A REPORT ON FIELD TESTS OF PRESTRESSED CONCRETE PILES DURING DRIVING

By

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INTRODUCTION

During the year 1960-61, engineers of the Texas Highway Department Bridge Division engaged the staff personnel at Texas A&M University to develop a computer program to accomplish the rigorous mathematical calculations for the analysis of pile behavior during driving. With the aid of Mr. Edward A. Smith* as a special consultant, a functioning computer program was developed and used successfully on a number of pile problems (2, 3, 4)**. This program was written for the IBM 709 Computer at the Texas A&M University Data Processing Center. With the use of this program it is practical to investigate theoretically the behavior of various type piles when driven by different equipment under different foundation conditions.

In order to properly use this theoretical solution of the wave equation, it was considered necessary to conduct field tests to obtain actual stress and displacement data to correlate with the theory. During the 1961-62 year, three prestressed concrete piles 95 ft. in length and two piles 92 ft. in length were instrumented with strain gages and displacement transducers and tested during driving. This work was done as a part of Research Project RP-27 and the field tests were conducted at the Nueces Bay Causeway at Corpus Christi, Texas (4). These five test piles were designated Test Piles 1, 2, 3, 4, and 5.

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* Formerly Chief Mechanical Engineer for Raymond International.

** Numbers thus (2, 3, 4) refer to corresponding items in the list of References.

During the 1962-63 year it was decided to field test two prestressed concrete piles 26 ft. in length in order to obtain data on the stresses and displacements of relatively short piles. These tests were conducted at the construction site of the McHard Underpass on Interstate Highway 45 South of Houston, Texas. These test piles have been designated Test Piles 6 and 7.

OBJECTIVES

Project No. 2-5-62-33 entitled "Study of Variables Which Affect Behavior of Concrete Piles During Driving," was initiated September 1, 1962. The objectives of the 1962-63 year were to

(1) make an orderly theoretical computer investigation of the influence of various factors on the behavior of piles during driving,

(2) present these results in the form of charts, diagrams, or tables for direct application by office design engineers, and

(3) instrument and field test at least two concrete piles to obtain data on dynamic stresses in piles to correlate with the computer results.

The results of objectives 1 and 2 were presented in a report entitled "Computer Study of Variables Which Affect the Behavior of Concrete Piles During Driving," which was prepared for the Bridge Division of the Texas Highway Department in August 1963. The results of objective 3 are presented in this report.

FIELD TESTS

General. Two precast prestressed concrete piles 26 ft. long and 16 in. square were instrumented with strain gages and tested during driving. The piles were driven in a firm coastal clay deposit at the site of the McHard Underpass on Interstate Highway 45 South of Houston, Texas. During driving, strain gage measurements and the pile displacements were recorded on a multi-channel recording oscillograph. These two test piles have been designated Test Piles 6 and 7.

Test Piles. The dimensions and design properties of the two test piles are presented in Figure 1. The piles were 16 in. square and 26 ft. long. Concrete specimens were obtained as the piles were being cast. Values of the unit weight, compressive strength, tensile strength, modulus of rupture, modulus of elasticity, and Poisson's ratio are presented in Table 1.

Table 1. Properties of Concrete in Test Piles (Test Piles 6 and 7)

Unit Weight, lb/cu ft	154
Compressive Strength, psi	
15 hour, 6" x 12" cyl	4690*
7 day, 6" x 12" cyl	6680
28 day, 3" x 4" x 16" prism	6570
Tensile Strength, psi	
28 day, 3" x 3" x 22" prism	520

* Concrete steam cured at 130° F. for 10 hours.

