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COMPARISON of DYNAMIC and UNCONFINED COMPRESSION STRENGTH for MACHINE FOOTING DESIGN

by

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INTRODUCTION and PURPOSE

In the design of machine foundations, the important factors to consider are the allowable deflections of the machines, and the natural frequency of the soil. Although there is argument as to the applicability of the present theories used to design machine foundations, the soil properties used in these theories are difficult to obtain accurately.

Since the only means of determining these soil properties necessary for the design of machine footings is with elaborate testing equipment, many soil investigation firms are excluded from machine foundation engineering. This study is designed to explore the possibility of approaching the design of machine footings by using a simplified model of field conditions and comparing these results with that obtained by a simple laboratory test. With this comparison, an allowable bearing capacity will be obtained directly.

LITERATURE REVIEW

Development of a rational approach to the design of machine foundations began with Eric Reissner and his colleagues in Germany in the 1930's. The first English translation on the subject did not appear until 1944, forming the basis of the so called elastic half-space solution (11).

The formulation of the problem was such that the stresses and displacements were sought for a rigid, massless, circular footing resting on a semi-infinite, homogeneous, isotropic, perfectly elastic soil mass. By use of Fourier-Bessel integral methods, a useful expression for the torque, T, was obtained as

$$T = \frac{16}{3}\mu r_o^3 \phi$$

where = modulus of rigidity of the soil

r_o = radius of footing

= angle of rotation of footing (expressed in radians)

The next exhaustive work on the subject did not appear until 1955 at which time Arnold, Bycroft, and Wharburton (1) considered a "rigid circular body on a homogeneous elastic medium of infinite surface area and constant depth which could be finite or infinite (1)."

The paper considered four modes of vibration: vertical translation, torsion, horizontal translation, and rocking. The investigation revealed that the amplitude of response to the vibrations were functions of f_1 and f_2 which in themselves were comples functions of the shearing modulus and poisson's ratio of the soil.

The equations for the vertical translation of a semi-infinite medium were as follows: for the displacement of the plate in the vertical direction, w,

$$w = \frac{P_z}{\mu r_o} (f_1 \cos pt - f_2 \sin pt)$$

and the amplitude of vibration, A,

$$A = \frac{P_z}{\mu r_o} (f_1^2 + f_2^2)^{\frac{1}{2}}$$

where P_z = amplitude of force in vertical direction

μ = modulus of rigidity

r_o = radius of footing

p = circular frequency of force applied to footing

t = time

In 1959, Bycroft (2) advanced a paper which showed that the elastic half-space theory could be applied with reasonable accuracy to out of balance machines.

In 1962, Hsieh (6) popularized the lumped mass system which proposed that a vibrating footing could be considered a mass-spring-dashpot system. The mass of the system was composed of the mass of the machine, the mass of the foundation, and the mass of a certain portion of the soil which is moving with the foundation. No suggestions were given as to how to treat the dashpot. The spring had as its constant the modulus of subgrade reaction.

The paper advanced mathematical expressions for calculating the amplitude of vibration of foundations using the lumped mass system and also expressions to determine the weight of the foundation necessary to reduce the amount of oscillations. The allowance made

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for a dynamic load was also proposed to be designated as a percentage of the static load.

The lumped mass system proved to be exact as long as the ground was considered as semi-infinite, isotropic, linearly elastic, and possessed no internal damping.

The general equation describing the amplitude of vertical vibration of a foundation was

$$A = \frac{Z}{\left[(K_{vs} - \omega^2 M)^2 + (\omega R_v)^2\right]^{.5}}$$

where

A = amplitude

Z = force

ω = frequency

$R_v = \dfrac{\text{depth of stratum}}{\text{radius of circular footing}}$

M = mass of system

K_{vs} = spring constant

$= \dfrac{4\, G\, r_o}{(1 - \nu)}$

with

G = shear modulus

r_o = radius of footing

ν = Poisson's ratio

The expression for the maximum amplitude of oscillation, A_o, was

$$A_o = \frac{.41\ b}{(b + 2)^{\frac{1}{2}}}\ \frac{\varepsilon}{M}$$

where ε is a constant defined by

$$Z = \varepsilon\omega^2$$

and b is the so called mass ratio defined by

$$b = \frac{M}{\rho r_o^3}$$

with ρ as the mass density of the soil.

The total dead weight necessary to prevent uplift with applied amplitude of vibration A_o is W. Noting that W = Mg then Hsieh gives the following expressions:

for $\nu = \frac{1}{2}$ $$W = \frac{0.41 \ b \ g \ \varepsilon}{(b + 2)^{\frac{1}{2}} \ A_o}$$

for $\nu = \frac{1}{4}$ $$W = \frac{0.52 \ b \ g}{0.42 + (b + 1)^{\frac{1}{2}} \ A_o} \varepsilon$$

for $\nu = 0$ $$W = \frac{0.61 \ b \ g}{0.24 + (b + .5)^{\frac{1}{2}} \ A_o} \varepsilon$$

g being the gravitational constant.

Richart (12) in 1962 prepared a paper which actually presented a method of designing machine foundations; not by designing a static foundation and then checking it for dynamic loads.

In this paper, by way of introduction, he defined three waves which are induced in soil by impact loads. There are horizontal waves of volume change, designated P-waves or compression waves. The wave of distortion of constant volume is the shear or S-wave. And a surface wave exists called the Rayleigh or R-wave.

Richart also compiled the work of other researchers to form a graph of allowable amplitude of vibrations versus frequency. Richart advocated that in the absence of specific instructions from the machine manufacturer, this chart could be used to find the limit of the vibration amplitude for a particular frequency.

This paper also formally defined resonance as encountered in soil as "the condition of vibration corresponding to a large increase in amplitude."

However, the main point of this paper was the development of curves to determine the resonant frequency of the soil and the maximum amplitude of oscillation for an idealized equivalent of the actual machine foundation. To use these curves the designer needs to know the unbalanced forces and the operating frequencies of the machine as well as approximate values of the shear modulus, poisson's ratio, and the density of the soil. The determination of the shear modulus and poisson's ratio is difficult at best, but knowing these parameters, the designer could choose the soil contact area and the necessary static weight of the footing to control resonant frequency and amplitude of oscillation.

In the following year, Hall and Richart (5) developed a machine and a complex testing procedure to determine the shear modulus of a soil. This testing equipment is not widely used and therefore limits many firm's participation in the design of machine foundations.

Lysmer and Richart (8) in 1966 developed several expressions from the elastic half-space model. For steady-state motion:

$$\delta = \frac{Q_o}{K} M \cos (\omega t + \phi)$$

where δ = deflection

Q_o = vertical load

K = spring constant

ω = frequency

t = time

and

$$M = \left(\frac{F_1^2 + F_2^2}{(1 - \frac{m\omega^2}{K} F_1)^2 + (\frac{m\omega^2}{K} F_2)^2} \right)^{\frac{1}{2}}$$

with F_1 and F_2 = functions of f_1 and f_2

m = mass of system

and $$\phi = \tan^{-1} \frac{F_2}{F_1 - \frac{m^2}{K}(F_1^2 + F_2^2)}$$

For the case of uniform periodic loading:

$$\zeta = - \frac{i\ s\ p_o\ e^{i\omega t}}{\omega(\rho G)^{\frac{1}{2}}}$$

with p_o = vertical load

and $$s = \left(\frac{1 - 2\nu}{2(1 - \nu)}\right)^{\frac{1}{2}}$$

The authors then introduced a simplified analog solution which was easier to apply but gave only limited agreement between theory and practice. The equations are as follows:

$$\zeta = - \frac{Q_o}{m\omega^2} e^{i\omega t}$$

For resonant frequency (f_r):

$$f_r = \frac{V_s}{r_o} \frac{(B - .36)^{\frac{1}{2}}}{B}$$

where V_s = shear wave velocity

$$B = \frac{1 - \nu}{4}$$

and the resonant amplitude (A_r):

$$A_r = \frac{Q_o}{K} \frac{B}{.85(B - .18)^{\frac{1}{2}}}$$

Funston and Hall (3) investigated the case of damping as applicable to the elastic half-space and the simplified lumped system methods of analysis. Damping is considered to be the loss of wave motion as the wave moves away from its point of origin. There are two types of damping; internal and geometric. Geometric damping is just the loss of wave amplitude due to distance from point of origin. Internal damping is loss of wave amplitude due to energy losses as soil particles are required to slide past each other. It was found that the elastic half-space model could not account for geometric damping but the lumped system could due to its approximations.

In 1967, shortly after the Funston and Hall (3) investigation, Richart and Whitman (13) compared actual footing vibration tests to the elastic half-space theory. They found that there was a very rough correlation between the theory and actual field results for accelerations less than 0.5g and oscillations less than 0.1 mil.

Whitman and Richart (16), also in 1967, published a comprehensive paper on the design of dynamically loaded foundations. One of the first paragraphs of the paper analyzed the state-of-the-art elegantly:

"The design of foundations subjected to dynamic loads is a trial and error procedure. Initial dimensions are selected considering such factors as the dimensions of the equipment or structure to be supported, the space available for the foundation, and the normal static bearing stress. The trial design must be analyzed to determine its response to the design dynamic loading, and then be adjusted and reanalyzed if necessary."

Also included in the paper is a formal explanation of the effect of dynamic loading on soils as " . . . sustained vibratory loads or repeated impacts can cause the internal structure of soil to change, causing settlements or loss in strength."

The design criteria for machine foundations listed typical operating frequencies of 200 - 12000 cpm, (3.33 - 200 Hz), with maximum velocities of 1 inch/second and maximum accelerations of 0.5g.

The lumped system was the design technique used exclusively in this paper. The two types of damping were listed as "the loss of energy through propagation of waves away from the vicinity of the footing," i.e. geometric damping and "the internal energy loss within soil due to hysteresis and viscous effects," i.e. internal damping.

To deal with geometric damping in footing design, the mass ratio was defined as

$$b = \frac{m_o}{\rho r_o^3}$$

for translation, with m_o = mass of foundation and machinery

ρ = density

r_o = radius of footing

and for rotation

$$b' = \frac{I_o}{\rho r_o^5}$$

with I_o being the dynamic moment of inertia of foundation and machinery.

Equations were also given to convert a rectangular footing into an equivalent circular footing as follows:

for translation $$r_o = \left(\frac{B\,L}{\pi}\right)^{\frac{1}{2}}$$

for rocking $$r_o = \left(\frac{B\,L^3}{3\pi}\right)^{\frac{1}{4}}$$

for twisting $$r_o = \left(\frac{B\,L\,(B^2 + L^2)}{6\pi}\right)^{\frac{1}{4}}$$

with B = footing width

L = footing length.

Charts were then constructed which gave geometric damping values as functions of the mass ratio.

An equation for internal damping was given as

$$\text{damping} = 4.5\,\gamma_{xz}^{0.2}\,\sigma_o^{-0.5}$$

with γ_{xz} = shearing strain

σ_o = confining pressure

In the absence of reliable internal damping data, the total damping was to be taken as:

the geometric damping + .05

This paper departed from accepted practice in that it argued for the exclusion of the mass of soil in motion under the footing as part of the mass ratio.

The paper also listed four ways of obtaining spring constants for the lumped mass system. The first was with laboratory testing, again unreliable due to ignorance of the shear modulus and Poisson's ratio. The second was by plate bearing test, next was vibration tests, and finally by determining the elastic subgrade modulus and using charts.

Equations were also given for the spring constants depending on the type of motion and footing configuration, as summarized below:

<u>circular footings</u>

motion	formula
vertical	$K_z = \frac{4\ G\ r_o}{1 - \nu}$
horizontal	$K_x = \frac{32\ (1 - \nu)\ G\ r_o}{7 - 8\nu}$
rocking	$K = \frac{8\ G\ r_o^3}{3\ (1 - \nu)}$
torsion	$K = \frac{16}{3}\ G\ r_o^3$

rectangular footings

motion	formula
vertical	$K_z = \frac{G}{1-\nu}\beta_z (B\ L)^{\frac{1}{2}}$
horizontal	$K_x = 2\ (1+\nu)\ G\ \beta_x\ (B\ L)^{\frac{1}{2}}$
rocking	$K = \frac{G}{1-\nu}\beta_\phi\ B\ L^2$

where β = a constant depending on motion type

In 1968, Karasudhi, Keer, and Lee (7) advanced a paper using Fredholm integral equations designed to prove mathematically that the motion of machine foundations was a complex function of the different properties of the soil.

McNeill (9) presented the most comprehensive work to date in 1969 at an international soil mechanics conference. The paper stated specifically that the design of machine foundations had one goal - to limit the motions of the machine. He also stated that the problems of design fell into two categories; "the response of the machine's own foundation;" and "the isolation design of another foundation which feels the first machine's motions."

He further stated:

"Many machine foundations today are designed by nonrational rules-of-thumb which are furnished by some machine manufacturers, found in many mechanical design papers and handbooks, or handed from father to son."

There are two types of load: the known load, such as from impact hammers or pistons; and the unknown load due to a machine imbalance.

The basic analytical tools necessary to analyze harmonic motion are: the natural frequency (f_n) defined by

$$f_n = \frac{1}{2\pi}\left(\frac{\text{stiffness}}{\text{inertia}}\right)^{\frac{1}{2}}$$

and the damped natural frequency f_d;

$$f_d = f_n \; (1 - D^2)^{\frac{1}{2}}$$

where D is the damping factor defined by

$$D = \frac{c}{2((\text{stiffness})(\text{inertia}))^{\frac{1}{2}}}$$

with c given by figures included in the paper. The operating frequency of the machine was designated by f_o, and the frequency of maximum response f_{fd} was

$$f_{fd} = \frac{f_n}{(1 - 2D^2)^{\frac{1}{2}}}$$

The maximum amplitude, Δ_{max}, was then

$$\Delta_{max} = \frac{R_{fd}}{2D \; (1 - D^2)^{\frac{1}{2}}}$$

where R_{fd} is called the machine ratio which is a function of each case of motion. The frequency of maximum response for a constant force, f_{cf}, is

$$f_{cf} = f_n \; (1 - 2D^2)^{\frac{1}{2}}$$

and the associated maximum response is

$$\Delta_{max} = \frac{R_{cf}}{2D \; (1 - D^2)^{\frac{1}{2}}}$$

with R_{cf} being the machine ratio for a constant force.

For nonharmonic analysis, special figures were given, while Richart's (5) tolerable motion chart was given as a good design guideline.

The velocities for the different types of earth waves were given as:

P-wave $$c_p = \left(\frac{E}{\rho} \frac{1 - \nu}{(1 - 2\nu)(1 + \nu)} \right)^{\frac{1}{2}}$$

S-wave $$c_s = \left(\frac{G}{\rho} \right)^{\frac{1}{2}}$$

R-wave $$c_r = c_s \, f(\nu)$$

and finally the laboratory "soil bar" wave c_b,

$$c_b = ((E) \, (\rho)^{-1})^{\frac{1}{2}}$$

where $f(\nu)$ simply stands for a function of Poisson's ratio.

The properties required for design of dynamic foundations are shear modulus and Poisson's ratio. The determination of these properties can be done by two laboratory techniques: the wave propagation, which is too complex and uncertain; and vibration techniques. The determination of the natural frequency f_n can be accomplished in three ways.

CASE I. Both ends of sample free or fixed.

$$f_n = \frac{n}{2\,L} c_w$$

CASE II. One end fixed, the other end free.

$$f_n = \frac{2n - 1}{4\,L} c_w$$

CASE III. One end fixed, while the other end has a weight of W_m with inertia of I_m attached.

$$\left(\frac{2\pi L}{c_w} f_n \right) \left(\tan \frac{2\pi L}{c_w} f_n \right) = \frac{W_b}{W_m} \text{ or } \frac{I_b}{I_m}$$

where
- n = integer dependent upon the mode of vibration
- L = length of sample
- c_w = velocity of generated wave
- W_b = weight of bar of soil
- I_b = inertia of bar of soil

Field tests, such as wave propagation and model measurements, can also be used to determine a soil's dynamic properties.

Finally, the paper advocated taking internal damping equal to 5% of geometric damping in the absence of other data.

The method of design advocated in the paper was the elastic half-space model with its assumptions of a circular foundation at the ground's surface which is stiff enough to track the soil's motions sitting on elastic, homogeneous, isotropic soil.

The design analysis of the footings was to be done using the charts included in the paper. The paper stressed that for layered soil, or embedded footings, the elastic half-space solution was unapplicable. The theory could not handle coupled motions, and deep pile foundations were a last resort, but when used, the piles should be bantered.

The paper also briefly mentioned isolation design as falling into two categories: active, where a massive footing is used; and passive, where a barrier is actually created.

Finally, after a machine is in operation and the footing fails due to vibration, there are three corrective measures: to alter the foundation configuration, to alter the subgrade properties, and to alter the machine loads.

Novak (10) in 1970 promptly called the applicability of the elastic half-space theory in doubt when he compared the theory with experiments and showed that the theory led to a large underestimation of the resonant amplitudes.

Richart, Woods, and Hall (14) promptly submitted a paper tracing the history of the elastic half-space theory, defended its applicability, and extended Lysmer and Richart's (8) work to include varying force machines as such: for the resonant frequency of a varying force (f_{mr}),

$$f_{mr} = \frac{V_s}{2\pi\, r_o}\left(\frac{.9}{B_z - .45}\right)^{\frac{1}{2}}$$

where

$$B_z = \frac{1 - \nu}{4}\;\frac{m}{\rho\, r_o^3}$$

with m = the mass of the footing and machine

The varying force Q_o is defined as

$$Q_o = m_e\, e\, \omega^2$$

where m_e = eccentric mass

e = eccentricity

The rotating force amplitude A_{zm}, was

$$A_{zm} = \frac{m_e\, e}{m} \qquad \frac{B_z}{.85\,(B_z - .18)^{\frac{1}{2}}}$$

The constant force resonant frequency (f_m) was rewritten slightly to become

$$f_m = \frac{1}{2\pi}\;\frac{V_s}{r_o}\left(\frac{B_z - .36}{B_z}\right)^{\frac{1}{2}}$$

while the constant force amplitude (A_m) was

$$A_m = \frac{B_z}{.85\,(B_z - .18)^{\frac{1}{2}}}\left(\frac{Q_o\,(1 - \nu)}{4\,G\,r_o}\right)$$

In 1971, Weismann (15) specifically modified the half-space equations to deal with torsion.

Finally, in 1979, Gazetas (4) qualified the elastic half-space solution with the following:

"The design of machine foundations is a trial-and-error procedure involving three interrelated steps: (1) Establishment of desired foundation performance ("failure") criteria; (2) determination of magnitude and characteristics of the dynamic loading; and (3) estimation of the anticipated translational and rotational motions of the machine-foundation-soil system."

"It is concluded that if the foundation has a high mass ratio and does not operate at very low frequencies, small errors in modeling the soil are unimportant and one can safely base the design on available half-space solutions."

Gazetas then gave an equation for finding the amplitude of motion $\tilde{\zeta}_o$ of a massive machine which was infinitely long as

$$\tilde{\zeta}_o = \frac{\zeta_o G}{P_o} \left[\frac{f_1^2 + f_2^2}{(1 - b\, a_o\, f_1)^2 + (b\, a_o\, f_2)^2} \right]^{\frac{1}{2}}$$

with ζ_o = amplitude of machine motion

P_o = force amplitude

$a_o = \frac{B}{V_s}$

while a_o is the frequency factor. To conclude his paper, Gazetas then summarized by stating that the elastic half-space solution should be used when dealing with rotating machinery.

DESIGN of EXPERIMENT

The experimental program was designed to equate the unconfined compression test with a dynamic load test to determine how the unconfined compression test results could be used for machine foundation design. This would be of great value to consulting engineers since the unconfined compressive strength is commonly run by all commercial testing labs at low costs while vibratory tests are difficult to obtain and very expensive.

The testing program was limited at the outset to twenty-one samples of silty clay. These samples, which normally cost fourteen dollars apiece, were obtained by Midcontinent Engineering and Testing (MET) at a site in Kansas City, Missouri, at no cost to the University. The silty clay samples obtained are typical of the altered loess which covers very large areas of the Upper Midwest. Silty clay was a good material to work with since sand requires a confining pressure such as in a triaxial shear machine, and pure clay is very sensitive to changes in frequency.

The design of the experiment consisted of testing a portion of each sample by unconfined compression procedures (ASTM - D-2166-66, 72) with a machine similar to that shown in Figure 1. A remaining portion was tested by dynamic loading using an MTS Model 483.01, System 90332, see Figure 2, which has no standard ASTM test method. All samples used came from Shelby tubes, having a diameter of $2^7/_8$ inches, (7.30 cm).

For the MTS machine testing, Richart's tolerable motion chart (12) was used, see Figure 3, to select a frequency range and amount of displacement. The most delicate recording range on the machine was used since even then, for most of the frequencies, the maximum displacement was in fractions of squares on the graphing paper.

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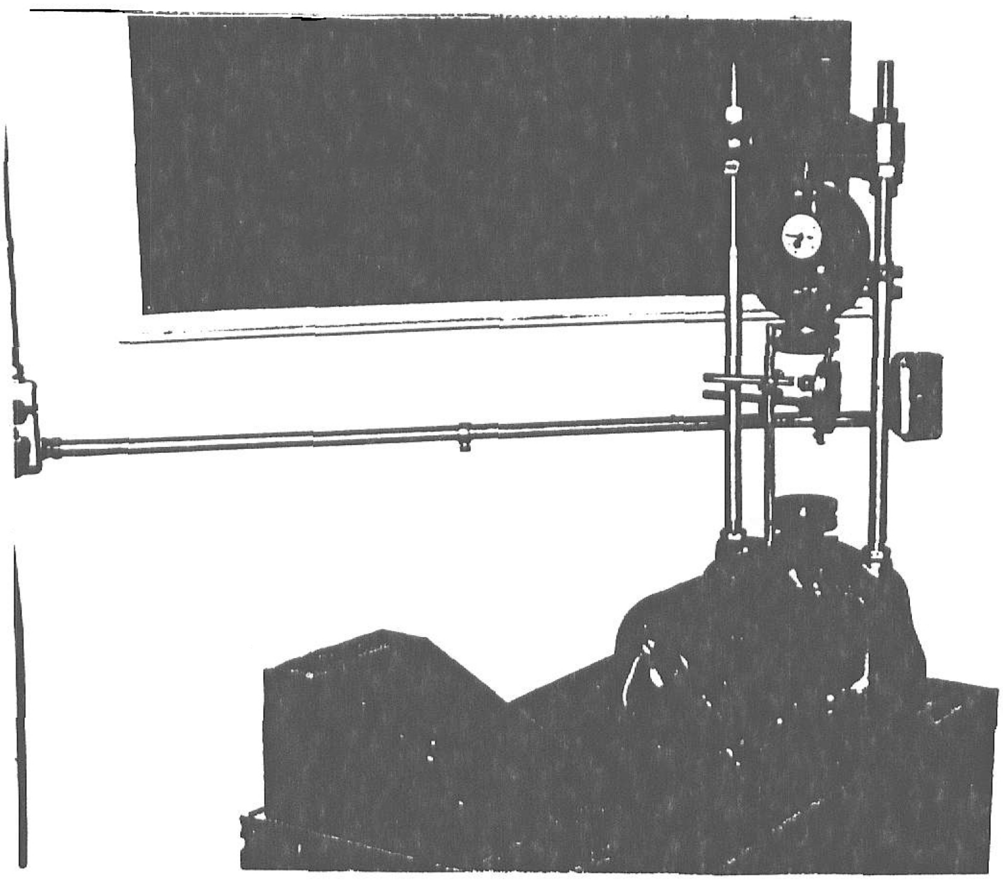


