the empirical correlations. Table III-6 lists the results and Figure III-28 shows the graphical comparison.

In Group 1 all four results from the hydraulic fracturing tests gave values lower than the values calculated using the "FEECON" program, however, the results are in quite good agreement as shown in Figure III-26. The comparisons in Groups 2 and 3 indicate five hydraulic test results lower than the results from the other methods, one result higher than the other results and four results within the range of values obtained from the other results as shown in Figure III-29. For the 5 tests where the clay is thought to be normally consolidated, the K_0 value from hydraulic fracturing ranged from 0.35 to 0.58 with a mean value of 0.49.

The primary objective of this thesis is to evaluate the hydraulic fracturing method as a means of measuring in situ stress. The primary difficulty with such an evaluation is that there is no way presently known to measure directly in situ stresses; consequently there are no direct values with which to compare the results of hydraulic fracturing. It has been shown that the values of horizontal stress obtained from hydraulic fracturing were within about the same range as those obtained from laboratory K_o-oedometer tests and empirical correlations. Further work will be necessary to determine how representative these values are with true in situ values.

The K_o values obtained from hydraulic fracturing scatter more widely than the K_o values obtained from other methods, as shown in Figure III-29. The hydraulic fracturing results do indicate the trend of increasing K_o with increasing OCR.

Generally the ability to calculate changes in stress due to a surface loading is accepted as being reliable. Results from Group 1 piezometers at the toes of the embankment at Station 263+00 and berm can be used to determine how reliably hydraulic fracturing detects a change in minor principal effective stress. The calculated $\bar{\sigma}_3$ using "FEECON" for piezometers 1 and 2 are 1.22 and 1.46 KSC respectively (Table III-3). The difference between these two values (0.24 KSC) represents the increment in $\bar{\sigma}_3$ caused by placing the embankment. The increment in $\bar{\sigma}_3$ measured by hydraulic fracturing for these two piezometers is 1.57 - 1.35 = 0.22 KSC. Likewise for piezometers 6 and 7, the calculated effective stress increment from the embankment is 1.11 - 1.01 = 0.10 KSC whereas the increment indicated by hydraulic fracturing is 1.2 - 1.15 = 0.05 KSC. There has not yet been enough time for significant consolidation and consequent effective stress increase, but these preliminary comparisons suggest that the reliability of hydraulic fracturing measurement may be checked by taking measurements where known effective stress increases can be obtained.

As mentioned in Section 3.3, the OCR of clays were computed from the stress history at Station 246+00 assuming the entire deposit has about the same stress history. In fact, it is known that the stress history varies somewhat along the 2.4 mile alignment. Leifer (1973) reports maximum past pressures at Cutler Circle Bridge, about 1 mile away from the Test Section, which are considerably different. However, no stress history information is available at the stations where hydraulic fracturing tests were performed.

Other factors which might cause errors in the hydraulic fracturing test include the following.

1) Installation of piezometers; Casagrande type piezometers were installed in a prebored hole and a sand filter was tamped around them. This may change the stress condition in the soil around the piezometer such that it would no longer compare with results predicted from laboratory tests. The Geonor Type M-206 piezometers were pushed into the ground which may have considerably increased the in situ horizontal stresses for

some distance around the piezometers. The effect of such disturbance has been considered somewhat by Bjerrum, et. al. This will not significantly affect the results reported here since the value at which the crack closes was used as an estimate of $\bar{\sigma}_3$.

2) The technique of measuring in situ minor principal stress; The technique adopted is to measure the "close up" pressure instead of the "fracturing" pressure. This method may be in error although it seems to give the most reasonable comparison obtained by other methods. Selecting the "close up" pressure is also reasonable because more uncertainties are involved in the "fracturing" pressure. As mentioned in Section 1.4, the "fracturing" pressure is higher than the "close up" pressure. The possible reasons for these differences are:

   a) Undrained shear strength of the soil may control the "fracturing" pressure in a short term test. This can be avoided by running a long term (drained) test but it is difficult to run such a test.

   b) The effective tensile strength of the soil, although it is generally negligible, may in some cases require higher "fracturing" pressure.

   c) The shape of hole, in case of Casagrande piezometer, may be a factor to the higher "fracturing" pressure.

Vaughan (1972) has discovered that uneven holes require higher "fracturing" pressure than smooth holes.

d) In the process of installing piezometers the stresses around the piezometer tip may have been increased.

Although it is very difficult to assess the validity of the hydraulic fracturing method without having other techniques which can measure accurate in situ lateral stress, the indications from this work are that the hydraulic fracturing method can measure reasonably reliable values of in situ minor principal stresses. This conclusion is based on the fact that the range of values obtained from the hydraulic fracturing agrees fairly closely with the range of values obtained from the other methods (See Figure III-29) and that changes in stress from an increase in load can be obtained which compare with predicted stress changes.

As mentioned in Section 3.2, two types of piezometers, Casagrande type and Geonor type, were used for testing, and the author could not find any differences of trends in the test results obtained from them.

CHAPTER IV

CONCLUSIONS AND RECOMMENDATIONS

Based on the results from hydraulic fracturing tests run at 14 different locations using two different types of piezometer, the author has drawn the following conclusions:

1) The hydraulic fracturing method for measuring in situ minor principal stress in feasible,

2) The results from hydraulic fracturing tests generally agree with the results obtained from K_0-oedometer laboratory tests and empirical correlations,

3) The hydraulic fracturing device developed for this testing program works well in supplying a wide range of pressures and flow rates. The equipment can also be used to run permeability tests on piezometers,

4) No differences of trends were found in the test results obtained from the geonor-type piezometers and the Casagrande-type piezometers.

The recommendations for further study include:

1) The addition of a volume change measuring device for an accurate measurement of water outflow at low flow rates would allow more accurate determination of permeability at low pressures before and after fracturing,

2) A series of test runs on the same piezometers and a comparison with the results from the first tests to see how reproducable the results are after time,

3) Improvement of the device for soils which are more permeable than clays,

4) Use of this device to measure change of in situ horizontal stress with consolidation to better determine how the results compare with field behavior. If hydraulic fracturing detects the same stress changes as those predicted by stress distribution analysis, then one can feel more confident that the measured stresses are representative of the actual in situ values.

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Piezometer No. (Type)	Station	Sensor Elevation (Ft.)	Top Elevation of Embankment
P-2-B (Casagrande)	209+00	-30.4	+35.0
P-10-B (Casagrande)	241+00	-61.5	+38.5
P-11-B (Casagrande)	251+00	-59.2	+38.5
P-12-B (Casagrande)	255+00	-65.0	+38.5
P-13-B (Casagrande)	259+00	-41.0	+38.5
P-16-B (Casagrande)	288+00	-41.3	+25.0
P-18-B (Casagrande)	312+00	-19.8	+22.3
P-1 (Geonor)	263+00	-57.0	+14.0
P-2 (Geonor)	263+00	-54.0	+16.0
P-3 (Geonor)	263+00	-35.5	+38.5
P-4 (Geonor)	263+00	-57.5	+38.5
P-5 (Geonor)	263+00	-79.5	+38.5
P-6 (Geonor)	263+00	-34.0	+14.0
P-7 (Geonor)	263+00	-40.0	+ 5.0

Table III-1

LIST OF PIEZOMETERS

Piez. No.	σ_v (KSC)	u_o (KSC)	$\bar{\sigma}_v$ (KSC)	$\bar{\sigma}_{vm}$ (KSC)	OCR	$\bar{\sigma}_h$ (KSC)	K_o or K
P-2-B	3.92	1.73	2.19	3.41	1.56	1.67	0.76
P-10-B	5.91	3.05	2.86	2.86	1.00	1.65	0.58
P-11-B	5.78	2.89	2.89	2.89	1.00	1.68	0.58
P-12-B	6.12	3.23	2.89	2.89	1.00	1.02	0.35
P-13-B	4.75	2.36	2.39	3.03	1.26	1.13	0.47
P-16-B	3.93	1.83	2.10	3.00	1.43	0.97	0.46
P-18-B	2.54	0.68	1.86	3.66	1.97	1.25	0.67
* P-1	4.15	2.27	1.88	2.34	1.24	1.58	0.84
* P-2	4.50	2.38	2.12	2.25	1.06	1.82	0.86
P-3	4.45	1.91	2.53	3.42	1.34	1.25	0.49
P-4	5.68	3.26	2.42	2.42	1.00	1.03	0.43
P-5	6.92	3.93	2.99	2.99	1.00	1.50	0.50
* P-6	3.50	1.61	1.89	3.08	1.63	1.37	0.72
* P-7	2.60	1.33	1.27	3.03	2.39	0.99	0.78

* Not one dimensional case (the $\bar{\sigma}_v$ and $\bar{\sigma}_h$ is calculated by extrapolating the results from two dimensional analysis).

Table III-2

RESULTS FROM HYDRAULIC FRACTURING TEST

Piez. No.	Sensor El.	$\bar{\sigma}_3$ From Hydraulic Fracture (KSC)	$\bar{\sigma}_3$ From "FEECON" (Draned) (KSC)	*Excess Pore Pressure (Δu) (KSC)	$\bar{\sigma}_3$ Partially Drained (KSC)	Location of Piezometer
P-1	-57.0	1.35	1.54	0.32	1.22	Toe of Berm
P-2	-54.0	1.57	1.98	0.52	1.46	Toe of Emb.
P-6	-34.0	1.20	1.42	0.31	1.11	Toe of Emb.
P-7	-40.0	1.15	1.22	0.21	1.01	Toe of Berm

Table III-3

COMPARISON BETWEEN HYDRAULIC FRACTURING TEST AND THE "FEECON"

Top Elevation of Embankment = +38.5'

Undisturbed Sample

Piez. No.	Sensor El.	$\bar{\sigma}_v$ (KSC)	OCR	Hydraulic Fracturing		K_o- Oedometer With Grease		K_o-Oedometer Without Grease	
				$\bar{\sigma}_h$ (KSC)	K_o	$\bar{\sigma}_h$ (KSC)	K_o	$\bar{\sigma}_h$ (KSC)	K_o
P-10-B	-61.5	2.86	1.00	1.65	0.58	1.72	0.60	1.57	0.55
P-11-B	-59.2	2.89	1.00	1.68	0.58	1.73	0.60	1.59	0.55
P-12-B	-65.0	2.89	1.00	1.02	0.35	1.73	0.60	1.59	0.55
P-13-B	-41.0	2.39	1.26	1.13	0.47	1.53	0.64	1.36	0.57
P-3	-35.5	2.54	1.34	1.25	0.49	1.65	0.65	1.47	0.58
P-4	-57.5	2.42	1.00	1.03	0.43	1.45	0.60	1.33	0.55
P-5	-79.5	2.99	1.00	1.50	0.50	1.79	0.60	1.64	0.55

Table III-4

COMPARISON BETWEEN HYDRAULIC FRACTURING TEST AND K_o-OEDOMETER

Top Elevation of Embankment = +38.5'

Piez. No.	Sensor El.	$\bar{\sigma}_v$ (KSC)	OCR	Hydraulic Fracturing		K_o-Oedometer Remolded Sample		Brooker & Ireland P.I. of BBC = 20.1%	
				$\bar{\sigma}_h$ (KSC)	K_o	$\bar{\sigma}_h$ (KSC)	K_o	$\bar{\sigma}_h$ (KSC)	K_o
P-10-B	-61.5	2.86	1.00	1.65	0.58	1.37	0.48	1.50	0.525
P-11-B	-59.2	2.89	1.00	1.68	0.58	1.39	0.48	1.52	0.525
P-12-B	-65.0	2.89	1.00	1.02	0.35	1.39	0.48	1.52	0.525
P-13-B	-41.0	2.39	1.26	1.13	0.47	1.27	0.53	1.34	0.562
P-3	-35.5	2.54	1.34	1.25	0.49	1.40	0.55	1.47	0.580
P-4	-57.5	2.42	1.00	1.03	0.43	1.16	0.48	1.27	0.525
P-5	-79.5	2.99	1.00	1.50	0.50	1.44	0.48	1.57	0.525

Table III-5

COMPARISON BETWEEN HYDRAULIC FRACTURING TEST AND OTHER METHODS

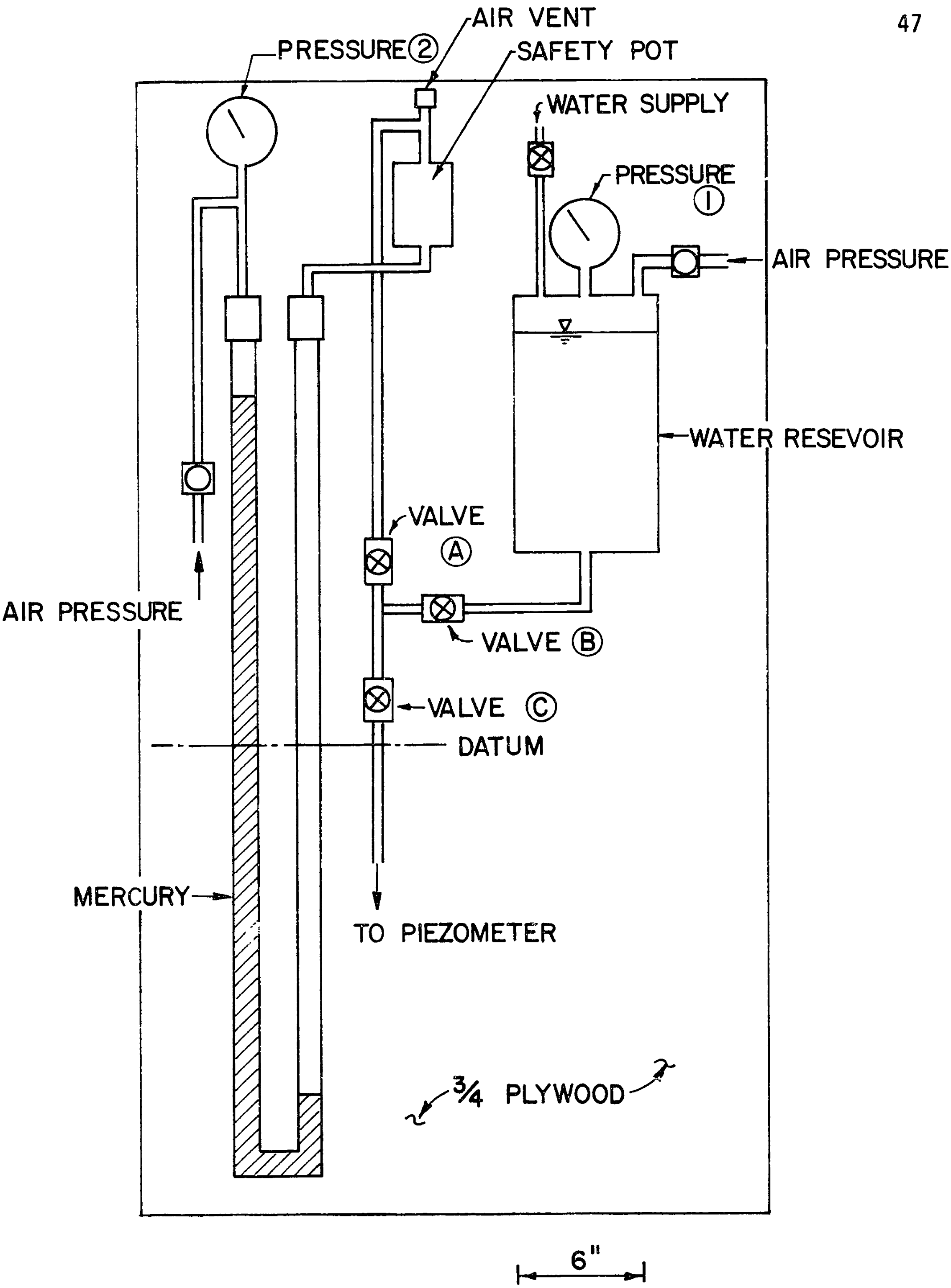
Top Elevation of Embankment Varies

Piez. No.	Sensor El.	$\bar{\sigma}_v$ (KSC)	OCR	Hydraulic Fracturing		K_o-Oedometer With Grease		K_o-Oedometer Without Grease	
				$\bar{\sigma}_h$ (KSC)	K_o	$\bar{\sigma}_h$ (KSC)	K_o	$\bar{\sigma}_h$ (KSC)	K_o
P-2-B	-30.4	2.19	1.56	1.67	0.76	1.51	0.69	1.31	0.60
P-16-B	-41.3	2.10	1.43	0.97	0.46	1.41	0.67	1.22	0.58
P-18-A	-19.8	1.86	1.97	1.25	0.67	1.43	0.77	1.19	0.64

Piez. No.	Sensor El.	$\bar{\sigma}_v$ (KSC)	OCR	Hydraulic Fracturing		K_o-Oedometer Remolded Sample		Brooker & Ireland P.I. of BBC = 20.1%	
				$\bar{\sigma}_h$ (KSC)	K_o	$\bar{\sigma}_h$ (KSC)	K_o	$\bar{\sigma}_h$ (KSC)	K_o
P-2-B	-30.4	2.19	1.56	1.67	0.76	1.33	0.61	1.31	0.60
P-16-B	-41.3	2.10	1.43	0.97	0.46	1.22	0.58	1.24	0.59
P-18-A	-19.8	1.86	1.97	1.25	0.67	1.28	0.69	1.20	0.645