Modulus of Rupture, psi	
28 day, center point 3" x 4" x 16" prism	1120
Modulus of Elasticity, psi	
28 day, Static	7.67×10^6
28 day, Dynamic	7.84×10^6
Poisson's Ratio	
28 day, Dynamic	.21

The modulus of elasticity of the concrete was required to transform the strain-gage readings into stress. Both the modulus of elasticity and unit weight values were used in setting up these pile problems for the theoretical solution by use of the digital computer. The strength properties were useful in interpreting the significance of the measured dynamic stresses. These two prestressed concrete piles were cast by Baass Brothers Concrete Company in Victoria, Texas.

Strain Gages and Instrumentation. Baldwin AS 9 constantan wire grid, Valore type brass foil envelope, strain gages were embedded parallel to the longitudinal axis of the prestressed concrete piles during the placing of the concrete. This was done about four weeks prior to the driving of the piles. The gages were located along the length of the pile at five points; i.e. the head of the pile, quarter point, mid-point, three-quarter point, and point of the pile. The precise locations are shown in Figure 1.

Each pile had two gages at each cross section near opposing steel strands. These two gages were hooked up as opposing arms of the

Wheatstone bridge used to measure the strain. In this manner any bending stresses present were eliminated and only direct tension or compression on the pile cross section was recorded at each gaged point along the pile. This was desirable for correlation purposes, because the computer solution of the wave equation does not take the bending stress into consideration.

The lead wires from the gages were Belden No. 8404, American wire gage No. 20, four conductor, shielded, vinyl plastic covered cable. These lead wires were run the length of the piles embedded in the concrete and were brought out near the pile head.

Since the length of the lead wires, gage locations, and manner of hook-up had been determined prior to the installation of the gages, all strain gage connections and connectors were prepared and water-proofed in the laboratory. No gluing, soldering, or water-proofing was done in the field. This was necessary since all instrumentation and tests had to be performed under field construction conditions in a manner such that the contractor would not be unduly delayed.

A Honeywell Type 1508 Visicorder oscillograph and a Honeywell Type 119 Amplifier system was used to amplify and record the dynamic strains. The oscillograph was equipped with Honeywell Type M1650 Galvonometers having a flat frequency response to 1000 cycles per second. Kodal Linograph direct print paper was used to record the data. The 110 volt, 60 cycle, electrical power was supplied by a portable generator.

A linear motion potentiometer with a 6 in. travel was used to record the dynamic displacements of the pile. These data were recorded on the oscillograph along with the strain gage data.

Soil Properties. In order to simulate the test piles for the computer solution, it was necessary to know the shear strength properties and identification of the soil into which the piles were being driven. Prior to the design of the McHard Underpass the foundation exploration crews of District 12 of the Texas Highway Department drilled several exploration holes at the site. The ultimate shear strength and description of the soil at various depths in the ground are given in Figure 2. In general, the soil was a firm marine clay deposit.

Pile Driver. A "Delmag" diesel pile hammer Type D-22 was used to drive the piles tested in this project. This hammer has a manufacturer's rated energy output per blow of 39,700 ft-lb. The technical data concerning this hammer are given in Figure 3. In addition to these data, the following supplemental information about the D-22 hammer has been determined from sketches and other literature concerning the equipment:

Compression ratio	=	13.7 to 1
Distance from striker head to center of exhaust port	=	1.25 ft
Diameter of ram	=	15 in.
Height of ram	=	8.45 ft

Diameter of anvil (average) = 15 in.

Height of anvil = 2 ft

Weight of helmet = 1200 lb

Duration of explosive pressure = 1/100 sec

The above information is approximate because detailed drawings of the hammer were not available. However, it is considered to be sufficiently accurate for setting up the computer program for the theoretical analysis.

The working principles of the diesel pile hammer are shown in Figure 4. The driving force delivered to the pile results from two events; (1) the impact of the ram on the anvil, and (2) the explosion of the diesel fuel. By far the greater of these two forces is the impact of the ram on the anvil. This force depends on the weight of the ram and its velocity at impact. In order to determine this velocity at impact, it is necessary to know the height of fall of the ram.

