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0396

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ABSTRACT

PRACTICAL APPLICATIONS OF STRESS-WAVE THEORY IN PILING DESIGN

By Charles H. Samson, Jr.,[1] M. ASCE, Farland C. Bundy,[2] M. ASCE
and Teddy J. Hirsch,[3] M. ASCE

Extensive efforts have been devoted in recent research studies to obtain a suitable means of applying wave theory to pile-driving problems for use by practicing Civil Engineers. Through use of a digital computer program devised to account for the significant factors, design engineers of the Texas Highway Department Bridge Division, working closely with research engineers of the Texas Transportation Institute at Texas A&M University, have applied this procedure in their pile-driving problems. Of special concern were long prestressed concrete piles driven in various causeway projects along the Texas Gulf Coast. In this paper are discussed various applications of stress-wave theory to practical piling problems.

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Background

Since 1961 an intensive research study has been underway at Texas A&M University on dynamic behavior of piling during driving. The work has been sponsored by the Bridge Division of the Texas Highway Department during this time, and for the past year cosponsored by the Bureau of Public Roads. The research was initiated as a result of difficulties encountered in driving prestressed concrete piling during the construction of the Lavaca Bay Causeway near Port Lavaca, Texas. Several piles 70 to 110 feet long developed transverse cracks and failed completely during driving.

In an effort to determine causes of the cracking and possible remedial action, a project was begun with the objective of developing a computer program based on the work of E. A. Smith (1).* Mr. Smith, who was formerly Chief Mechanical Engineer for Raymond International, Inc., but now retired, worked extensively over a period of several years to

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* Numbers in parentheses refer to list of References at the end of the article.

adapt one-dimensional wave theory to the dynamic behavior of piling under the impact of a pile-driver ram.

In 1961 a computer procedure was developed at the Texas Transportation Institute (2) at Texas A&M University as the first phase of the research.. The program was designed for the IBM 709 Digital Computer at the A&M Data Processing Center. Mr. Smith served as a special consultant on this work.

Since 1961, the research has continued in several directions. Several piles have been instrumented and field tested for stress, strain, and displacement data. This work has been described in two reports published by the Texas Transportation Institute (3, 4). A general discussion of dynamic behavior of piling as predicted by wave theory, based on this research, has been published by Institute research engineers (5). A study is also under way in which the computer procedure is used to determine the influence of different variables on the behavior of piles during driving. An interim report (6) has been prepared on the portion of the study thus far completed.

Stress-wave Theory

The stress-wave theory used in this research is based on classical one-dimensional wave theory, details of which may be found in References (7) and (8).

Figure 1a illustrates a typical pile-driving situation. Figure 1b shows the idealized representation used by Smith in developing his procedure. The pile is pictured as series of discrete weights and springs, properties of which are determined from the properties of the actual pile. Load-deformation characteristics of the pile material, the soil, the cap-block, and the cushion block must be predetermined. The weight and impact velocity of the ram must also be predetermined.

In the application of this method a relatively small increment of time is selected. The analysis becomes a step-by-step operation initiated by the impact velocity of the ram, assumed to remain constant during the first interval. In sequence, then, displacements, internal forces, and soil reactions may be determined for all segments. With this information, it is possible to calculate new velocities for all segments for use in the next time interval. This sequence of computations may be continued until the desired information is obtained. Information that would prove of interest to a design engineer might be, for example:

1. Permanent set per blow.
2. Maximum compressive stress and maximum tensile stress that might occur during one blow in any segment.
3. Variation of stress with time for each segment.
4. Variation of displacement with time for each segment.

Such data may be readily extracted from the computer solution.

Example

As an illustration of theory, the pile of Figure 2a is shown, along with the idealization used in the computer. The pile properties are taken from Reference (1). The total ultimate soil resistence is arbitrarily taken to be 200 kips uniformily distributed along the side. The resistance for this particular case is considered to be entirely side friction. Other pertinent information is given as follows:*

Hammer energy = 12,000 ft lb

Impact velocity of ram = 12.4 ft per sec

Weight of pile cap = 700 lb

Capblock stiffness = 2×10^6 lb per in.

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* The reader is referred to References (1), (2), and (5) for more detailed discussion of these factors.

Coefficient of restitution of capblock = 0.50

Soil quake (maximum elastic deformation) = 0.1 in.

Soil damping coefficient = 0.05 sec per ft

Figure 2b illustrates a typical displacement versus time plot obtained from the computer solution. The particular curve shown is for the point of the pile. Figure 2c shows a plot of force versus time for the mid-point of the pile. The set per blow was determined to be 0.39 in.

Practical Application

Bridge Division engineers of the Texas Highway Department have put stress-wave theory to work in solving practical field problems. In the construction of both the Lavaca Bay Causeway and the Nueces Bay Causeway on the Texas Gulf Coast, long prestressed concrete piles (70 to 110 feet) were driven. Cross sections of piles used on the projects are given in Figure 3. A general view of the partially completed Nueces Bay job is shown in Figure 4. The rig that appears in the foreground is taking soil borings. Piles were driven by a diesel hammer having the following characteristics:

Rated Hammer energy = 39,800 ft lb

Weight of ram = 4,850 lb

Figure 5 shows a pile positioned for driving at the Nueces Bay site.

On the Lavaca Bay project, cushioning was provided by a circular one-inch plywood capblock and a three-inch thick oak cushion block. The cushioning on the Nueces Bay project consisted of the same type capblock as used in the Lavaca Bay work and a six-inch oak cushion block.

On both projects there occurred some instances of transverse cracking in the piles of the sort shown in the close-up view of Figure 6. Here the crack has been filled with an epoxy adhesive in an effort to permit redriving of the pile. Obvious questions were: What was the cause? What corrective measures could be taken?

As a means of shedding more light on the problem, a specific pile installation at the Lavaca Bay site was selected for theoretical investigation. Based on soil data acquired through soil borings, the soil resistance for a 90-foot pile at a penetration of 30 feet was estimated to be that shown in Figure 7.

In order to observe the effect of different hammer energies and different ram velocities, three different sets of hammer characteristics, representative of different pile drivers in use, were selected for study. These were as follows:

Hammer A:

Diesel hammer
Ram weight = 4,850 lb
Rated energy per blow = 39,800 ft lb
Maximum explosive pressure on pile = 158,700 lb

Hammer B:

Single-acting steam hammer
Ram weight = 9,300 lb
Rated energy per blow = 30,225 ft lb

Hammer C:

Single-acting steam hammer
Ram weight = 14,000 lb
Rated energy per blow = 37,500 ft lb

Figure 8 provides curves of maximum tensile stress versus impact velocity. The maximum tensile stress is the maximum developed in the pile considering all time intervals and all pile segments. For a given hammer, different impact velocities correspond to different hammer efficiencies.

In the plot of Figure 8, two curves are given for Ram A: one accounting for the internal explosive pressure, one allowing for no internal pressure. Since there is some question among engineers as to the degree of influence of explosive pressure, the two curves bound the range of effectiveness.