Figure 1. Unconfined Compression Testing Machine

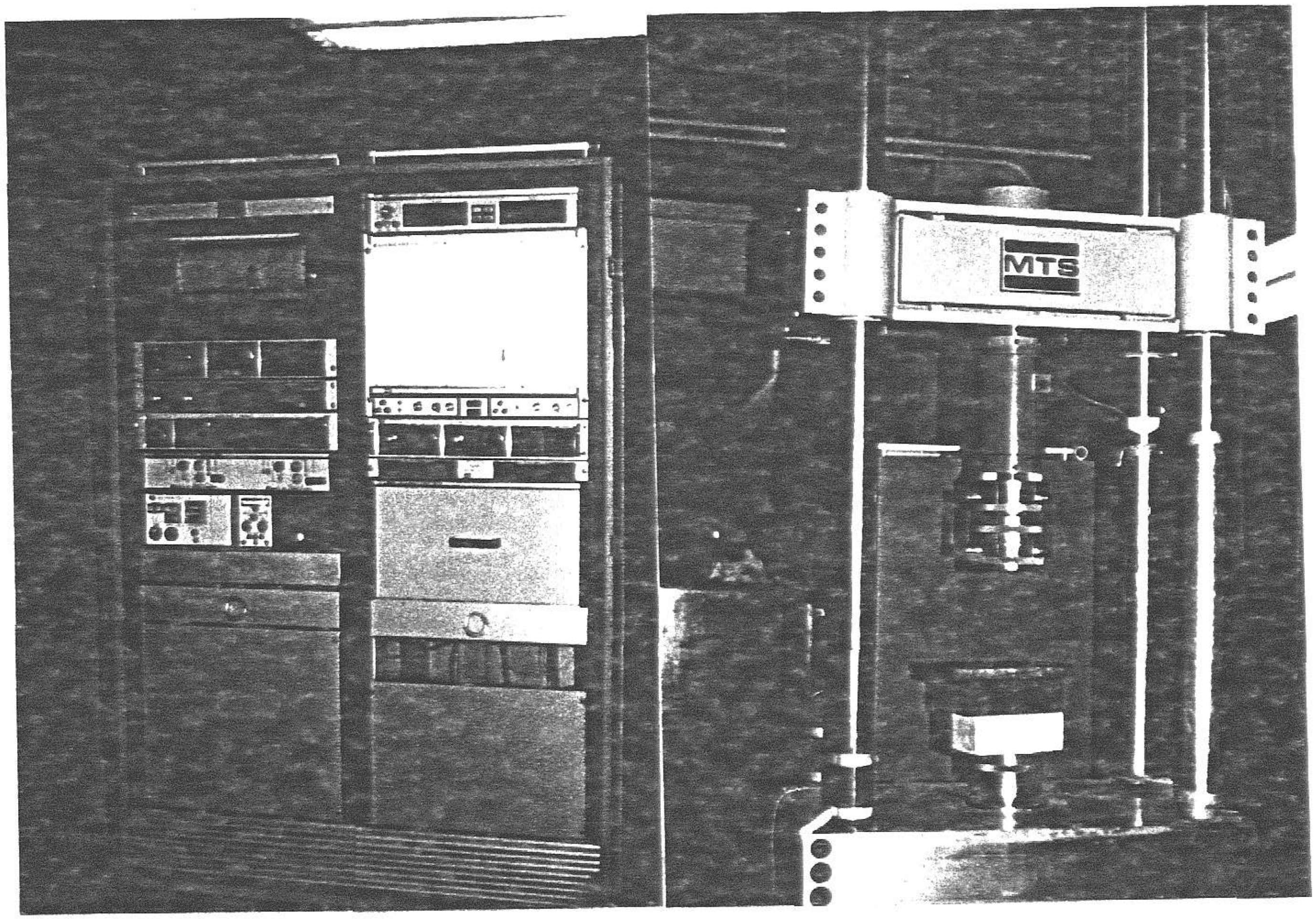


Figure 2. MTS Machine

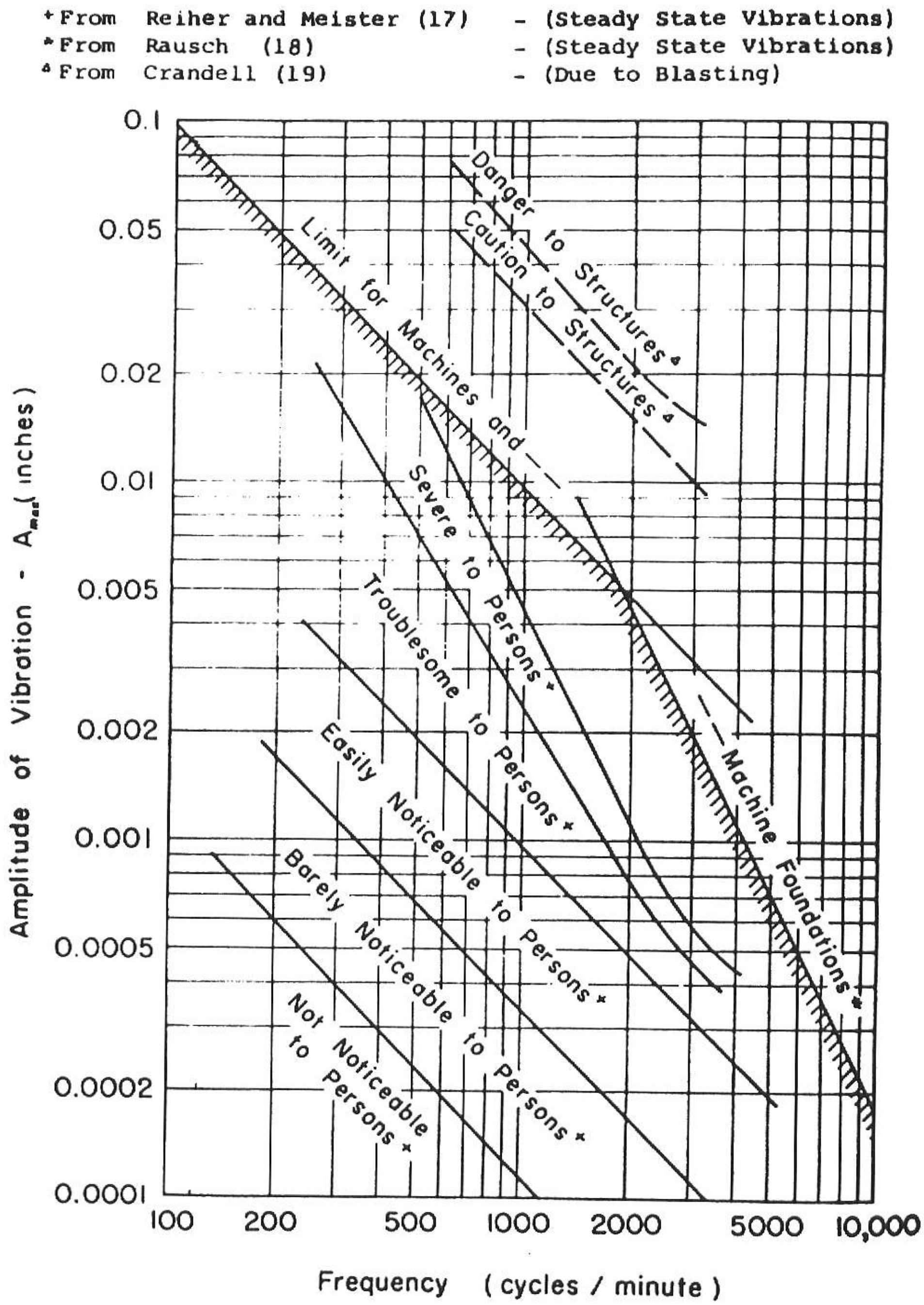


FIG. 2.—ALLOWABLE VERTICAL VIBRATION AMPLITUDE FOR A PARTICULAR FREQUENCY OF VIBRATION

Figure 3. Richart's Tolerable Motion Chart

Using this criteria, and looking at Table 1 which was composed from Pichart's chart, 5 Hz was selected as the testing frequency since this was in the range of machine operating frequencies (3.33 - 200 Hz). This frequency also had as its maximum displacement .03", which is ten squares on the MTS graph paper using the most delicate recording setting.

Samples were loaded onto the MTS machine, adjusting the height of the head of the machine so that the testing being performed was in the range of the plotter. The machine was then zeroed and put into the run program mode using stroke control. The span was increased until the proper displacement was indicated on the plotter. This test was run an optimum number of three times per sample in order to obtain a good average of the force required to deform the sample the specified amount (.03") at 5 Hz.

A half-sine function was chosen as the loading function since this most closely resembled a footing in place. A full sine function would have been inappropriate since under the premise of this experiment such a loading function would have been a footing jumping clear of the ground and imparting an impact load.

Finally, there was concern about the length of sample being worked with would not be long enough to allow full waves to form in the sample. This length would not give any allowance for damping either.

However, it was decided that the waves that would be generated would be reflected back into the sample by the steel plates, and with the absence of damping, the testing condition would be conservative.

Thus, the condition being worked with could be likened to a machine operating at low frequencies above bedrock at a shallow depth.

A condition where the elastic half-space solution breaks down totally according to Gazetas (4).

Any variability between samples would be non-consequential since the dynamic strength of the sample was compared with its unconfined compressive strength.

By reading the force required to deform the samples the specified amount, the bearing capacity of the soil for the specified frequency-displacement characteristic was obtained.

This bearing capacity was then compared with the bearing capacity from the unconfined compression test.

TABLE 1. LIMITS of TOLERANCE

Frequency (Hz)	Displacement (inches)	cycles/minute	Squares on plotting paper *
1.67	.1	100	33.33
5	.03	300	10.0
16.67	.0095	1000	3.16
33.33	.0047	2000	1.57
50	.002	3000	.667
83.3	.0007	5000	.233
166.67	.00018	10000	.06

* using the .5% plotting range

ANALYSIS and PRESENTATION of DATA

Since many of the samples available for use in this study were of only sufficient length for one test, use was made of unconfined compression data and results furnished by either Midcontinent Engineering and Testing or by the Advanced Soil Testing class, spring 1983, whenever possible. The data furnished from these sources appears on sheets marked with the MET logo. Nine unconfined compression tests are original data and are presented as such. For the information furnished by either MET or the Advanced Soils Testing class, see Appendix A.

By observing an unconfined compression test data sheet, as in Appendix B, one may follow how data is compiled.

Columns one and three are read directly during the test as according to the ASTM procedure. Column two is compiled by dividing column one by the original length of the sample being tested.

Column four is obtained by multiplying column three by .31 in this test case.

Column five is the original area of the specimen divided by the quantity of one minus column two.

Column six is column four divided by column five.

The mathematical formulas are shown on the data sheets.

Column six versus column two is then graphed, multiplying column two by one hundred for convenience. The peak of this graph is the ultimate strength, q_u. The graphs for nine unconfined compression tests follow this discussion.

By referring to an MTS data sheet, in Appendix C, it is seen that the data was much easier to use in this test. The horizontal axis is the displacement axis. With a machine setting of .5% on the plotter, each small square is .003 inches (.0762 mm). The vertical axis is the load axis, again with a plotter sensitivity of .5%, each small square is 10 pounds (44.5 newtons).

During the test, the span was increased until the horizontal distance on the plotter was ten squares from the starting point. When the vertical distance had stabilized at this point, the loading was ceased. The vertical distance from the original point was then determined and multiplied by ten to give the load on the soil. This load is defined herein to be the "dynamic strength," Q_d.

Table 2 was constructed as follows. A gage length of 5 inches (12.7 cm) was selected. For the required specimen deformation of .03 inches (.762 mm), the corresponding strain was .006. The length of each individual dynamic specimen was then multiplied by .006 to find the required deformation. This deformation was found on the MTS data sheet, and the corresponding load determined. This load was then divided by an area of 6.53 inches^2 (421 cm^2) which is the original area divided by one minus the strain. Since there was an optimum number of three trials per specimen, the load which was used was an average of the three. This procedure gave Q_d, the "dynamic strength."

The corresponding static unconfined compression strength at a .006 strain level was read directly from the compression versus strain curves. This was designated as $q_{.03}$. The ultimate unconfined compression strength, q_u, is the peak of the compression versus strain curve.

The ultimate strength was divided by the dynamic strength, q_u/Q_d, to see what sort of a "safety factor" would be used. There proved to be a wide scatter of points with a mean of 4.32, a standard deviation of 3.18, and a coefficient of variation of 73.6%.

The ultimate strength was then divided by the compressive strength at .006 strain, $q_u/q_{.03}$, to see if there would be a corresponding manner of scattering comparing a static test to a static test. The mean of this set of figures was 4.53, with a standard deviation of 2.15, and a coefficient of variation of 47.6%.

The dynamic strength was then divided by the static strength at .006 strain level, $Q_d/q_{.03}$, to determine if under identical strains which type of loading showed a higher "strength." The mean of this set of figures was 1.28, the standard deviation was .74, and the coefficient of variation of 57.8%.

Table 3 is a comparison of the dynamic strength obtained by using different safety factors on the unconfined compressive strength. An asterisk indicates the computed "dynamic strength" is less than the tested "dynamic strength." The row below the list of borings is the percent of time the "safety factor" yields a dynamic strength value below the tested value. The bottom row lists the percent increment between consecutive safety factors.

Table 4 lists the densities and water contents of the samples.

The samples which did not prove to be acceptable for a safety factor of six were checked for similarities. The depths of the samples were checked and found to be unrelated.

The densities were next compared and found to bary between 89.5 and 105.7 pounds/ft^3, (1434 to 1693 kg/m^3.) This was well within the acceptable range so it was discounted as a factor.

The natural water contents were next checked and found to range between 22.5 to 23.3 percent. Again this was discounted as a factor due to being well within the range of acceptable tests.

Finally, the lengths of the samples were next compared and found to have no relevance on the safety factor.

Figure 4.
Compression vs. Strain
#2, 20' - 22'
James Brennan
6/21/83

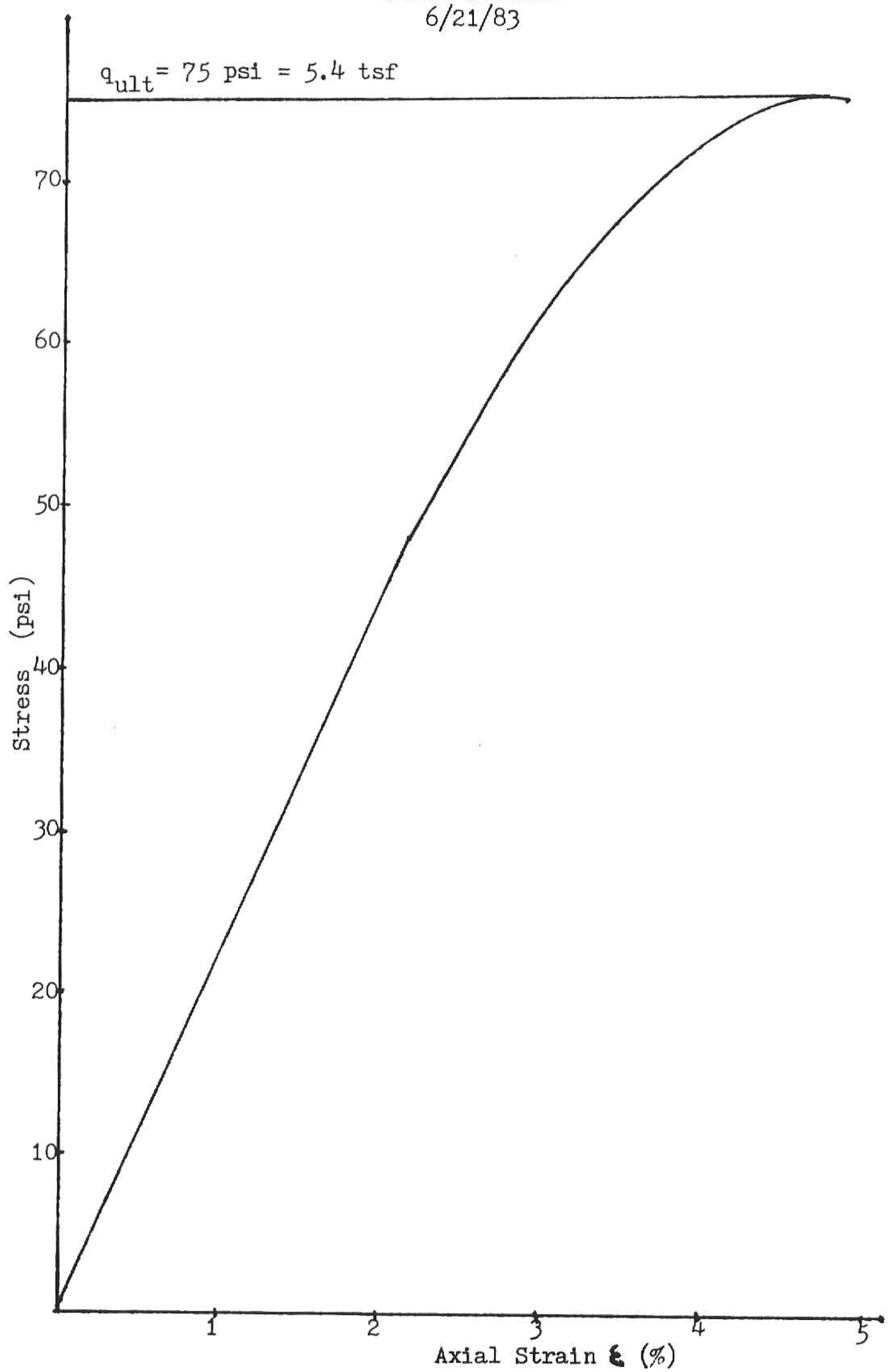
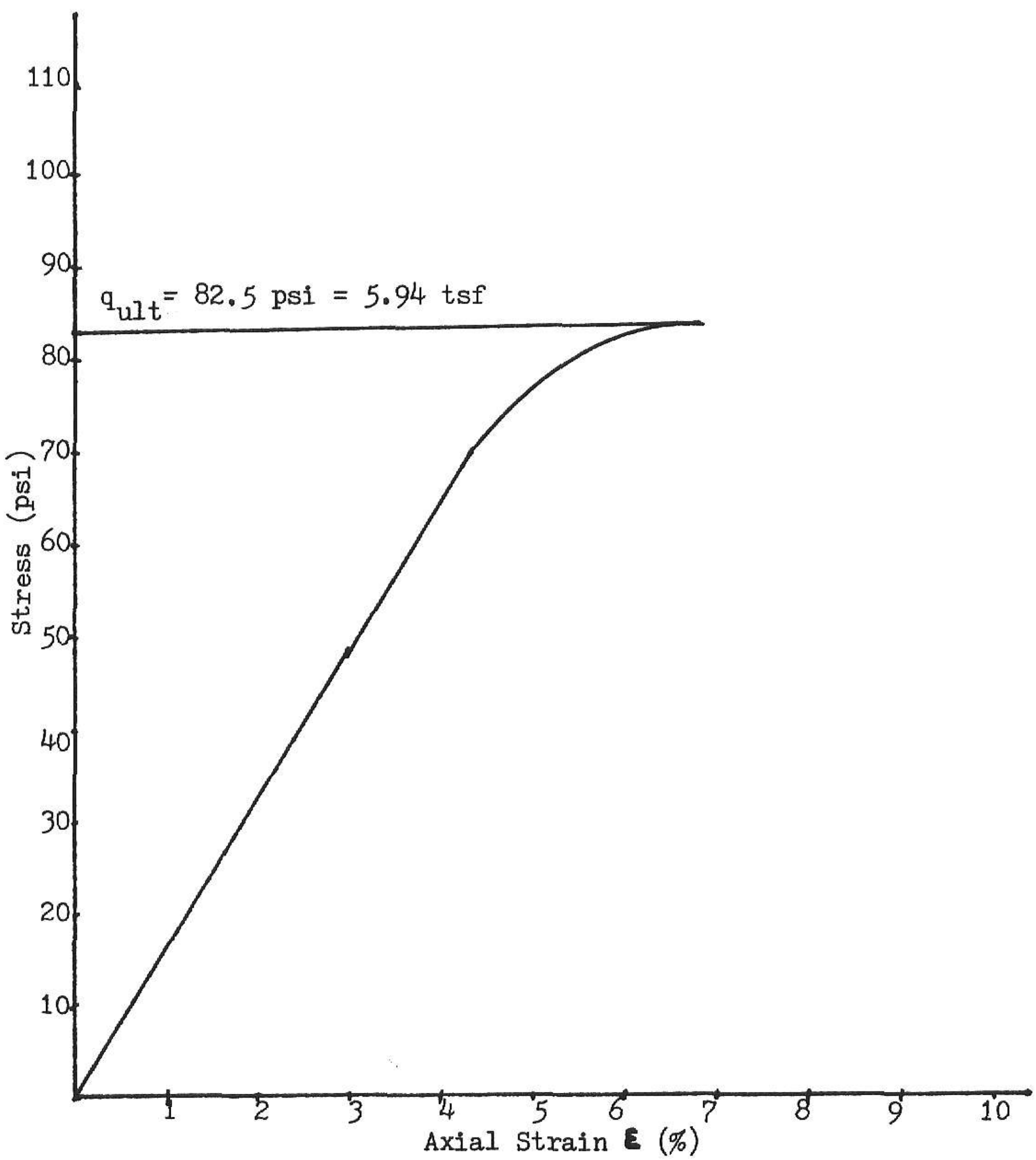


Figure 5.
Compression vs. Strain
#2, 30' - 32'
James Brennan
6/21/83



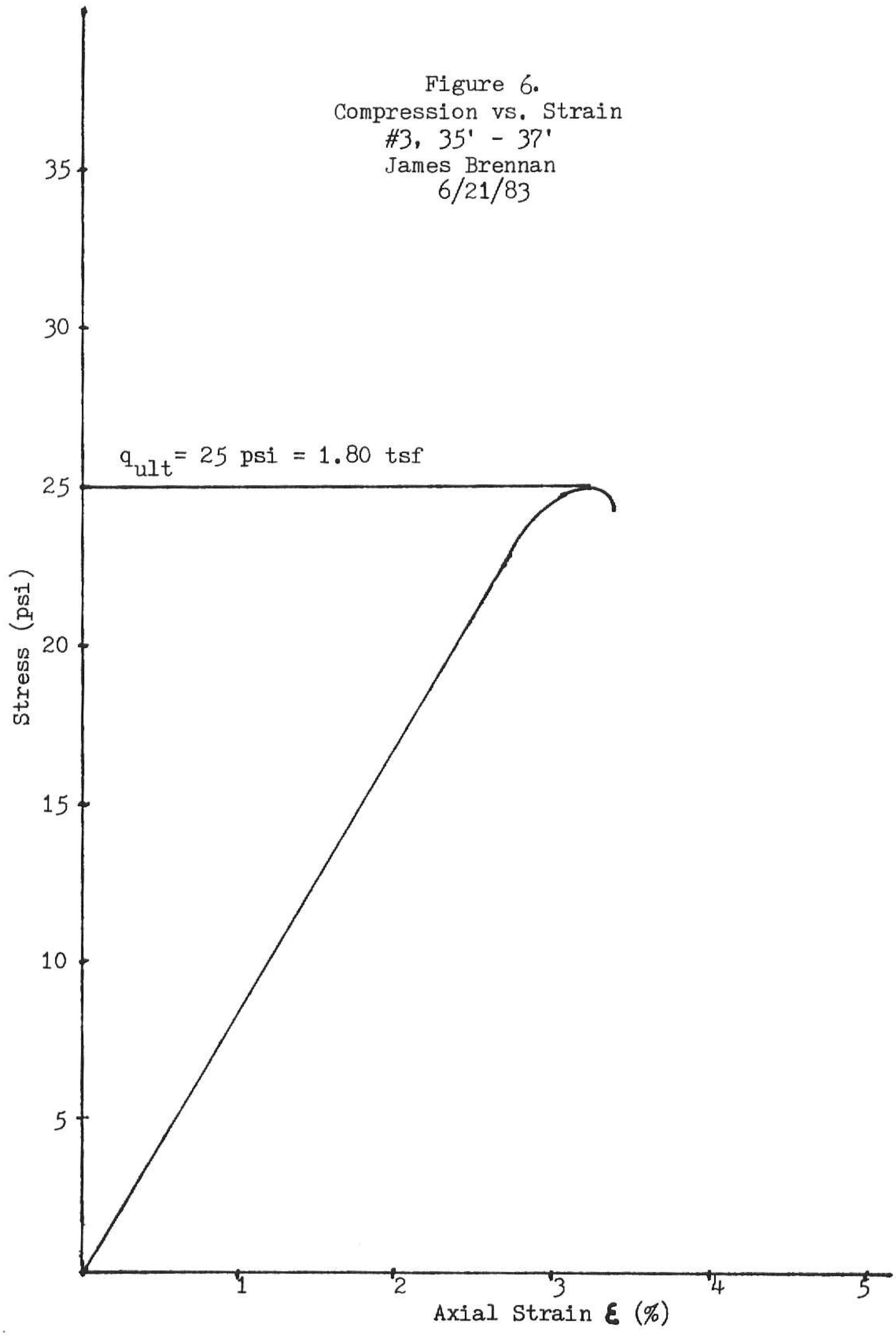


Figure 6.
Compression vs. Strain
#3, 35' - 37'
James Brennan
6/21/83
q_{ult} = 25 psi = 1.80 tsf
Stress (psi)
35
30
25
20
15
10
5
1
2
3
4
5
Axial Strain ε (%)

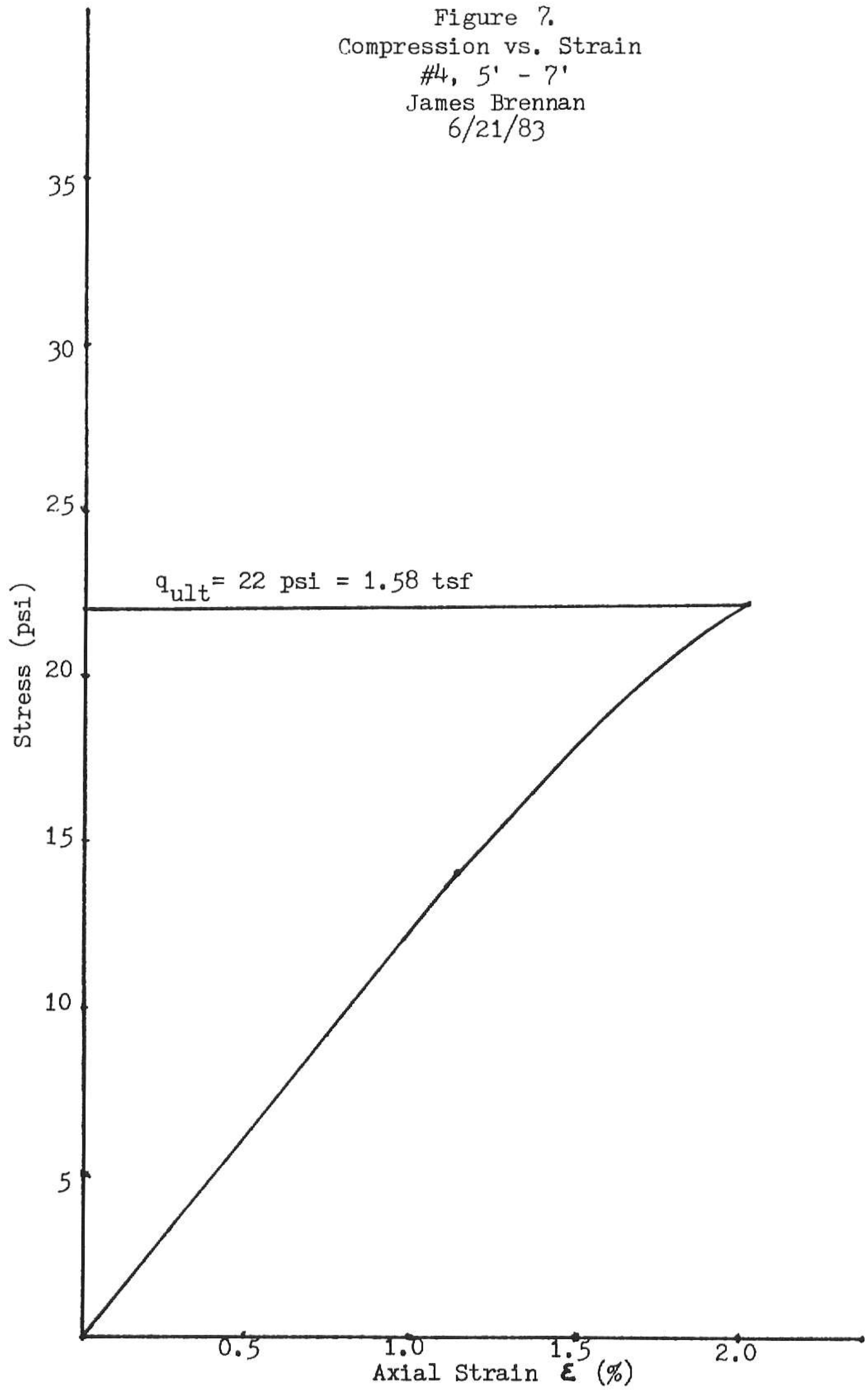


Figure 7.
Compression vs. Strain
#4, 5' - 7'
James Brennan
6/21/83
35
30
25
q_{ult} = 22 psi = 1.58 tsf
20
15
10
5
Stress (psi)
0.5
1.0
1.5
2.0
Axial Strain ε (%)