Pile Driving and Test Procedure. When the test piles arrived at the driving site by truck, the strain gages had been previously cast in them and several feet of lead wires with connectors attached were protruding from the concrete near the pile head. Shielded cable extensions were connected to these wires at the head of the pile, and the pile was then raised into position in the leads of the pile driver rig. The extension cables were connected to the recording oscillograph, and each strain gage channel was then balanced and calibrated prior to the driving of the pile.

The piles had been previously measured and marked off at one foot intervals so that the penetration of the pile into the ground could be determined by inspection. As the pile was driven continuously into the ground, the recording oscillograph was turned on intermittently at different depths of penetration. In general, the recorder was run for periods of three to five seconds. By doing this the stresses from three to five consecutive blows could be recorded along with the time interval between blows. This time interval was desired, because the height of the ram fall could be more accurately determined from it than from direct visual observation.

After the pile had penetrated the ground and the permanent set was about one inch per blow, the pile driver was stopped so the displacement transducer could be attached. The transducer was attached to the pile with a clamp and its base was attached to a timber resting on piles previously driven. The pile driver was then started for about six or seven consecutive blows and the dynamic stresses and displacements were recorded.

The entire field procedure was designed such that the data could be obtained in a manner such that the contractor would not be unduly delayed. This was necessary since the contractor received no special monetary compensation for his cooperation in this pile research.

Test Data. Figures 5 and 6 are typical examples of the oscillograph records of the dynamic strains and displacements for a single blow on Test

Pile 7. This pile had penetrated 19 ft into the ground. Gage 1 was located at the head of the pile, gage 2 at the quarter point, gage 3 at the mid-point, gage 4 at the three-quarter point, and gage 5 at the point of the pile. As shown in Figure 5, the maximum compressive stress occurred at gage 1 and is about 2576 psi. The maximum tensile stress occurred at gage 3 and is about 464 psi. The maximum displacement is about 0.7 in. and the permanent set about 0.6 in. The temporary elastic compression of the ground and pile is then about 0.1 in. in this case. The vertical scale of strain or stress shown on the figures is only approximate, since each strain gage channel had a separate calibration.

The vertical lines on the figures are time lines and are spaced at 0.01sec. intervals. The longitudinal frequency of vibration is seen to be about 290 cycles per sec.

These two piles had a final prestress of about 710 psi (Figure 1) and the concrete had an additional tensile strength of about 520 psi (see Table 1). This indicates these piles should withstand a measured tensile stress of about 1230 psi without failure. The maximum compressive strength of the concrete was about 6570 psi (see Table 1). Keeping this in mind, it is interesting to note Table 2 which gives a summary of the maximum tensile and compressive stresses recorded in both the test piles. The maximum tension recorded was 696 psi in Test Pile 7; however, values of around 400 to 500 psi were more common. The maximum compressive stress recorded was 3350 psi in Test Pile 6; however, values of around 2500 to 2900 psi were more common.

A complete tabulation of the maximum tensile and compressive stress recorded by each gage from each blow of the hammer is presented in the Appendix.

Table 2. Maximum Measured Stresses in Prestressed Concrete Piles (Test Piles 6 and 7)

Depth of Pile in Ground (ft)	Computed Hammer Drop (ft)	Average Penetration per Blow in In.		Maximum Stresses*	
		Set	Quake	Compression (psi)	Tension (psi)
Test Pile No. 6 (26' long)					
2	3.80			1514 (gage 2)	358 (gage 4)
3	5.13			2452	472 (gage 4)
12	4.87			2208	488 (gage 4)
15	5.22			2543	358 (gage 4)
19	5.68			2772	407 (gage 4)
20	5.70			2818	358 (gage 4)
22.5	6.34	0.69	0.15	3350	495
24	6.06			2970	456
Test Pile No. 7 (26' long)					
2	3.45			1818	618
11	4.79			2121	464
15	5.27			2379	464
18	6.56	0.66	0.16	2906 (gage 2)	541
19	5.60	0.53	0.15	2906 (gage 2)	696
20	5.68			2738 (gage 2)	387
25	5.27			2788	541

*Maximum compressive stress occurred at head of pile (gage 1) unless noted otherwise. Maximum tensile stress occurred at center of pile (gage 3) unless noted otherwise.