Also in Figure 8, for each ram two cushion blocks were considered. The dashed curves represent results based on cushioning provided by a new block. Because continued use of a cushion block continues to permanently compress it, leading to a considerably increased stiffness, comparision curves are shown for cushioning having a stiffness arbitrarily increased ten times over that of an unused block. That a great amount of compression occurs is evidenced by Figure 9, in which the capblock and cushion block used at Nueces Bay are shown before and after driving. The charred appearance of the capblock and the "squashed" appearance of the cushion block in Figure 9b attest to the amount of compression that has taken place.

No attempt will be to generalize on the basis of the limited results shown. It should be noted that the soil resistance in Figure 7 is extremely light and indicative of resistance encountered during the initial stage of driving. In Figure 8, all tensile stresses plotted exceed the 800-psi prestress in the pile. Of course, adding the tensile strength of the concrete to the prestress would provide perhaps 1,200 or 1,300-psi total ultimate strength.

The results of Figure 8 support certain conclusions, at least for the circumstances considered:

1. In conditions of low soil resistance, very high tensile stresses may be developed.
2. A softer cushion block will in general lead to lower maximum tensile stress.
3. The influence of diesel hammer explosive pressure on the maximum tensile stress does not appear significant.
4. For a given ram, the development of high tensile stress is greatly influenced by the impact velocity.

One immediate step that was taken by Bridge Division engineers was to insist that new cushion blocks be installed at least with each new pile being driven. On the Nueces Bay project, this step alone appeared largely to eliminate the cracking problem. A further practice that was followed in some cases of very light driving was the pre-drilling of a slightly undersized shaft to a depth offering firmer bearing. Since the pile could be positioned at this depth without impact driving, the period of greatest danger of tensile cracking was avoided.

Other remedial steps that suggest themselves on the basis of Figure 8 are the use of a thicker cushion block and the decrease of impact velocity by decreasing the stroke of the pile-driver ram during periods of low soil resistance.

It is some interest to note the influence of modulus of elasticity on the magnitude of maximum tensile stress developed. Figure 10 is a plot of maximum tensile stress versus ram impact velocity for the diesel hammer. It is noted that a significant increase in tensile stress occurs with an increase in modulus of elasticity. These results suggest the interesting possibilities of use of lightweight aggregate concrete, with its characteristic lower modulus of elasticity, as a material for prestressed concrete piles.

Field Tests

Field tests performed during the research at Texas A&M are described in detail in References (3), (4), and (7). These tests have provided data on a recording oscillograph (see Figure 11) for stress variation with time at different points along the pile and displacement variation with time at the head of the pile. Electric resistance strain gages embedded in the pile measured strains. A linear voltage displacement, shown in a close-up view in Figure 12, transducer measured the travel at the head of the pile. Figure 13 shows one set of oscillographic data taken from the recorder. Experimental data acquired in this way are being correlated with corresponding theoretical solutions obtained by the computer stress-wave program. Figure 14 shows one such correlation, taken from Reference (5). Considering the uncertainty associated with soil factors, the correlation is thought to be encouraging. Further research related to dynamic soil behavior is expected to improve the correlation.

Future Applications of Stress-wave Theory

The objective throughout the Texas A&M research has been to develop an effective and reliable method for use by practicing engineers concerned with designing piling and determining driving conditions. It is expected that practical use may be achieved in several ways:

1. Where a large computer is available and where soil information is known, specific problems can be solved, as illustrated by Figure 8.
2. Where a computer is not locally available, a central service organization may be used. Texas A&M, for example, has such facilities.

3. Parameter studies currently in progress should lead to much generalized information that can be of direct benefit to the practicing engineer. For example, Figure 15 has been extracted from Reference (6), in which a whole family of such plots has been developed. Plots of maximum stress versus impact energy for the conditions indicated are given. A separate curve is provided for each ram weight. Figure 16, also taken from Reference (6), provides curves of permanent set versus impact energy.

4. Extensive comparison of stress-wave solutions with those from simplified formulas should define more clearly the limitations and the ranges of applicability of the formulas.

Future Research at Texas A&M

Future research planned includes continued field tests to provide correlation information; additional generalized paramenter studies; isolated experimental studies of piling and cushioning material, pile-drivers, and soil factors; and continued refinement of the computer program. The research engineers hope that their efforts will lead to a reliable means of theoretically determining driving stresses, displacements, and ultimate bearing capacity by means of a digital-computer analysis.

References

(1) Smith, E. A. L., "Pile-driving Analysis by the Wave Equation," Transactions, ASCE, Vol. 127, 1962, Part I, p. 1145.

(2) Samson, Charles H., Jr., "Pile-driving Analysis by the Wave Equation (Computer Procedure), Report RP 25, Texas Transportation Institute, Texas A&M University, May, 1962.

(3) Hirsch, Teddy J., "Stresses in Long Prestressed Concrete Piles During Driving," Report RP 27, Texas Transportation Institute, Texas A&M University, September, 1962.

(4) Hirsch, Teddy J., "Field Tests of Prestressed Concrete Piles During Driving," Report RP 27-2, Texas Transportation Institute, Texas A&M University, August, 1963.

(5) Samson, Charles H., Jr., Hirsch, Teddy J., and Lowery, Lee L., Jr., "Computer Study of Dynamic Behavior of Piling," Journal of the Structural Division, ASCE, Vol. 89, No. ST5, August, 1963.

(6) Hirsch, Teddy J., "Computer Study of Variables Which Affect the Behavior of Concrete Piles During Driving," Report RP 27-3, Texas Transportation Institute, Texas A&M University, August, 1963.

(7) Hirsch, Teddy J., Samson, Charles H., Jr., and Lowery, Lee L., Jr., "Driving Stresses in Prestressed Concrete Piles," presented at National Meeting ASCE, San Francisco, California, October 7, 1963.