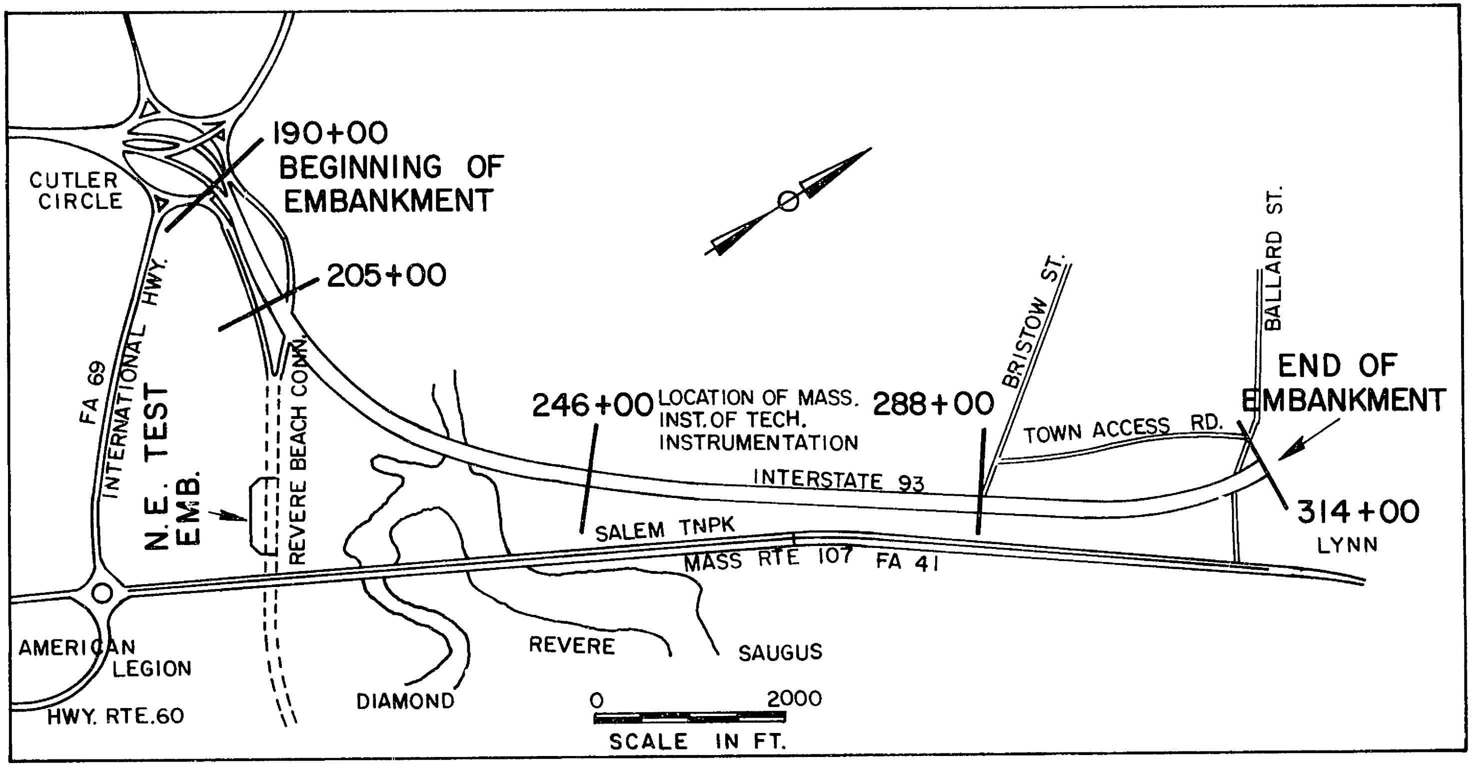
Table III-6

COMPARISON BETWEEN HYDRAULIC FRACTURING AND OTHER METHODS



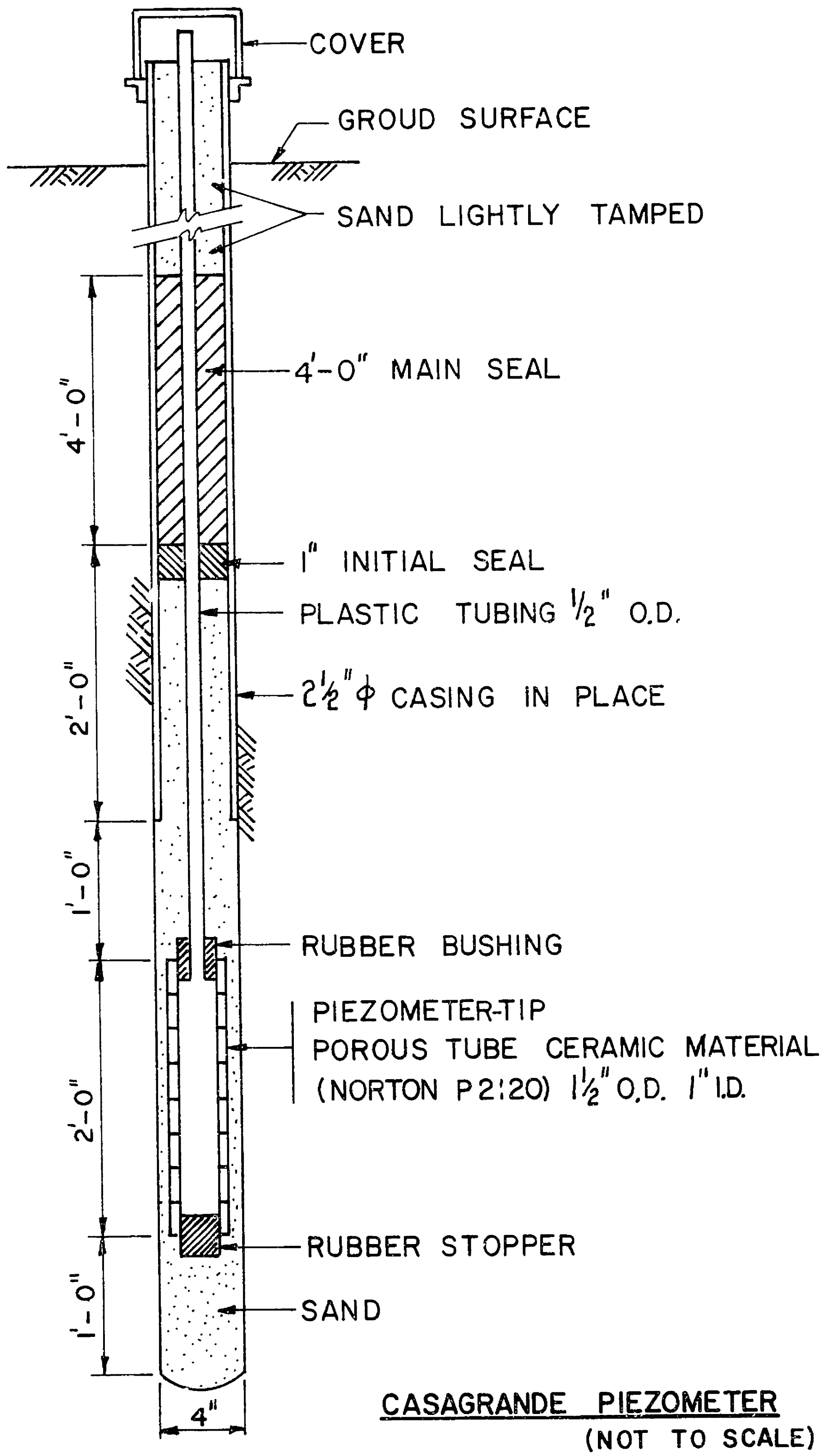
HYDRAULIC FRACTURING DEVICE

FIGURE II-1



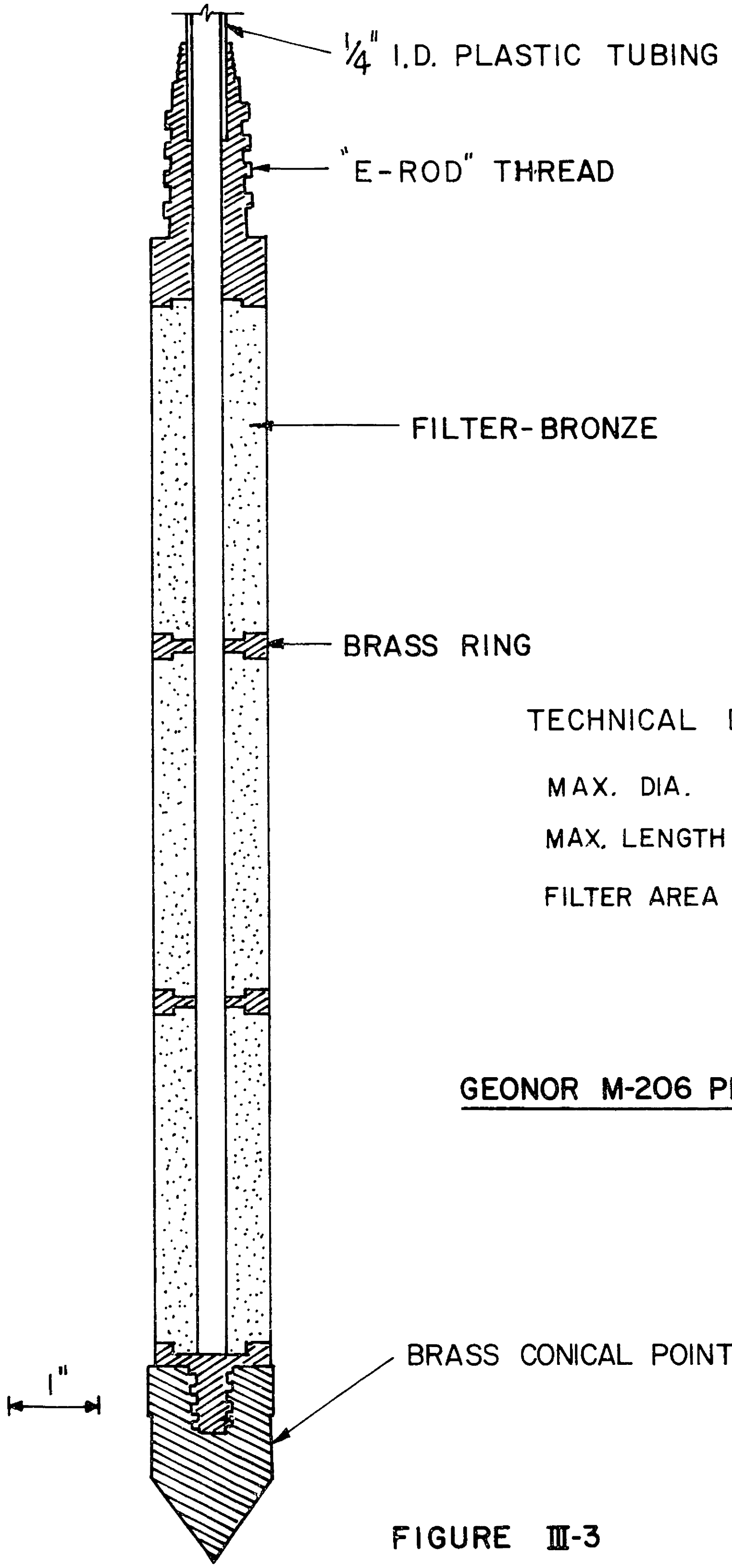
PLAN - I-95 REVERE - SAUGUS AREA

FIGURE III-I



CASAGRANDE PIEZOMETER
(NOT TO SCALE)

FIGURE III-2



TECHNICAL DATA

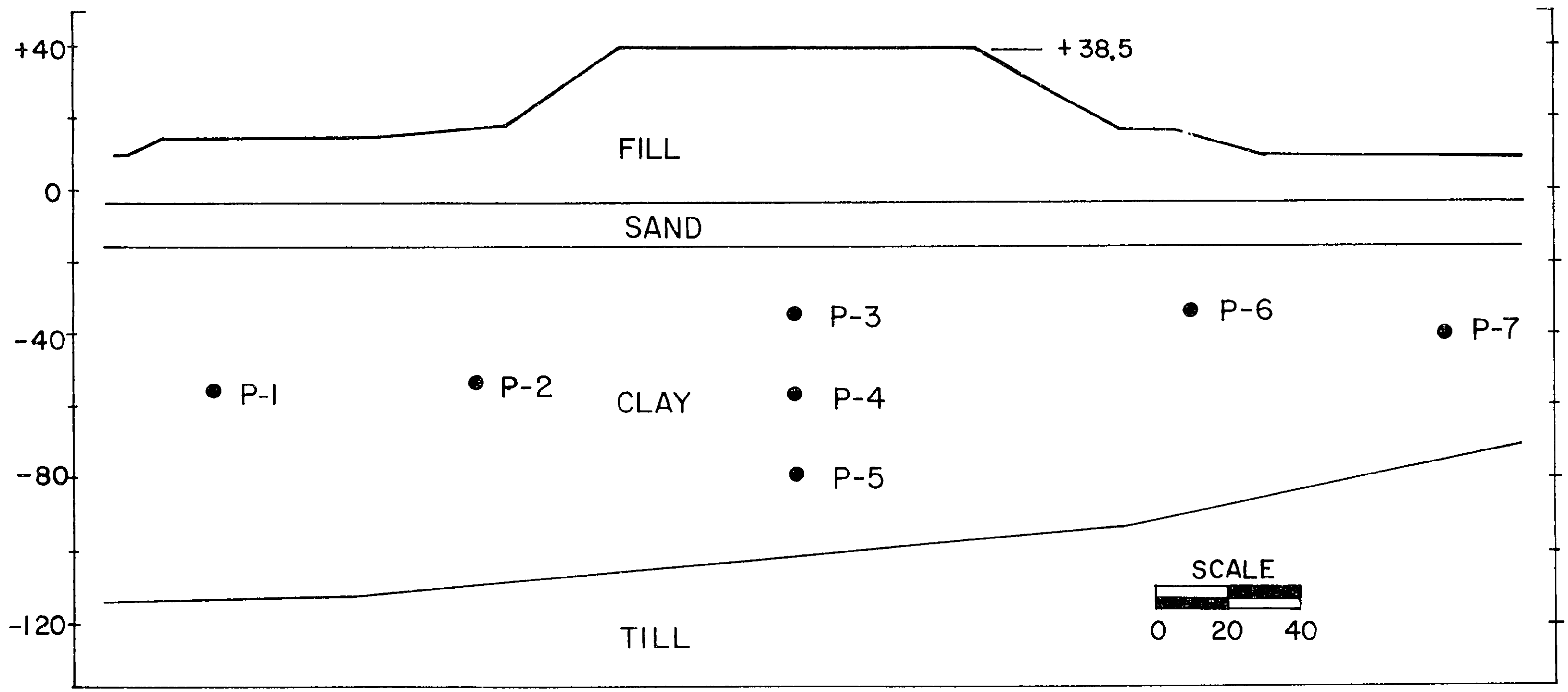
MAX. DIA. : 1 5/16 IN.

MAX. LENGTH : 17 IN

FILTER AREA : 35 SQ. IN.