COMPUTER CORRELATION

Problem Simulation. For the digital computer solution of these pile problems, the actual pile is simulated as shown in Figure 7. In order to accomplish this simulation (1), various physical data concerning the ram, anvil, capblock, etc. were obtained from either the pile driver manufacturer, field observations, laboratory tests, or estimated using engineering judgment.

Ram. The weight of the steel ram, W(1), was 4850 lb. It was about 15 in. in diameter and about 8.45 ft. high. Its stiffness was calculated to be

$$K(\text{ram}) = \frac{AE}{L} = 50 \times 10^6 \text{ lb/in.}$$

where

K = stiffness in lb/in.,

A = cross-sectional area in square in.,

L = length in in., and

E = modulus of elasticity in psi (30×10^6 psi for steel).

Its coefficient of restitution was assumed to be $e = 1.0$.

The velocity of the ram at impact with the anvil was computed from its height of fall in the following manner. Referring to Figure 4, it can be seen that the ram is free falling until it passes the exhaust ports on the side of the diesel cylinder. After a mathematical investigation of the effect of the compressed diesel fuel on the ram velocity, it was concluded that the velocity of the ram at impact was essentially the same as the free-fall velocity at the instant it passed the exhaust ports. Therefore the ram velocity at impact is found by

$$V = \sqrt{2g\,(h-1.25)}$$

where

V = ram velocity in ft/sec.,

g = acceleration due to gravity (32.2 ft/sec^2),

h = total fall of ram in ft, and

1.25 = distance from center of exhaust port to anvil striker face in ft.

In addition to the energy transmitted to the pile by the falling ram, the explosion pressure from the diesel fuel was also included. This was accomplished by holding the maximum explosion pressure of 158,700 lb. (see Figure 3 for D-22 technical data) on top of the anvil for a period of 1/100 sec. after the ram impact.

Anvil. The weight of the steel anvil, W(2), was 1150 lb. It was about 15 in. in diameter and about 24 in. high. Its stiffness is calculated to be

$$K(\text{anvil}) = \frac{AE}{L} = 210 \times 10^6 \text{ lb/in.}$$

In this problem the spring stiffness K(1) was assigned a composite stiffness of both the ram and the anvil. Thus

$$K(1) = \frac{K(\text{ram}) \, . \, K(\text{anvil})}{K(\text{ram}) \times K(\text{anvil})} = 40.5 \times 10^6 \text{ lb/in.}$$

Also

$$e(1) = 1.0.$$

Capblock. The composition of the capblock was unknown, but it was assumed to be a one in. thick plywood disk with a contact diameter of 19.74 in. because this type capblock is commonly used with this make hammer. Since the driving force was perpendicular to the grain of the wood it was assumed to be compressed to a thickness of 1/2 in. and laboratory tests previously performed on this type material indicated that its modulus of elasticity would probably be about 40,000 psi. Its spring stiffness $K(2)$ was calculated to be

$$K(2) = \frac{AE}{L} = 24.5 \times 10^6 \text{ lb/in.}$$

The coefficient of restitution of this well-compressed wood was assumed to be

$$e(2) = 0.5$$

Pile Cap (Helmet). The weight of the helmet, $W(3)$ was estimated to be 1200 lb.