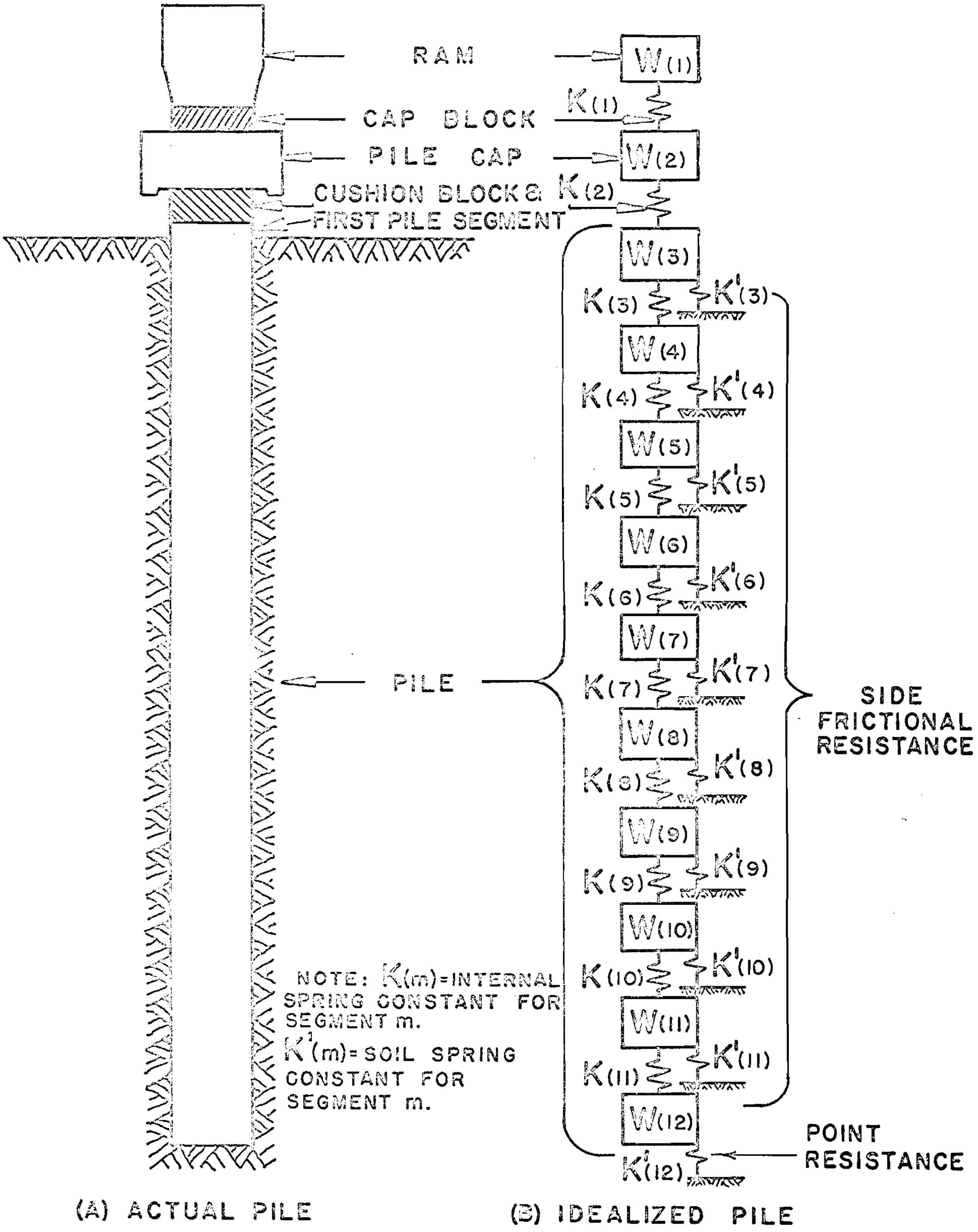
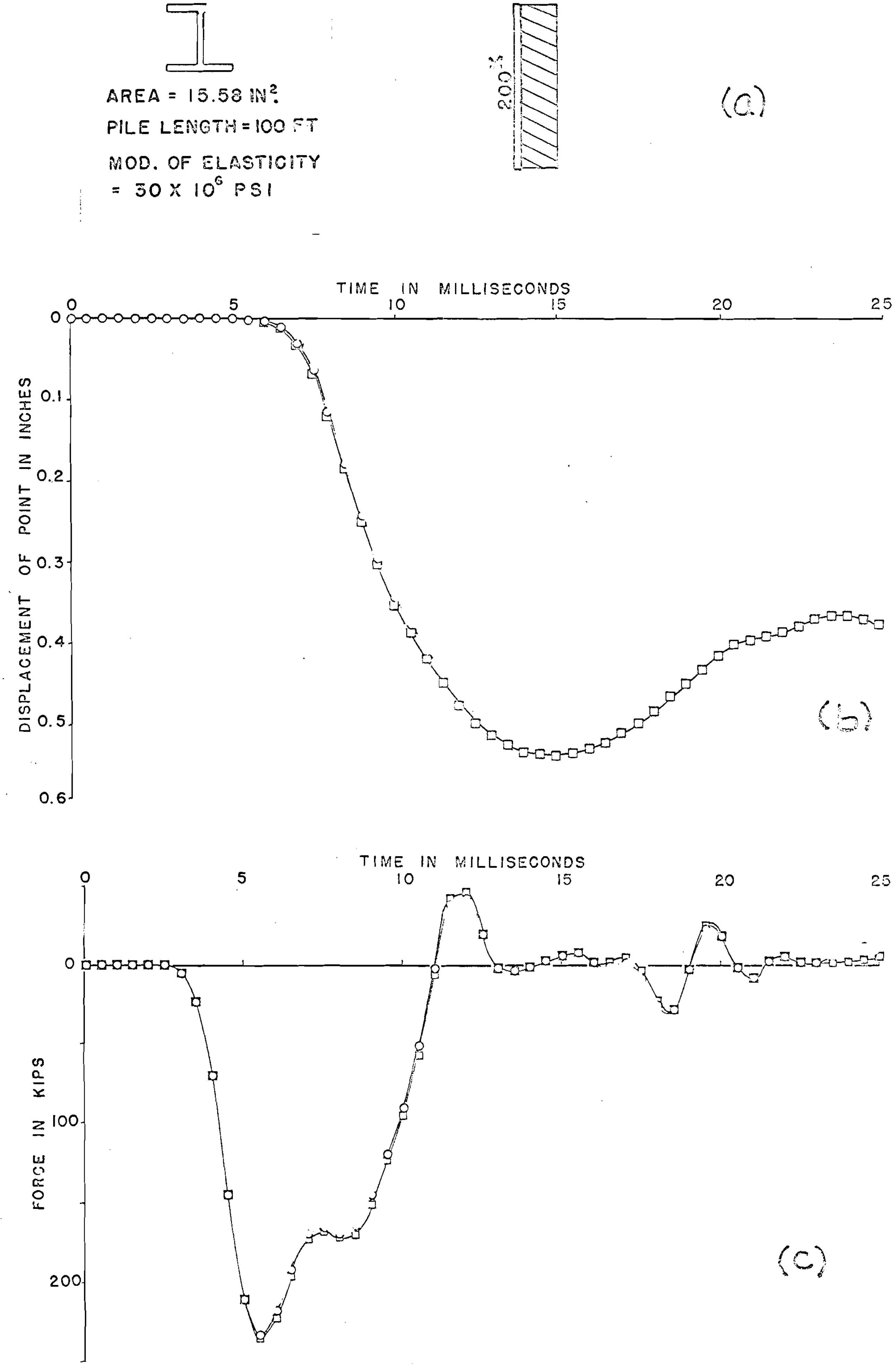
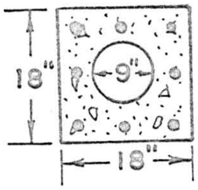