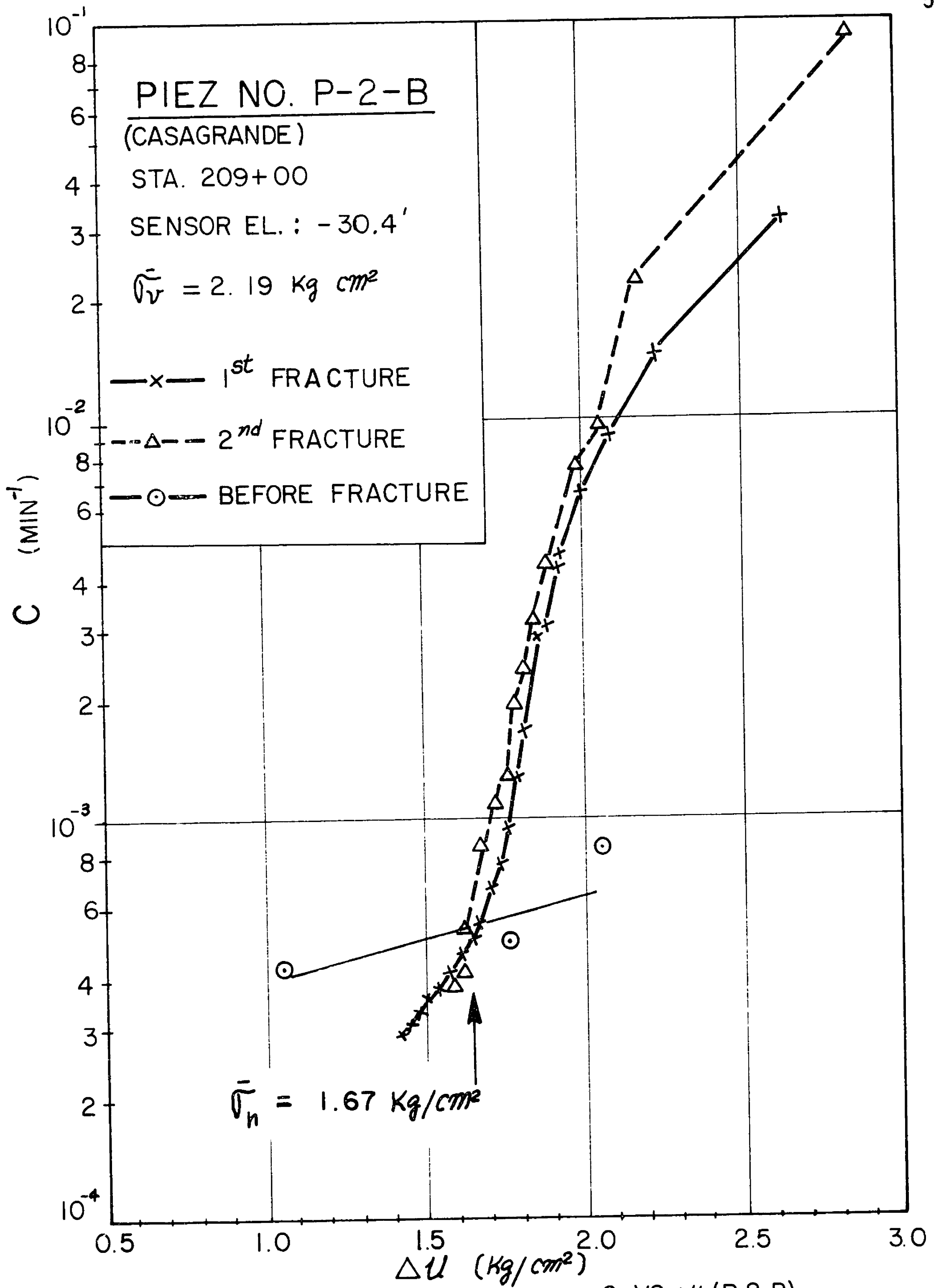
GEONOR M-206 PIEZOMETER

FIGURE III-3



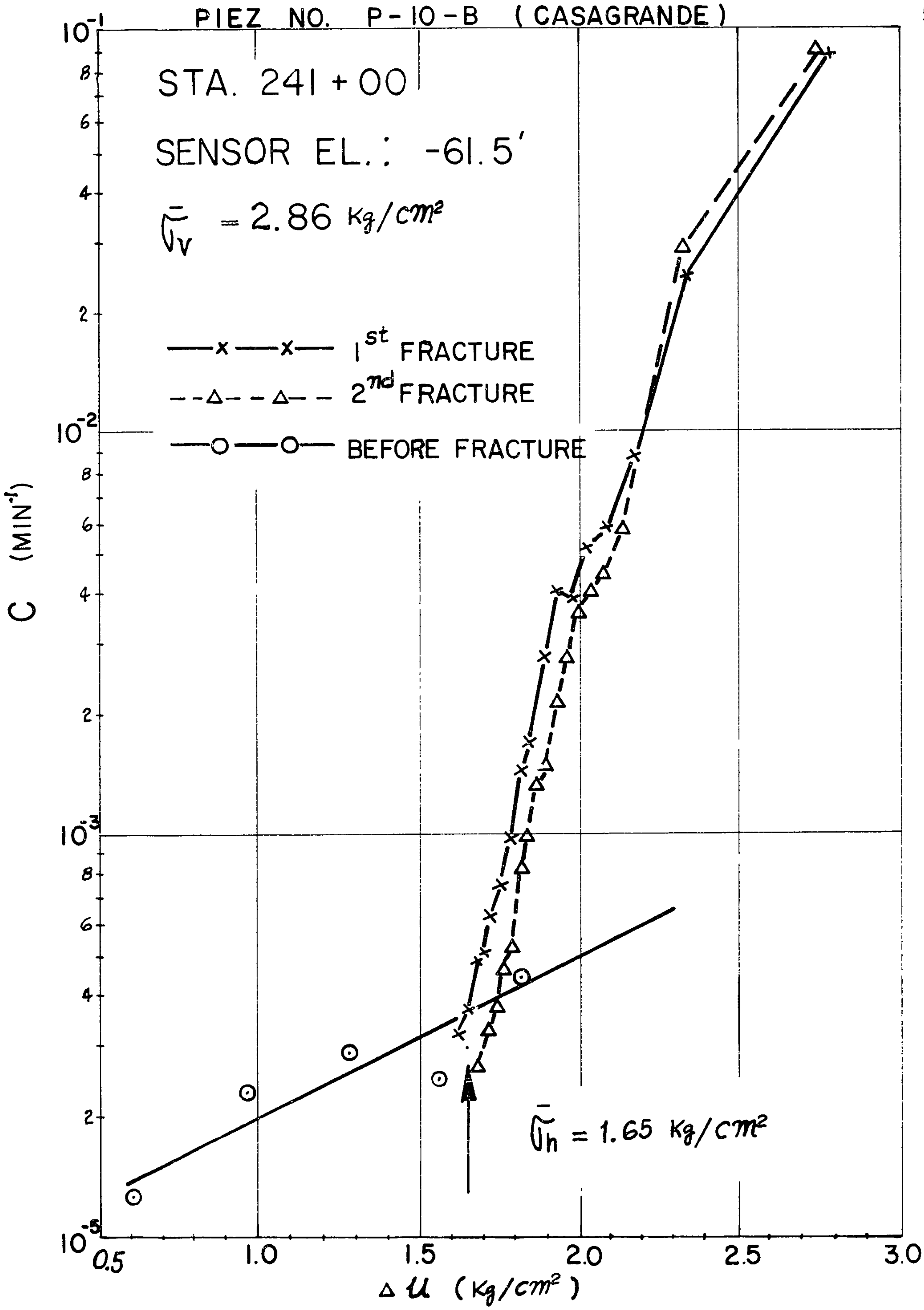
GEONOR PIEZOMETERS AT STA. 263+00

FIGURE III-4



C VS. ΔU (P-2-B)

FIGURE III-5



C VS. Δu (P-10-B)

FIGURE III - 6

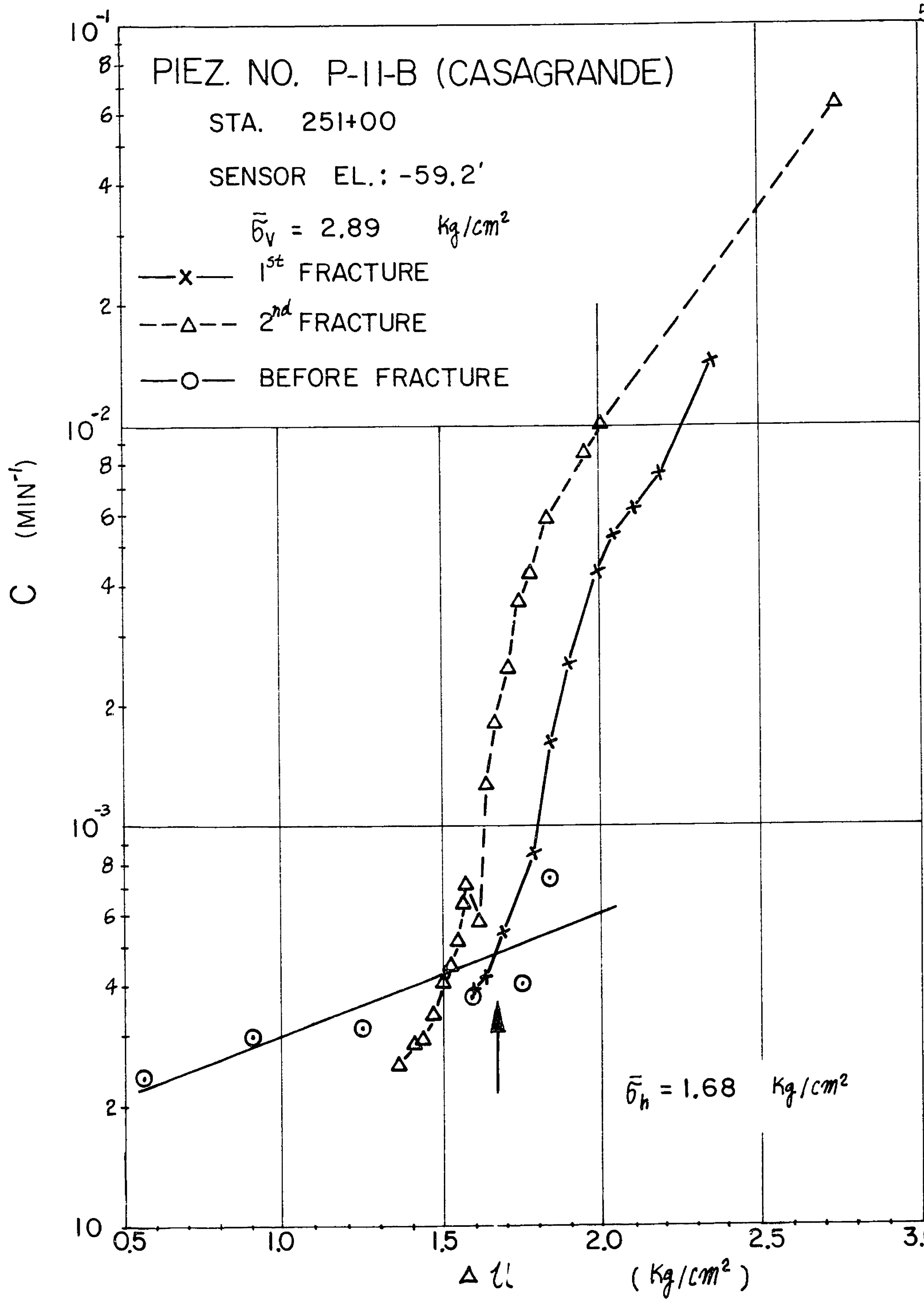


FIGURE III-7 C VS. Δu (P-11-B)

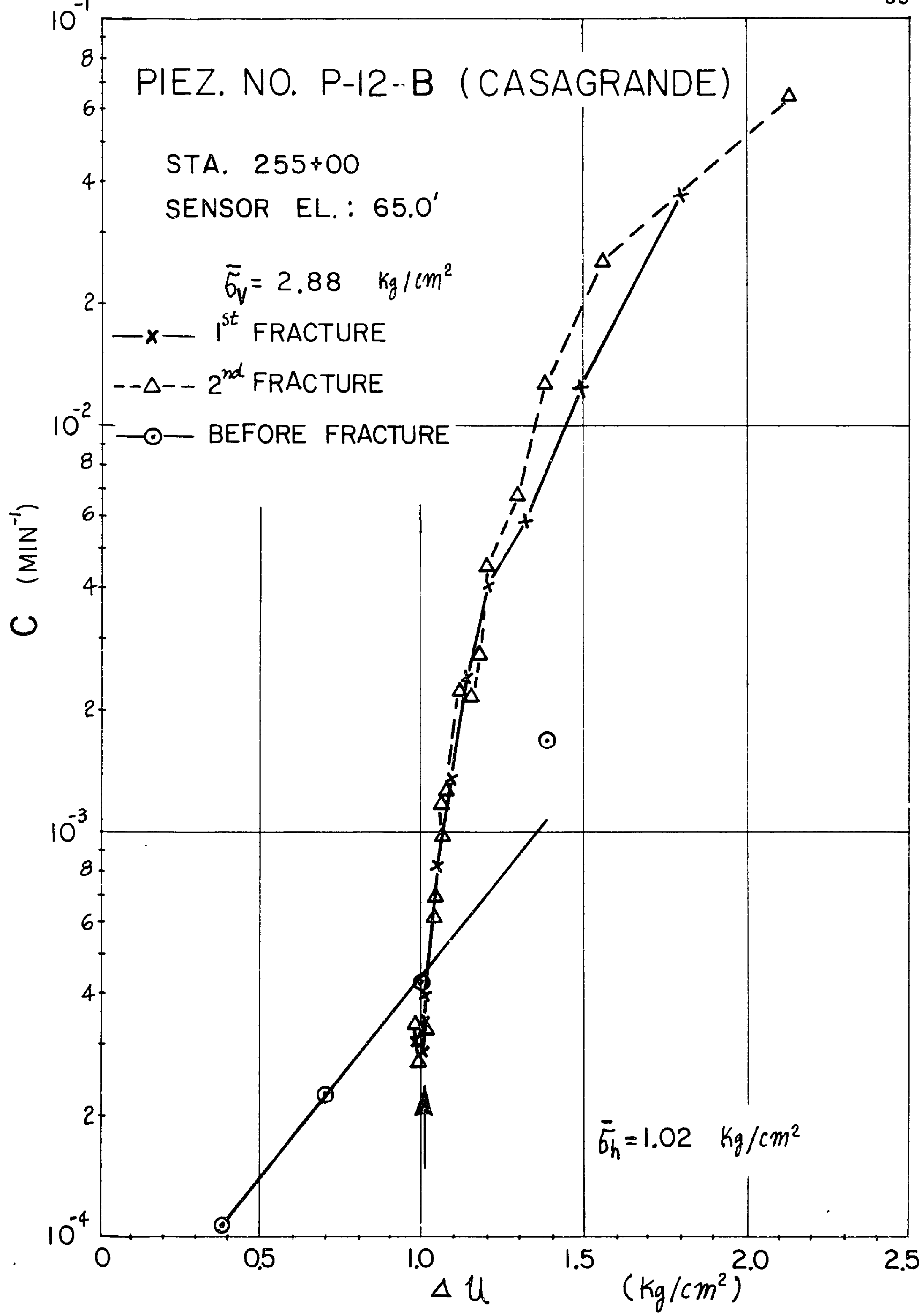
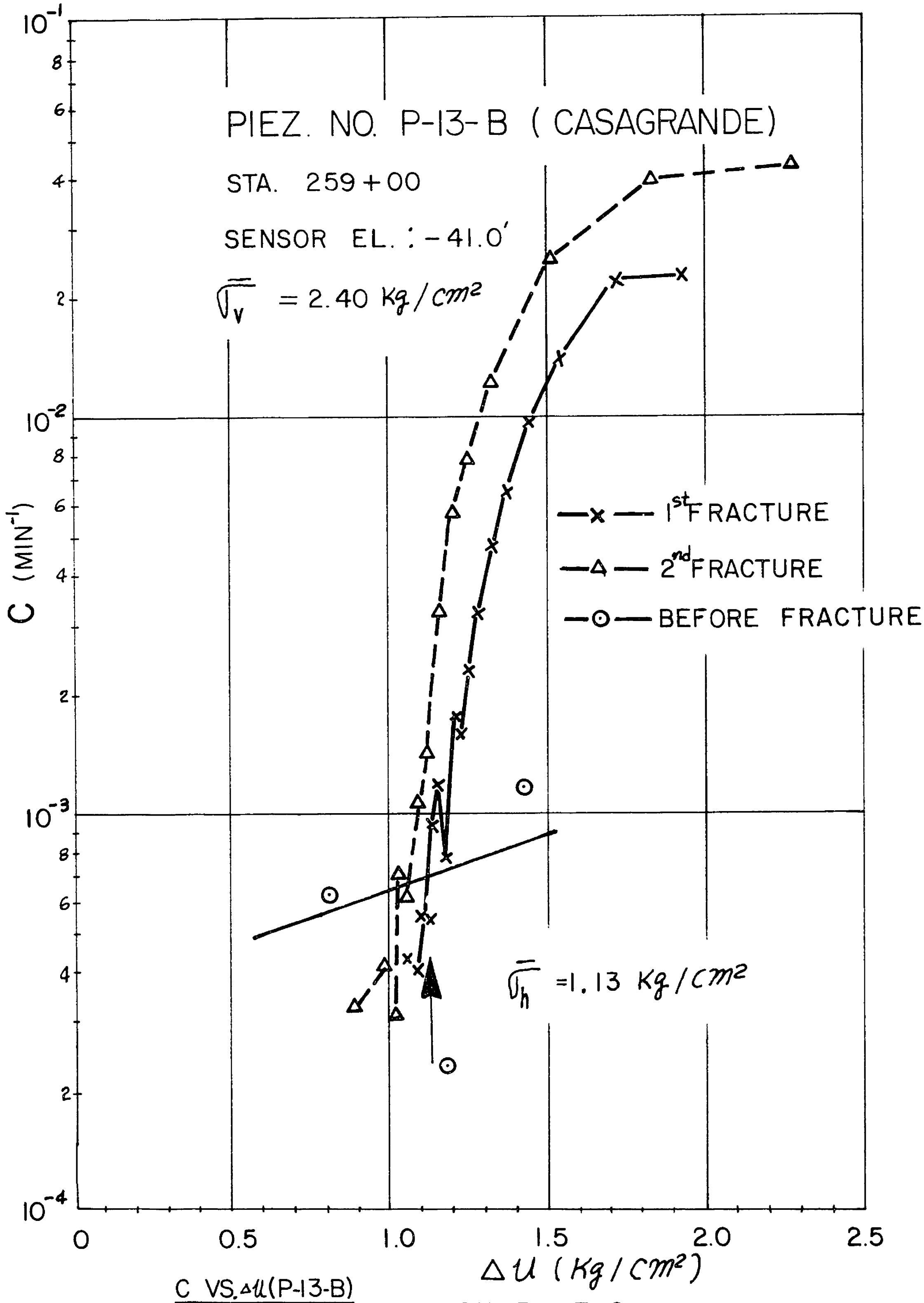