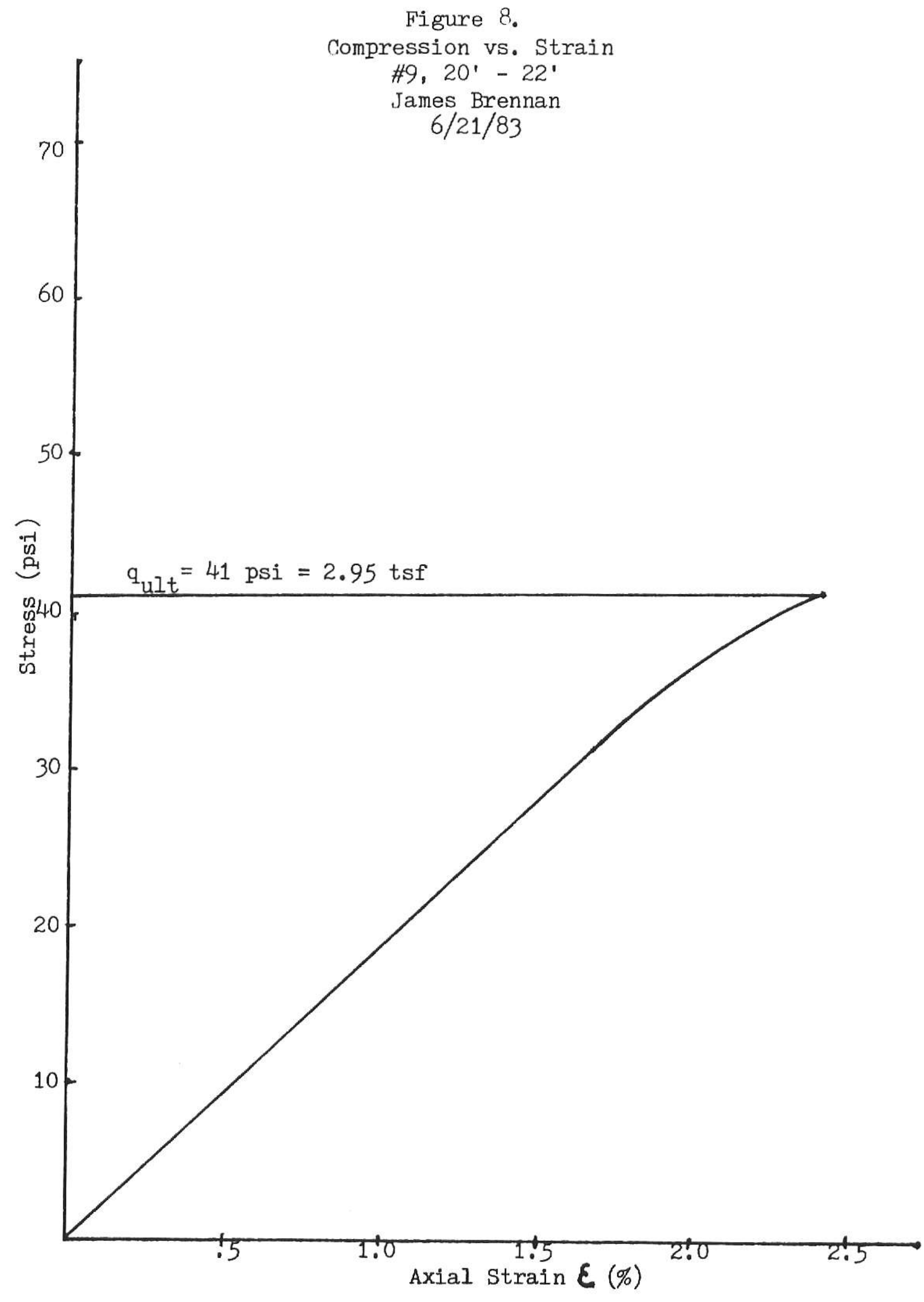


Figure 8.
Compression vs. Strain
#9, 20' - 22'
James Brennan
6/21/83
Stress (psi)
70
60
50
40
30
20
10
q_{ult} = 41 psi = 2.95 tsf
.5
1.0
1.5
2.0
2.5
Axial Strain ε (%)

Figure 9.
Compression vs. Strain
#12, 20' - 22'
James Brennan
6/21/83

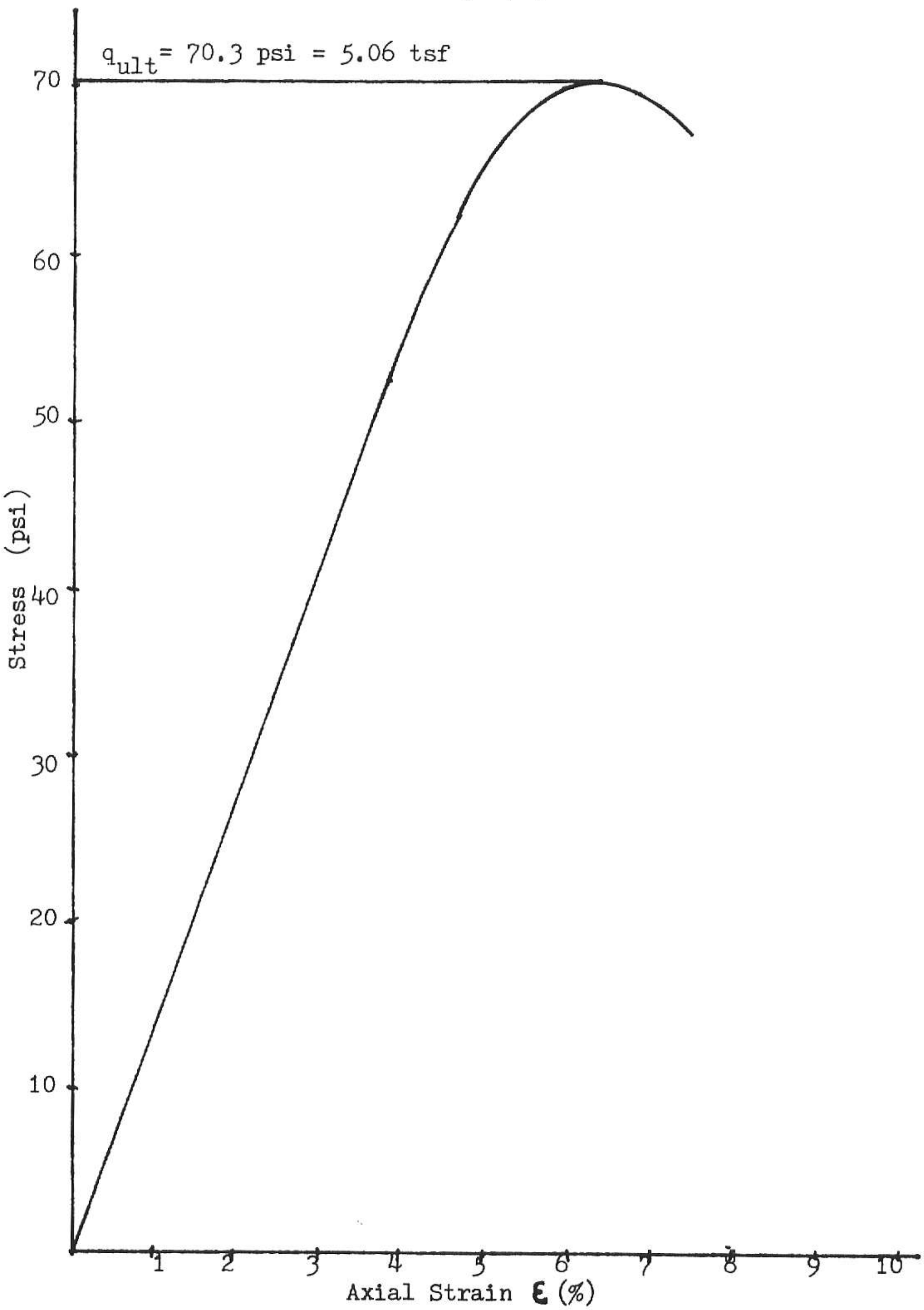


Figure 10.
Compression vs. Strain
#13, 20' - 22'
James Brennan
6/21/83

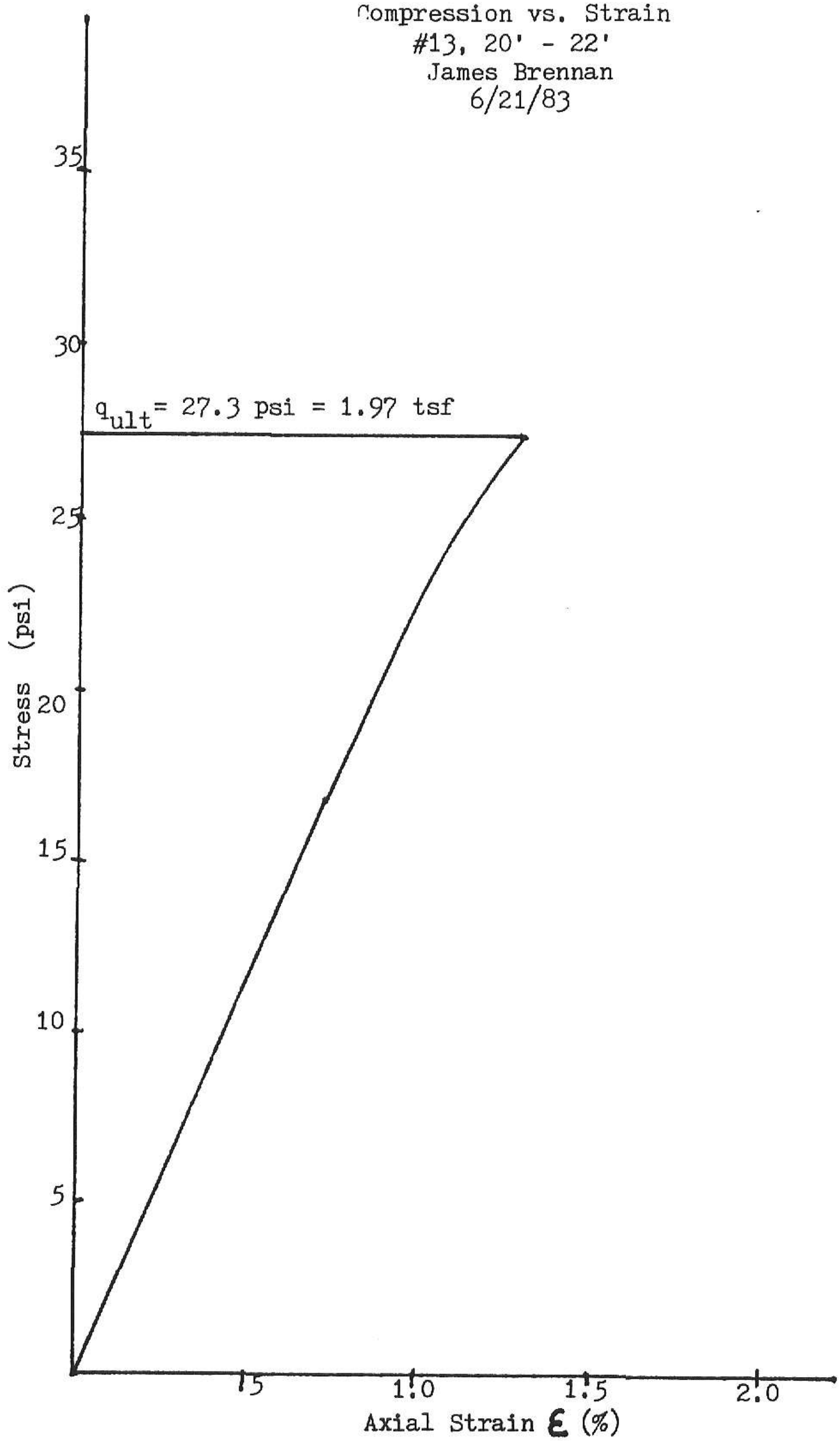


Figure 11.
Compression vs. Strain
#23, 10' - 12'
6/21/83

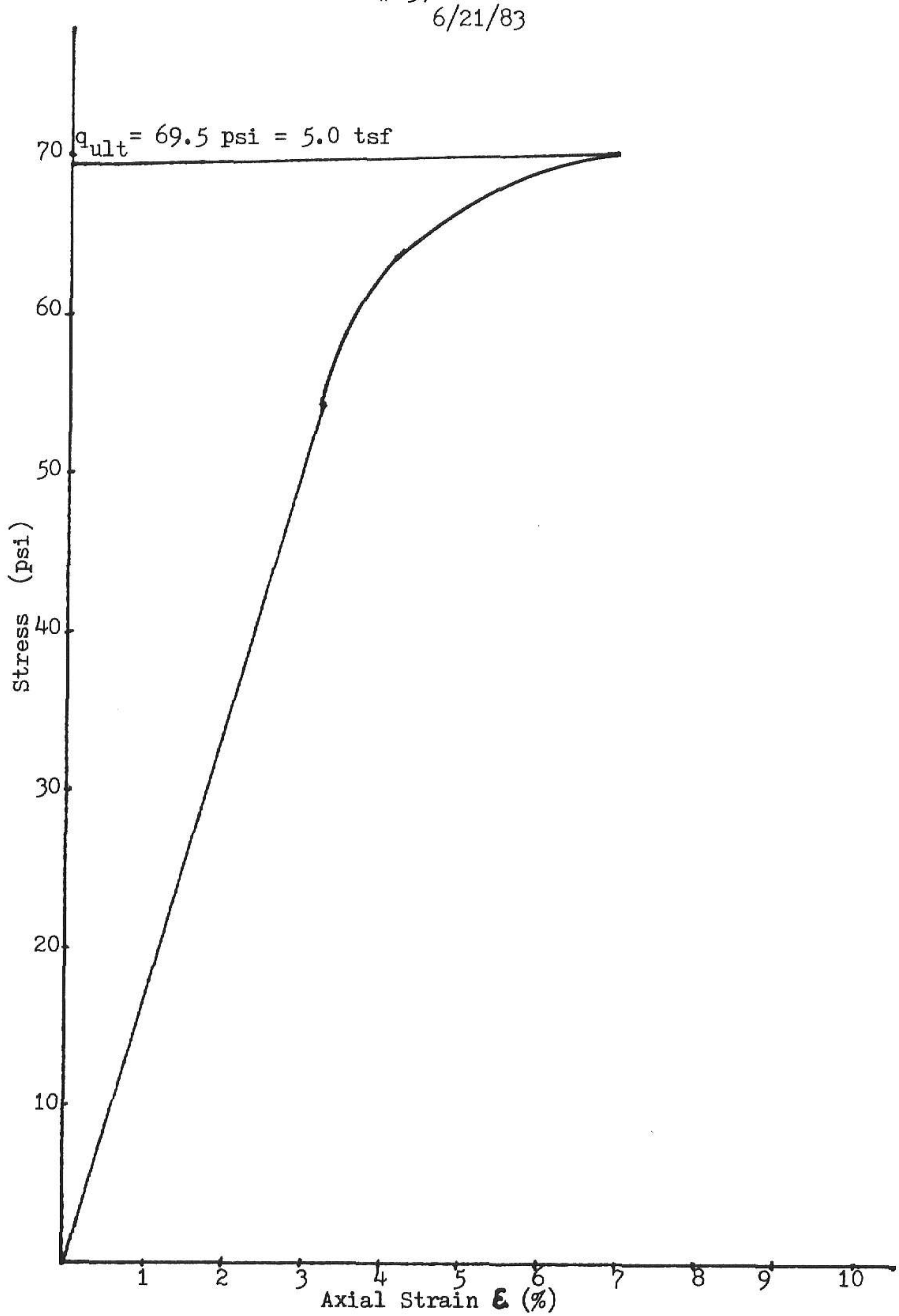


Figure 12.
Compression vs. Strain
#26, 30' - 32'
James Brennan
6/21/83

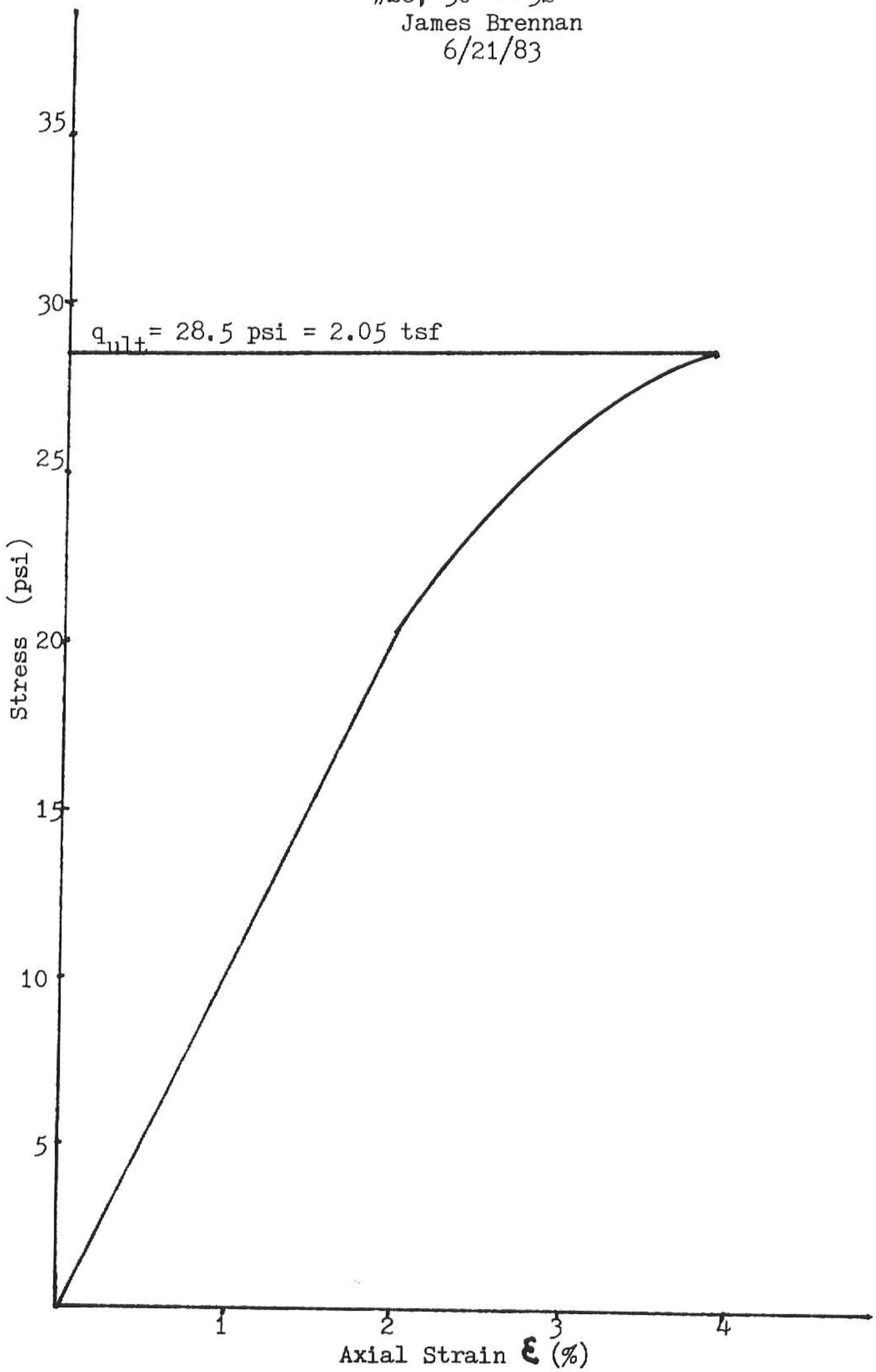


TABLE 2. STATIC and DYNAMIC STRENGTH COMPARISON

Boring #	Depth ft. (m)	Q_d (1)	$q_{.03}$ (1)	q_u	q_u/Q_d	$q_u/q_{.03}$	$Q_d/q_{.03}$
2	14 - 16	.33	.48	2.55	7.73	5.31	.69
2	20 - 22	1.98	.83	5.40	2.73	6.51	2.39
2	30 - 32	1.25	.72	5.94	4.75	8.25	1.74
2	32 - 34	.75	.55	1.68	2.24	3.05	1.36
3	5 - 7	.68	NA	1.21	1.78	NA	NA
3	10 - 12	.37	.20	1.09	2.95	5.45	1.85
3	35 - 37	.67	.36	1.80	2.69	5.0	1.86
4	5 - 7	.48	.49	1.58	3.29	3.22	.98
4	10 - 12	.33	.24	.90	2.73	3.75	1.38
9	10 - 12	.22	.48	.95	4.32	1.98	.46
9	20 - 22	1.84	.76	2.95	1.60	3.88	2.42
12	5 - 7	1.94	NA	2.81	1.45	NA	NA
12	20 - 22	.35	.58	5.06	14.46	8.72	.60
13	20 - 22	.33	.99	1.97	5.97	1.99	.33
13	25 - 27	.33	.12	.76	2.30	6.33	2.75
13	30 - 32	.44	.37	.83	1.89	2.24	1.19
16	15 - 17	.66	.63	1.89	2.86	3.0	1.04
17	20 - 22	.26	.18	.84	3.23	4.67	1.44
18	20 - 22	.15	.96	1.38	9.2	1.44	.16
23	10 - 12	.67	.68	5.0	7.46	7.35	.99
26	30 - 32	.40	.52	2.05	5.13	3.94	.77
mean					4.32	4.53	1.28
standard deviation					3.18	2.15	.74
coefficient of variation					73.6%	47.6%	57.8%

(1) Values obtained at a strain of 0.6% corresponding to a 0.03 inch deformation in a 5 inch gage length.

NA - not available

All values shown are in tons/ft^2.

1 foot is .3048 meters.
1 ton/$_{ft^2}$ is 96487 newtons/$_{m^2}$

TABLE 3. DYNAMIC STRENGTH for DIFFERENT SAFETY FACTORS

Boring #, Depth ft. (m)		Safety Factors 3.0	3.5	4.0	4.5	5.0	6.0
		Q_d tsf ($N/_{m}2$)					
2	14 - 16	.85	.73	.64	.57	.51	.43
2	20 - 22	1.80*	1.54*	1.35*	1.20*	1.08*	.90*
2	30 - 32	1.98	1.69	1.49	1.32	1.19*	.99*
2	32 - 34	.56*	.48*	.42*	.37*	.34*	.19*
3	5 - 7	.40*	.35*	.30*	.27*	.24*	.20*
3	10 - 12	.36*	.31*	.27*	.24*	.22*	.18*
3	35 - 37	.60*	.51*	.45*	.40*	.36*	.30*
4	5 - 7	.53	.45*	.40*	.35*	.32*	.26*
4	10 - 12	.30*	.26*	.23*	.20*	.18*	.15*
9	10 - 12	.32	.27	.24	.21*	.19*	.16*
9	20 - 22	.98*	.84*	.74*	.66*	.59*	.49*
12	5 - 7	.94*	.80*	.70*	.62*	.56*	.47*
12	20 - 22	1.69	1.45	1.27	1.12	1.01	.84
13	20 - 22	.66	.56	.49	.44	.39	.33
13	25 - 27	.25*	.22*	.19*	.17*	.15*	.13*
13	30 - 32	.28*	.24*	.21*	.18*	.17*	.14*
16	15 - 17	.63	.54*	.47*	.42*	.38*	.32*
17	20 - 22	.28	.24*	.21*	.19*	.17*	.14*
18	20 - 22	.46	.39	.35	.31	.28	.23
23	10 - 12	1.67	1.43	1.25	1.11	1.00	.83
26	30 - 32	.68	.59	.51	.46	.41	.34*
Percent of time acceptable		52.3	61.9	61.9	66.7	71.4	81.0
Increase in percent of time acceptable			9.6	0	4.8	4.7	9.6

1 foot is .3048 meters.
1 $ton/_{ft}2$ is 96487 $newtons/_{m}2$

* means acceptable

TABLE 4. DENSITIES and WATER CONTENTS

Boring #	Depth ft.	Lenth of Dynamic Sample (inches)	Dry Density pcf	Water Content %
2	14 - 16*	5 3/8	102.3	23.3
2	20 - 22	5 1/4	102.5	23.6
2	30 - 32	5 5/8	114.0	18.2
2	32 - 34	5 3/8	109.7	22.2
3	5 - 7	5 1/4	92.4	32.2
3	10 - 12	3 3/8	98.8	25.9
3	35 - 37	5	91.0	32.9
4	5 - 7	4 1/4	92.6	30.9
4	10 - 12	4 1/8	98.6	24.1
9	10 - 12	5	99.4	25.1
9	20 - 22	5 1/4	98.8	23.8
12	5 - 7	5 1/4	106.7	21.7
12	20 - 22*	4 1/8	105.7	23.0
13	20 - 22	2 5/8	102.9	23.5
13	25 - 27	4 1/2	97.6	23.3
13	30 - 32	4 3/8	105.7	18.7
16	15 - 17	6 3/4	109.1	22.8
17	20 - 22	4 7/8	99.0	25.7
18	20 - 22*	5 5/8	89.5	22.8
23	10 - 12*	5 1/4	103.4	22.5
26	30 - 32	5 1/4	100.7	26.9

* were not acceptable with a safety factor of 6

1 foot is .3048 meters.
1 pcf is 16.018 kg/m^3
1 inch is 2.54 cm.

CONCLUSIONS

Table 2 shows that soil behaves in a similar manner as any other material in that at identical strains, soil can withstand a higher dynamic load than it can a static load.

For the testing conditions described herein, there seems to be evidence that using a safety factor of six will result in a safe allowable dynamic bearing capacity 81 percent of the time using the unconfined compression test as a standard.

The study presented herein hopefully suggests a starting point for the correlation of unconfined with dynamic strength. The work indicated that at low frequencies on a silty clay material, the minimum safety factor to be used would be six. Further research is necessary in this area before anything definitive can be stated.

RECOMMENDATIONS for FURTHER RESEARCH

A beginning has been made on the design of machine foundations using simple testing methods and not relying on sophisticated, and possibly misleading, testing procedures.

The next step of the research should try to show an actual relationship between the unconfined compressive strength and the bearing capacity actually used in the field on successful machine foundations.

This should be accompanied by extensive laboratory testing of different type material specimens at different frequencies.

Identical samples should be obtained so that a statistical analysis could be performed to determine the typical scatter from an unconfined compression test. Identical samples would also be needed so that a statistical analysis of the typical scatter from the dynamic testing could be determined.

A literature review should be done in order to ascertain the variability of the shear modulus tests now being performed.

Finally, the sensitivity of the MTS machine should be improved so that other frequencies can be tested other than 5 Hz. All this testing requires a large number of samples so that the tests will have statistical significance.

This work will allow a more precise safety factor to be used, which in turn will appreciably assist in machine foundation design.

ACKNOWLEDGMENTS

I would like to thank Dr. Cooper for willingly answering my questions when Prof. Williams was not available.