Cushion Block. The cushion block was 18 in. square and 4 in. thick. It was made of pine plywood and the driving force was applied perpendicular to its grain. After several hundred hammer blows it was compressed to about a three in. thickness and previous laboratory tests on this type material indicated its modulus of elasticity was about 40,000 psi. Its contact area with the pile was equal to the cross-sectional area of the pile. Its spring stiffness was

$$K(\text{cushion}) = \frac{AE}{L} = 3.39 \times 10^6 \text{ lb/in.}$$

The coefficient of restitution of the pine plywood cushioning material was assumed to be 0.5.

Concrete Pile. Test Piles 6 and 7 were 26 ft. long and they had a cross-sectional area of 254 square in. The concrete weighed 154 lb. per cubic ft. and had a modulus of elasticity of 7.84×10^6 psi. For computer simulation the pile was divided into eight segments of equal length, 3.25 ft. each. The weights of the segments, W(4) through W(11), were 883 lb. each. The spring stiffnesses of the pile segments were

$$K(\text{pile}) = \frac{AE}{L} = 51.0 \times 10^6 \text{ lb/in.}$$

E. A. Smith (1) suggested that the use of internal damping in the pile material might be desirable to account for energy losses resulting from stress-strain hysteresis. Although no data were available that suggested a value for such a damping property, it was found that a better agreement between the measured and computed stresses resulted when a value of $B = 0.002 \frac{\text{in.-sec}}{\text{ft}}$ was used. Consequently, this value was used to compute the results presented in this paper.

Referring to Figure 7, it can be seen that spring K(3) should have a composite stiffness of both the cushion block and the first concrete pile segment. Thus

$$K(3) = \frac{K(\text{cushion}) \cdot K(\text{pile})}{K(\text{cushion}) + K(\text{pile})} = 3.39 \times 10^6 \text{ lb/in. (approx.)}$$

and

$$e(3) = 0.5 \text{ (approx.)}.$$

All other springs, K(4) through K(11), have stiffnesses equal to that of the pile segments, 51.0×10^6 lb/in.

Soil Resistance. In order to complete the simulation of this pile problem, certain values must be assigned to certain constants that describe the soil resistance on the pile during driving. The values presently defined are the ultimate static soil resistance, Ru, the damping capacity of the soil, J or J', and the soil "quake" or elastic deformability. Up to the present time no experiments have been performed to determine accurately the last two constants, damping and "quake". The values of the ultimate frictional soil resistance are given in Figure 2. Although these were predominantly skin-friction piles, some point resistance was also present. The point resistance was computed by the equation

$$Ru(\text{point}) = 1.3 \times 5.7 \times C \times A$$

where

Ru(point) = point resistance on pile in kips,

C = cohesion of clay in ksf, and

A = area of pile point in sq ft

Since no tests have been developed for determining the damping constants or "quake" for soils, the following values were assumed:

"quake" in friction	= 0.02 in.
"quake" at point	= 0.10 in. (1)
friction damping constant	= 0.05 (1)
point damping constant	= 0.15 (1)

Computer Results. In this investigation the problems were run by an IBM 709 Computer program of the wave equation. This program was essentially the same as that described by E. A. Smith (1) and C. H. Samson (2), except that it has been modified to include the effect of gravity and separate frictional and point forces on the last pile segment. A comparison of the computed stresses with those measured in the field is given by Table 3. Since the strain gages were located at various points along the length of the pile, the computed stress shown was taken from the corresponding segments of the pile. The compressive stresses tabulated were taken from the gage nearest the head of the pile and the tensile stresses tabulated were taken from the gage nearest the mid-length of the pile unless noted otherwise. For the exact location of these gages, reference is made to Figure 1. This was done because, in general, the maximum measured compressive stress was near the head of the pile and the maximum measured tensile stress was near the mid-length of the pile. This is not to be construed to mean that these were the maximum stresses present in the pile. The measured stresses shown are the average of several consecutive blows (see Appendix).

The computed tensile stresses may appear high, but in view of the unknown dynamic properties of the soil, concrete, and wood materials involved in the problem and also the variable nature of the foundation, the quantitative comparisons made in Table 3 are considered good.