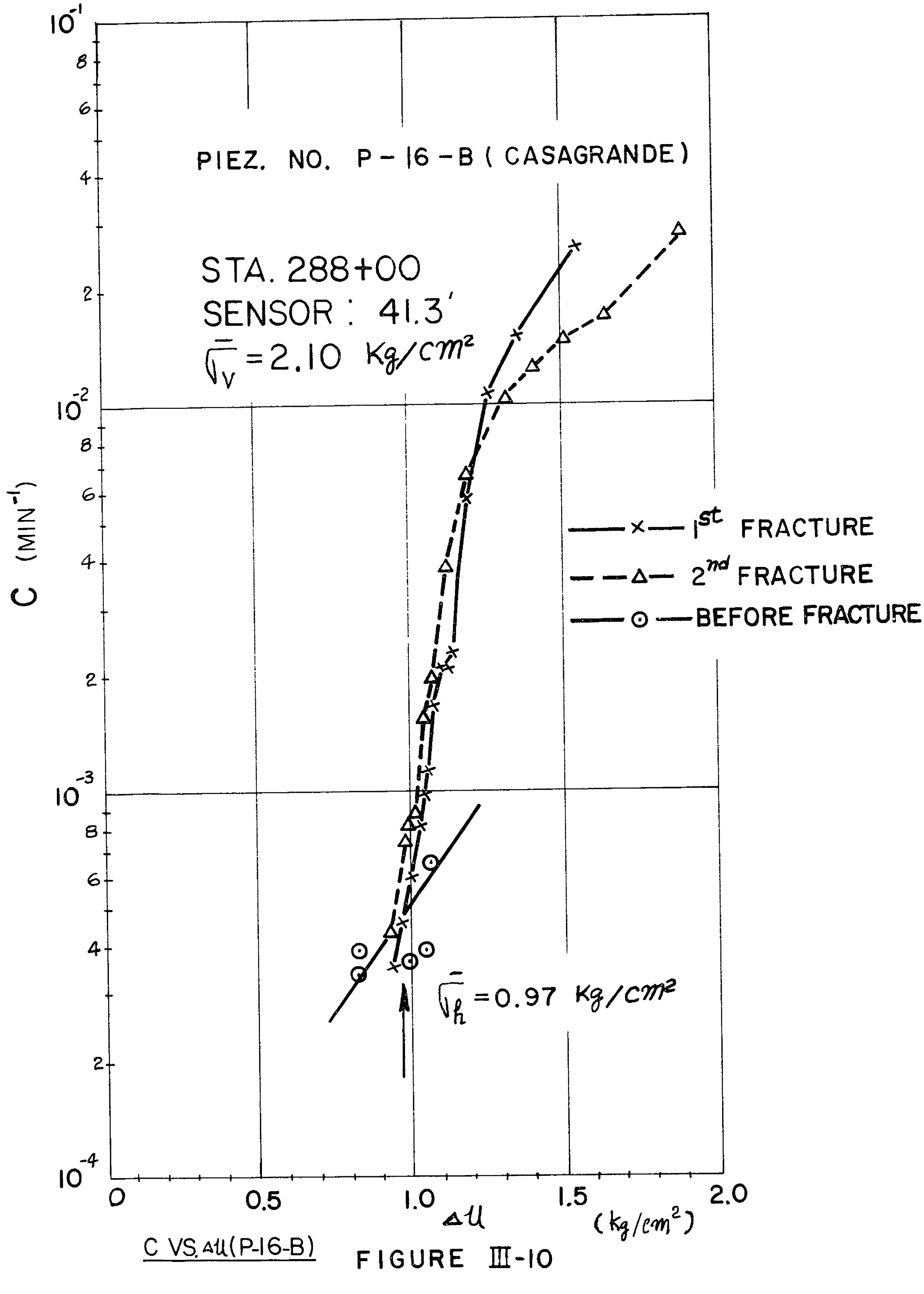
FIGURE III-8

C VS. Δu (P-12-B)



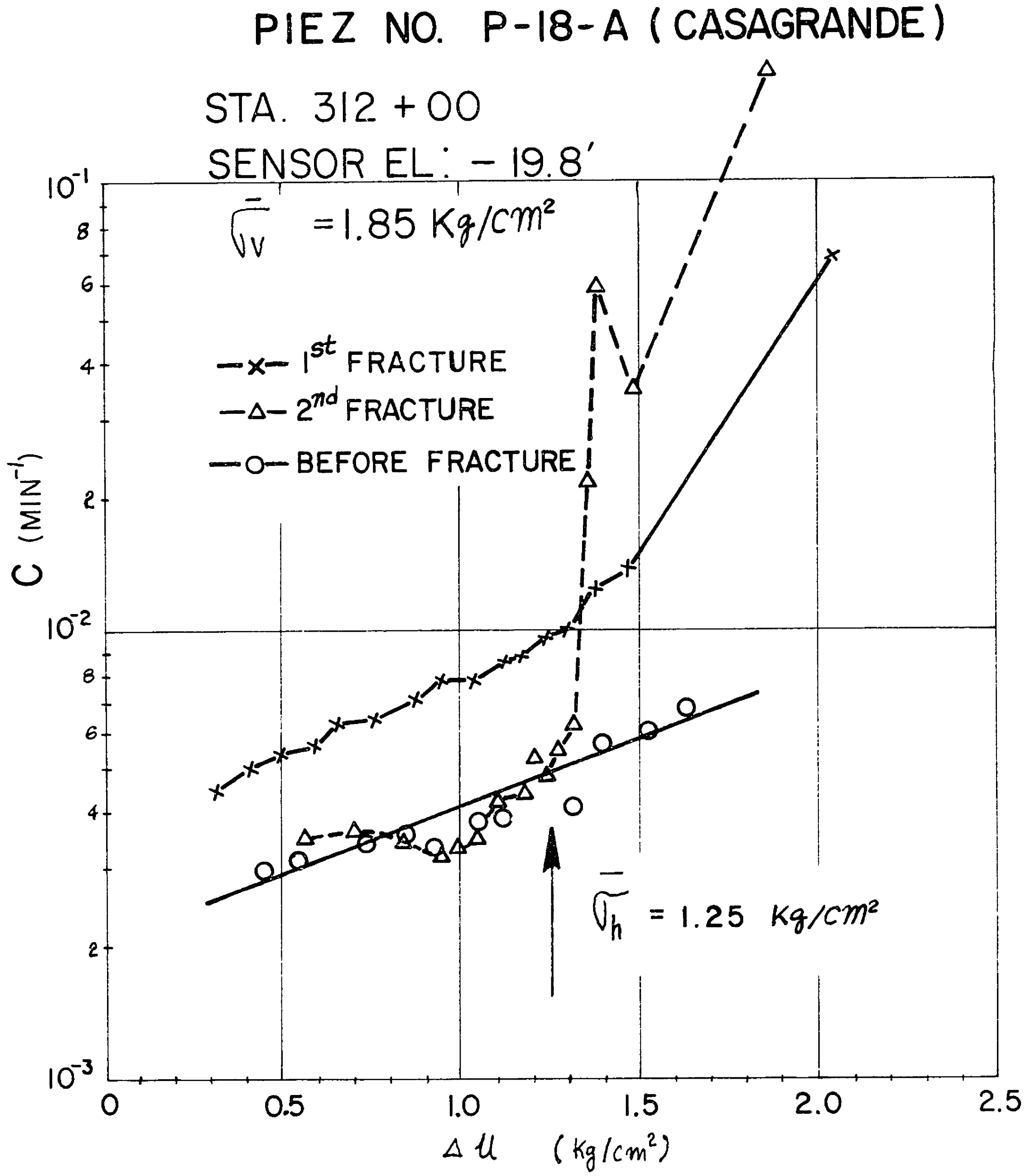
C VS. Δu (P-13-B)

FIGURE III-9



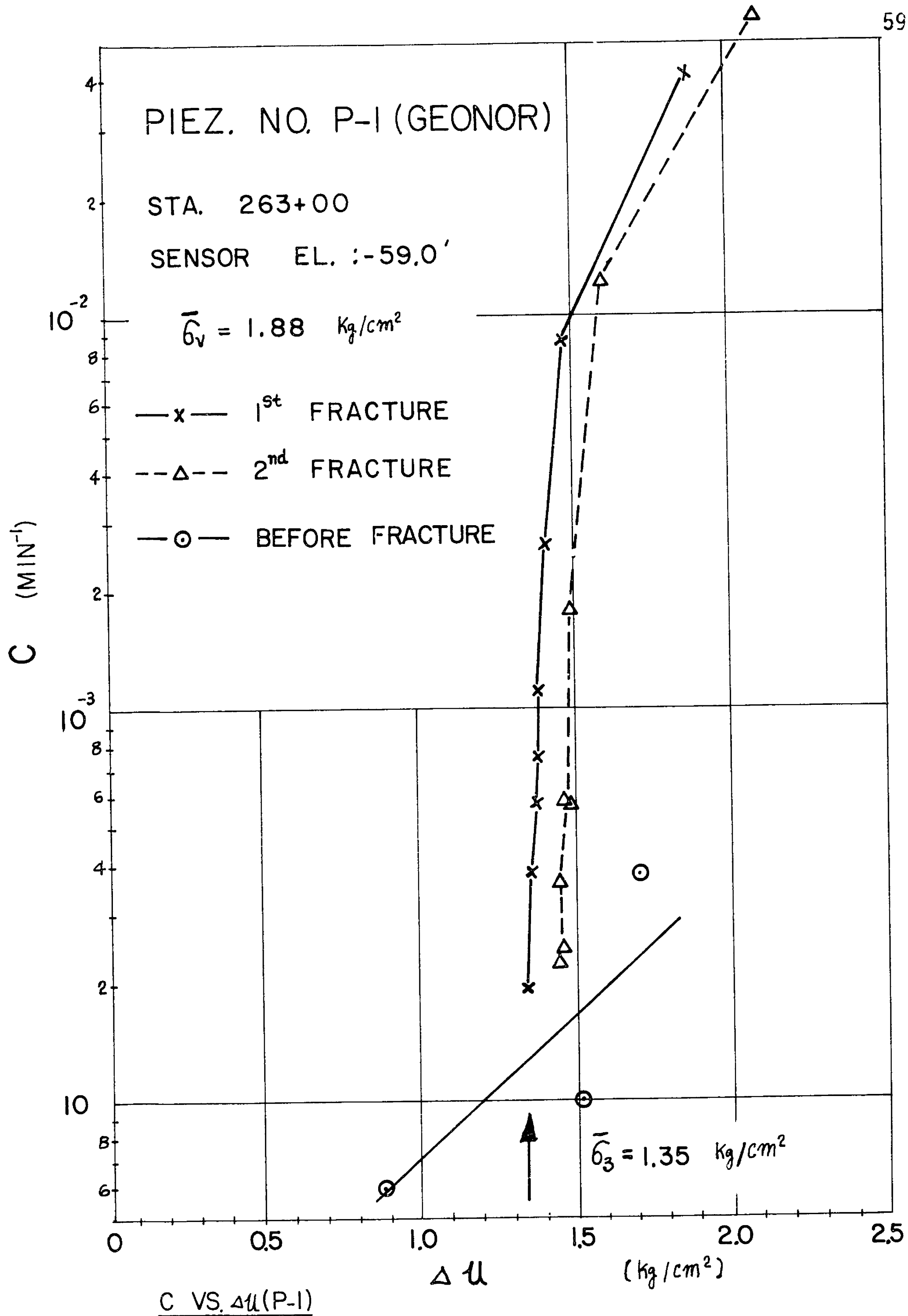
C VS. Δu (P-16-B)

FIGURE III-10



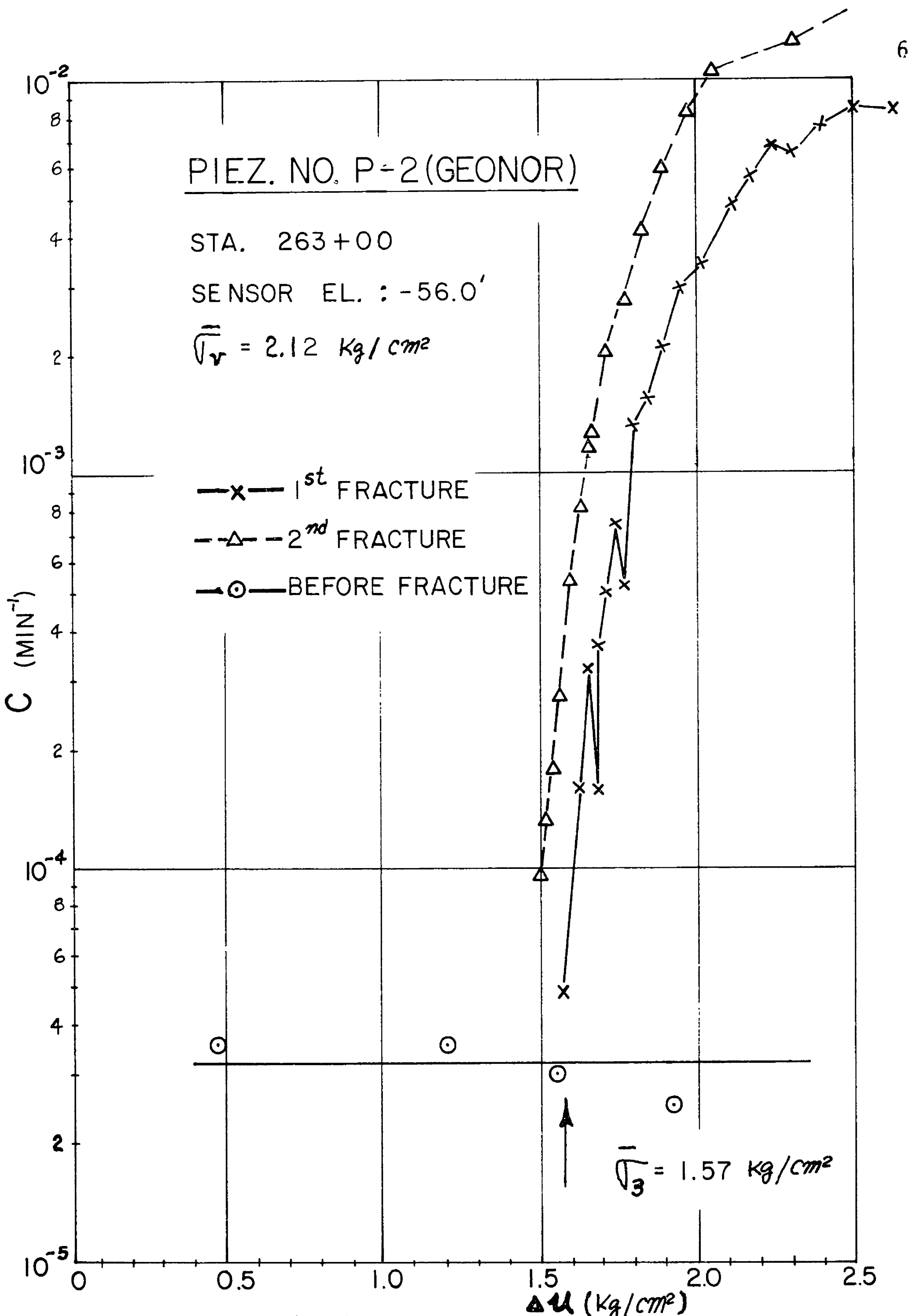
C VS. Δu (P-18-A)

FIGURE III-11



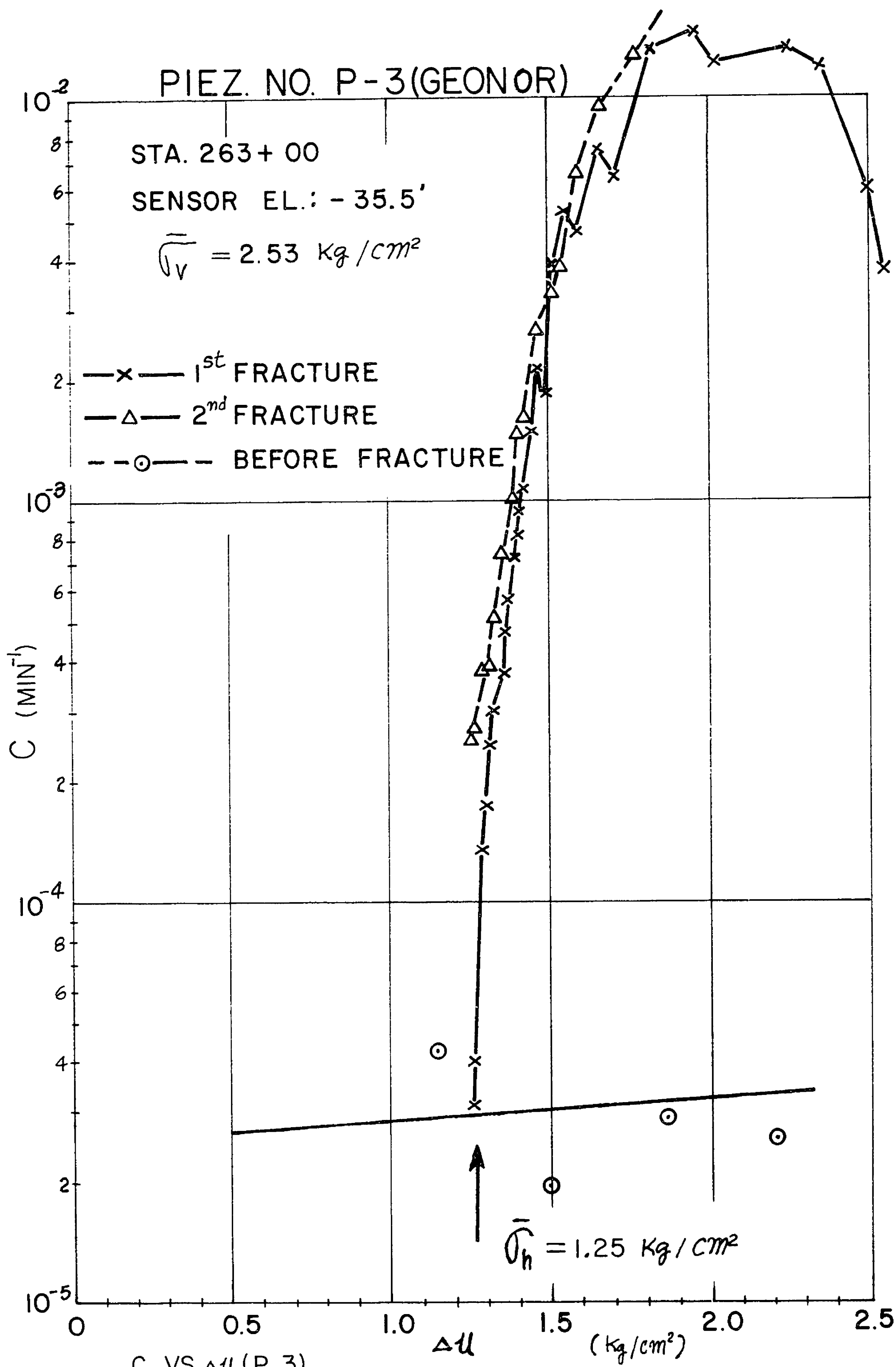
C VS. Δu (P-1)