Dr. Swartz should also be recognized for his willing devotion of time to my efforts.

I would like to thank Midcontinent Engineering and Testing and the Advanced Soil Testing class, spring 1983, for the use of their test data.

I would like to recognize Prof. Williams for being a constant source of inspiration.

Finally, I salute Dr. Henry Beck, who agreed to be on my committee, and carefully proofread my manuscript.

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APPENDIX A
MET and Advanced Soils Testing Class Data

MIDCONTINENT ENGINEERING & TESTING CO.

MET #20799

Project Great Midwest Factory Outlet Mall

Project No.

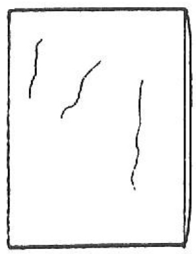
SUMMARY OF SOIL TESTS

Laboratory Midcontinent Engineering & Testing Co.

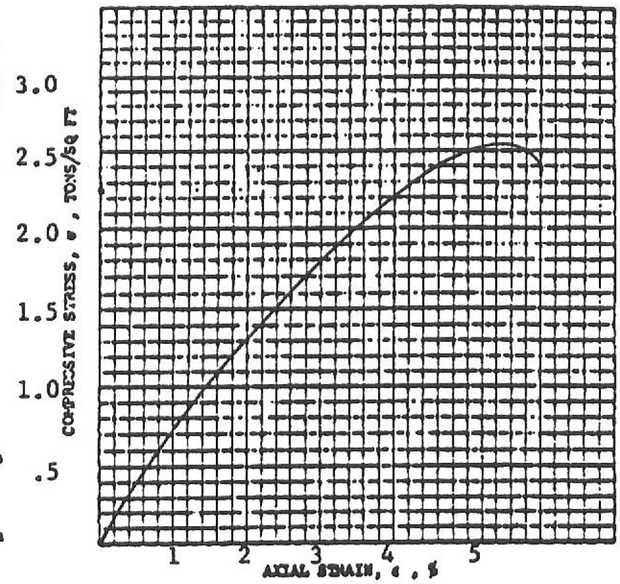
Date March 16, 1983

BORING NUMBER	SAMPLE NUMBER	DEPTH OR ELEVATION	CLASSIFICATION	W %	DRY UNIT WT. PCF.	ATTERBERG LIMITS			UNCONFINED COMPRESSION					REMARKS
						LL	PL	PI	TSF	% E				
2	9	40'0"-40'9"	Weathered tan and gray shale	18.9										
3	1	0'-2'	Brown clayey silt	26.7	94.9				0.74	4.5				
3	2	5'-7'	Brown silt	32.2	92.4				1.21					
3	3	10'-12'	Brown silt	25.9	98.8				1.09	3.6				
3	4	15'-17'	Light brown silt	24.2	100.0				1.68					
3	5	20'-22'	Brown mottled with tan clayey silt with some sand and pebbles	22.2	105.0				1.06	3.6				
3	6	25'-27'	Yellow-tan interbedded with rust-tan and gray coarse silt	25.7	92.0				0.72	1.8				
3	7	30'-32'	Tan mottled with gray silt	16.6	93.6									
3	8	35'-37'	Tan mottled with rust-tan silt with a few pebbles and sand	32.9	91.0									
3	9	40'0"-40'9"	Rust-tan mottled with gray clayey silt with sandy silt	24.9										

MET
MIDCONTINENT ENGINEERING & TESTING CO.



Sketch of specimen after failure



TEST TYPE
(Check one)

☐ Controlled-stress

☒ Controlled-strain

Type of specimen ☒ Undisturbed ☐ Remolded			Test No.	Test No.	Test No.	Test No.
Initial	Water content	w_o	23.3 %	%	%	%
	Void ratio	e_o				
	Saturation	S_o	%	%	%	%
	Dry density, lb/cu ft	γ_d	102.3			
Time to failure, min		t_f				
Unconfined compressive strength, tons/sq ft		q_u	2.55			
Undrained shear strength, tons/sq ft		s_u				
Sensitivity ratio		S_t				

Classification Light brown mottled with some tan silt

LL	PL	PI	G_s

Specimen diam 2.850 in. Specimen Height 5.563 in.

Remarks MET #20799

Project Great Midwest Factory Outlet Mall

Area

Boring No. 2 Sample No.

Depth, ft 15'9"-16'3" Date Mar. 8, 1983

UNCONFINED COMPRESSION TEST REPORT

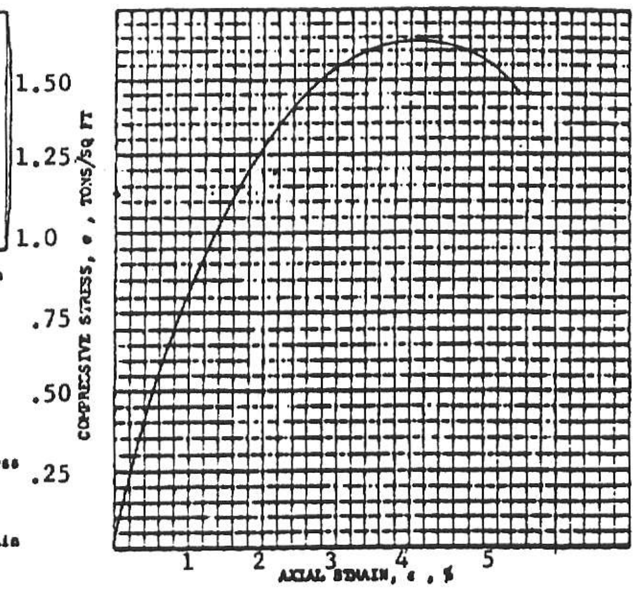
MIDCONTINENT ENGINEERING & TESTING CO.

Sketch of specimen after failure

TEST TYPE
(Check one)

☐ Controlled-stress

☒ Controlled-strain



Type of specimen	☒ Undisturbed ☐ Remolded		Test No.	Test No.	Test No.	Test No.
Initial	Water content	w_o	22.2 %	%	%	%
	Void ratio	e_o				
	Saturation	S_o	%	%	%	%
	Dry density, lb/cu ft	γ_d	109.7			
Time to failure, min		t_f				
Unconfined compressive strength, tons/sq ft		q_u	1.68			
Undrained shear strength, tons/sq ft		s_u				
Sensitivity ratio		S_t				

Classification: Yellow-tan mottled with rust-tan and gray clayey silt with some silty clay, sand and pebbles

LL	PL	PI	G_s

Specimen diam 2.850 in. Specimen Height 5.563 in.

Remarks MET #20799

Project Great Midwest Factory Outlet Mall

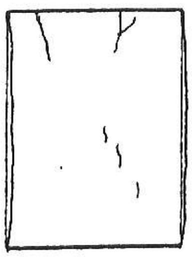
Area

Boring No. 2 Sample No.

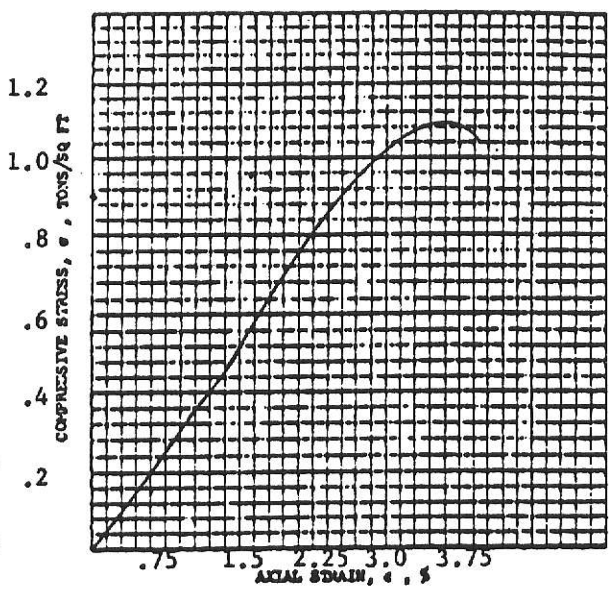
Depth, ft 32'9"-33'3" Date Mar. 8, 1983

UNCONFINED COMPRESSION TEST REPORT

MIDCONTINENT ENGINEERING & TESTING CO.



Sketch of specimen after failure



TEST TYPE (Check one)

☐ Controlled-stress

☒ Controlled-strain

Type of specimen ☐ Undisturbed ☐ Remolded			Test No.	Test No.	Test No.	Test No.
Initial	Water content	w_o	25.9 %	%	%	%
	Void ratio	e_o				
	Saturation	S_o	%	%	%	%
	Dry density, lb/cu ft	γ_d	98.8			
Time to failure, min		t_f				
Unconfined compressive strength, tons/sq ft		q_u	1.09			
Undrained shear strength, tons/sq ft		s_u				
Sensitivity ratio		S_t				

Classification Brown silt

LL	PL	PI	G_s

Specimen diam 2.850 in. | Specimen Height 5.563 in.

Remarks MET #20799

Project Great Midwest Factory Outlet Mall

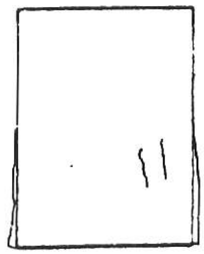
Area

Boring No. 3 Sample No.

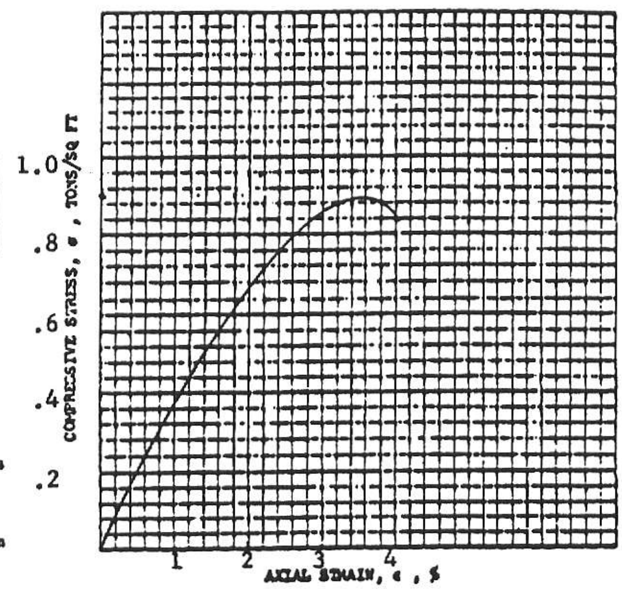
Depth, El 11'0"-11'6" Date Mar. 8, 1983

UNCONFINED COMPRESSION TEST REPORT

MIDCONTINENT ENGINEERING & TESTING CO.



Sketch of specimen after failure



TEST TYPE
(Check one)

☐ Controlled-stress

☒ Controlled-strain

Type of specimen	☒ Undisturbed ☐ Remolded		Test No.	Test No.	Test No.	Test No.
Initial	Water content	w_o	24.1 %	%	%	%
	Void ratio	e_o				
	Saturation	S_o	%	%	%	%
	Dry density, lb/cu ft	γ_d	98.6			
Time to failure, min		t_f				
Unconfined compressive strength, tons/sq ft		q_u	0.90			
Undrained shear strength, tons/sq ft		s_u				
Sensitivity ratio		S_t				

Classification Reddish brown silt

LL | PL | PI | G_s

Specimen dia 2.850 in. | Specimen Height 5.563 in.

Remarks MET #20799

Project Great Midwest Factory Outlet Mall

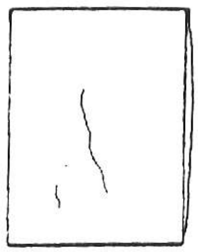
Area

Boring No. 4 Sample No.

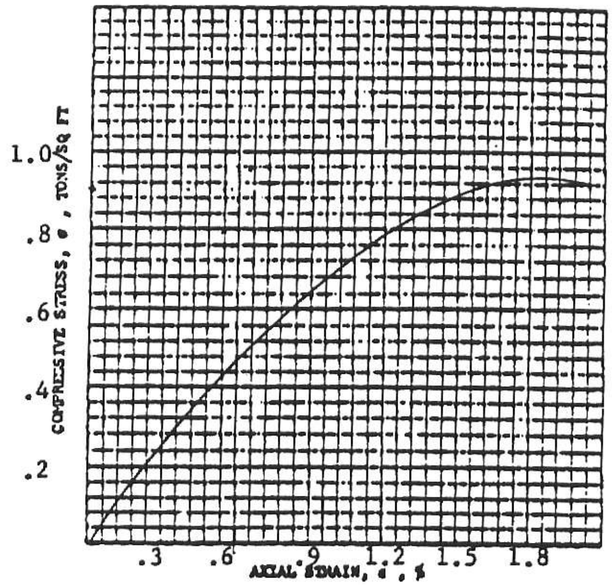
Depth, ft 11'0"-11'6" Date Mar. 8, 1983

UNCONFINED COMPRESSION TEST REPORT

MIDCONTINENT ENGINEERING & TESTING CO.



Sketch of specimen after failure



TEST TYPE
(Check one)

☐ Controlled-stress

☒ Controlled-strain

Type of specimen	☒ Undisturbed ☐ Remolded		Test No.	Test No.	Test No.	Test No.
Initial	Water content	w_o	25.1 %	%	%	%
	Void ratio	e_o				
	Saturation	S_o	%	%	%	%
	Dry density, lb/cu ft	γ_d	99.4			
Time to failure, min		t_f				
Unconfined compressive strength, tons/sq ft		q_u	0.95			
Undrained shear strength, tons/sq ft		s_u				
Sensitivity ratio		S_t				

Classification Light brown mottled with some tan brown silt

LL	PL	PI	G_s

Specimen diam 2.850 in. | Specimen Height 5.563 in.

Remarks MET #20799

Project Great Midwest Factory Outlet Mall

Area

Boring No. 9 Sample No.

Depth, ft 10'0"-10'6" Date Mar. 9, 1983

UNCONFINED COMPRESSION TEST REPORT

MIDCONTINENT ENGINEERING & TESTING CO.

MET #20799

SUMMARY OF SOIL TESTS

Project Great Midwest Factory Outlet Mall

Project No. ______

Laboratory Midcontinent Engineering & Testing Co.

Date March 16, 1983

BORING NUMBER	SAMPLE NUMBER	DEPTH OR ELEVATION	CLASSIFICATION	W %	DRY UNIT WT. PCF	ATTERBERG LIMITS			UNCONFINED COMPRESSION					REMARKS
						LL	PL	PI	TSF	% E				
9	8	35'0"-36'8"	Rust-tan clayey sand with some pebbles	6.7										
9	9	40'0"-40'9"	Tan-rust with yellow-tan silty sand	16.5										
9	10	45'0"-46'6"	Rust-tan mottled with tan-gray silty sand	25.7	92.0									
12	1	0'-2'	A mixture of brown, rust and gray clayey silt	22.2	104.7				1.11	10.8				
12	2	5'-7'	Reddish-tan mixed with dark brown and gray clayey silt	21.7	106.7				2.81					
12	3	10'-12'	Rust and gray clay to dark gray silt	23.6	104.9				1.62	3.6				
12	4	15'-17'	Brown-tan mottled with some gray silt	22.9	96.5				0.85	1.8				
12	5	20'-22'	Brown-tan mottled with some gray silt	23.0	105.7									
13	1	0'-2'	Dark gray-brown mixed with tan, rust silt with pieces of Ls. & shale	20.3	104.1									
13	2	5'-7'	A mixture of yellow-tan, rust & gray silty clay with pieces of Ls.	21.9										

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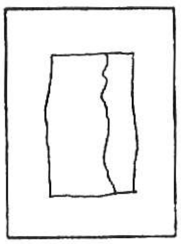
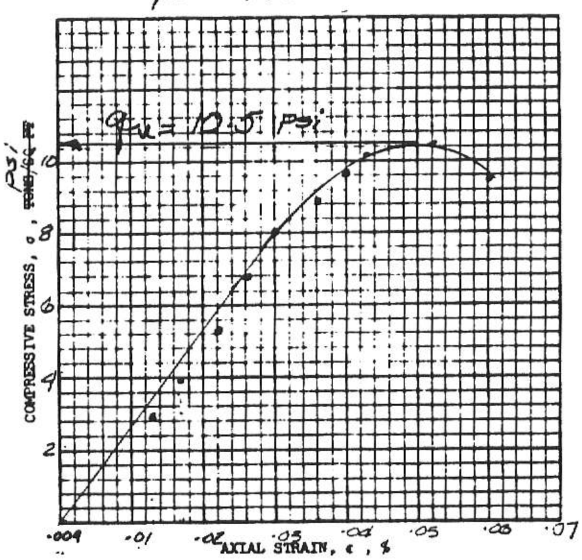
MIDCONTINENT ENGINEERING & TESTING CO.

qu = 0.756 TSF.



Sketch of specimen after failure



TEST TYPE
(Check one)

☐ Controlled-stress

☐ Controlled-strain

Type of specimen	☐ Undisturbed ☐ Remolded		Test No.	Test No.	Test No.	Test No.
Initial	Water content	w_o	23.88%	%	%	%
	Void ratio	e_o	.7606			
	Saturation	S_o	%	%	%	%
	Dry density, lb/cu ft	γ_d	93.92			
Time to failure, min		t_f				
Unconfined compressive strength, tons/sq ft		q_u				
Undrained shear strength, tons/sq ft		s_u				
Sensitivity ratio		S_t				

Classification

LL	PL	PI	G_s

Specimen Diam cm in. | Specimen Height cm in.

Remarks ______

Project ______ Job No.

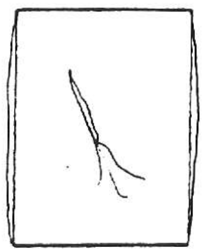
Area ______

Boring No. ______ Sample No. 13

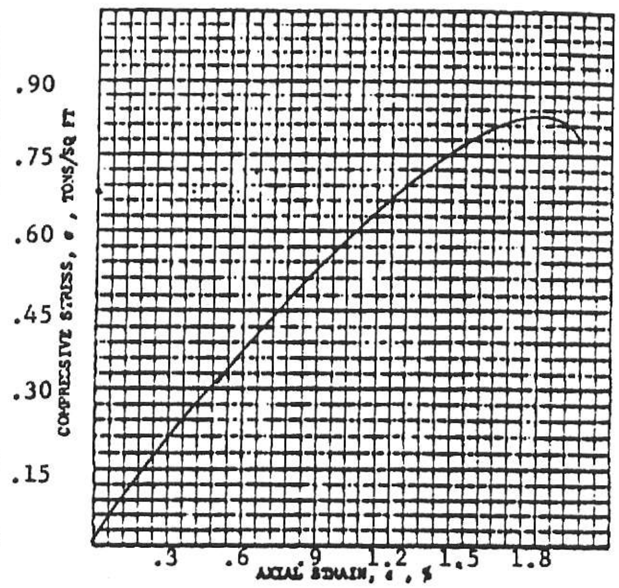
Depth, El 25-27' Date 3-10-83

UNCONFINED COMPRESSION TEST REPORT

MIDCONTINENT ENGINEERING & TESTING CO.



Sketch of specimen after failure



TEST TYPE
(Check one)

☐ Controlled-stress

☒ Controlled-strain

Type of specimen	☒ Undisturbed ☐ Remolded		Test No.	Test No.	Test No.	Test No.
Initial	Water content	w_o	18.7 %	%	%	%
	Void ratio	e_o				
	Saturation	S_o	%	%	%	%
	Dry density, lb/cu ft	γ_d	105.7			
Time to failure, min		t_f				
Unconfined compressive strength, tons/sq ft		q_u	0.83			
Undrained shear strength, tons/sq ft		s_u				
Sensitivity ratio		S_t				

Classification Yellowish-tan silty sandy clay

LL	PL	PI	G_s

Specimen Dia 2.850 in. Specimen Height 5.563 in.

Remarks MET #20799

Project Great Midwest Factory Outlet Mall

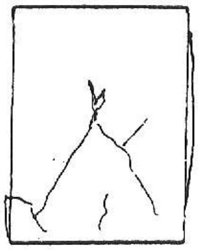
Area

Boring No. 13 Sample No.

Depth, El 31'0"-31'6" Date Mar. 10, 1983

UNCONFINED COMPRESSION TEST REPORT

MIDCONTINENT ENGINEERING & TESTING CO.

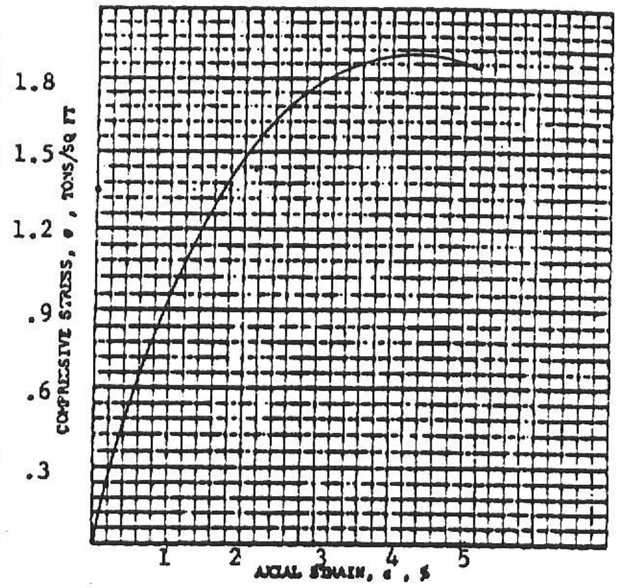


Sketch of specimen after failure

TEST TYPE
(Check one)

☐ Controlled-stress

☒ Controlled-strain



Type of specimen	☒ Undisturbed ☐ Remolded		Test No.	Test No.	Test No.	Test No.
Initial	Water content	w_o	22.8 %	%	%	%
	Void ratio	e_o				
	Saturation	S_o	%	%	%	%
	Dry density, lb/cu ft	γ_d	109.1			
Time to failure, min		t_f				
Unconfined compressive strength, tons/sq ft		q_u	1.89			
Undrained shear strength, tons/sq ft		s_u				
Sensitivity ratio		S_t				

Classification Weathered grayish-green shale

LL	PL	PI	G_s

Specimen diam 2.850 in. Specimen Height 5.563 in.

Remarks MET #20799

Project Great Midwest Factory Outlet Mall

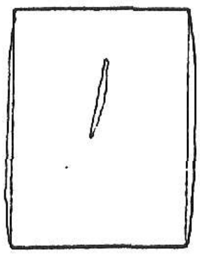
Area

Boring No. 16 Sample No.

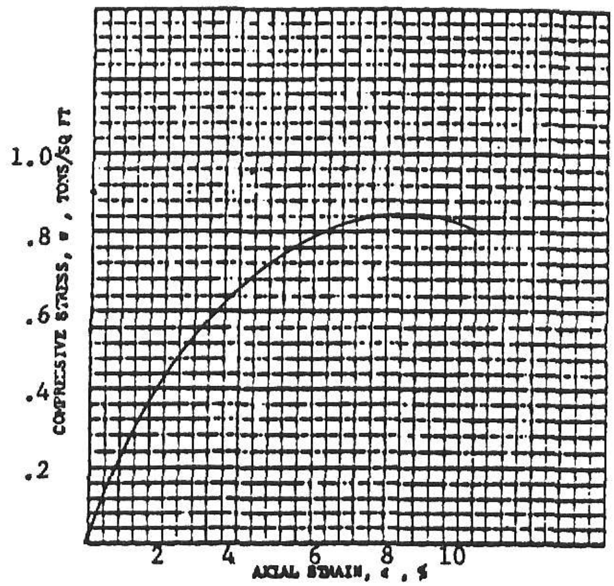
Depth, ft 15'9"-16'3" Date Mar. 10, 1983

UNCONFINED COMPRESSION TEST REPORT

MIDCONTINENT ENGINEERING & TESTING CO.



Sketch of specimen after failure



TEST TYPE
(Check one)

☐ Controlled-stress

☒ Controlled-strain

Type of specimen	☒ Undisturbed ☐ Remolded		Test No.	Test No.	Test No.	Test No.
Initial	Water content	w_o	25.7 %	%	%	%
	Void ratio	e_o				
	Saturation	S_o	%	%	%	%
	Dry density, lb/cu ft	γ_d	99.0			
Time to failure, min		t_f				
Unconfined compressive strength, tons/sq ft		q_u	0.84			
Undrained shear strength, tons/sq ft		s_u				
Sensitivity ratio		S_t				

Classification Greenish-gray silt

LL	PL	PI	G_s

Specimen Dia 2.850 in. Specimen Height 5.563 in.

Remarks MET #20799

Project Great Midwest Factory Outlet Mall

Area

Boring No. 17 Sample No.

Depth, ft 21'0"-21'6" Date Mar. 10, 1983

UNCONFINED COMPRESSION TEST REPORT

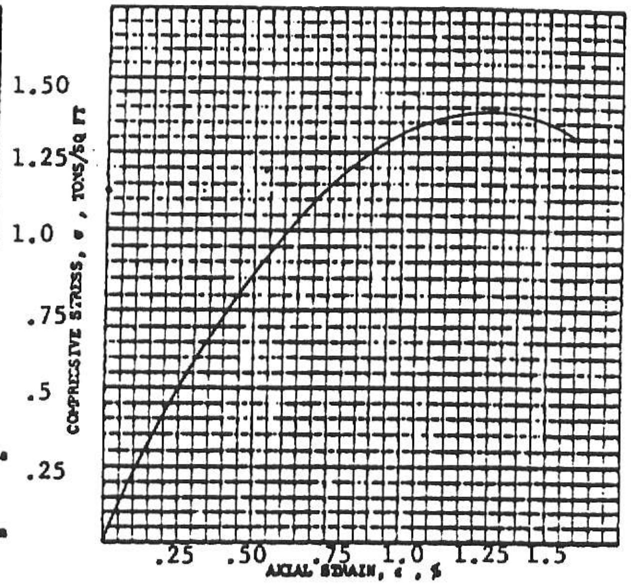
MIDCONTINENT ENGINEERING & TESTING CO.

Sketch of specimen after failure

TEST TYPE
(Check one)

☐ Controlled-stress

☒ Controlled-strain



Type of specimen		☒ Undisturbed ☐ Remolded	Test No.	Test No.	Test No.	Test No.
Initial	Water content	w_o	22.8 %	%	%	%
	Void ratio	e_o				
	Saturation	S_o	%	%	%	%
	Dry density, lb/cu ft	γ_d	89.5			
Time to failure, min		t_f				
Unconfined compressive strength, tons/sq ft		q_u	1.38			
Undrained shear strength, tons/sq ft		s_u				
Sensitivity ratio		S_t				

Classification Brown mottled with some gray silt

LL	PL	PI	G_s

Specimen diam 2.850 in. Specimen Height 5.563 in.