To illustrate the computer output of the theoretical stresses, Table 4 shows the computer listing of the compressive and tensile stresses in certain segments of Test Pile 7 at 19 ft penetration into the ground. The time shown is in 1/10,000 sec. Referring again to Figure 7, it can be seen that segment 3 is at the head of the pile, segment 5 is at the quarter point, segment 7 is at the mid-point, segment 9 the three-quarter point, and segment 11 is at the point. Table 5 shows the maximum compressive and tensile stresses computed in each pile segment(segments 4 through 11). For this particular problem the absolute maximum compressive stress, -3022 psi, was at segment 3 at the head of the pile and the absolute maximum tensile stress, + 804 psi, was at segment 7 the mid-point.

In order to make qualitative and quantitative comparison of the computer results with the recorded oscillograph stress data, Figures 8, 9, and 10 are presented. These figures show a computer plot of the stress versus time (in 1/10,000 seconds) for segments at the head, mid-length, and tip of Test Pile 7 (segments 3, 7, and 11, respectively). The solid line is the computed stress and the dashed line the measured stress. Figure 11 shows a comparison of the computed displacement of the pile head with the measured displacement.

While it may be argued that these comparisons leave something to be desired, they are reasonable considering the number of unknown factors which must be estimated. It is believed that when experimental values of material properties are available, the computer solution of the wave equation will more accurately predict pile behavior.

Table 3. Comparison of Computed Stresses with Average Measured Stresses

Depth of Pile in Ground (ft)	Computed Hammer Drop(ft)	Permanent Set Per Blow (in.)		Comparison of Computed Stresses with Average Measured Stresses -Compression* psi		+Tension* psi	
		Computed	Measured	Computed	Measured	Computed	Measured
Test Pile No. 6							
2	3.73			-2248	-1404	+ 578	+293 (gage 4)
3	5.07			-2951	-2433	+1201	+433
12	4.54			-2760	-2104	+ 798	+342
15	5.22			-3004	-2512	+ 476	+358 (gage 4)
19	5.58			-3127	-2681	+ 496	+354 (gage 4)
20	5.57			-3125	-2761	+ 499	+346 (gage 4)
22.5	6.10	0.72	0.69	-3300	-3038	+1004	+402
24	5.91			-3250	-2894	+ 987	+407
Test Pile No. 7							
2	3.28			-2251	-1394	+ 798	+452
11	4.74			-2835	-2050	+ 933	+361
15	5.23			-3004	-2333	+ 476	+441
18	5.48	0.85	0.66	-3105	-2470	+ 854	+415
19	5.27	0.74	0.53	-3022	-2486	+ 804	+442
20	5.50			-3101	-2592	+ 842	+428
25	5.50			-3124	-2680	+ 941	+449

*Maximum compressive stress occurred at head of pile (gage 1) unless noted otherwise. Maximum tensile stress occurred at center of pile (gage 3) unless noted otherwise.

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                         TEXAS A&M UNIVERSITY
                         PILE DRIVING ANALYSIS
                         CASE NUMBER   HPS 27
                         PROBLEM NUMBER    19

INPUT DATA

OPTIONS    1  2  3  4  5  6  7  8  9  10
           1  1  2  2  2 -0 -0 -0 -0  -0

1/(DEL T) =    10000.0000                     RU(TOTAL) =     -0.

RU(POINT) =       -0.                          IVEL(1) =     16.13000

        P =       11       QUAKE = 0.020        QUAKP  =      0.100

        J =        0.15000                      JPRIME =      0.05000

  ERES(1) =        1.00000                      ERES(2)=      0.50000

  GAMMA 1 =   158700.00                         GAMMA 2=     -0.