FIGURE III - 12



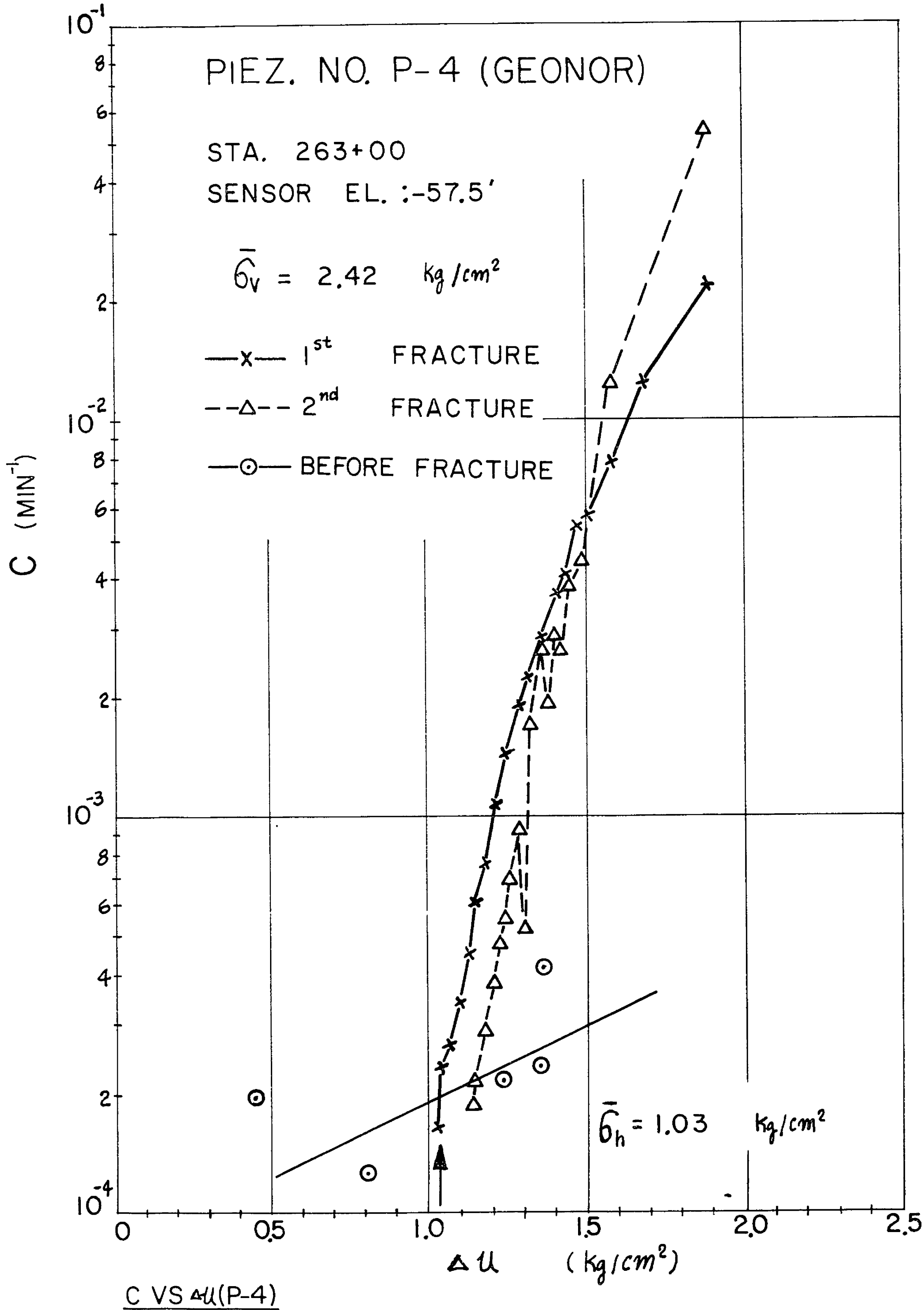
C VS. Δu (P-2)

FIGURE III-13



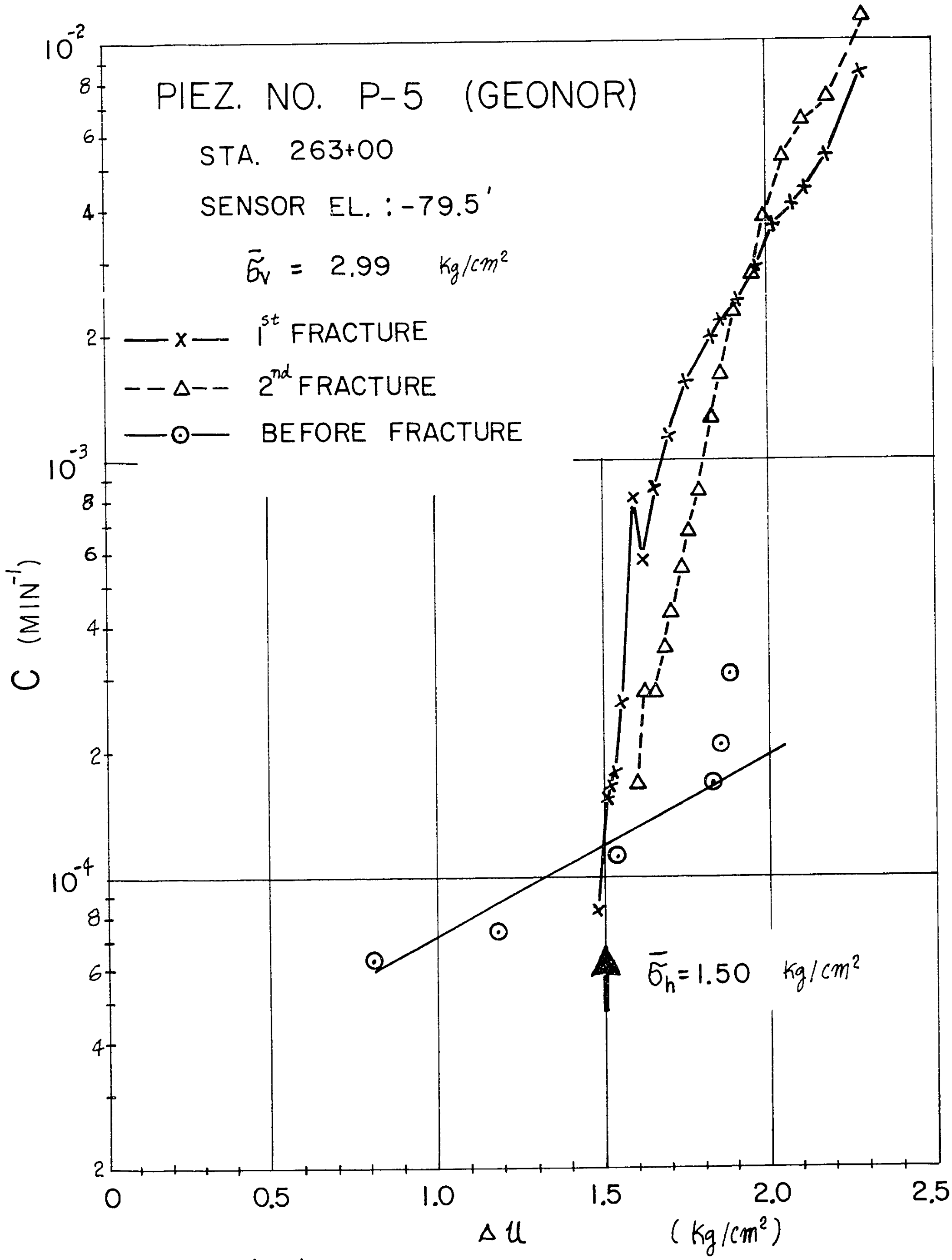
C VS. Δu (P-3)

FIGURE III-14



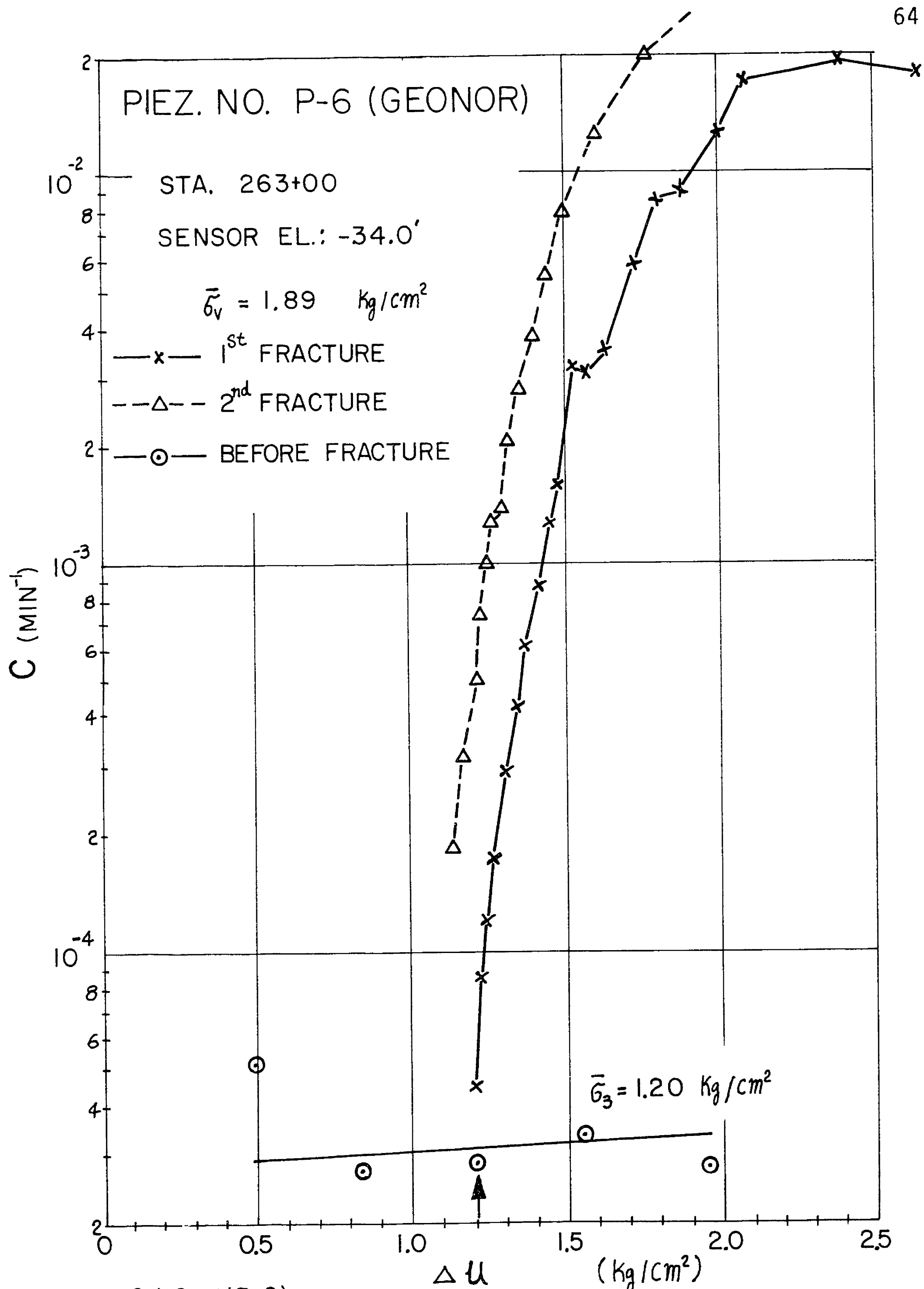
C VS Δu(P-4)

FIGURE III-15



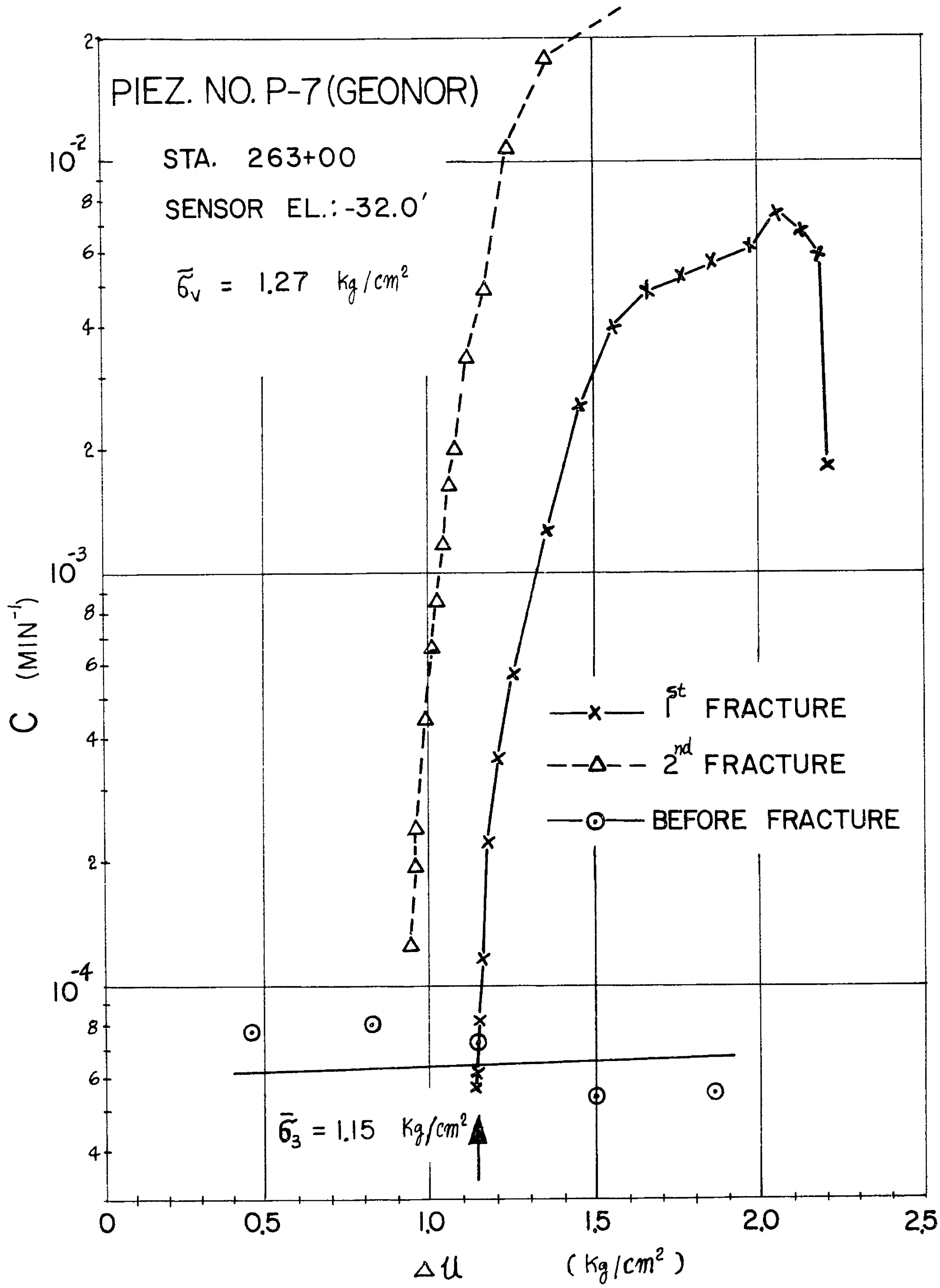
C VS. Δu (P-5)

FIGURE III-16



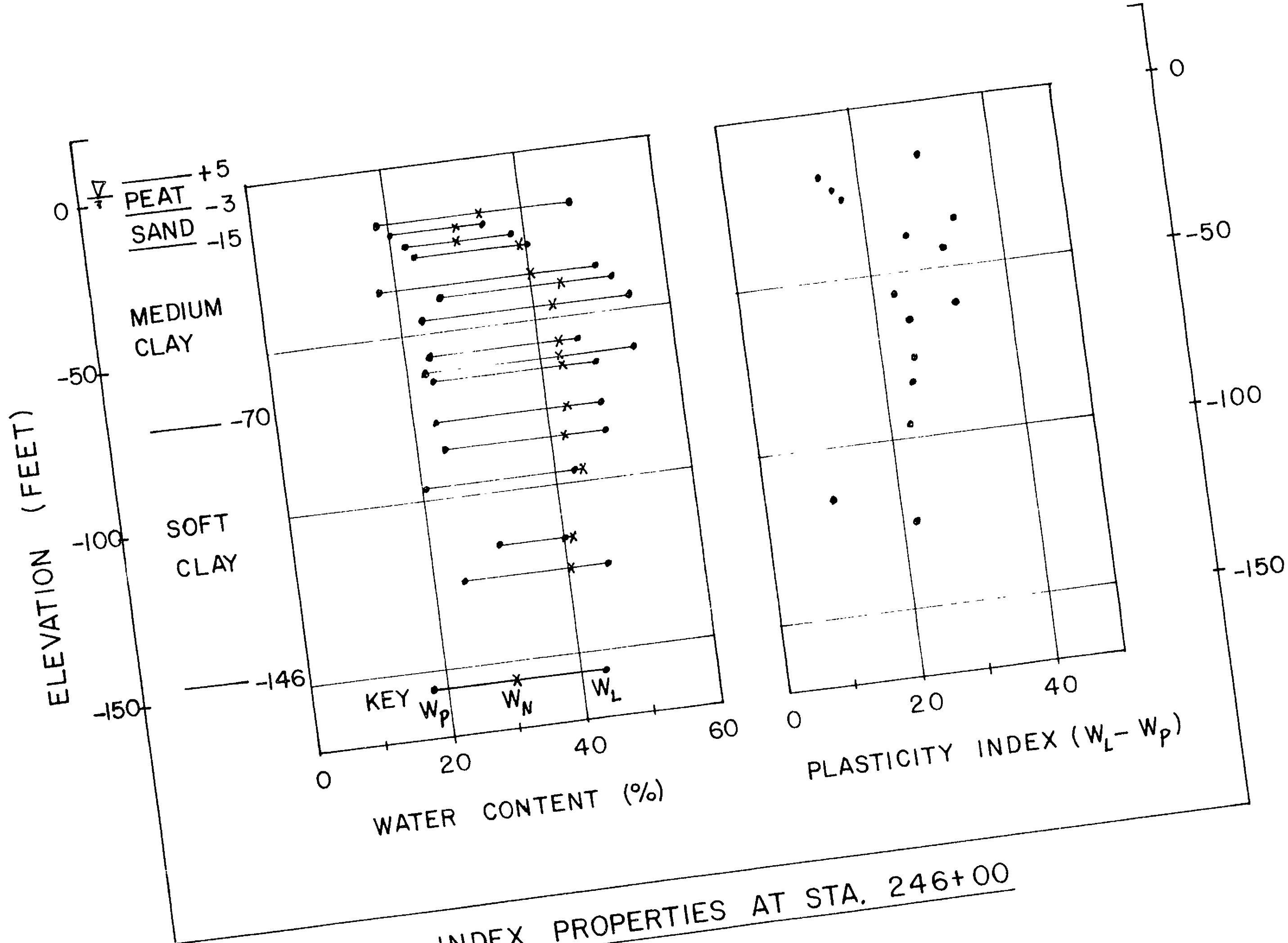
C VS. Δu (P-6)