RAM
W(1)
K(1)
CAP BLOCK
PILE CAP
W(2)
CUSHION BLOCK &
FIRST PILE SEGMENT
K(2)
W(3)
K(3)
K'(3)
W(4)
K(4)
K'(4)
W(5)
K(5)
K'(5)
W(6)
K(6)
K'(6)
W(7)
K(7)
K'(7)
PILE
SIDE FRICTIONAL RESISTANCE
W(8)
K(8)
K'(8)
W(9)
K(9)
K'(9)
W(10)
K(10)
K'(10)
W(11)
K(11)
K'(11)
W(12)
K'(12)
POINT RESISTANCE
NOTE: K(m)=INTERNAL SPRING CONSTANT FOR SEGMENT m.
K'(m)= SOIL SPRING CONSTANT FOR SEGMENT m.
(A) ACTUAL PILE
(B) IDEALIZED PILE

FIG. 1



AREA = 15.58 IN^2.
PILE LENGTH = 100 FT
MOD. OF ELASTICITY
= 30 X 10^6 PSI
200^K
(a)
TIME IN MILLISECONDS
0
5
10
15
20
25
DISPLACEMENT OF POINT IN INCHES
0
0.1
0.2
0.3
0.4
0.5
0.6
(b)
TIME IN MILLISECONDS
0
5
10
15
20
25
FORCE IN KIPS
0
100
200
(c)

FIG. 2

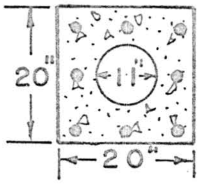


AREA = 269.1 IN.^{2}

PILE LENGTH = 95 FT

MOD. OF ELASTICITY
= 8.18 X 10^6 PSI

(a)



AREA = 302.97 IN.^{2}

PILE LENGTH = 90 FT

MOD. OF ELASTICITY
= 4.95 X 10^6 PSI

(b)

FIG. 3

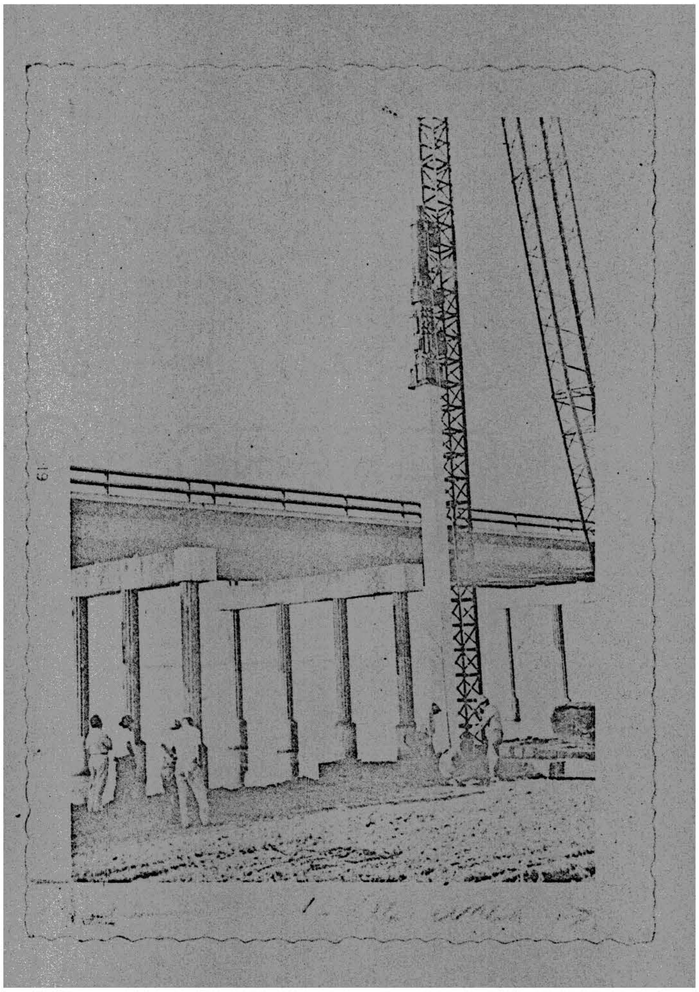


Figure 4.