Remarks MET #20799

Project Great Midwest Factory Outlet Mall

Area

Boring No. 18 Sample No.

Depth, ft 20'-20'6" Date Mar. 11, 1983

UNCONFINED COMPRESSION TEST REPORT

APPENDIX B
Unconfined Compression Test Data Sheets

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UNCONFINED COMPRESSION TEST

Description of soil ____

Specimen No. #2 Location 20-22'

Moist weight of specimen ____ moisture content ____ ____%

Length of specimen, L 6 3/8" Diameter, D 2 7/8" Area, A_0 6.49 □"

Proving ring calibration factor: 1 div. = .31

Specimen deformation = ΔL in. (1)	Vertical strain, $\epsilon = \frac{\Delta L}{L}$ (2)	Proving ring dial reading −No. of small divisions (3)	Load = Col. 3 x calibration factor of proving ring (lb) (4)	Corrected area = $A_c = \frac{A_0}{1-\epsilon}$ (in²) (5)	Stress = $\frac{\text{Col. 4}}{\text{Col. 5}}$ (lb/in²) (6)
.01	.001569	71	22	6.50	3.38
.02	.00314	144	44.6	6.51	6.85
.03	.00471	221	68.5	6.52	10.51
.04	.00627	289	90	6.53	13.78
.05	.00784	369	114.4	6.54	17.49
.06	.00941	441	136.7	6.55	20.9
~~.08~~ .07	.01098	~~480~~ 520	161.2	6.56	24.6
.08	.01255	580	179.8	6.57	27.4
.09	.01411	681	211	6.58	32.1
.10	.01569	732	227	6.59	34.4
.11	.01725	799	248	6.60	37.6
.12	.01882	862	267	6.61	40.4
.13	.0204	920	285	6.63	43.0
.14	.0220	977	303	6.64	45.6
.15	.0235	1061	329	6.65	49.5
.16	.0251	1141	354	6.66	53.2
.17	.0267	1208	374	6.67	56.1
.18	.0282	1270	394	6.68	59.0
.19	.0298	1310	406	6.69	60.7
.20	.0314	1372	425	6.70	63.4

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UNCONFINED COMPRESSION TEST

Description of soil ______

Specimen No. 2 Location 20-22'

Moist weight of specimen ______ moisture content ______ %

Length of specimen, L 6 3/8" Diameter, D 2 7/8" Area, A_0 6.49□"

Proving ring calibration factor: 1 div. = .31

Specimen deformation = ΔL in. (1)	Vertical strain, $\epsilon = \frac{\Delta L}{L}$ (2)	Proving ring dial reading —No. of small divisions (3)	Load = Col. 3 x calibration factor of proving ring (lb) (4)	Corrected area = $A_c = \frac{A_0}{1-\epsilon}$ (in^2) (5)	Stress = $\frac{\text{Col. 4}}{\text{Col. 5}}$ (lb/in^2) (6)
.21	.0329	1422	441	6.71	65.7
.22	.0345	1463	454	6.72	67.6
.23	.0361	1501	465	6.73	69.1
.24	.0376	1530	474	6.74	70.3
.25	.0392	1567	486	6.75	72
.26	.0408	1593	494	6.77	73
.27	.0424	1610	499	6.78	73.6
.28	.0439	1630	505	6.79	74.3
.29	.0455	1640	508	6.80	74.7
.30	.0471	1651	512	6.81	75.2
.31	.0486	1655	513	6.82	75.2

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UNCONFINED COMPRESSION TEST

Description of soil

Specimen No. #2 Location 30'-32'

Moist weight of specimen _____ moisture content _____ %

Length of specimen, L 5 3/8" Diameter, D 2 7/8" Area, A_0 6.49□"

Proving ring calibration factor: 1 div. = _____

Specimen deformation = ΔL in. (1)	Vertical strain, $\epsilon = \frac{\Delta L}{L}$ (2)	Proving ring dial reading —No. of small divisions (3)	Load = Col. 3 x calibration factor of proving ring (lb) (4)	Corrected area = $A_c = \frac{A_0}{1-\epsilon}$ (in²) (5)	Stress = $\frac{\text{Col. 4}}{\text{Col. 5}}$ (lb/in²) (6)
.01	16	.001860	4.96	6.50	.763
.02	62	.00372	19.22	6.51	2.95
.03	130	.00558	40.3	6.53	6.17
.04	209	.00744	64.7	6.54	9.89
.05	293	.00930	90.8	6.55	13.86
.06	369	.01116	114.4	6.56	17.44
.07	441	.01302	136.7	6.58	20.78
.08	515	.01488	159.7	6.59	24.2
.09	591	.01674	183.2	6.60	27.8
.10	672	.01860	208	6.61	31.5
.11	752	.0205	233	6.63	35.1
.12	821	.0223	255	6.64	38.4
.13	903	.0242	280	6.65	42.1
.14	962	.0260	298	6.66	44.7
.15	1011	.0279	313	6.68	46.9
.16	1093	.0298	339	6.69	50.7
.17	1125	.0316	349	6.70	52.1
.18	1193	.0335	370	6.71	55.1
.19	1263	.0353	392	6.73	58.2
.20	1331	.0372	413	6.74	61.3

UNCONFINED COMPRESSION TEST

Description of soil ______

Specimen No. 2 Location 30'-32'

Moist weight of specimen ______ moisture content ______ %

Length of specimen, L 5 3/8" Diameter, D 2 7/8" Area, A_0 6.49□"

Proving ring calibration factor: 1 div. = .31

Specimen deformation = ΔL in. (1)	Vertical strain, $\epsilon = \frac{\Delta L}{L}$ (2)	Proving ring dial reading —No. of small divisions (3)	Load = Col. 3 x calibration factor of proving ring (lb) (4)	Corrected area = $A_c = \frac{A_0}{1-\epsilon}$ (in^2) (5)	Stress = $\frac{Col. 4}{Col. 5}$ (lb/in^2) (6)
.21	.0391	1362	422	6.75	62.5
.22	.0409	1411	437	6.77	64.5
.23	.0428	1541	478	6.78	70.5
.24	.0447	1582	490	6.79	72.2
.25	.0465	1622	503	6.81	73.9
.26	.0484	1672	518	6.82	76.0
.27	.0502	1705	529	6.83	77.5
.28	.0521	1741	540	6.85	78.8
.29	.054	1773	550	6.86	80.2
.30	.0558	1793	556	6.87	80.9
.31	.0577	1811	561	6.89	81.4
.32	.0595	1822	565	6.90	81.9
.33	.0614	1832	568	6.93	82.0
.34	.0633	1852	574	6.94	82.7
.35	.0651	1852	574	6.96	82.5
.36	.0670	1852	574	6.97	82.3

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UNCONFINED COMPRESSION TEST

Description of soil ______

Specimen No. #3 Location 35'-37'

Moist weight of specimen ______ moisture content ______ ______%

Length of specimen, L 5 5/8" Diameter, D 2 7/8" Area, A_0 6.49□"

Proving ring calibration factor: 1 div. = .31

Specimen deformation = ΔL in. (1)	Vertical strain, $\epsilon = \frac{\Delta L}{L}$ (2)	Proving ring dial reading —No. of small divisions (3)	Load = Col. 3 x calibration factor of proving ring (lb) (4)	Corrected area = $A_c = \frac{A_0}{1-\epsilon}$ (in²) (5)	Stress = $\frac{\text{Col. 4}}{\text{Col. 5}}$ (lb/in²) (6)
.01	0	.001778	0	6.50	0
.02	15	.00356	4.65	6.51	.714
.03	39	.00533	12.09	6.52	1.854
.04	83	.00711	26.7	6.54	4.08
.05	125	.00889	38.8	6.55	5.92
.06	172	.01067	53.3	6.56	8.13
.07	211	.01244	65.4	6.57	9.95
.08	242	.01422	75.0	6.58	11.40
.09	287	.016	89.0	6.60	13.48
.10	329	.01778	102.0	6.61	15.43
.11	364	.01956	112.8	6.62	17.04
.12	392	.0213	121.5	6.63	18.33
.13	427	.0231	132.4	6.64	19.94
.14	452	.0249	140.1	6.66	21.0'
.15	483	.0267	149.7	6.67	22.4
.16	508	.0284	157.5	6.68	23.6
.17	532	.0302	164.9	6.69	24.6
.18	541	.032	167.7	6.70	25.0
.19	530	.0338	164.3	6.72	24.4

UNCONFINED COMPRESSION TEST

Description of soil ______

Specimen No. B-4 Location 5'-7'

Moist weight of specimen ______ moisture content ______ %

Length of specimen, L 5 1/2'' Diameter, D 2 7/8'' Area, A_0 6.49□''

Proving ring calibration factor: 1 div. = .31

Specimen deformation $= \Delta L$ in. (1)	Vertical strain, $\epsilon = \frac{\Delta L}{L}$ (2)	Proving ring dial reading —No. of small divisions (3)	Load = Col. 3 x calibration factor of proving ring (lb) (4)	Corrected area = $A_c = \frac{A_0}{1-\epsilon}$ (in^2) (5)	Stress = $\frac{\text{Col. 4}}{\text{Col. 5}}$ (lb/in^2) (6)
.01	~~31~~ 43	.001818	13.33	6.50	2.05
.02	~~66~~ 100	.00364	.31	6.51	4.76
.03	~~101~~ 134	.00545	41.5	6.53	6.36
.04	~~135~~ 198	.00727	61.4	6.54	9.39
.05	~~167~~ 237	.00909	73.5	6.55	11.22
.06	281	.0109	87.1	6.56	13.28
.07	332	.01272	102.9	6.57	15.66
.08	372	.01455	115.3	6.59	17.50
.09	409	.01636	126.8	6.60	19.21
.10	446	.01818	136.4	6.61	20.6
.11	470	.02	145.7	6.62	22.0
.12					
.13					
.14					
.15					
.16					
.17					
.18					

179

UNCONFINED COMPRESSION TEST

Description of soil ____

Specimen No. 9 Location 20' - 22'

Moist weight of specimen ____ moisture content ____ %

Length of specimen, L 7 1/2" Diameter, D 2 7/8" Area, A_0 6.49□"

Proving ring calibration factor: 1 div. = 0.31

Specimen deformation = ΔL in. (1)	Vertical strain, $\epsilon = \frac{\Delta L}{L}$ (2)	Proving ring dial reading —No. of small divisions (3)	Load = Col. 3 x calibration factor of proving ring (lb) (4)	Corrected area = $A_c = \frac{A_0}{1-\epsilon}$ (in²) (5)	Stress = $\frac{\text{Col. 4}}{\text{Col. 5}}$ (lb/in²) (6)
.01	.001333	2'	.62	6.50	.095
.02	.002667	45	13.95	6.51	2.14
.03	.004	91	28.21	6.52	4.33
.04	.00533	142	44.02	6.52	6.75
.05	.00667	192	59.52	6.53	9.11
.06	.008	242	75.02	6.54	11.47
.07	.00933	311	96.41	6.55	14.72
.08	.01067	381	118.1	6.56	18.00
.09	.012	445	138	6.57	21.00
.10	.0133	503	155.9	6.58	23.69
.11	.01467	562	174.2	6.59	26.43
.12	.016	631	195.6	6.60	29.6
.13	.01733	687	213	6.60	32.3
.14	.01867	737	228	6.61	34.5
.15	.02	782	242	6.62	36.6
.16	.0213	824	255	6.63	38.5
.17	.0227	861	267	6.64	40.2
.18	.024	885	274	6.65	41.2

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UNCONFINED COMPRESSION TEST

Description of soil

Specimen No. 12 Location 20'-22'

Moist weight of specimen ______ moisture content ______ ______%

Length of specimen, L 5 3/8" Diameter, D 2 7/8" Area, A_0 6.49□"

Proving ring calibration factor: 1 div. = .31

Specimen deformation = ΔL in. (1)	Vertical strain, $\epsilon = \frac{\Delta L}{L}$ (2)	Proving ring dial reading —No. of small divisions (3)	Load = Col. 3 x calibration factor of proving ring (lb) (4)	Corrected area = $A_c = \frac{A_0}{1-\epsilon}$ (in²) (5)	Stress = $\frac{\text{Col. 4}}{\text{Col. 5}}$ (lb/in²) (6)
.01	.001860	41	12.71	6.50	1.96
.02	.00372	91	28.2	6.51	4.33
.03	.00558	138	42.8	6.53	6.55
.04	.00744	214	66.3	6.54	10.13
.05	.00930	283	87.7	6.55	13.39
.06	.01116	337	104.5	6.56	15.93
.07	.01302	394	122.1	6.58	18.56
.08	.01488	445	138	6.59	20.9
.09	.01674	511	158.4	6.60	24
.10	.01860	568	176.1	6.61	26.6
.11	.0205	621	192.5	6.63	29.0
.12	.0223	663	206	6.64	31.0
.13	.0242	708	219	6.65	32.9
.14	.0260	752	233	6.66	35.0
.15	.0279	817	253	6.68	37.9
.16	.0298	892	277	6.69	41.4
.17	.0316	943	292	6.70	43.6
.18	.0335	1001	310	6.71	46.2
.19	.0353	1061	329	6.73	48.9
.20	.0372	1118	347	6.74	51.5

179

UNCONFINED COMPRESSION TEST

Description of soil

Specimen No. 12 Location 20′-22′

Moist weight of specimen ______ moisture content ______ %

Length of specimen, L 5 3/8″ Diameter, D 2 7/8″ Area, A_0 6.49

Proving ring calibration factor: 1 div. = .31

Specimen deformation = ΔL in. (1)	Vertical strain, $\epsilon = \frac{\Delta L}{L}$ (2)	Proving ring dial reading —No. of small divisions (3)	Load = Col. 3 x calibration factor of proving ring (lb) (4)	Corrected area = $A_c = \frac{A_0}{1-\epsilon}$ (in^2) (5)	Stress = $\frac{\text{Col. 4}}{\text{Col. 5}}$ (lb/in^2) (6)
.21	.0391	1173	364	6.75	53.9
.22	.0409	1231	382	6.77	56.4
.23	.0428	1292	401	6.78	59.1
.24	.0447	1340	415	6.79	61.1
.25	.0465	1381	428	6.81	62.8
.26	.0484	1411	437	6.82	64.1
.27	.0502	1433	444	6.83	65.0
.28	.0521	1482	459	6.85	67.0
.29	.054	1493	463	6.86	67.5
.30	.0558	1510	468	6.87	68.1
.31	.0577	1530	474	6.89	68.8
.32	.0595	1551	481	6.90	69.7
.33	.0614	1569	486	6.91	70.3
.34	.0633	1572	487	6.93	70.3
.35	.0651	1572	487	6.94	70.1
.36	.0670	1572	487	6.96	70.0
.37	.0688	1583	491	6.97	70.4
.38	.0707	1583	491	6.98	70.3
.39	.0726	1579	489	7.00	69.9
.40	.0744	1510	468	7.01	66.8

179

UNCONFINED COMPRESSION TEST

Description of soil ______

Specimen No. 13 Location 20-22'

Moist weight of specimen ______ moisture content ______ ______%

Length of specimen, L 5 3/8" Diameter, D 2 7/8" Area, A_0 6.49□"

Proving ring calibration factor: 1 div. = .31

Specimen deformation = ΔL in. (1)	Vertical strain, $\epsilon = \frac{\Delta L}{L}$ (2)	Proving ring dial reading —No. of small divisions (3)	Load = Col. 3 x calibration factor of proving ring (lb) (4)	Corrected area = $A_c = \frac{A_0}{1-\epsilon}$ (in²) (5)	Stress = $\frac{\text{Col. 4}}{\text{Col. 5}}$ (lb/in²) (6)
.01	~~11~~ 96	.001860	27.9	6.50	4.29
.02	~~60~~ 160	.00372	49.6	6.51	7.62
.03	~~128~~ 260	.00558	80.6	6.53	12.34
.04	~~236~~ 340	.00744	105.4	6.54	16.11
.05	~~376~~ 410	.00936	127.1	6.55	19.40
.06	~~410~~ 530	.01116	155	6.56	23.6
.07	580	.01302	179.8	6.58	27.3
.08					
.09					
.10					
.11					
.12					
.13					
.14					
.15					
.16					
.17					
.18					

UNCONFINED COMPRESSION TEST

Description of soil

Specimen No. 13 Location 25-27'

Moist weight of specimen moisture content %

Length of specimen, L_o 4 5/8 Diameter, D 2 7/8" Area, A_0 6.49 in²

Proving ring calibration factor: 1 div. =

.31 cal factor

Specimen deformation $= \Delta L$ in. (1)		Vertical strain, $\epsilon = \frac{\Delta L}{L}$ (2)	Proving ring dial reading —No. of small divisions (3)	Load = Col. 3 x calibration factor of proving ring (lb) (4)	Corrected area = $A_c = \frac{A_0}{1-\epsilon}$ (in²) (5)	Stress = $\frac{\text{Col. 4}}{\text{Col. 5}}$ (lb/in²) (6)
.01	10	0.0022	1.0	0.31	6.504	0.048
.02	20	0.0043	13	4.03	6.518	0.62
.03	30	0.0065	24	7.44	6.532	1.14
.04	40	0.0086	35	10.85	6.546	1.66
.06	60	0.013	62	19.22	6.575	2.92
.08	80	0.017	84	26.04	6.602	3.94
.10	100	0.022	115	35.65	6.636	5.37
.12	120	0.026	144	44.64	6.663	6.70
.14	140	0.030	173	53.63	6.690	8.02
.16	160	0.036	195	60.45	6.732	8.98
.18	180	0.039	2~~0~~2	65.72	6.753	9.73
.20	200	0.043	224	69.44	6.782	10.24
.24	240	0.052	232	71.92	6.846	10.505
.28	280	0.060	215	66.65	6.904	9.654
.32	320					
.36	360					

179

UNCONFINED COMPRESSION TEST

Description of soil ____________

Specimen No. 23 Location 10'-12'

Moist weight of specimen ________ moisture content ________ ____%

Length of specimen, L 6 3/4" Diameter, D 2 7/8" Area, A_0 6.49 in^2

Proving ring calibration factor: 1 div. = .31

Specimen deformation = ΔL in. (1)	Vertical strain, $\epsilon = \frac{\Delta L}{L}$ (2)	Proving ring dial reading —No. of small divisions (3)	Load = Col. 3 x calibration factor of proving ring (lb) (4)	Corrected area = $A_c = \frac{A_0}{1-\epsilon}$ (in^2) (5)	Stress = $\frac{\text{Col. 4}}{\text{Col. 5}}$ (lb/in^2) (6)
.01	.001481	55	17.05	6.50	2.62
.02	.00296	106	32.9	6.51	5.05
.03	.00444	156	48.4	6.52	7.42
.04	.00593	209	64.8	6.53	9.92
.05	.00741	268	83.1	6.54	12.71
.06	.00889	341	105.7	6.55	16.14
.07	.01037	402	124.6	6.56	18.99
.08	.01185	472	146.3	6.57	22.3
.09	.01333	528	163.7	6.58	24.9
.10	.01481	592	183.5	6.59	27.8
.11	.01630	628	194.7	6.60	29.5
.12	.01778	692	215	6.61	32.5
.13	.01926	751	233	6.62	35.2
.14	.0207	751	233	6.63	35.1
.15	.0222	782	242	6.64	36.4
.16	.0237	811	251	6.65	37.7
.17	.0252	881	273	6.66	41.0
.18	.0267	952	295	6.67	44.2
.19	.0281	1012	314	6.68	47.0
.20	.0296	1071	332	6.69	49.6

179

UNCONFINED COMPRESSION TEST

Description of soil ______

Specimen No. 23 Location 10' - 12'

Moist weight of specimen ______ moisture content ______ %

Length of specimen, L 6 3/4" Diameter, D 2 7/8" Area, A_0 6.49□"

Proving ring calibration factor: 1 div. = .31

Specimen deformation = ΔL in. (1)	Vertical strain, $\epsilon = \frac{\Delta L}{L}$ (2)	Proving ring dial reading —No. of small divisions (3)	Load = Col. 3 x calibration factor of proving ring (lb) (4)	Corrected area = $A_c = \frac{A_0}{1-\epsilon}$ (in²) (5)	Stress = $\frac{\text{Col. 4}}{\text{Col. 5}}$ (lb/in²) (6)
.41	.0607	1538	477	6.91	69.0
.42	.0622	1540	477	6.92	68.9
.43	.0637	1548	480	6.93	69.3
.44	.0652	1551	481	6.94	69.3
.45	.0667	1562	484	6.95	69.6
.46	.0681	1571	487	6.96	70.0
.47	.0696	1562	484	6.98	69.3

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UNCONFINED COMPRESSION TEST

Description of soil ____________________

Specimen No. 23 Location 10'-12'

Moist weight of specimen ________ moisture content ________ ____%

Length of specimen, L 6 3/4" Diameter, D 2 7/8" Area, A_0 6.49□"

Proving ring calibration factor: 1 div. = .31

Specimen deformation = ΔL in. (1)	Vertical strain, $\epsilon = \frac{\Delta L}{L}$ (2)	Proving ring dial reading —No. of small divisions (3)	Load = Col. 3 x calibration factor of proving ring (lb) (4)	Corrected area = $A_c = \frac{A_0}{1-\epsilon}$ (in^2) (5)	Stress = $\frac{\text{Col. 4}}{\text{Col. 5}}$ (lb/in^2) (6)
.21	.0311	1130	350	6.70	52.2
.22	.0326	1181	366	6.71	54.5
.23	.0341	1230	381	6.72	56.7
.24	.0356	1272	394	6.73	58.5
.25	.0370	1310	406	6.74	60.2
.26	.0385	1348	418	6.75	61.9
.27	.04	1362	422	6.76	62.4
.28	.0415	1382	428	6.77	63.2
.29	.0430	1410	437	6.78	64.5
.30	.0444	1421	441	6.79	64.9
.31	.0459	1432	444	6.80	65.3
.32	.0474	1432	441	6.81	64.8
.33	.0489	1451	450	6.82	66.0
.34	.0504	1462	453	6.83	66.3
.35	.0519	1482	459	6.85	67.0
.36	.0533	1492	463	6.86	67.5
.37	.0548	1502	466	6.87	67.8
.38	.0563	1512	469	6.88	68.2
.39	.0578	1522	472	6.89	68.5
.40	.0593	1528	474	6.90	68.7

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UNCONFINED COMPRESSION TEST

Description of soil ______
Specimen No. #26 Location 30-32'
Moist weight of specimen ______ moisture content ______ %
Length of specimen, L 6 5/8" Diameter, D 2 7/8" Area, A_0 6.49 in²
Proving ring calibration factor: 1 div. = .31

Specimen deformation = ΔL in. (1)	Vertical strain, $\epsilon = \frac{\Delta L}{L}$ (2)	Proving ring dial reading —No. of small divisions (3)	Load = Col. 3 x calibration factor of proving ring (lb) (4)	Corrected area = $A_c = \frac{A_0}{1-\epsilon}$ (in²) (5)	Stress = $\frac{\text{Col. 4}}{\text{Col. 5}}$ (lb/in²) (6)
.01	41	.001509	12.71	6.50	1.955
.02	83	.00302	25.7	6.51	3.95
.03	121	.00453	37.5	6.52	5.75
.04	165	.00604	51.2	6.53	7.84
.05	201	.00755	62.3	6.54	9.53
.06	231	.00906	71.6	6.55	10.93
.07	263	.01057	81.5	6.56	12.42
.08	293	.01208	90.8	6.57	13.82
.09	321	.01358	99.5	6.58	15.12
.10	354	.01509	109.7	6.59	16.65
.11	371	.01660	115	6.60	17.42
.12	392	.01811	121.5	6.61	18.38
.13	425	.01962	131.8	6.62	19.91
.14	460	.0211	142.6	6.63	21.51
.15	475	.0226	147.3	6.64	22.18
.16	502	.0242	155.6	6.65	23.40
.17	511	.0257	158.4	6.66	23.78
.18	531	.0272	164.6	6.67	24.7
.19	550	.0287	170.5	6.68	25.5
.20	569	.0302	176.1	6.69	26.3

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UNCONFINED COMPRESSION TEST

Description of soil ____________

Specimen No. 26 Location 30-32'

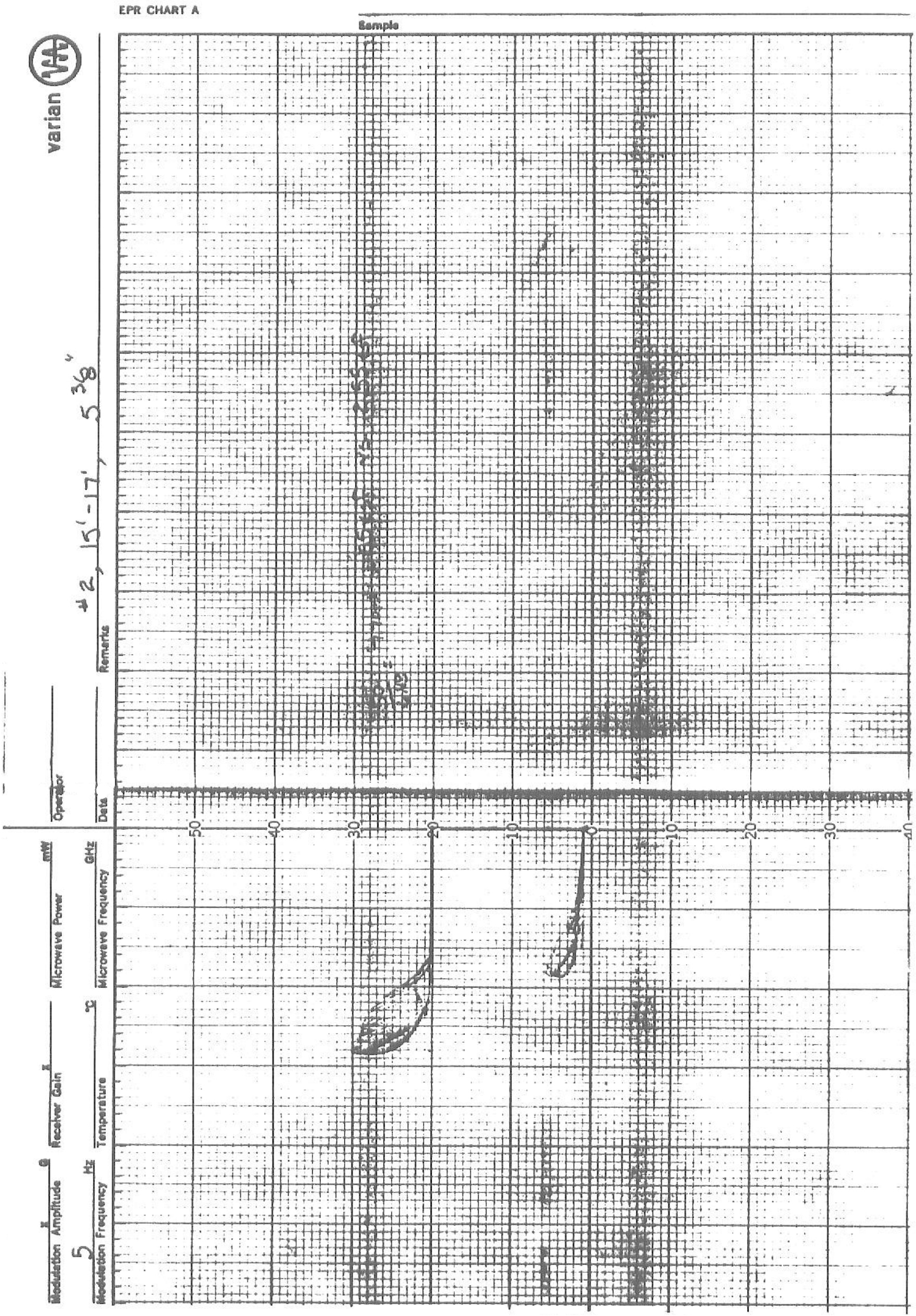
Moist weight of specimen ______ moisture content ______ ____%