FIGURE III-17



C VS. Δu (P-7)

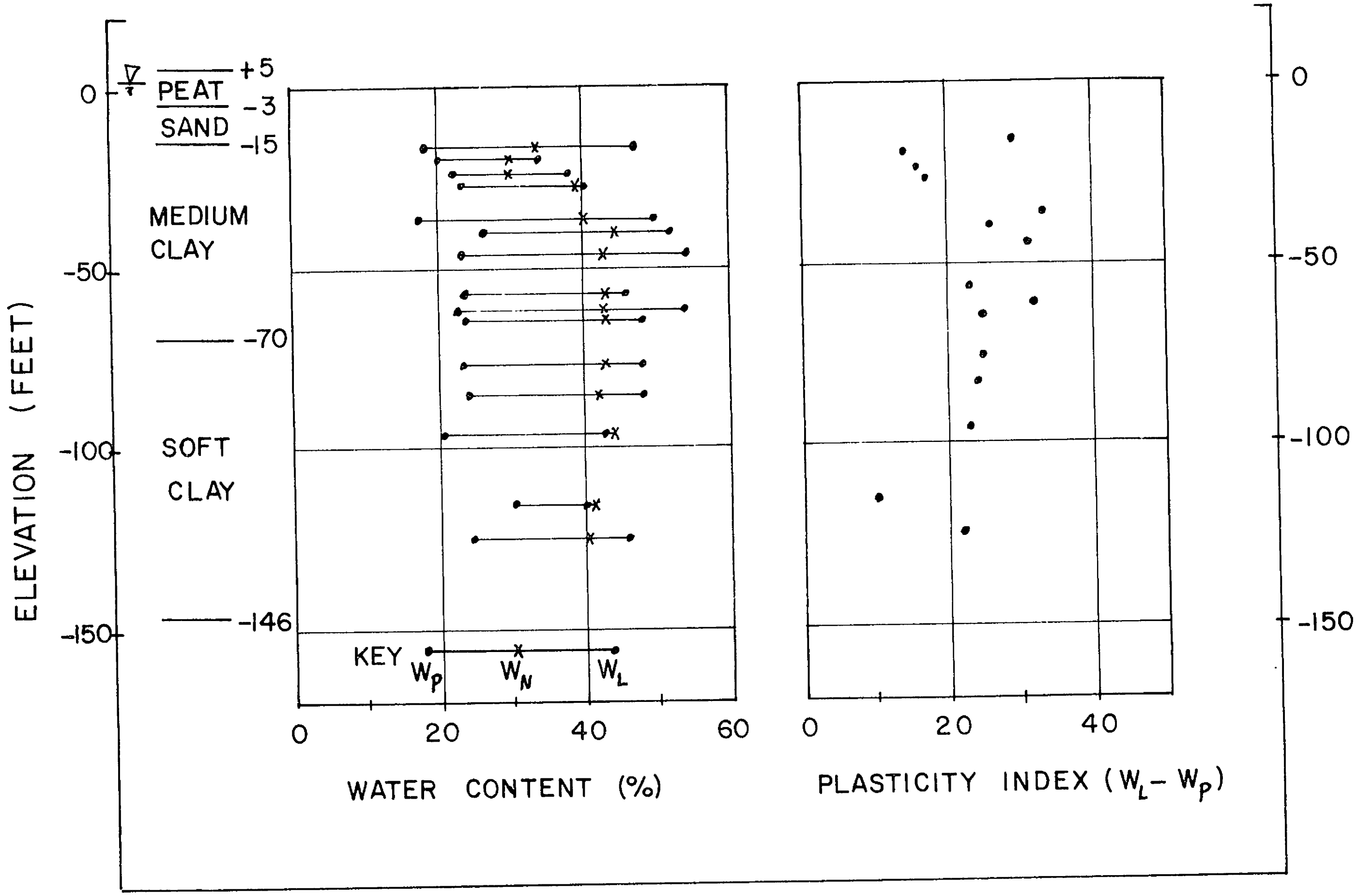
FIGURE III-18



INDEX PROPERTIES AT STA. 246+00

(FROM RECKER, 1973)

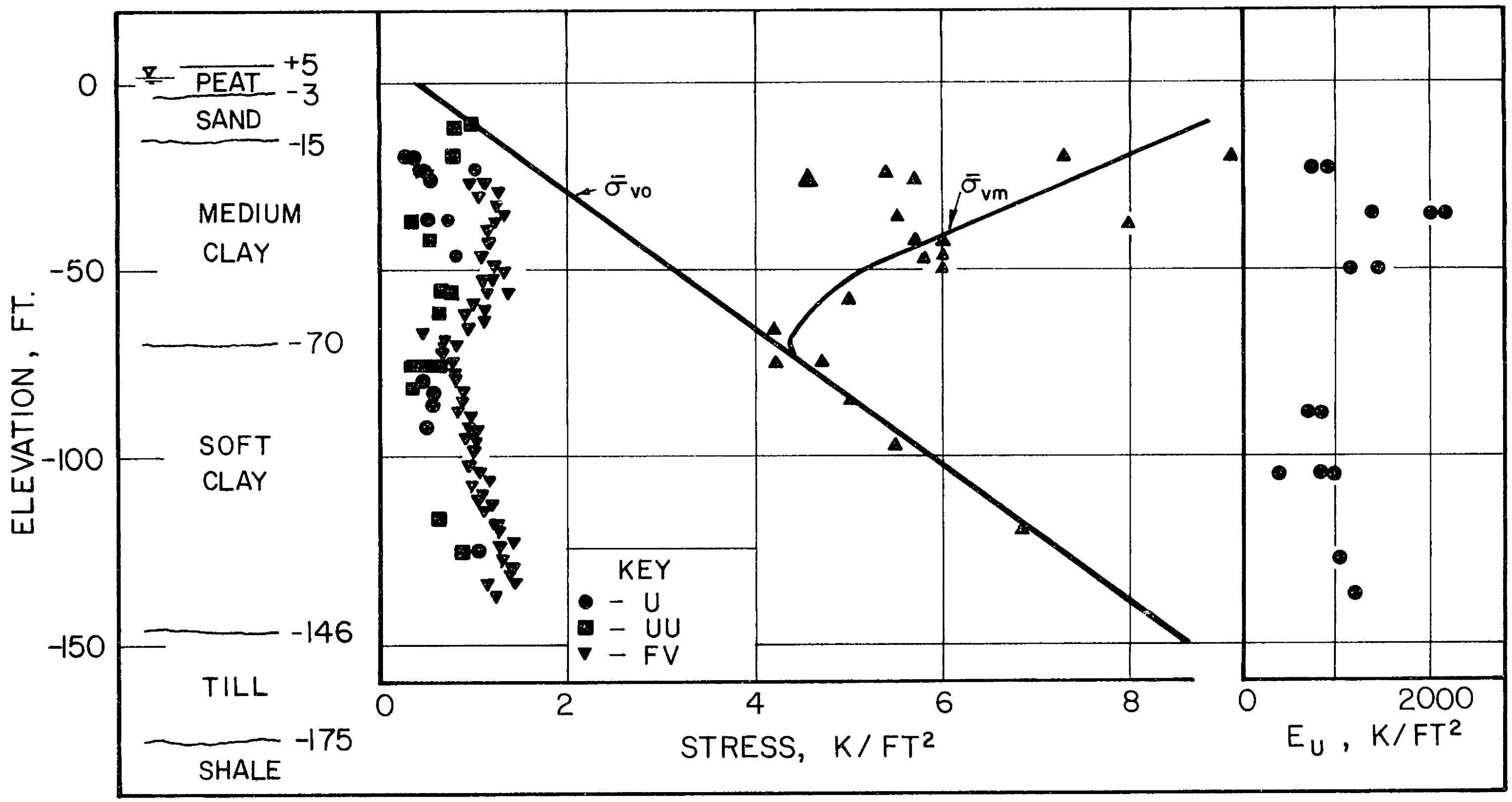
FIGURE III-19



INDEX PROPERTIES AT STA. 246+00

(FROM RECKER, 1973)

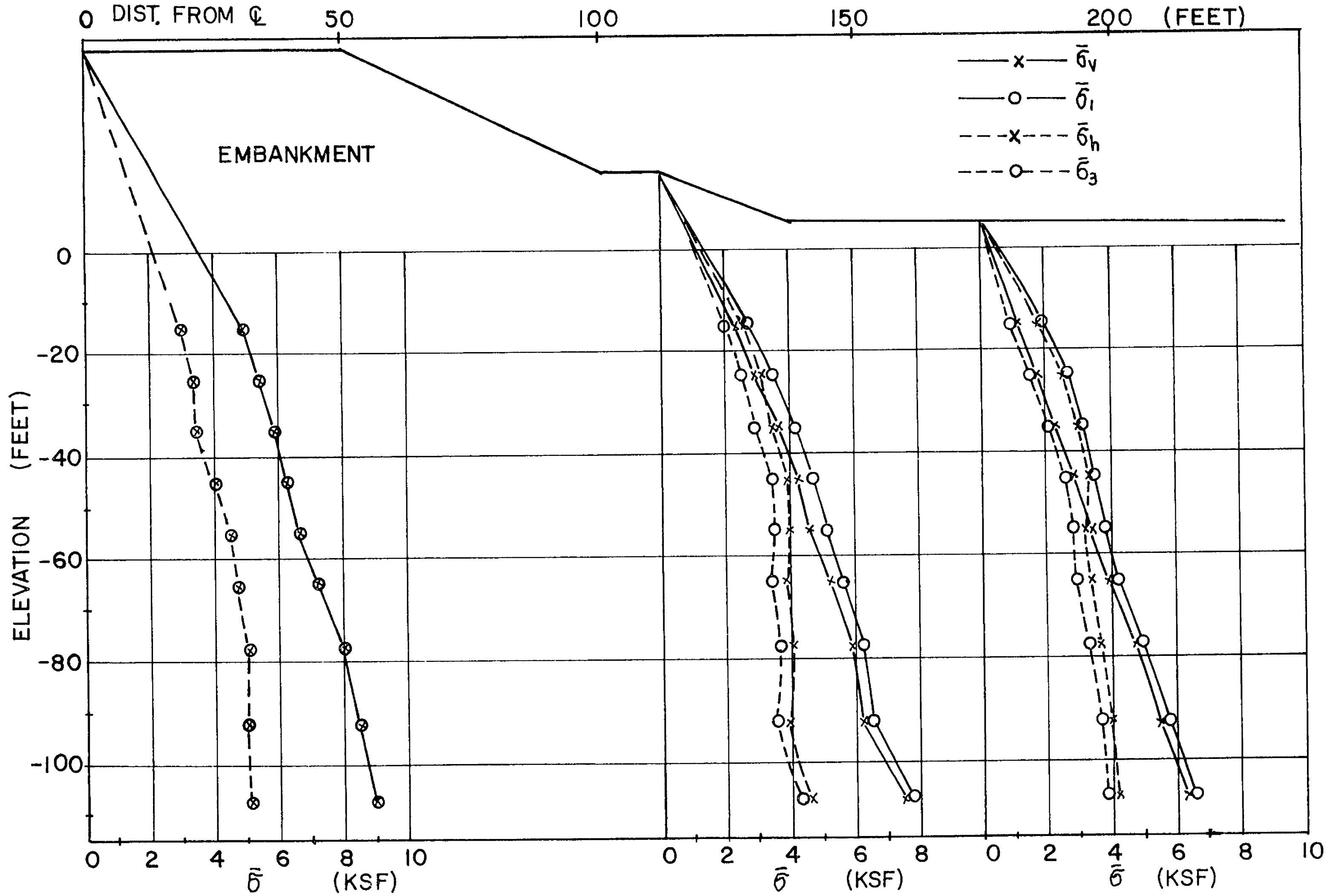
FIGURE III-19



(FROM RECKER, 1973)

SUBSOIL PROPERTIES AT STA. 246+00

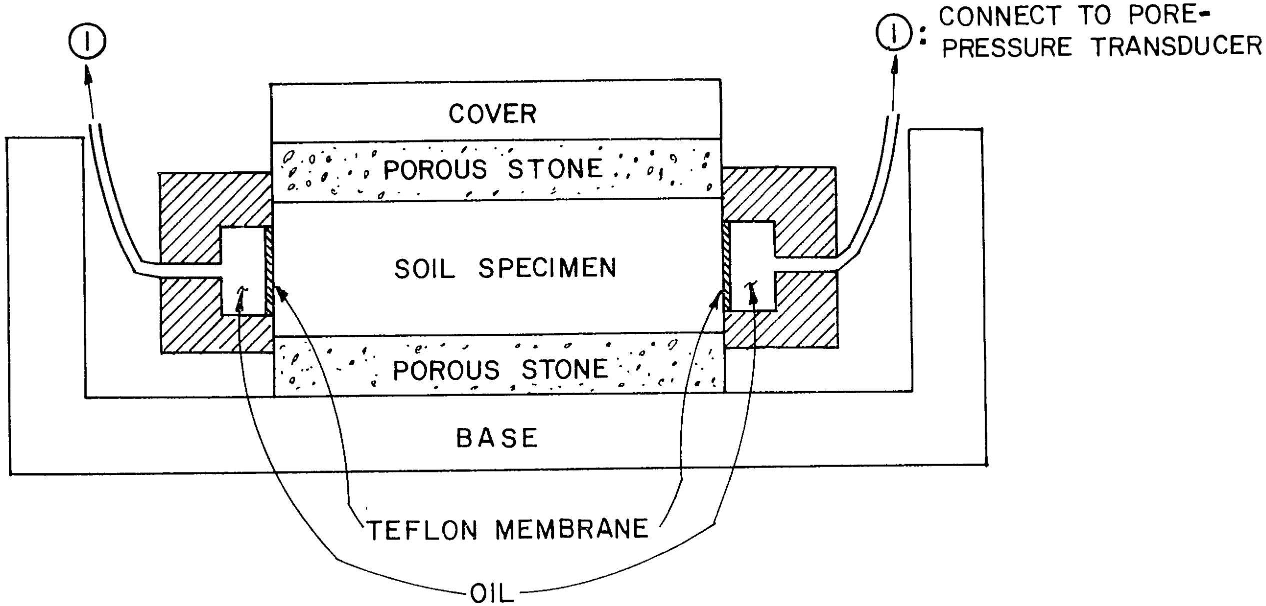
FIGURE III-20



EFFECTIVE FINAL STRESS DISTRIBUTION AT STA. 246+00 (FEECON)

FIGURE III-21

(RUN BY J. WHITTLE)