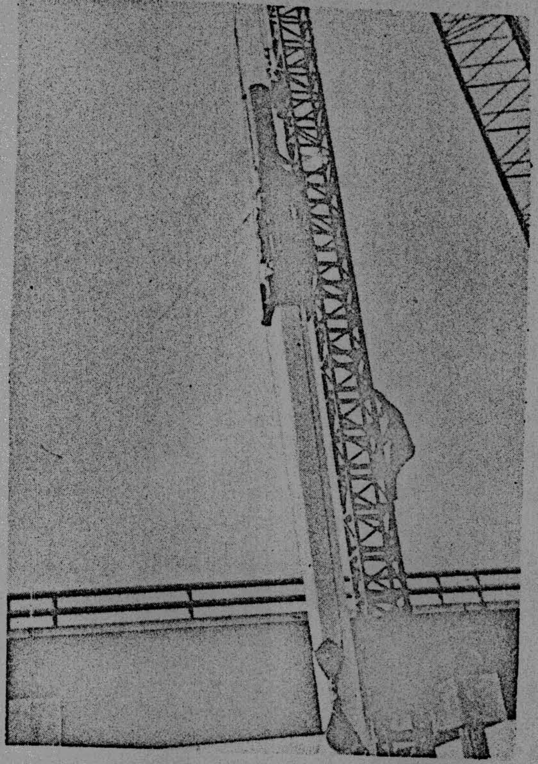


FIG. 5

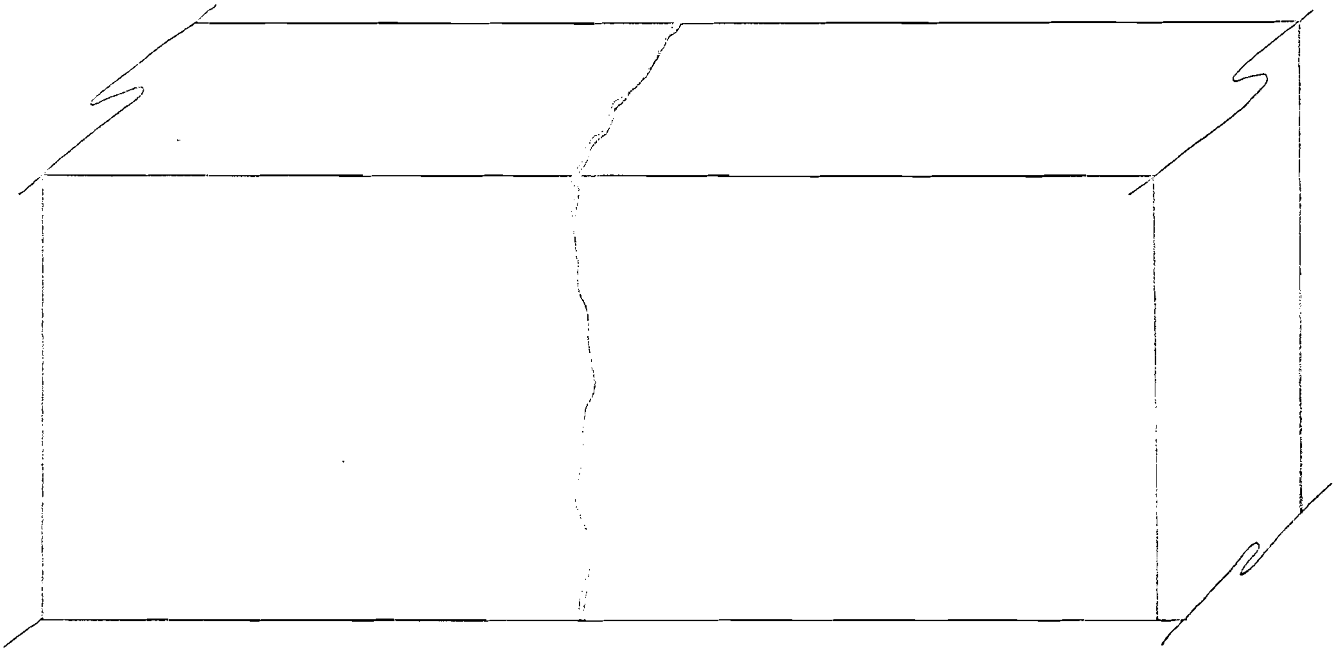


Figure 6.

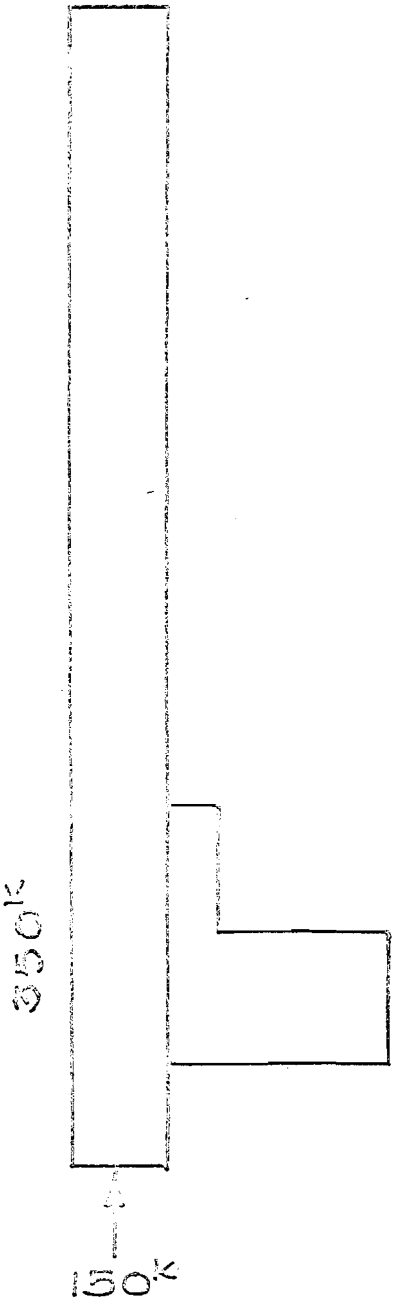


FIG. 7

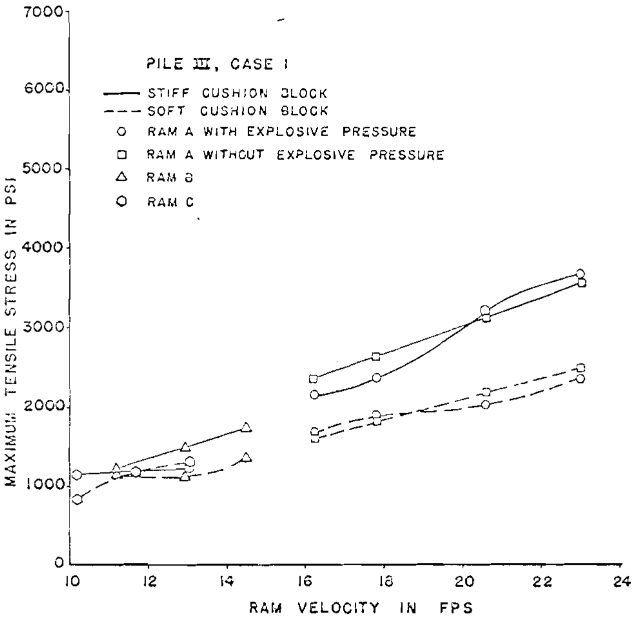

PILE III, CASE 1
STIFF CUSHION BLOCK
SOFT CUSHION BLOCK
○ RAM A WITH EXPLOSIVE PRESSURE
□ RAM A WITHOUT EXPLOSIVE PRESSURE
△ RAM B
○ RAM C
MAXIMUM TENSILE STRESS IN PSI
0
1000
2000
3000
4000
5000
6000
7000
10
12
14
16
18
20
22
24
RAM VELOCITY IN FPS

FIG. 8

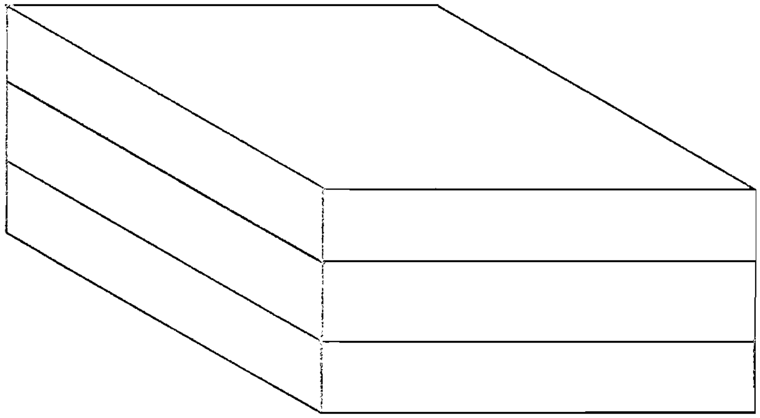


Figure 9a.

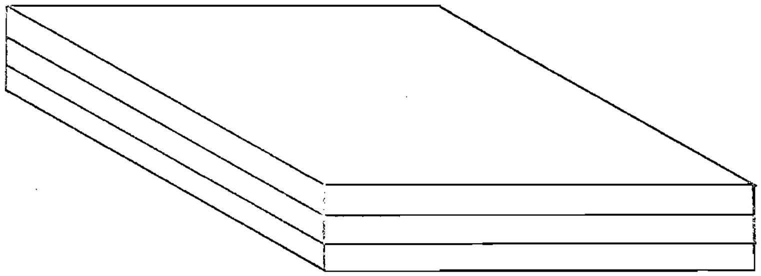


Figure 9b.