Length of specimen, L 6 5/8'' Diameter, D 2 7/8'' Area, A_0 6.490''

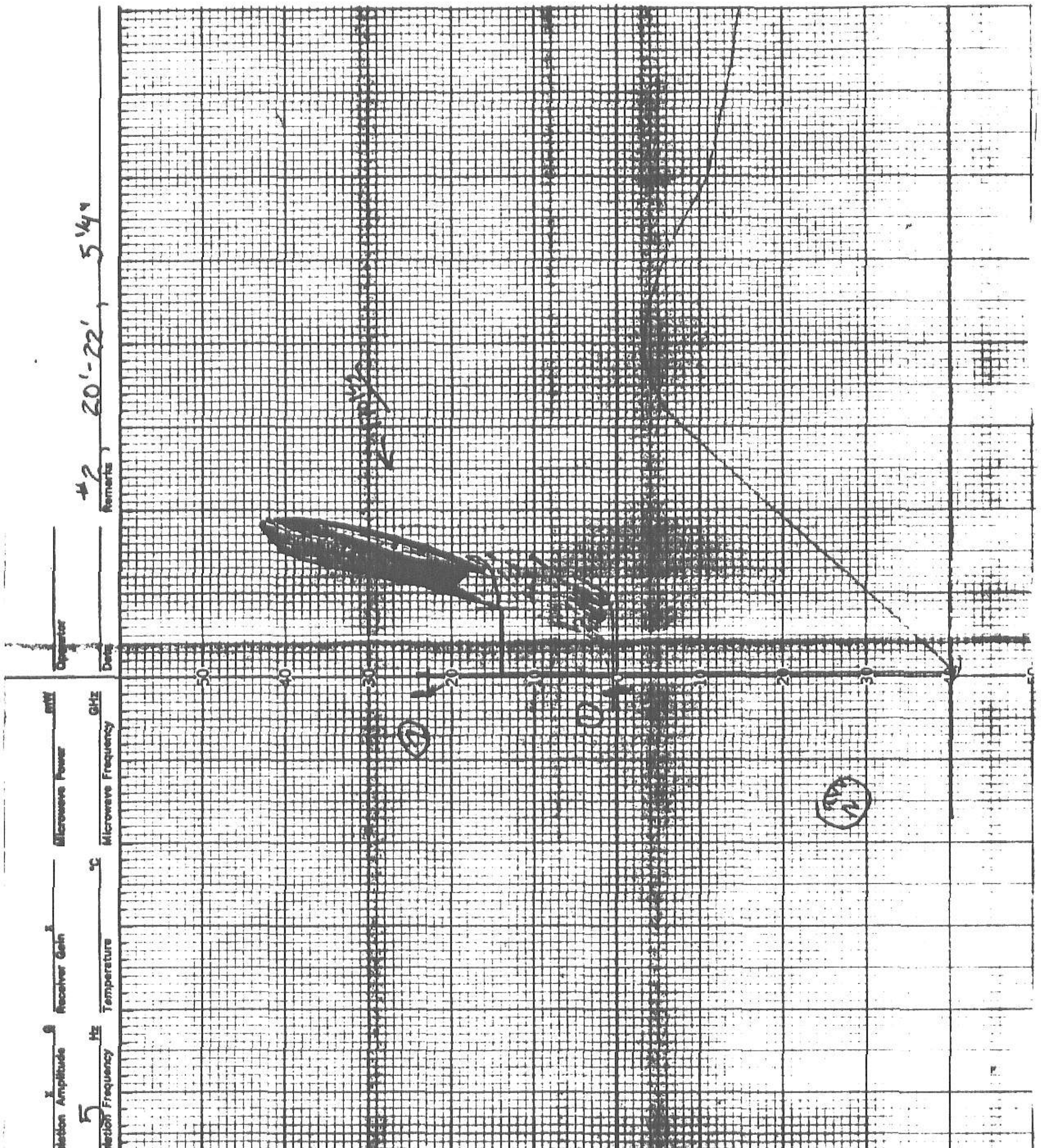
Proving ring calibration factor: 1 div. = .31

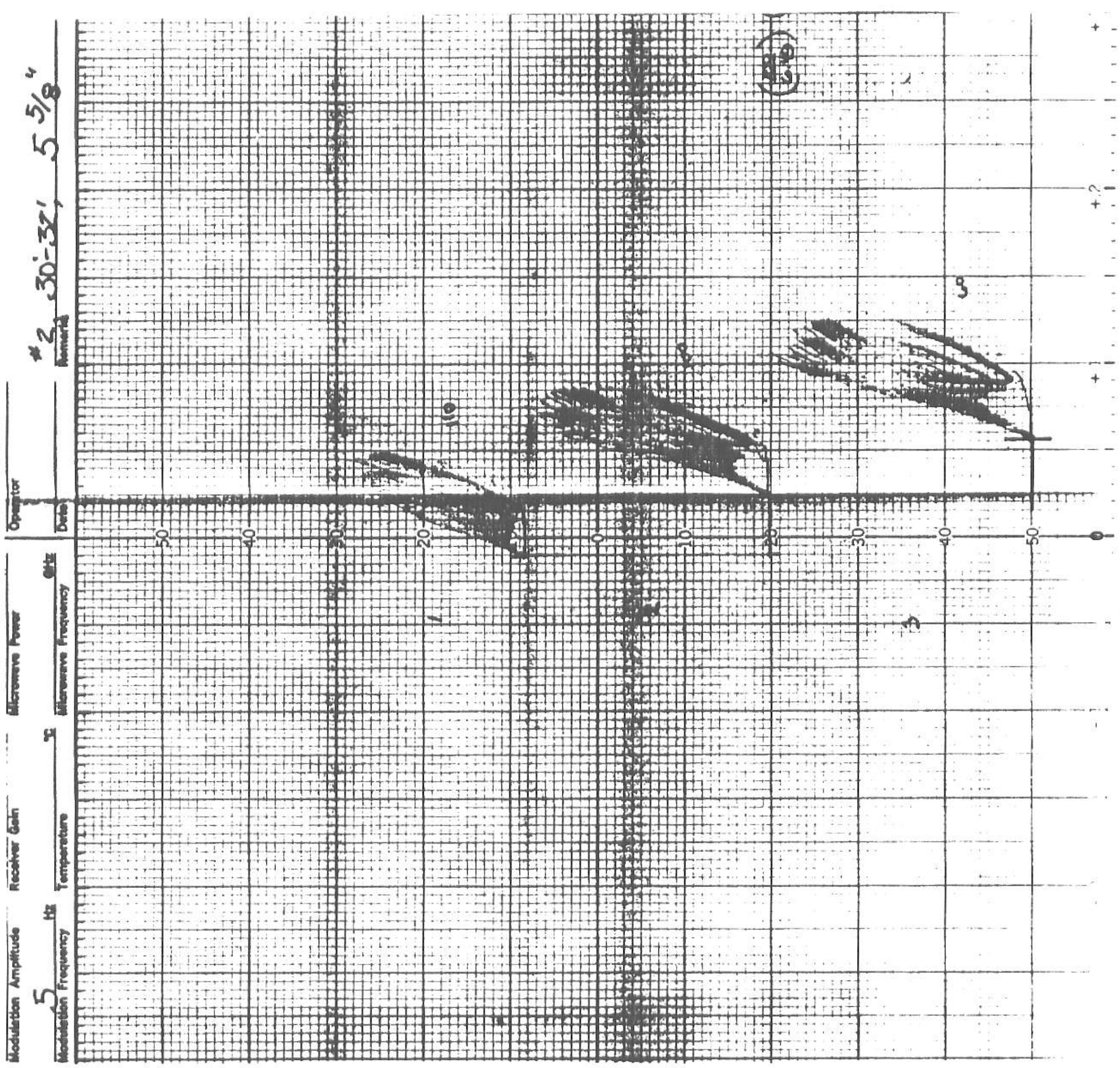
Specimen deformation = ΔL in. (1)	Vertical strain, $\epsilon = \frac{\Delta L}{L}$ (2)	Proving ring dial reading —No. of small divisions (3)	Load = Col. 3 x calibration factor of proving ring (lb) (4)	Corrected area = $A_c = \frac{A_0}{1-\epsilon}$ (in^2) (5)	Stress = $\frac{\text{Col. 4}}{\text{Col. 5}}$ (lb/in^2) (6)
.21	.0317	581	180.1	6.70	26.9
.22	.0332	593	183.8	6.71	27.4
.23	.0347	592	183.5	6.72	27.3
.24	.0362	613	190.0	6.73	28.2
.25	.0377	631	195.6	6.74	29.0
.258	.0389	609	188.8	6.75	28.0

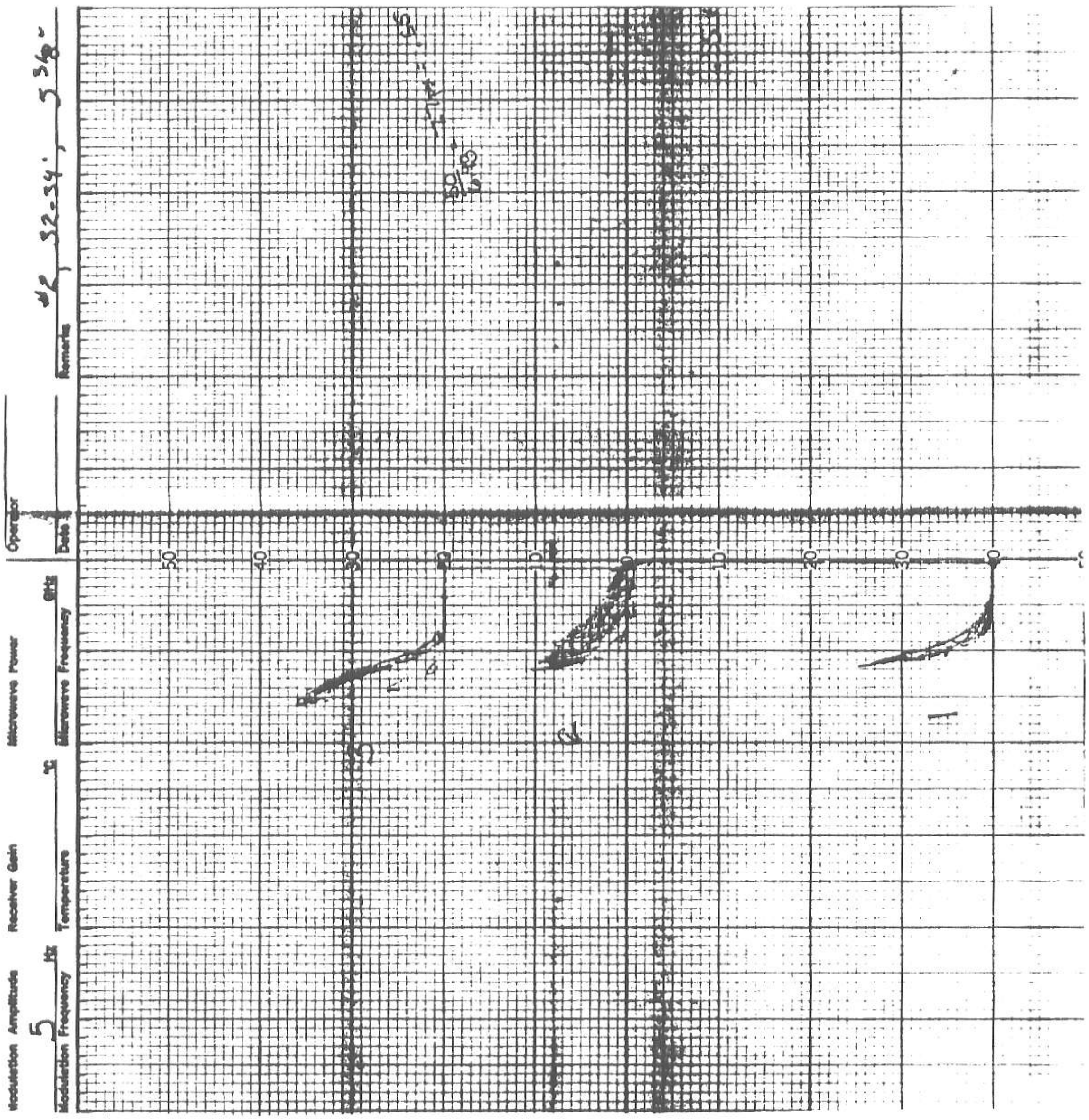
APPENDIX C
MTS Data Sheets

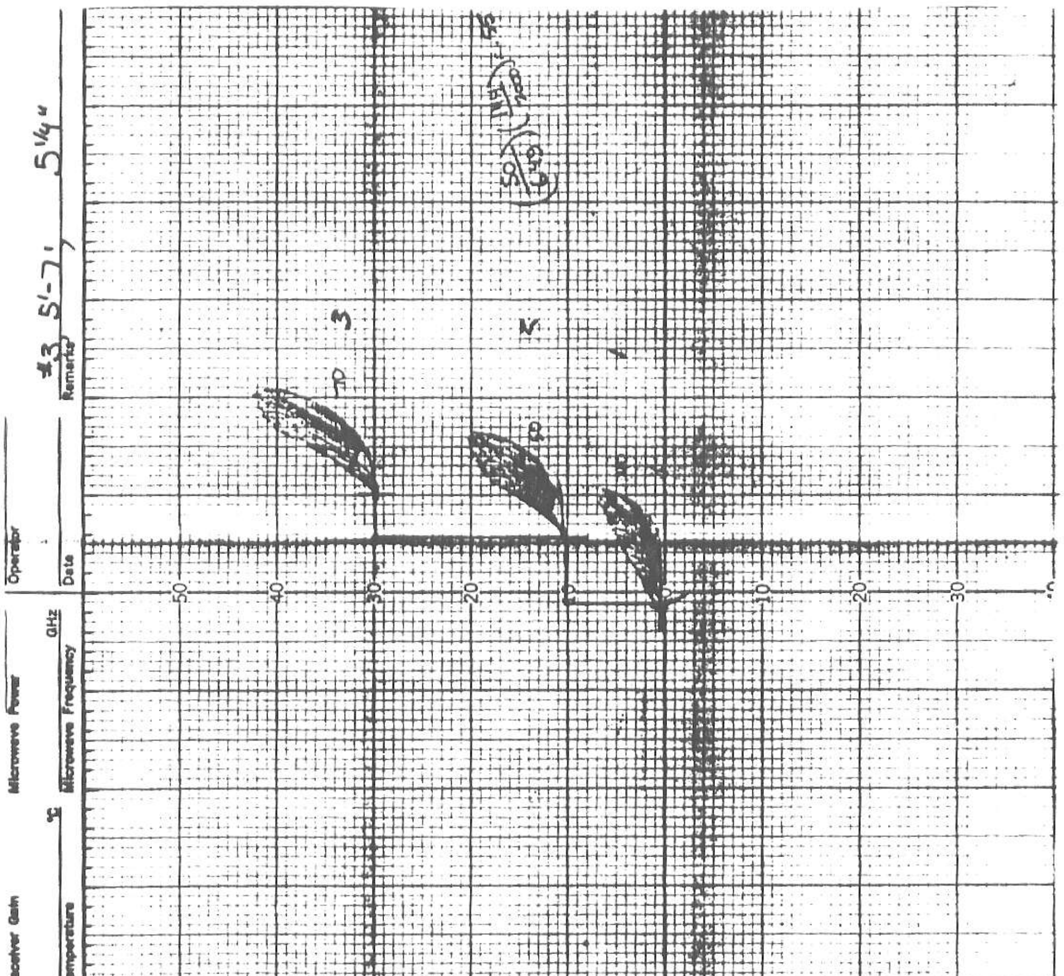


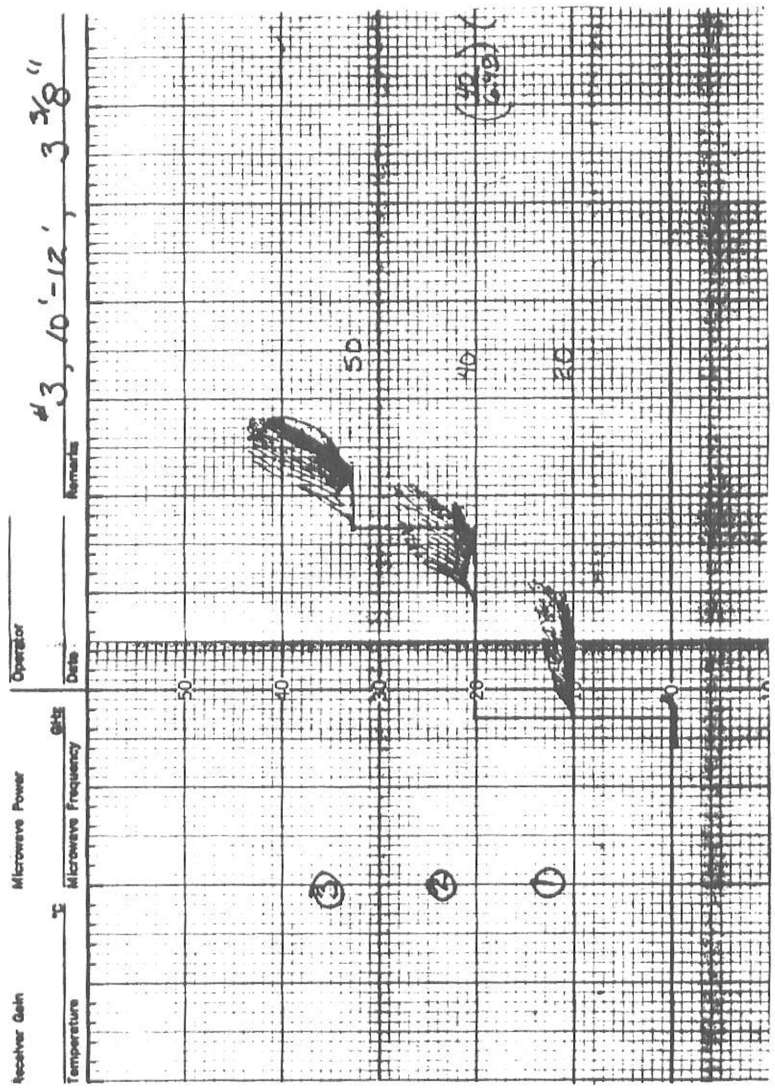
EPR CHART A
Sample
varian
#2, 15'-17', 5 3/8"
Remarks
Operator
Date
Modulation Amplitude G
Receiver Gain
Microwave Power mW
5
Modulation Frequency Hz
Temperature °C
Microwave Frequency GHz

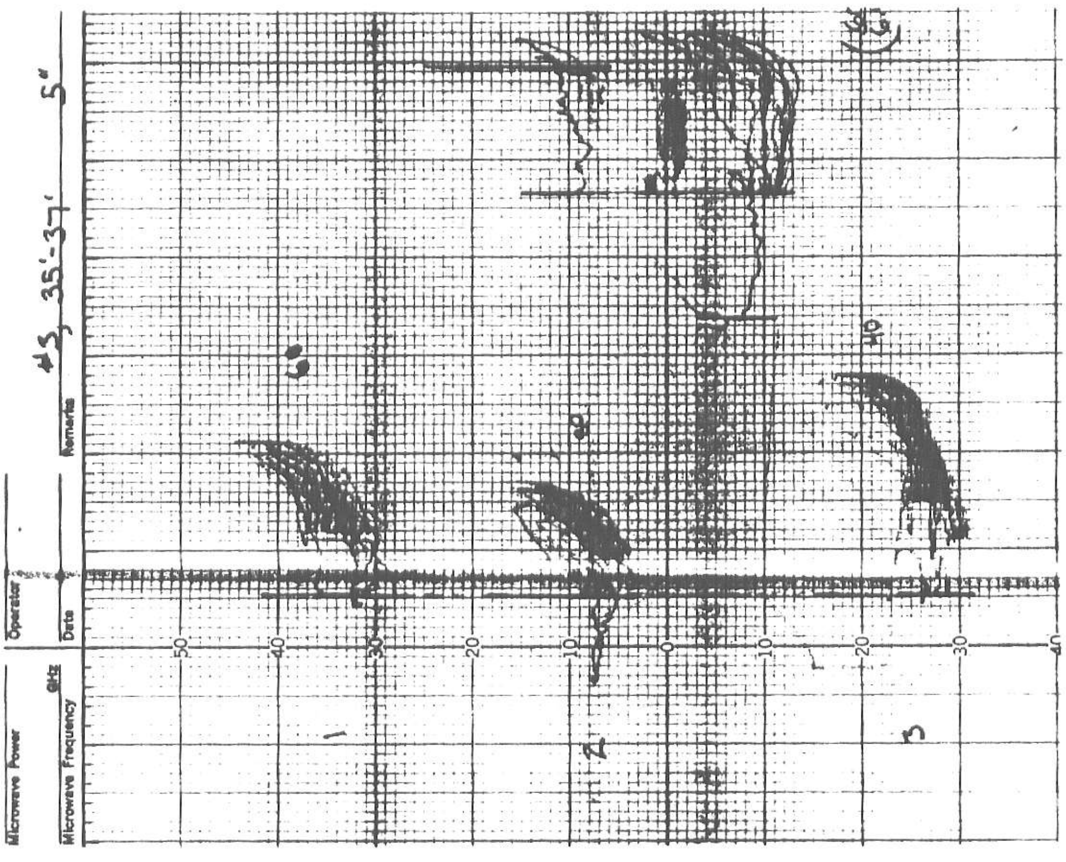

Modulation Amplitude
Modulation Frequency
5
Receiver Gain
Temperature
Microwave Power
Microwave Frequency
GHz
Operator
Date
Remarks
#2 20'-22', 5 1/4"
50
40
30
20
10
0
10
20
30

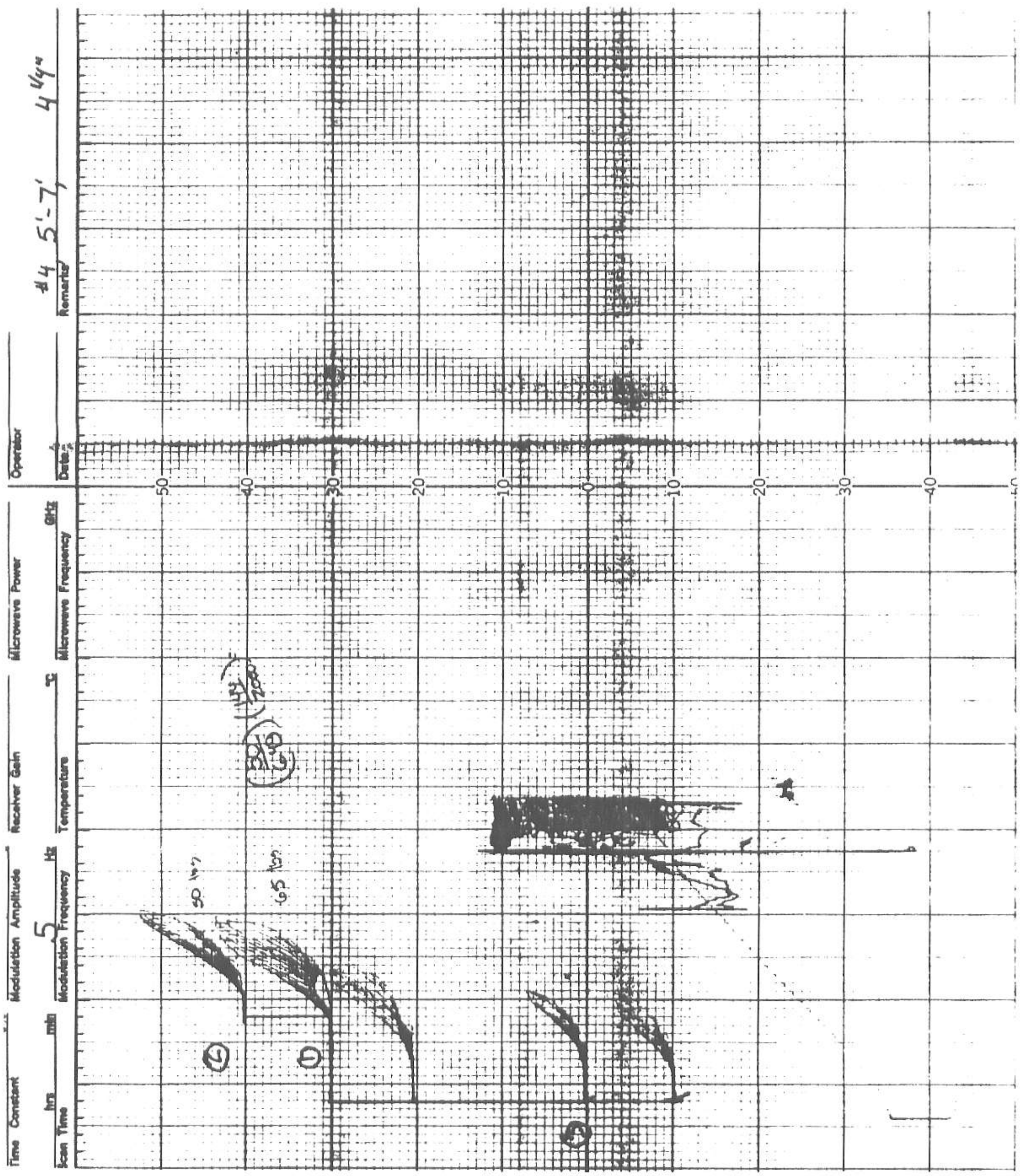

Modulation Amplitude
5
Modulation Frequency Hz
Receiver Gain
Temperature °C
Microwave Power
Microwave Frequency GHz
Operator
Date
Remarks
#2 30'-32', 5 5/8"
50
40
30
20
10
0
10
20
30
40
50