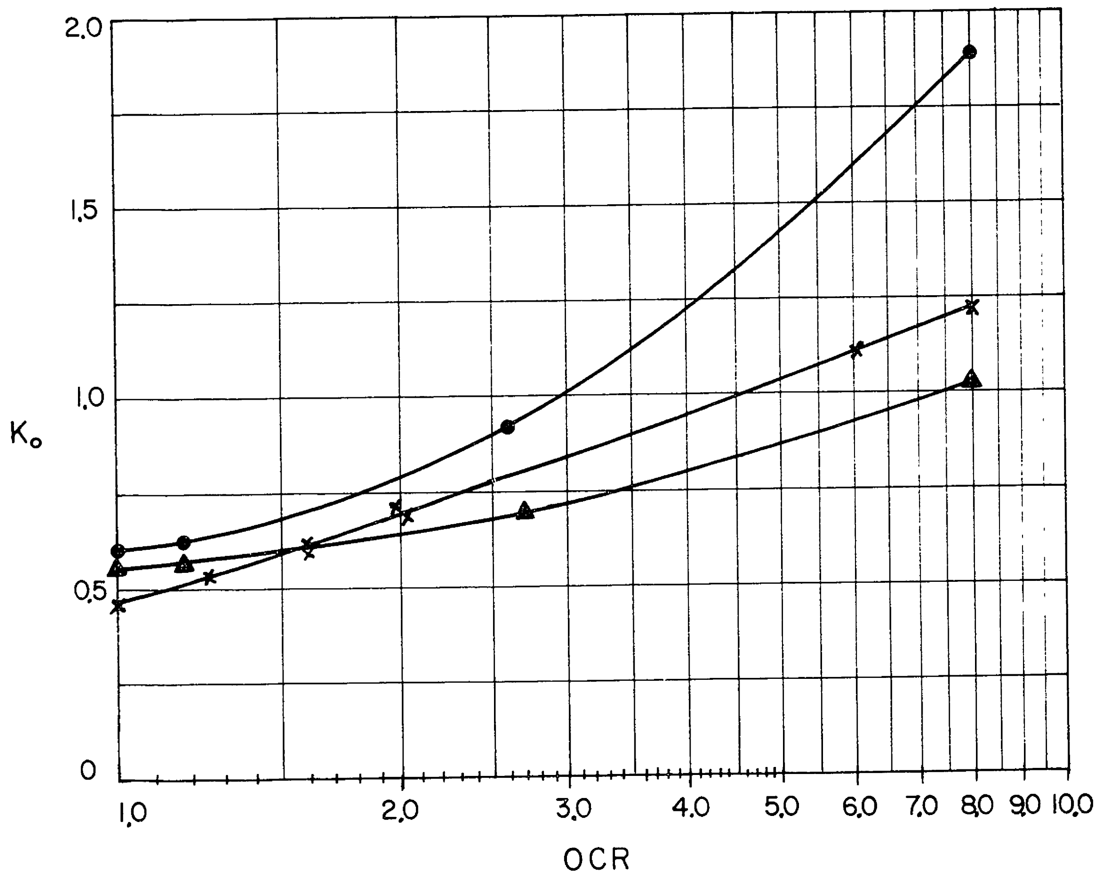


FLOATING RING K_o-OEDOMETER

FIGURE III-22

SAMPLE : BOSTON BLUE CLAY

TEST : LABORATORY K_o-OEDOMETER

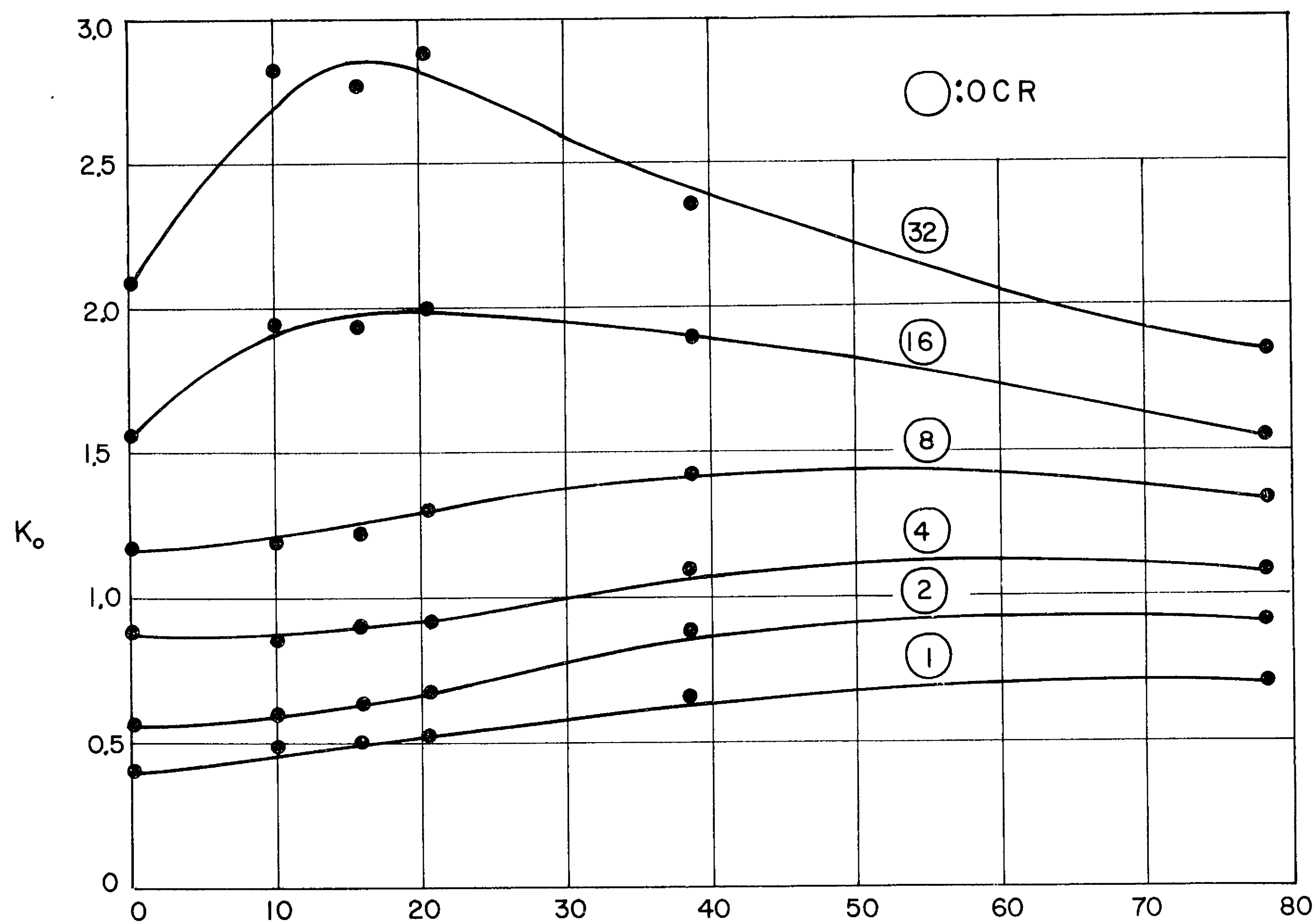


UNDISTURBED SAMPLE TESTED BY F. SILVA

REMOLDED SAMPLE DATA FROM LADD(1965)

K_o VERSUS OCR

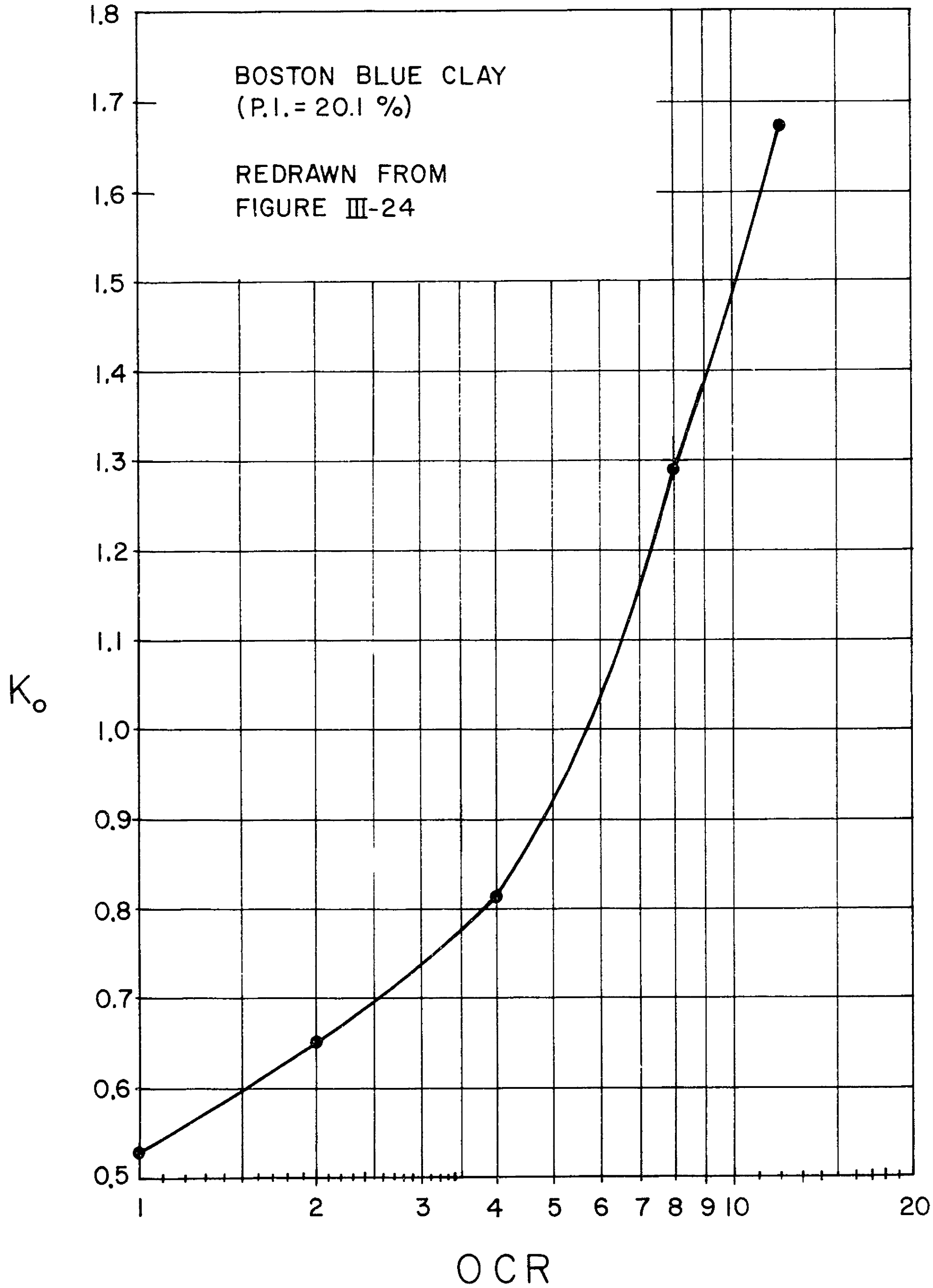
FIGURE III-23



FROM BROOKER & IRELAND (1965)   PLASTICITY INDEX (%)

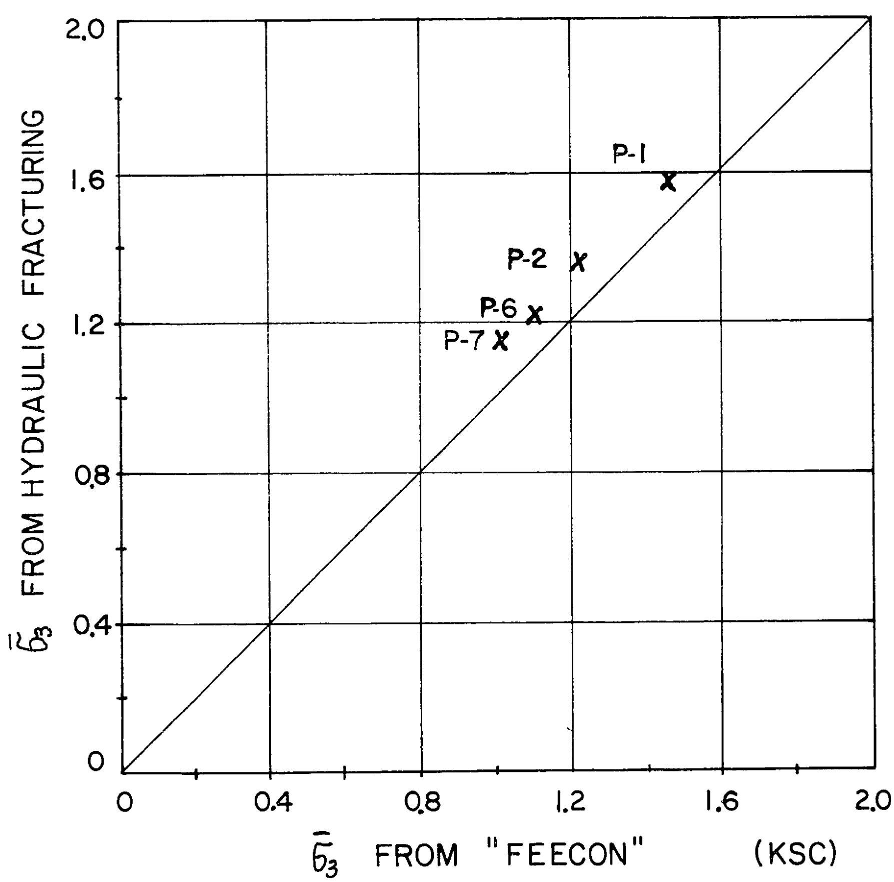
K_o VERSUS P.I.

FIGURE III-24



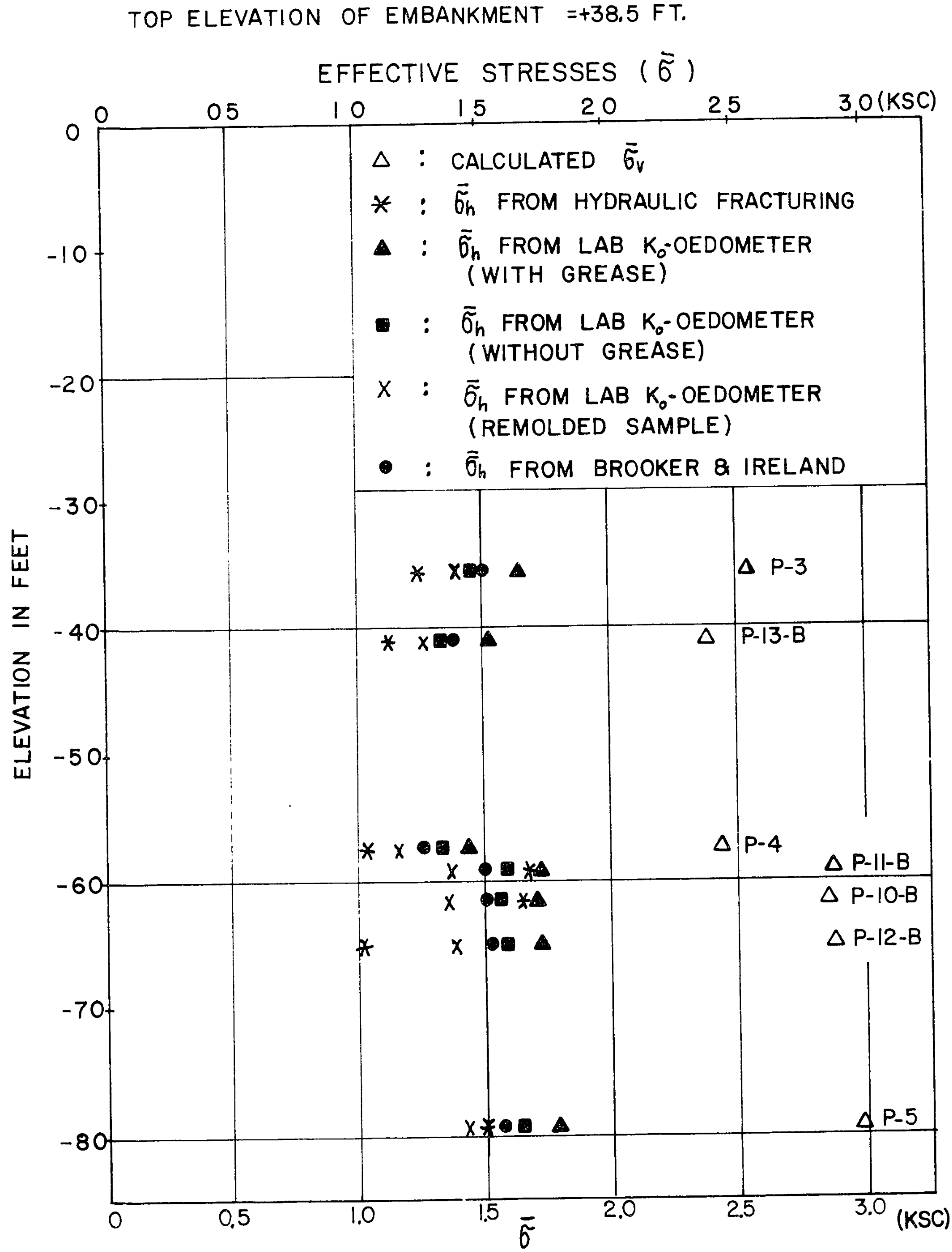
K_o VERSUS OCR FOR B.B.C.