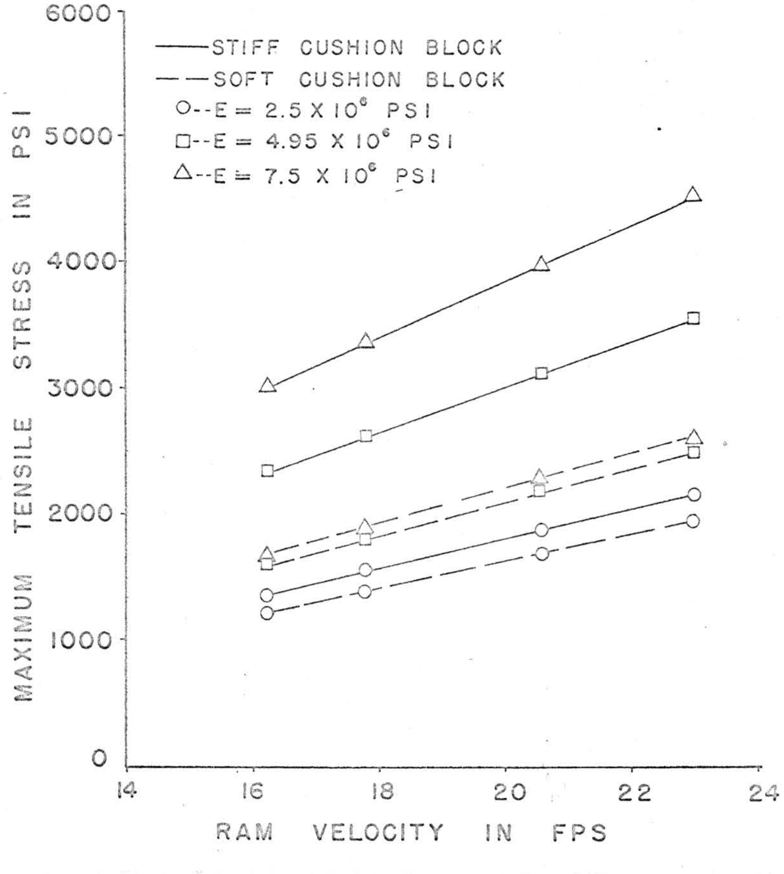

6000
5000
4000
3000
2000
1000
0
MAXIMUM TENSILE STRESS IN PSI
14
16
18
20
22
24
RAM VELOCITY IN FPS
——STIFF CUSHION BLOCK
— —SOFT CUSHION BLOCK
○--E = 2.5 X 10⁶ PSI
□--E = 4.95 X 10⁶ PSI
△--E = 7.5 X 10⁶ PSI

FIG. 10

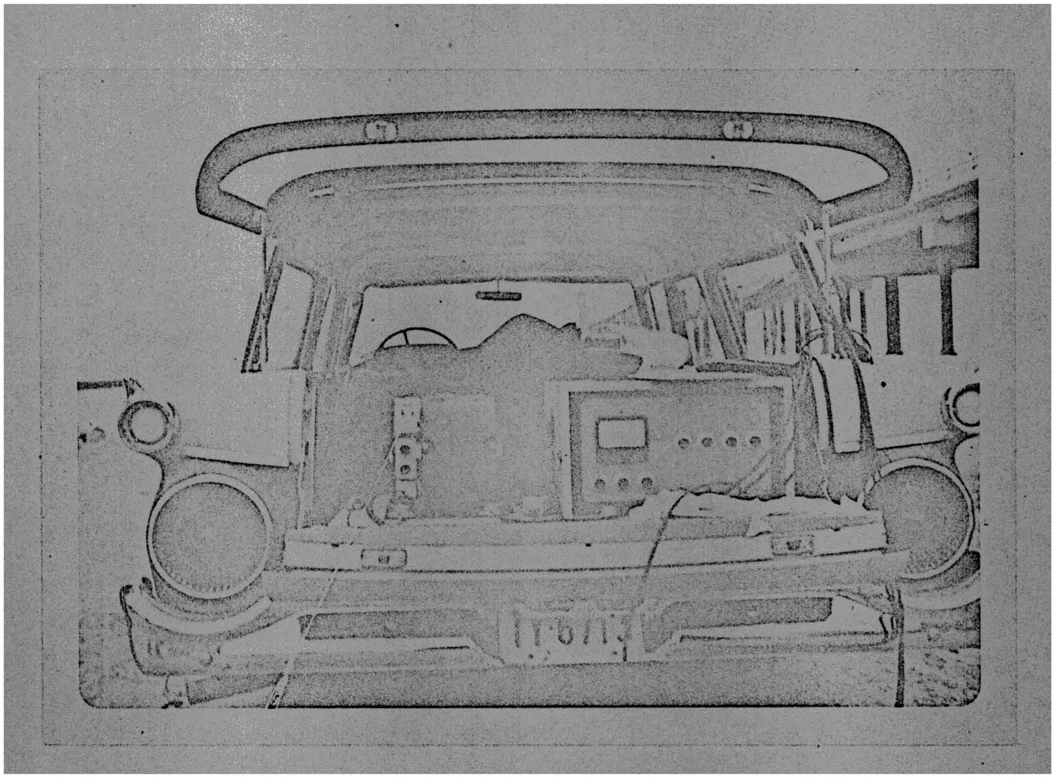


Figure 11.