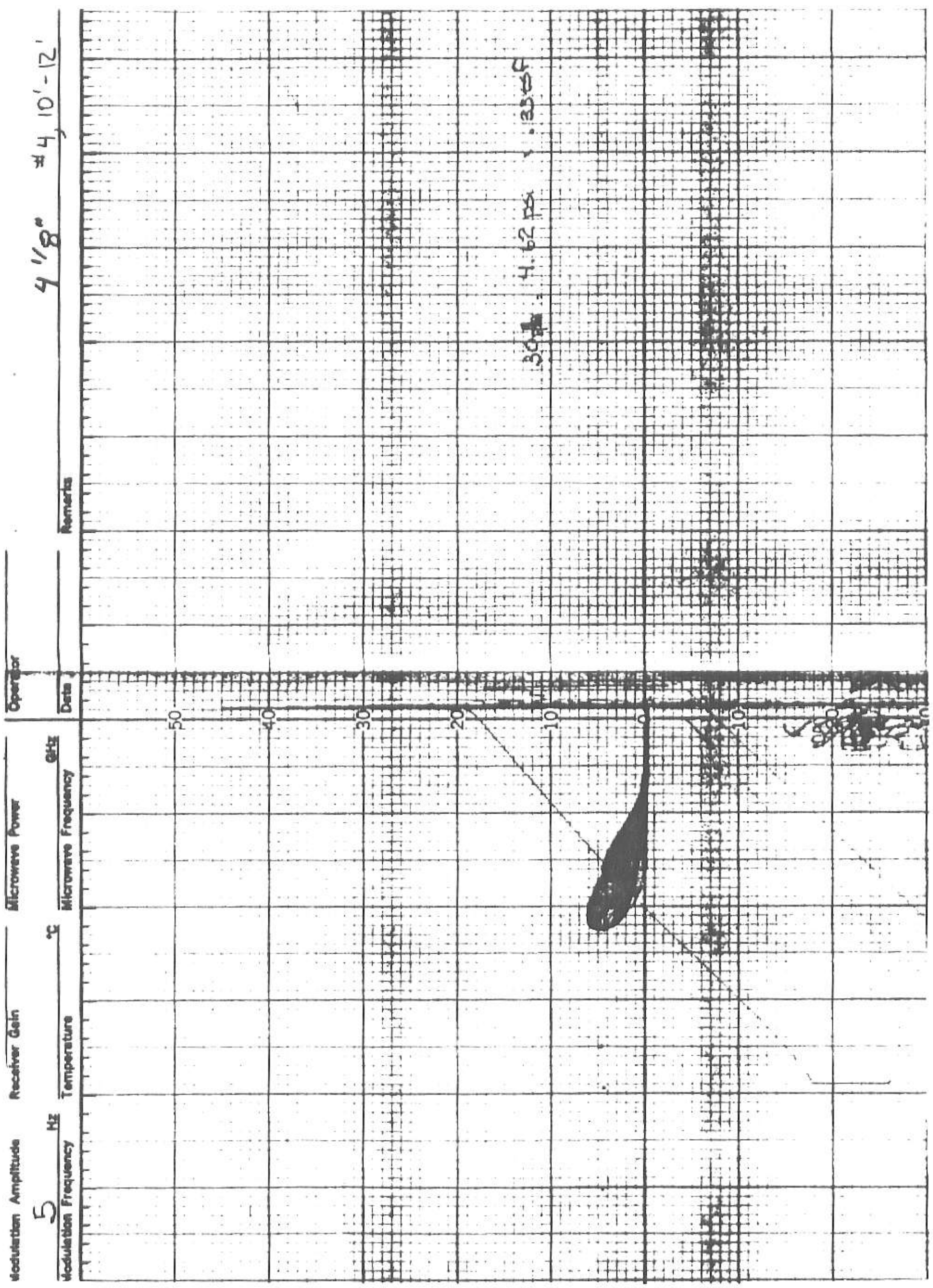



Modulation Amplitude
5
Modulation Frequency
Hz
Receiver Gain
Temperature
°C
Microwave Power
Microwave Frequency
GHz
Operator
Date
Remarks
#3 5'-7' 5 1/4"
50
40
30
20
10
10
20
30

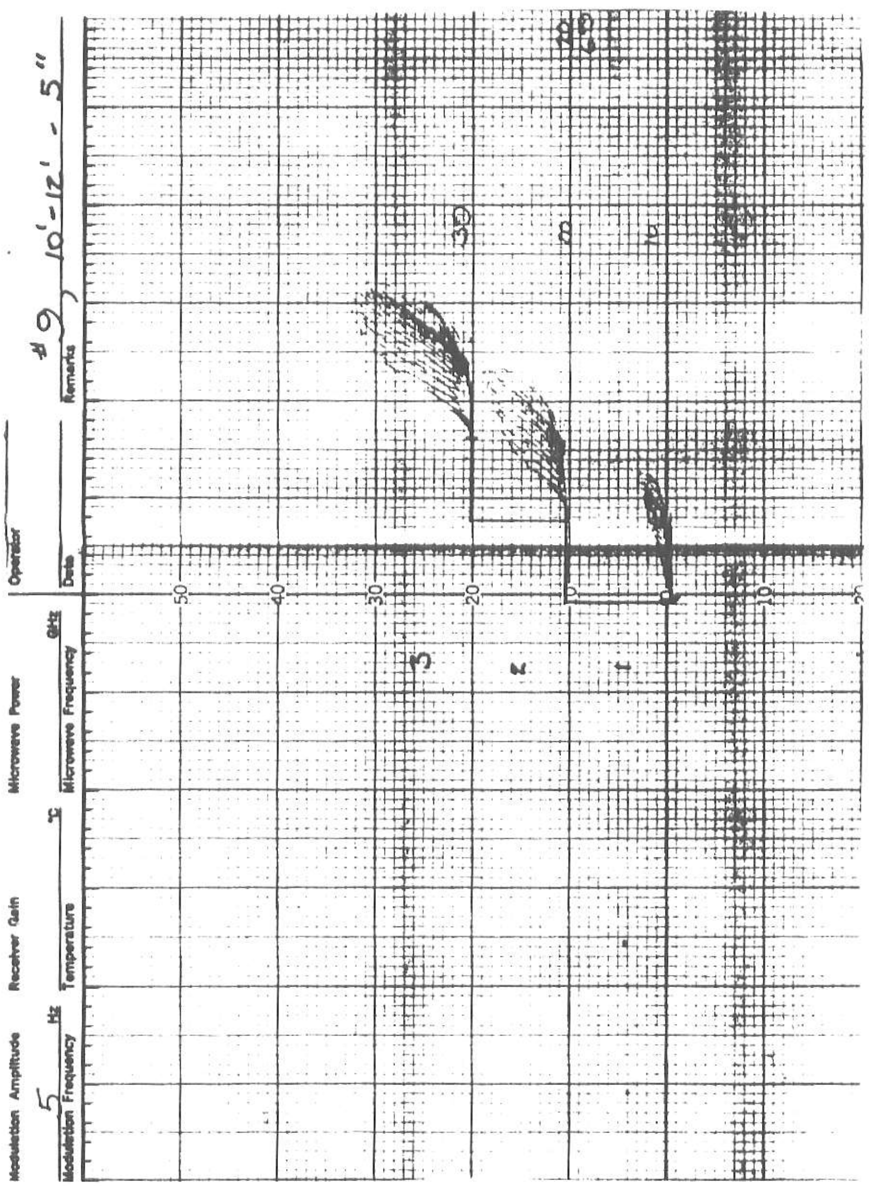

Receiver Gain
Temperature
Microwave Power
Microwave Frequency
GHz
Operator
Date
Remarks
#3, 10'-12', 3 3/8"

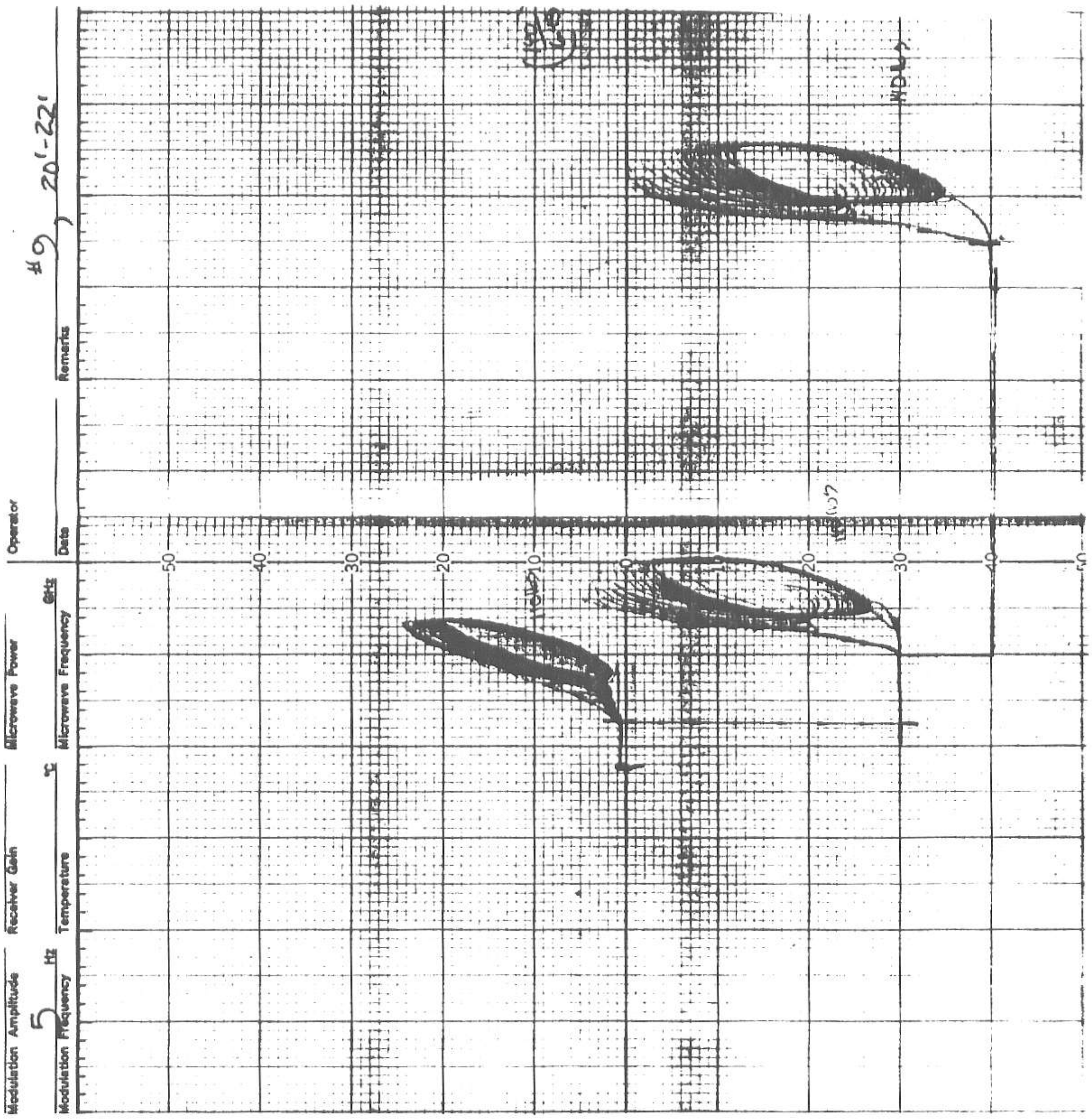

Microwave Power
Microwave Frequency GHz
Operator
Date
Remarks
#3, 35'-37' 5"
50
40
30
20
10
0
10
20
30
40
1
2
3

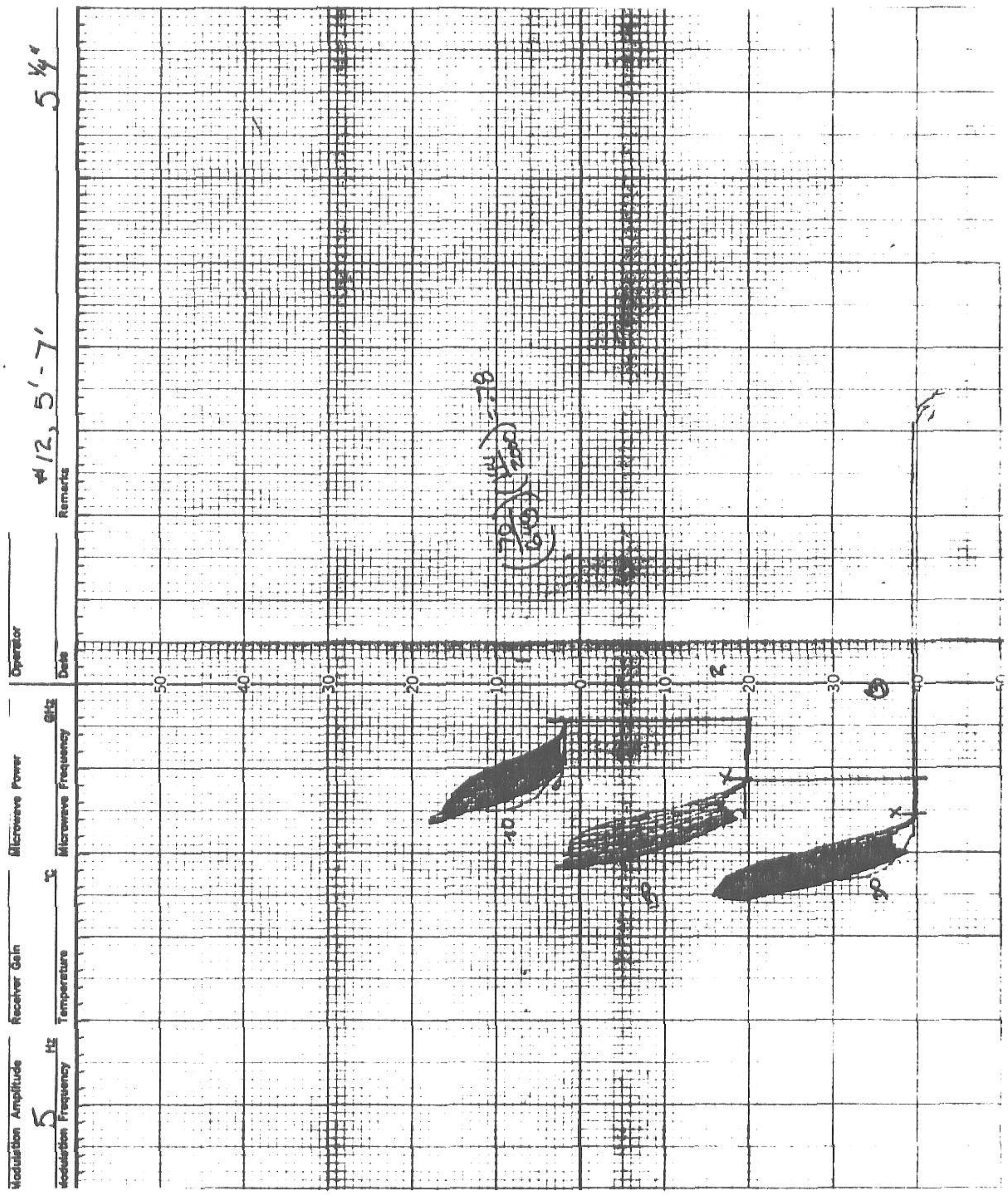

Time Constant
Scan Time
hrs
min
Modulation Amplitude
5
Modulation Frequency
Hz
Receiver Gain
Temperature
Microwave Power
Microwave Frequency
GHz
Operator
Date
Remarks
#4 5'-7' 4 1/4"
50
40
30
20
10
0
10
20
30
40



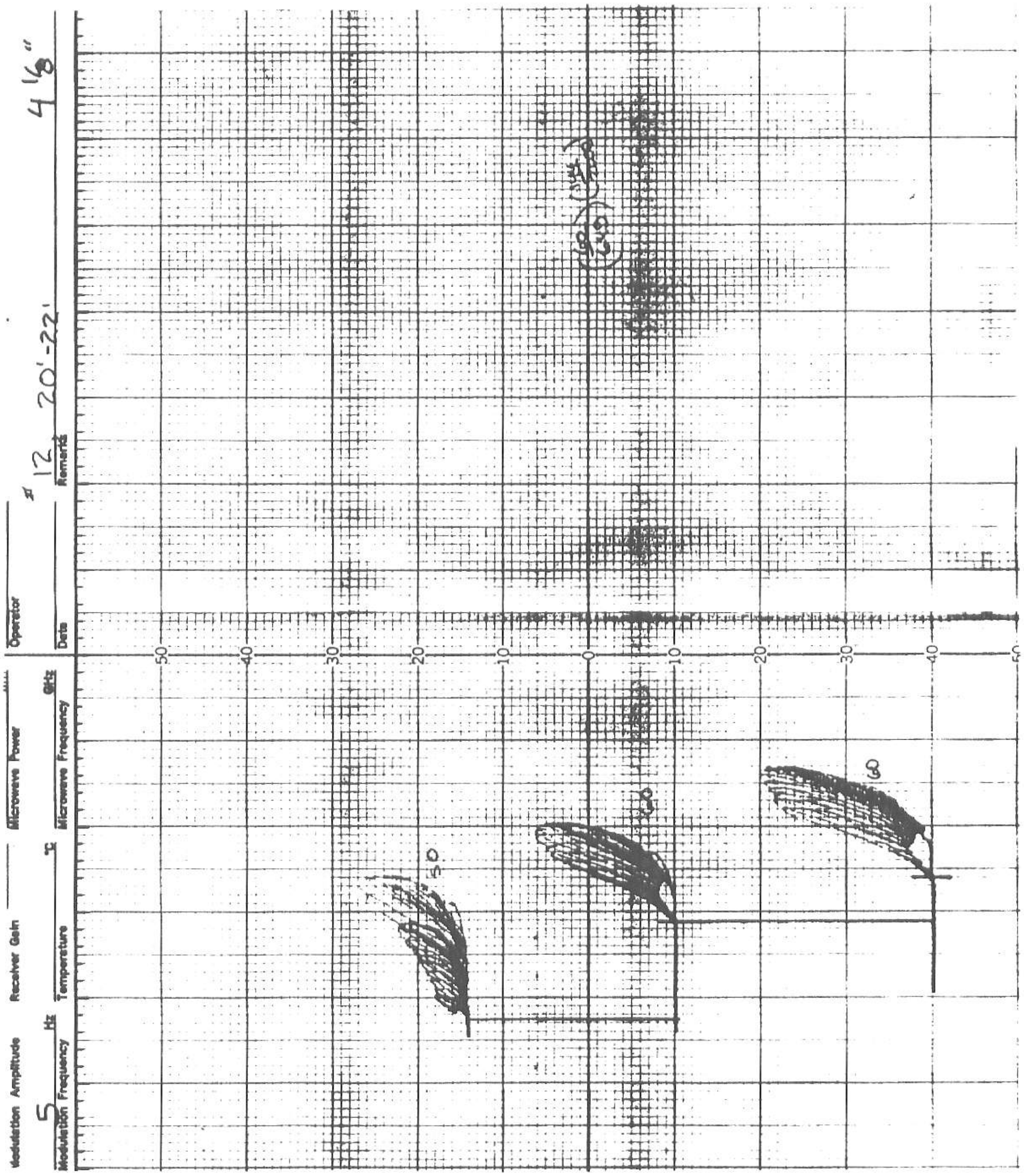
Modulation Amplitude
5
Modulation Frequency
Hz
Receiver Gain
Temperature
°C
Microwave Power
Microwave Frequency
GHz
Operator
Date
Remarks
4 1/8" #4, 10'-12'
50
40
30
20
10
10
20

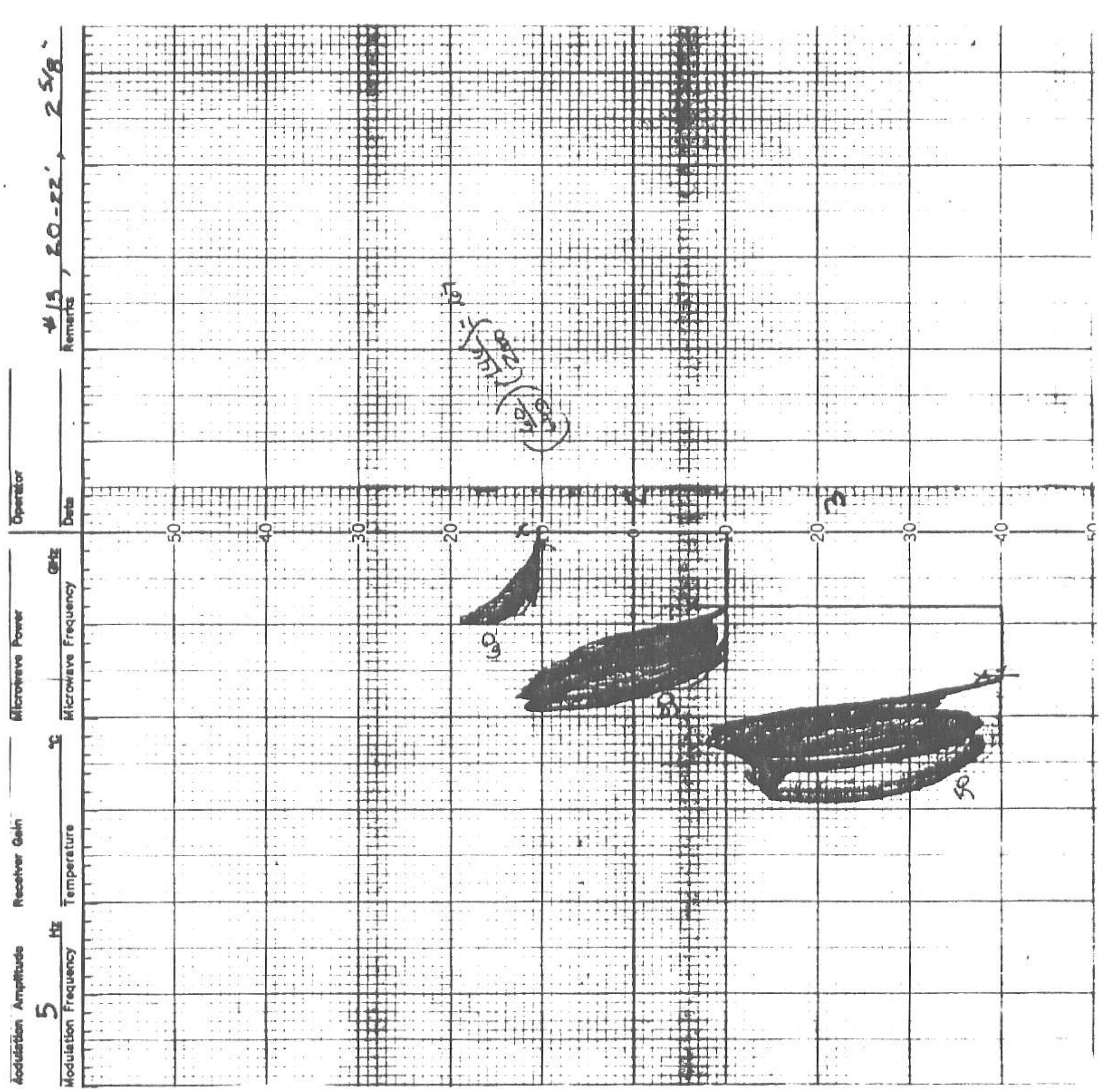

Modulation Amplitude
5
Modulation Frequency
Hz
Receiver Gain
Temperature
°C
Microwave Power
Microwave Frequency
GHz
Operator
Date
Remarks
#9 10'-12' - 5"
50
40
30
20
10

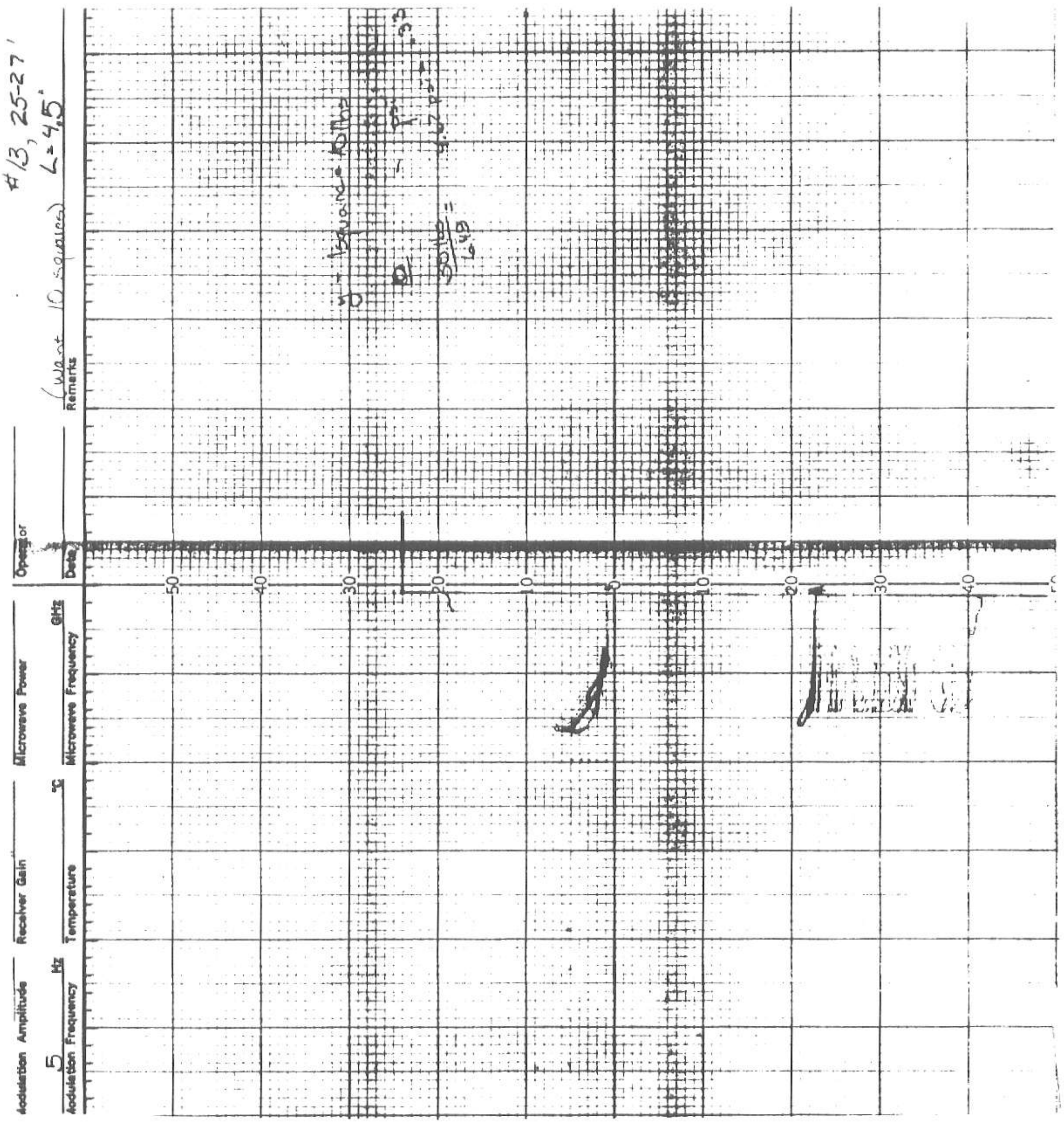

Modulation Amplitude
5
Modulation Frequency
Hz
Receiver Gain
Temperature
°C
Microwave Power
Microwave Frequency
GHz
Operator
Date
Remarks
#9, 20'-22'
50
40
30
20
10
0
10
20
30
40
50