FIGURE III-25



COMPARISON BETWEEN RESULTS FROM HYDRAULIC FRACTURING & "FEECON"

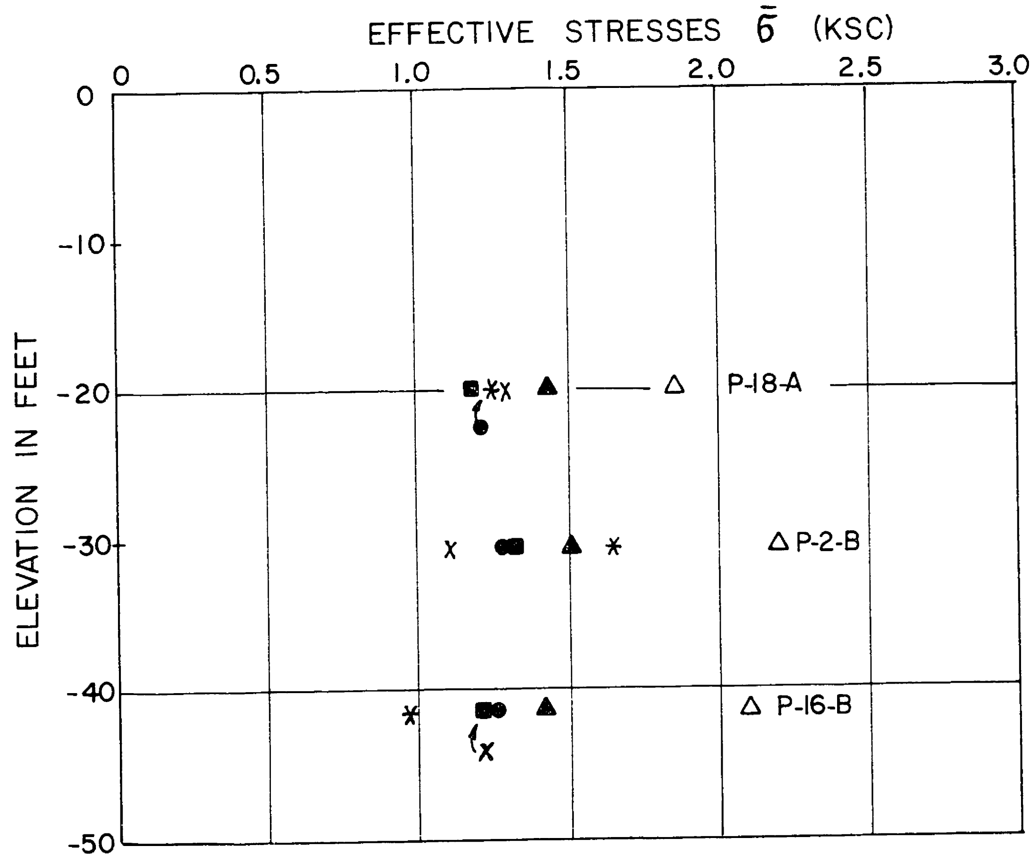
FIGURE III-26



COMPARISON OF $\bar{\sigma}_h$ FROM VARIOUS METHODS (I)

FIGURE III-27

TOP ELEVATION OF EMBANKMENT VARIES



△ : CALCULATED $\bar{\sigma}_v$

✱ : $\bar{\sigma}_h$ FROM HYDRAULIC FRACTURING

▲ : $\bar{\sigma}_h$ FROM LAB K_o-OEDOMETER (WITH GREASE)

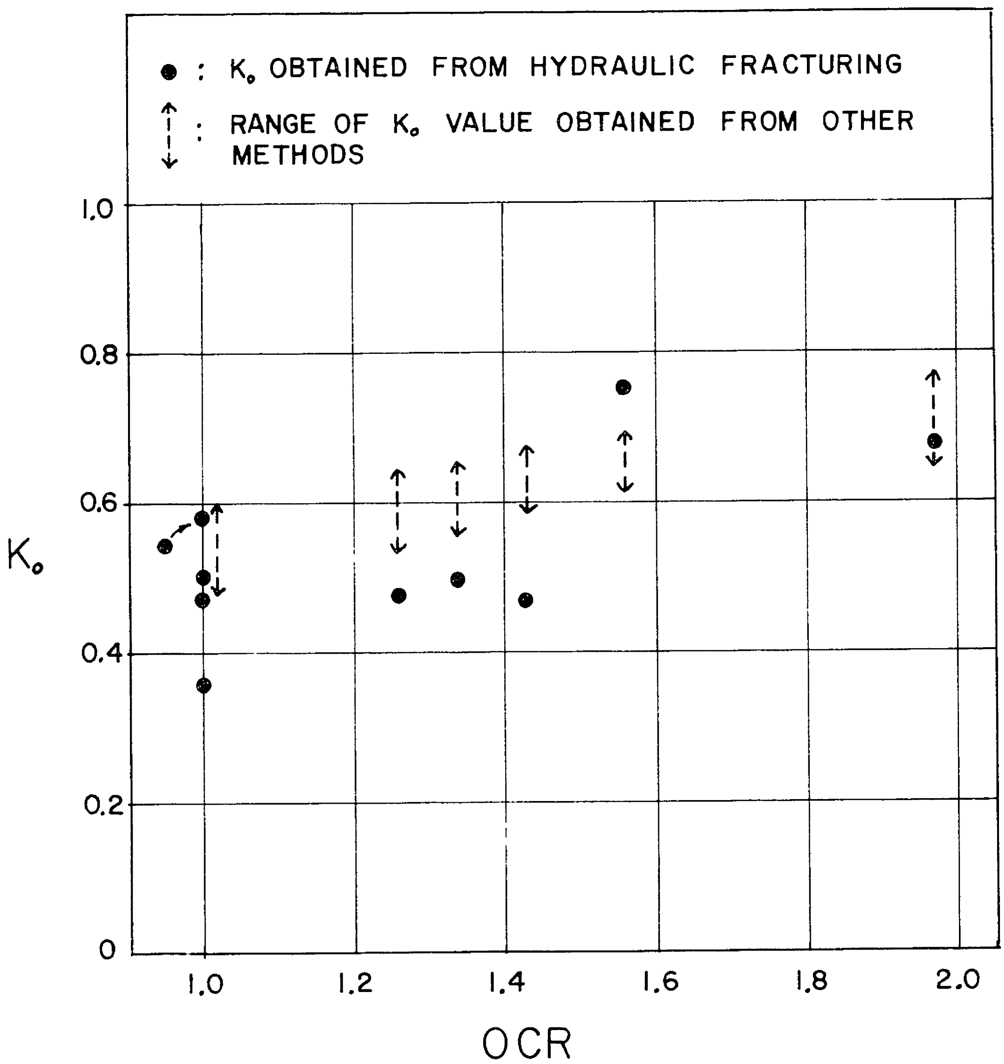
■ : $\bar{\sigma}_h$ FROM LAB K_o-OEDOMETER (WITHOUT GREASE)

× : $\bar{\sigma}_h$ FROM LAB K_o-OEDOMETER (REMOLDED SAMPLE)

● : $\bar{\sigma}_h$ FROM BROOKER & IRELAND

COMPARISON OF $\bar{\sigma}_h$ FROM VARIOUS METHODS (II)

FIGURE III-28



RANGE OF K_o

FIGURE III-29

APPENDIX A

This Appendix describes the hydraulic fracturing test device developed for the study and the recommended test procedures.

DEVICE

The overall dimension of the device, as shown in Figure II-1, was 2'-6" in width and 4'-6" in height. The height was limited to 4'-6" because tests had to be run inside a 5'-0" diameter instrumentation tunnel. The water reservoir was cylindrical, 6" in diameter and 1'-0" in height, and contained 1.2 U.S. gallons of water. Two pieces of 1" diameter and 3'-3" long Lexan tubing was used as mercury manometer, and fittings were made of stainless steel in order to prevent mercury corrosion. Half inch diameter plastic tube was used to connect the system and the connectors used were Swagelok tube fittings. The three valves used were Whitey ball valve Type 45S8. The use of fittings for plastic tubing was minimized in order to reduce head loss across fittings. The device withstood a maximum pressure of 120 psi.

RECOMMENDED PROCEDURES

It takes about two hours to finish a test and requires at least two persons to run it. The quantities of water required to run a test are approximately 1/2 gallon for Geonor piezometer and one gallon for Casagrande piezometer. The rate of water outflow at fracture is approximately 5 cm/sec and at this flow rate one must pay careful attention to obtain accurate readings. In the tests at I-95 clays were fractured at pressures from 20 to 40 psi.

APPARATUS AND SUPPLIES

Special:

1) Hydraulic fracturing device (Figure II-1)
2) Support frame for the device
3) Piezometer reader
4) Flushing tube (if flushing of piezometer is necessary before testing)
5) Fittings to connect the device to piezometers.

General:

1) Two wrenches (8 to 10 in.)
2) Level
3) Supply of clean water
4) Water pump (5 gal. tank)
5) Air pressure supply
6) Stop watch
7) Knife
8) Field record book
9) Flash light (for tests inside the tunnel).

PROCEDURES

1) Record the reading of a piezometer to be tested.
2) Set the device plumb and level on the ground, and measure

the distance between the ground level and the center of the manometer. The center of the manometer is datum.

3) Saturate the device and the piezometer, and connect them to each other. Deairing can be easily done by using the air vent.
4) Open valves (a), (b) and (c).
5) Increase the pressure (1) to 5 psi. In case the piezometer head is far below the ground level (about 15 feet), do not increase the pressure (1).
6) After the mercury manometer is equalized with the head of the reservoir, record the manometer reading.
7) Run variable head test by closing valve (b).
8) Observe the time and manometer reading for about 5 minutes.
9) Open valve (b) slowly after the test.
10) Increase the pressure (1) at 5 psi increment and run variable head test, as steps 6, 7 and 8, at each pressure increment. The rate of water outflow will gradually increase as the pressure (1) increases.
11) When mercury reaches near the top of the manometer under equilibrium with the pressure (1) (15 psi for the device), increase the pressure (2) the same amount as the pressure (1) for the next pressure increment to prevent mercury from being forced out of the manometer.

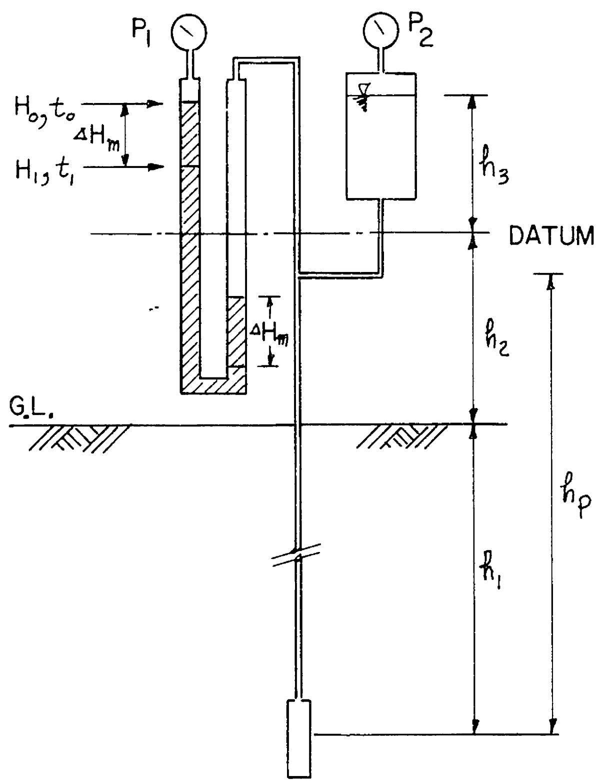
12) Continue the test as steps 6 through 11 until an abrupt increase of water outflow (drop of mercury at a rate about 5 cm/sec) is observed. This sudden increase of water outflow indicates the clay has been fractured. The rapid outflow of water will slow down suddenly as the fracture closes. Run this test long enough so that the rate of water outflow returns back to near the value before the clay was fractured.

13) Run another test at the same fracturing pressure. The purpose of this step is to compare the results with the results from step 11.

APPENDIX B

This Appendix describes a sample computation for C value, the head loss due to flow of water through tubing and fittings, and the effect of volume change of tubing to the test results.

SAMPLE COMPUTATION FOR C VALUE



H_o: Initial excess head (cm of H_2O)

H_1: Excess heat at t_1 (cm of H_2O)

ΔH_m: Drop of mercury in t_1 (cm)

h_p: Initial pore water pressure

p_1, p_2: Pressure in psi.

$$C = \frac{\ln \frac{H_o}{H_1}}{t_1 - t_o} \quad (\sec^{-1})$$

Units:

H:in cm of water

t:in second

$$H_o = (P_1 \times 70.307 + h_1 + h_2 + h_3) - h_p \text{ (cm)}$$

$$H_1 = H_o - 2\gamma_m \Delta H_m + \gamma_w \Delta H_m \text{ (cm)}$$

$$= H_o - 26.2\ \Delta H_m$$

where: γ_m = Unit weight of mercury

γ_w = Unit weight of water

HEAD LOSS

Head loss due to friction is computed assuming that flow of water in manometer is 1 cm/min. This flow rate is typical for conditions at closure of crack. Darcy equation was used to compute the head loss due to flow through tubing and an equation recommended by the manufacturer of the ball valve was used to compute the head loss across the valves.

1) Head Loss Through Tubing

This computation assumed the length of 3/8 in. O.D. tubing is 100 feet and the friction factor (f) is equal to 0.1 based on the fact that flow is laminar.

$$h_L = f \frac{\ell}{d} \frac{V^2}{g} \quad \text{(Darcy equation)}$$

in which

f = friction factor

ℓ = length of tubing

d = I.D. of tubing

V = flow velocity

g = acceleration due to gravity

$$V_{tubing} \cdot = \frac{(0.75 x 2.54)^2}{(0.25 x 2.54)^2} \cdot V_m$$

V_m = flow rate in manometer

I.D. tubing = 0.25 in.

I.D. manometer = 0.75 in.

$$= 9 \text{ cm/min} = 0.15 \text{ cm/sec}$$

$$h_L = 0.1 \times \frac{(100 x 30.48) \cdot (0.15)^2}{(0.25 x 2.54) \cdot (30.48 x 32.2)}$$

$$= 0.01 \text{ cm} = 1 \times 10^{-5} \text{ KSC.}$$

2) Head Loss Across Two Valves

$$\Delta p = \frac{Q^2\ (SG)}{c_v^{\,2}} \quad \text{(recommended by manufacturer)}$$

where Δp = pressure drop in psi

Q = flow in U.S. Gal/min

SG = specific gravity of fluid (water = 1)

c_v = valve coefficient (= 9.8)

$$Q = \frac{(0.75 \times \frac{1}{12} \times \frac{1}{2})^2 \times \pi \times 0.033}{0.134} = 7.55 \times 10^{-4} \text{ Gal/min}$$

$$\Delta p = \frac{(7.55\text{x}10^{-4})^2 \text{ x } 1}{(9.8)^2} = 5.9\text{x}10^{-9} \text{ psi}$$

$$= 4\text{x}10^{-10} \text{ KSC}$$

$$\text{Total Loss} = 4\text{x}10^{-10}\text{x}2 = 8\text{x}10^{-10} \text{ KSC}$$

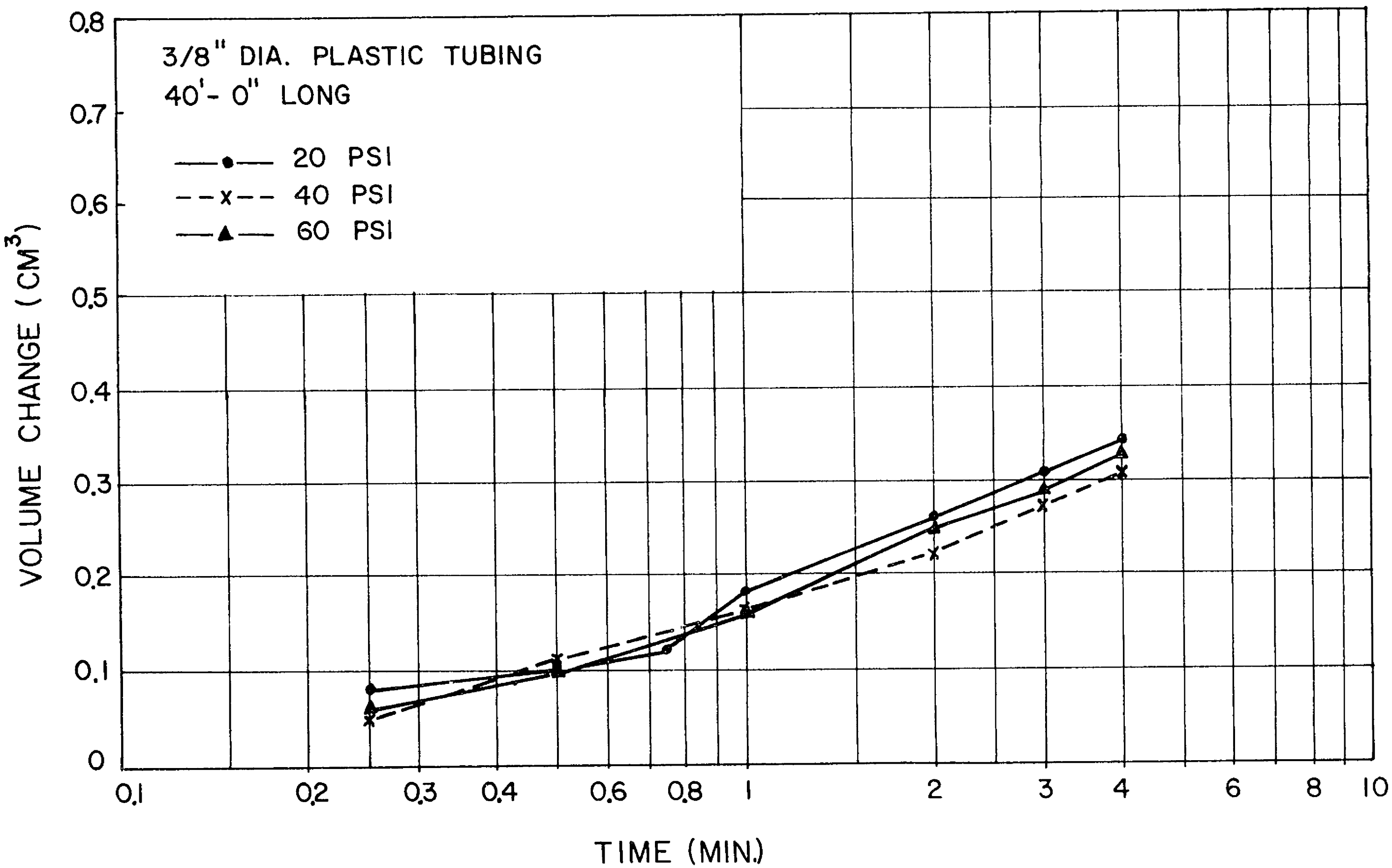
VOLUME CHANGE

Volume change of plastic tubing under high pressures was calibrated as shown in Figure B-1. The effect of volume change to the mercury manometer reading was computed assuming that the maximum volume change at the "close up" (u_c) pressure was 0.8 cubic centimeter.

$$\Delta H_m = \frac{0.8}{(0.75x2.54x0.5)^2 \times \pi} = 0.28 \text{ cm of } H_g$$

$$= 3.8x10^{-3} \text{ KSC}$$

in which ΔH_m is the change of mercury manometer due to the volume change of tubing.



VOLUME CHANGE CALIBRATION OF 3/8" DIA. TUBING

FIGURE B-1