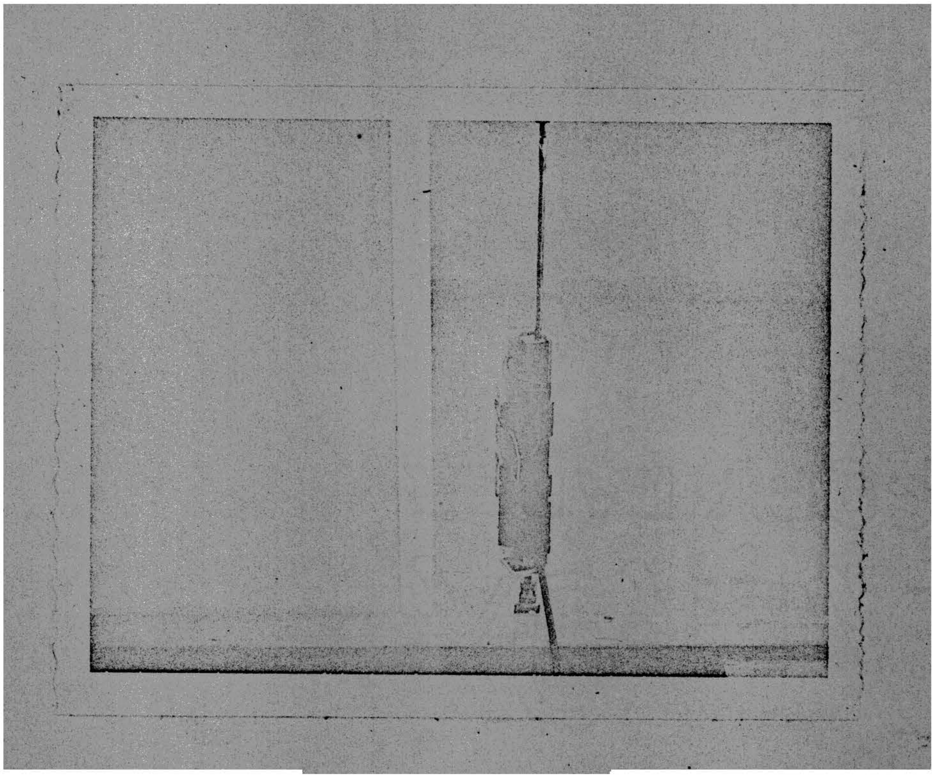


FIG. 12

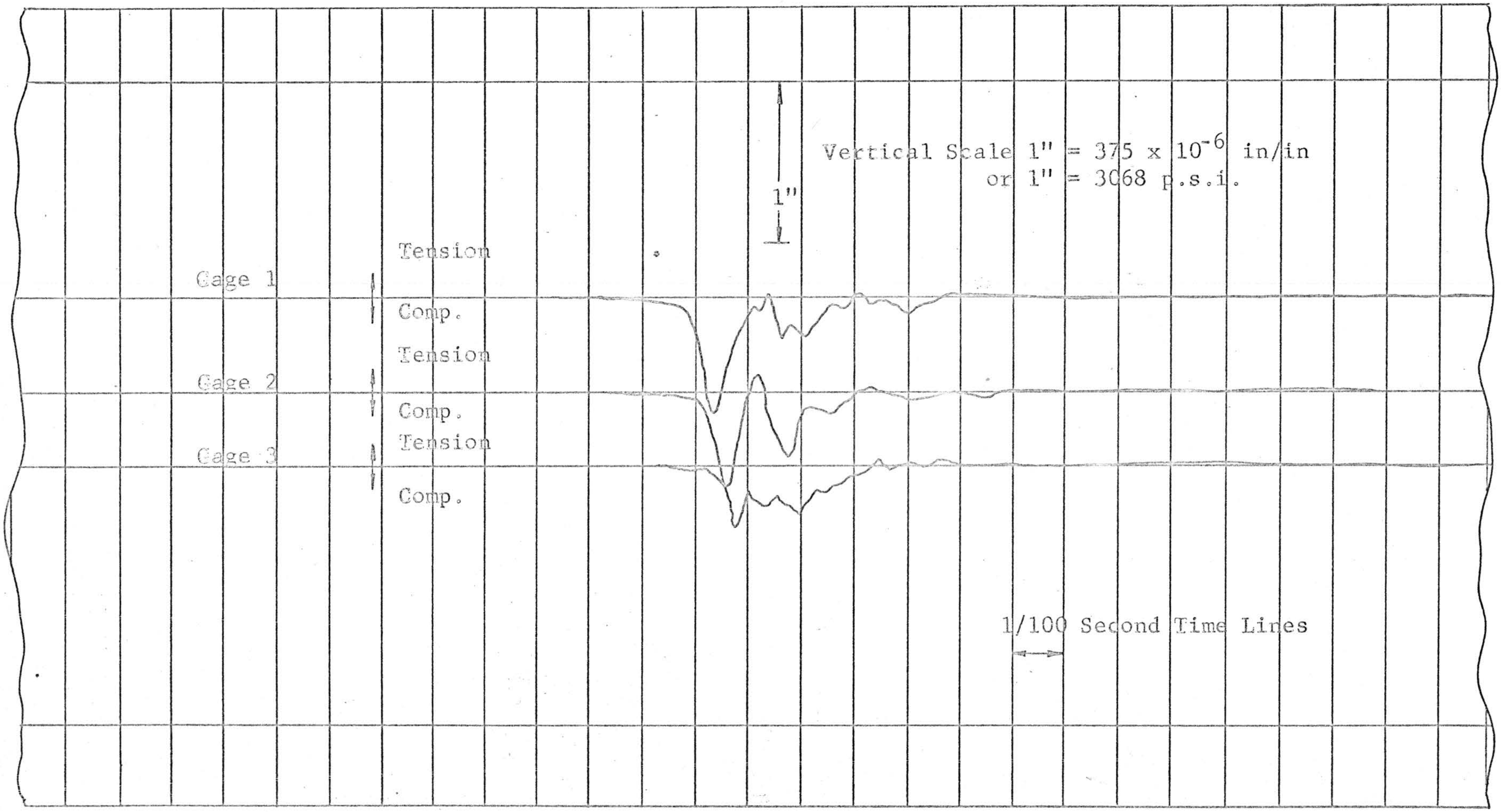
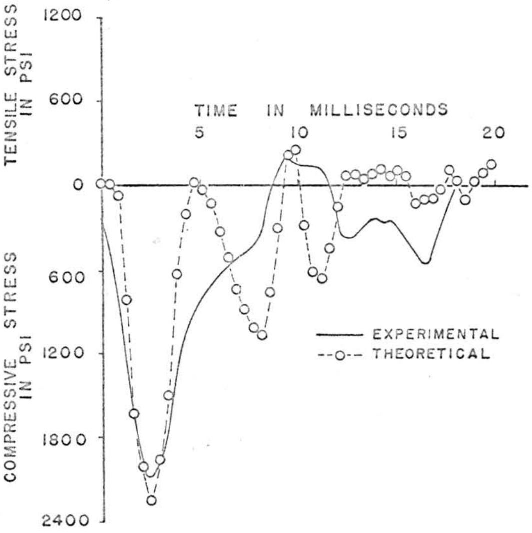

Vertical Scale 1" = 375 x 10^{-6} in/in
or 1" = 3068 p.s.i.
1"
Gage 1
Tension
Comp.
Gage 2
Tension
Comp.
Gage 3
Tension
Comp.
1/100 Second Time Lines

FIG. 13



TENSILE STRESS IN PSI
COMPRESSIVE STRESS IN PSI
1200
600
0
600
1200
1800
2400
TIME IN MILLISECONDS
5
10
15
20
EXPERIMENTAL
--o-- THEORETICAL