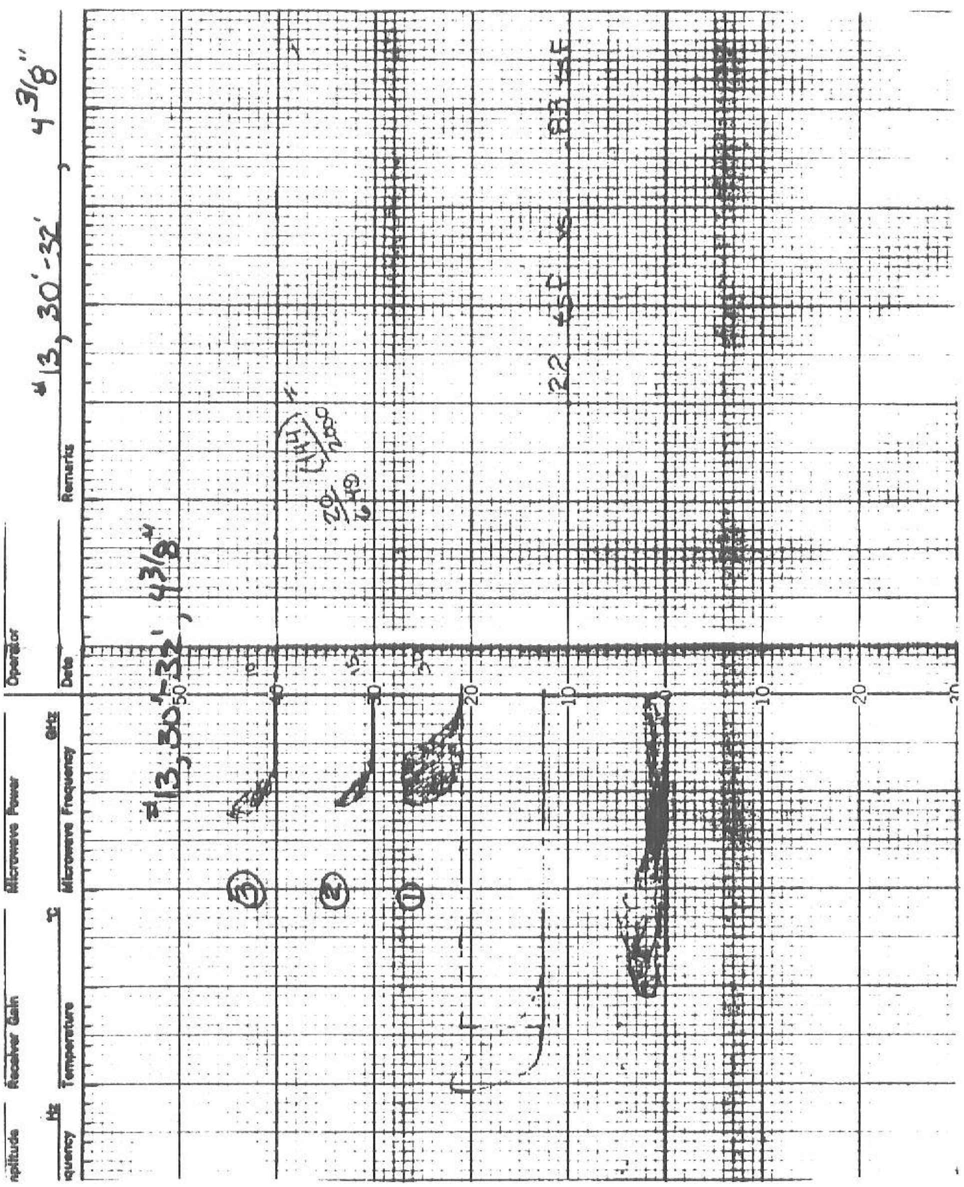


Modulation Amplitude
5
Modulation Frequency
Hz
Receiver Gain
Temperature
°C
Microwave Power
Microwave Frequency
GHz
Operator
Date
Remarks
#12, 5'-7'
5 1/4"

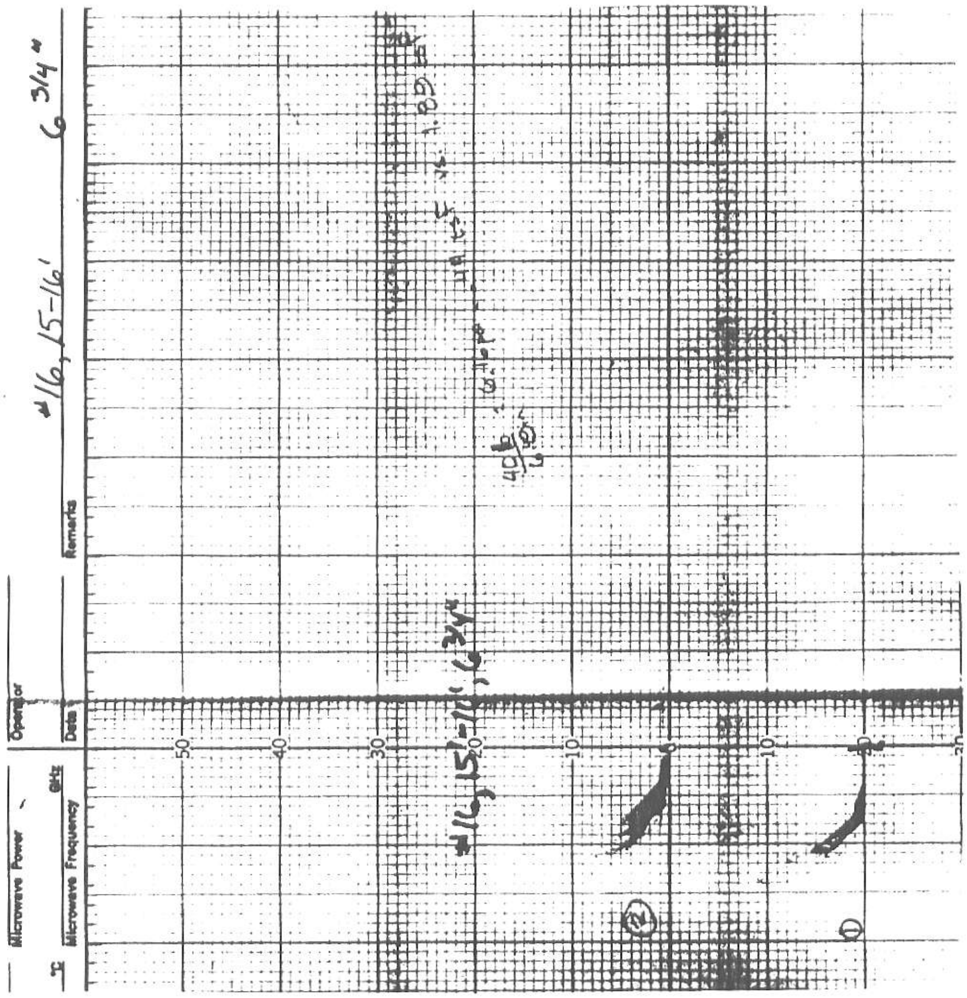

Modulation Amplitude
Receiver Gain
Microwave Power
Operator
Modulation Frequency
Hz
Temperature
K
Microwave Frequency
GHz
Date
Remarks
5
12
20'-22'
4 1/8"
50
40
30
20
10
0
10
20
30
40
50


Modulation Amplitude
5
Receiver Gain
Microwave Power
Operator
Modulation Frequency Hz
Temperature °C
Microwave Frequency GHz
Date
Remarks
#13, 20-22', 2 5/8"
50
40
30
20
10
0
10
20
30
40
50


Modulation Amplitude
5
Modulation Frequency
Hz
Receiver Gain
Temperature
°C
Microwave Power
Microwave Frequency
GHz
Operator
Date
Remarks
(want 10 squares)
#13, 25-27'
L=4.5'
50
40
30
20
10
5
10
20
30
40



Remarks
Operator
Date
Microwave Power
Microwave Frequency
Receiver Gain
Temperature
#13, 30'-32', 4 3/8"


Microwave Power
Microwave Frequency
GHz
Operator
Date
Remarks
#16, 15-16'
6 3/4"
#16, 15'-16', 6 3/4"
50
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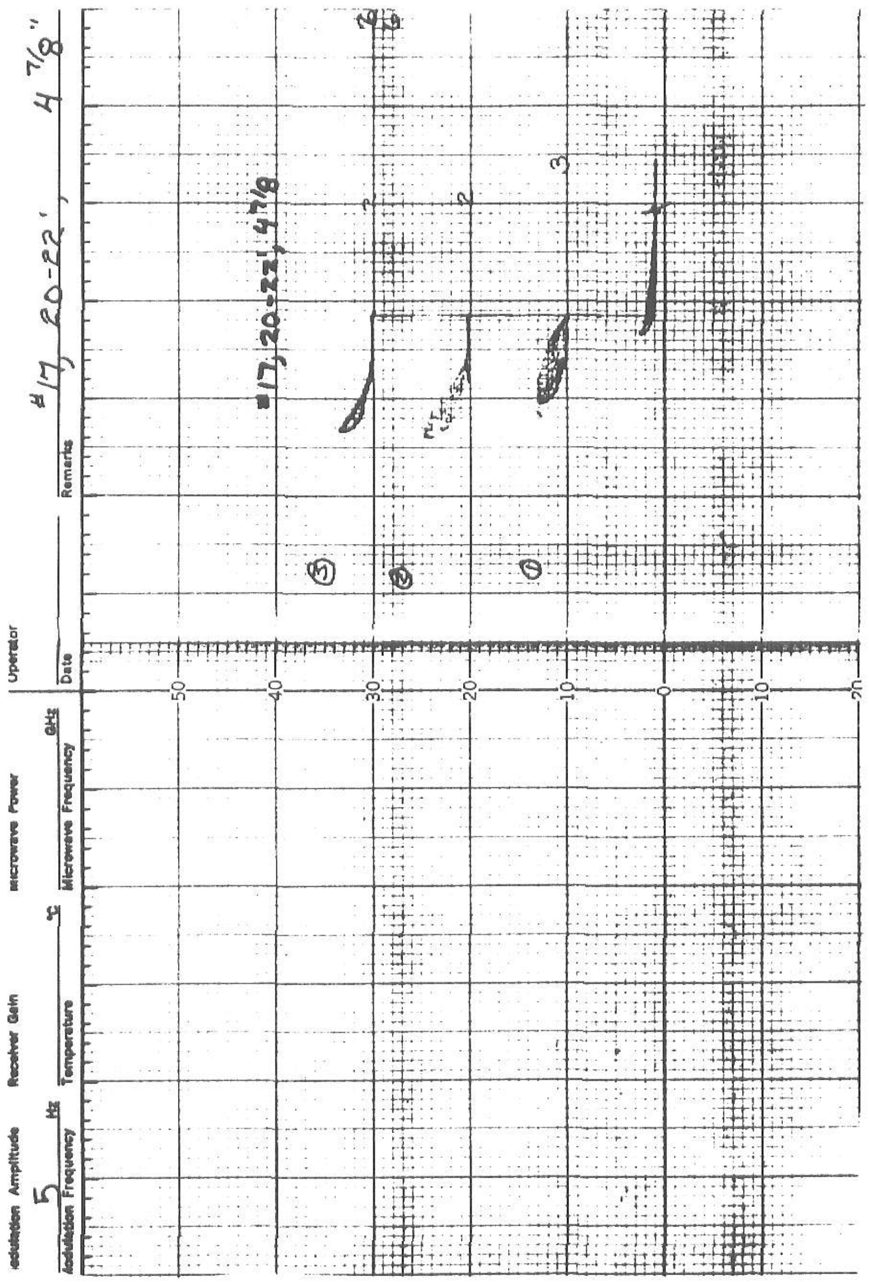
10
0
10
20
30


Amplitude
5
Hz
Receiver Gain
Temperature
Microwave Power
Microwave Frequency
GHz
Operator
Date
Remarks
#17, 20-22', 4 7/8"
#17, 20-22', 4 7/8
50
40
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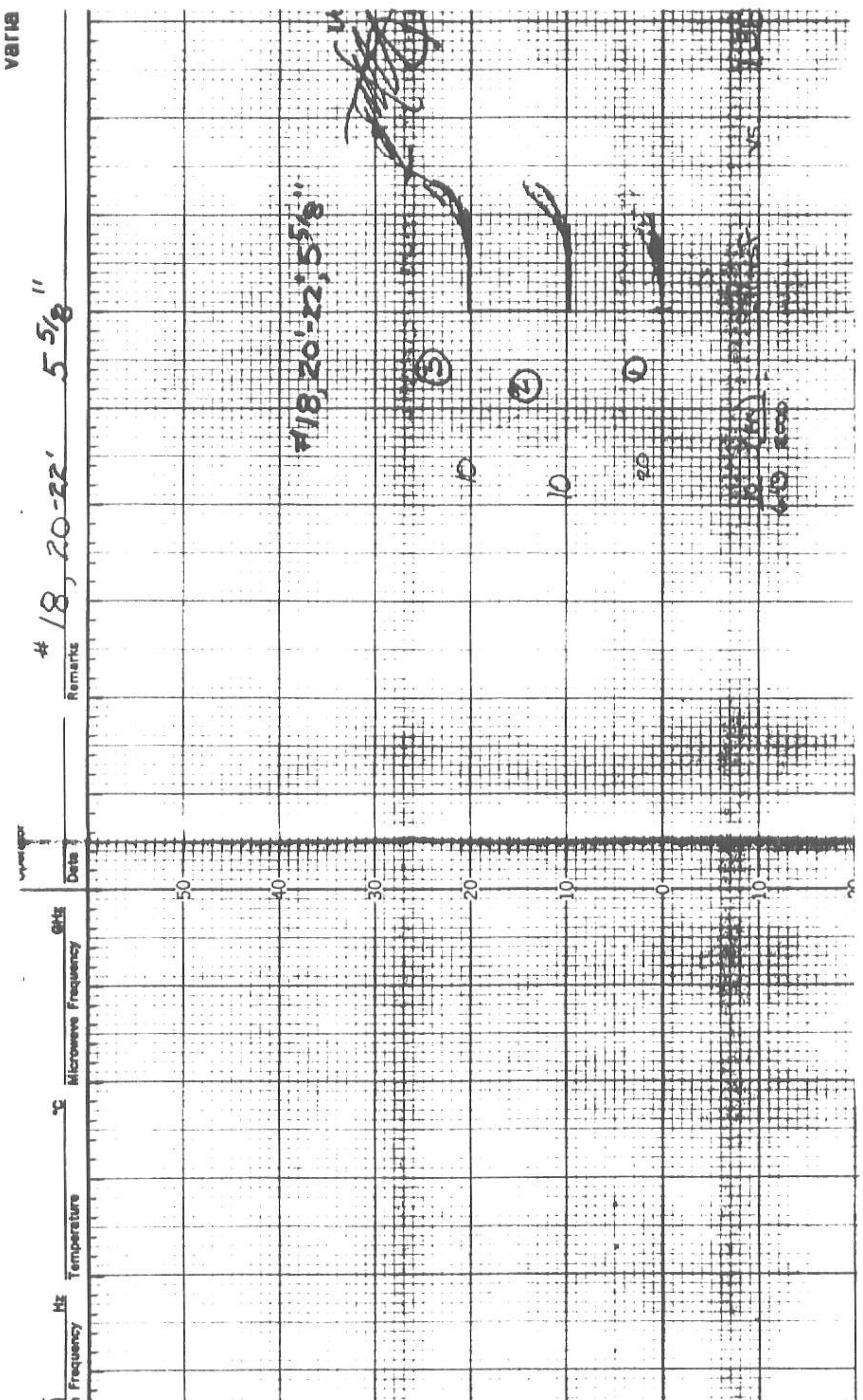
20
10
0
10
20


varian
Modulation Frequency 5 Hz
Temperature °C
Microwave Frequency GHz
Date
Remarks # 18, 20-22', 5 5/8"
#18, 20'-22', 5 5/8"
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40
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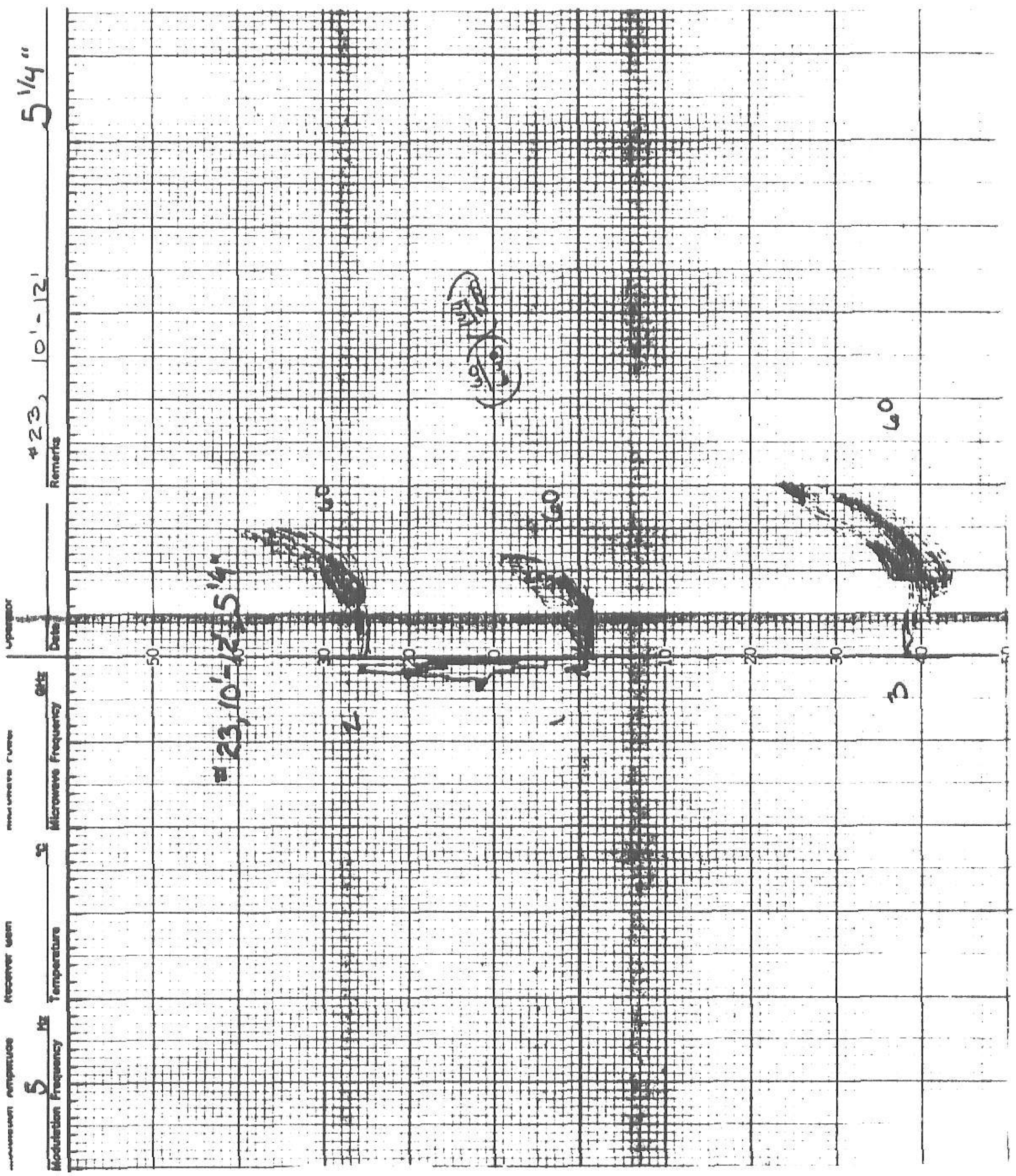
20
10
0
10
20


#23, 10'-12'
5 1/4"
Remarks
Date
Microwave Frequency
GHz
Temperature
Modulation Frequency
Hz
5
#23, 10'-12', 5 1/4"
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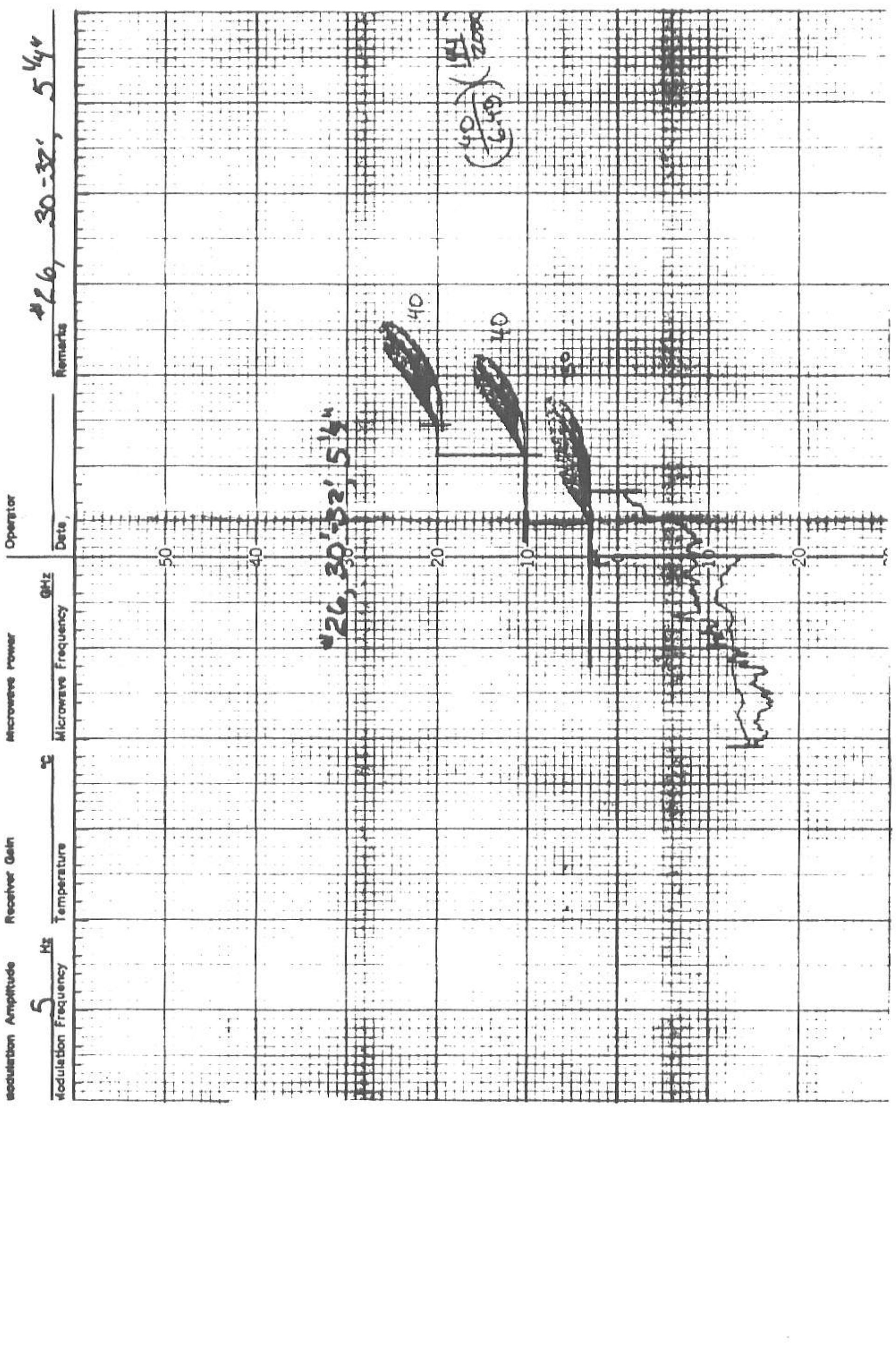
0
10
20
30
40



Modulation Amplitude
5
Modulation Frequency
Hz
Receiver Gain
Temperature
°C
Microwave Power
Microwave Frequency
GHz
Operator
Date
Remarks
#26, 30-32', 5 1/4"
#26, 30'-32', 5 1/4"
40
50
40
30
20
10
10
20

COMPARISON of DYNAMIC and UNCONFINED COMPRESSION STRENGTH for MACHINE FOOTING DESIGN

by

JAMES J. BRENNAN

B. S., Kansas State University, 1982

AN ABSTRACT OF A MASTER'S THESIS

submitted in partial fulfillment of the

requirements for the degree

MASTER OF SCIENCE

Department of Civil Engineering

KANSAS STATE UNIVERSITY
Manhattan, Kansas

1983

ABSTRACT

A study of machine foundations bearing capacity as related to the unconfined compressive strength is presented. The purpose of the study was to determine an appropriate factor of safety to use with the unconfined compression test so that the elaborate and expensive laboratory analysis necessary in the present design of machine foundations could be avoided.

A literature review was conducted which traced the history and development of the present machine footing design procedure in vogue. Some of the information thus gathered was used to determine parameters in the laboratory research.

The results of the study showed that for a silty clay soil, supporting a machine foundation vibrating at low frequencies, the factor of safety to decrease the unconfined compressive strength by to obtain an allowable dynamic strength was 5, which was adequate 85% of the time.

This study was meant to be a beginning since extensive research is necessary relating unconfined compressive strength to dynamic bearing strength of footings already in place, and further laboratory work is necessary for different soil materials at different frequencies.