FIG. 14

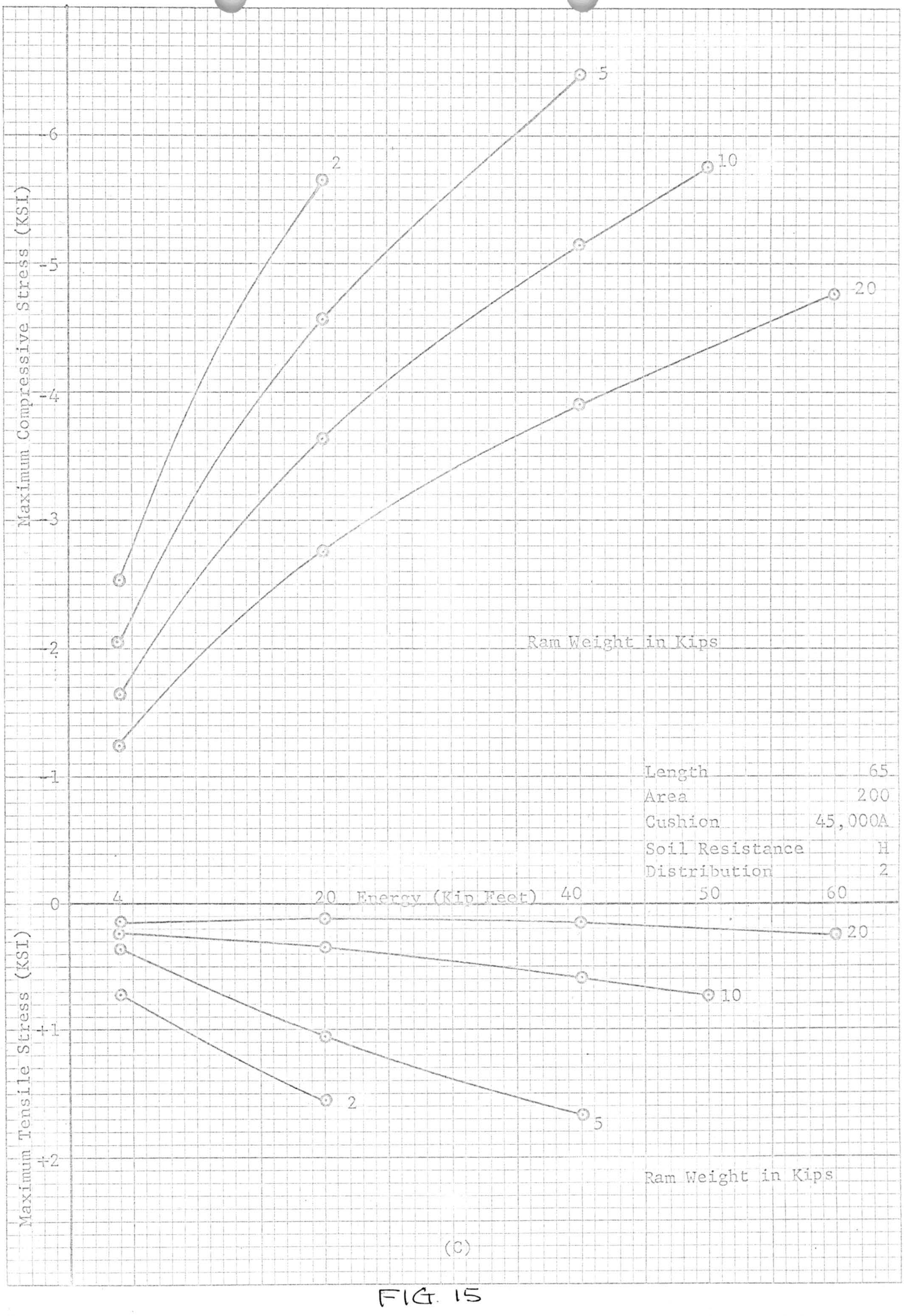
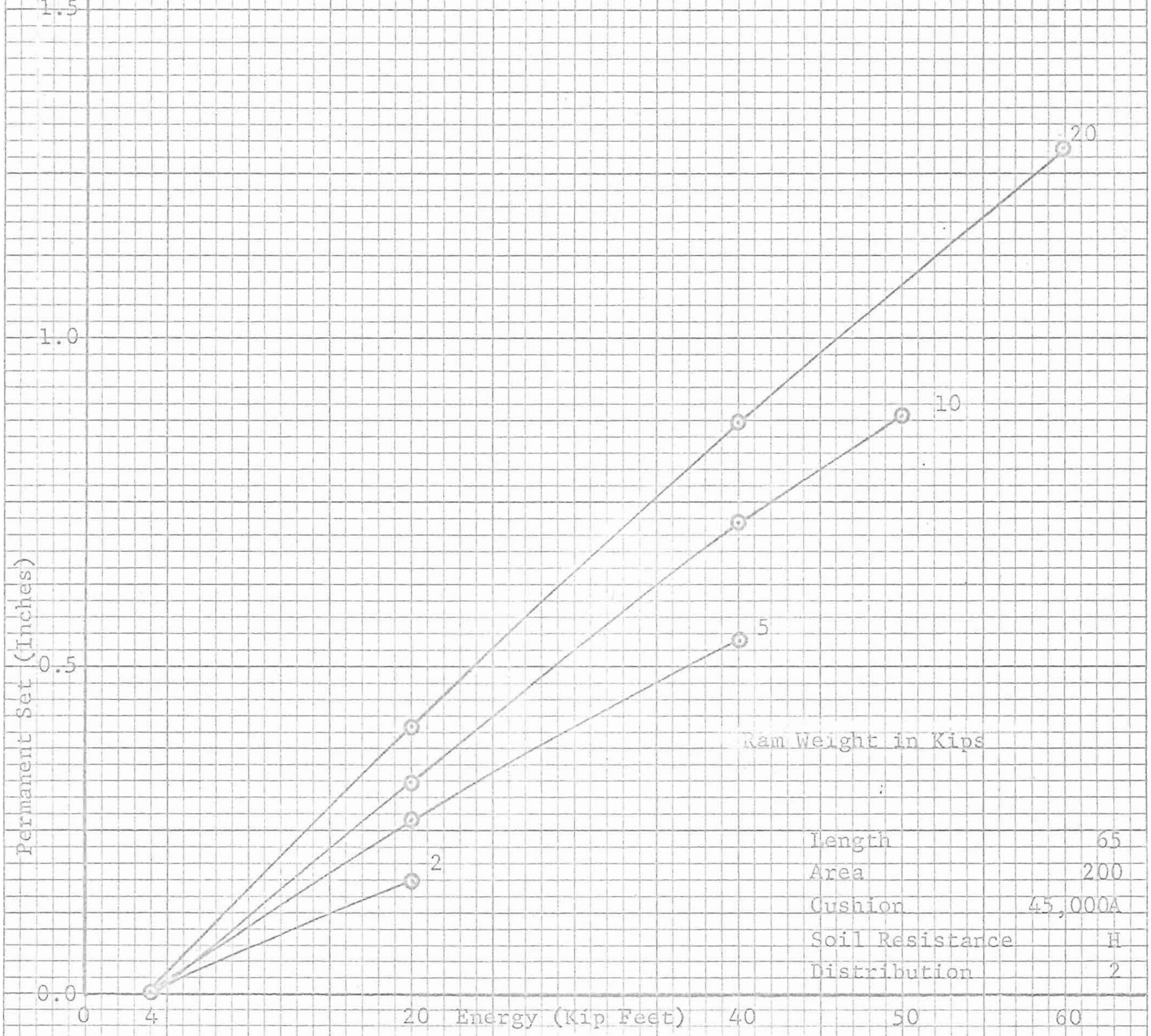

Maximum Compressive Stress (KSI)
-6
-5
-4
-3
-2
-1
0
+1
+2
Maximum Tensile Stress (KSI)
4
20
Energy (Kip Feet)
40
50
60
2
5
10
20
Ram Weight in Kips
Length 65
Area 200
Cushion 45,000A
Soil Resistance H
Distribution 2
20
10
2
5
Ram Weight in Kips
(C)

FIG. 15


1.5
1.0
0.5
0.0
Permanent Set (Inches)
0
4
20
Energy (Kip Feet)
40
50
60
20
10
5
2
Ram Weight in Kips
Length 65
Area 200
Cushion 45,000A
Soil Resistance H
Distribution 2

(C)

FIG. 16