      N 1 =      200                               N2 =    1000

       MO =       -0

M       K(M)             B(M)                    ERES(M)           GAMMA(M)

 1   40500000.0      -0.                         1.00000           158700.00
 2   24500000.0      -0.                         0.50000               -0.
 3    3390000.0      -0.                         0.50000               -0.
 4   51000000.0       0.00200000                 0.                   -1.00
 5   51000000.0       0.00200000                 0.                   -1.00
 6   51000000.0       0.00200000                 0.                   -1.00
 7   51000000.0       0.00200000                 0.                   -1.00
 8   51000000.0       0.00200000                 0.                   -1.00
 9   51000000.0       0.00200000                 0.                   -1.00
10   51000000.0       0.00200000                 0.                   -1.00
11     226000.0       0.                         0.                    0.

M         W(M)            RU(M)                  IVEL(M)

 1       4850.000          -0.                  16.13000
 2       1150.000          -0.                   0.
 3       1200.000          -0.                   0.
 4        883.000          -0.                   0.
 5        883.000          -0.                   0.
 6        883.000        6360.00                 0.
 7        883.000       10570.00                 0.
 8        883.000       17480.00                 0.
 9        883.000       23200.00                 0.
10        883.000       29900.00                 0.
11        883.000       29900.00                 0.
12         0.           22600.00                 0.
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TABLE 4

TEXAS A&M UNIVERSITY
PILE DRIVING ANALYSIS
CASE NUMBER HPS 27
PROBLEM NUMBER 19

STRESSES IN PSI (-COMPRESSION, +TENSION) FOR SEGMENTS 3, 5, 7, 9, 11

TIME	SEGMENT 3	SEGMENT 5	SEGMENT 7	SEGMENT 9	SEGMENT 11
0	-9.	-16.	-19.	-15.	-2.
2	-9.	-16.	-19.	-15.	-2.
4	-25.	-16.	-19.	-15.	-2.
6	-132.	-22.	-19.	-15.	-2.
8	-442.	-69.	-21.	-15.	-2.
10	-996.	-230.	-41.	-16.	-2.
12	-1588.	-570.	-121.	-24.	-2.
14	-2079.	-1041.	-316.	-63.	-2.
16	-2448.	-1537.	-640.	-167.	-2.
18	-2687.	-1987.	-1049.	-361.	-2.
20	-2814.	-2350.	-1480.	-631.	-3.
22	-2880.	-2606.	-1873.	-923.	-6.
24	-2939.	-2770.	-2193.	-1177.	-11.
26	-3005.	-2871.	-2393.	-1359.	-21.
28	-2977.	-2928.	-2478.	-1472.	-36.
30	-2555.	-2907.	-2464.	-1486.	-58.
32	-1904.	-2654.	-2364.	-1417.	-87.
34	-1349.	-2137.	-2124.	-1299.	-124.
36	-1081.	-1502.	-1701.	-1135.	-168.
38	-902.	-947.	-1144.	-897.	-202.
40	-568.	-523.	-577.	-575.	-206.
42	-137.	-129.	-104.	-211.	-206.
44	-0.	275.	273.	104.	-201.
46	-0.	550.	578.	315.	-192.
48	-0.	637.	770.	426.	-181.
50	-0.	596.	792.	454.	-169.
52	-0.	467.	657.	393.	-157.
54	-9.	281.	410.	248.	-147.
56	-13.	61.	104.	41.	-139.
58	-14.	-160.	-215.	-188.	-133.
60	-20.	-356.	-504.	-402.	-131.
62	-46.	-511.	-731.	-570.	-132.
64	-105.	-619.	-872.	-674.	-135.
66	-202.	-684.	-924.	-710.	-139.
68	-329.	-716.	-899.	-683.	-143.
70	-464.	-728.	-820.	-610.	-146.
72	-599.	-725.	-718.	-515.	-149.
74	-733.	-717.	-618.	-422.	-150.
76	-870.	-718.	-540.	-349.	-149.
78	-1008.	-741.	-501.	-308.	-148.
80	-1140.	-798.	-509.	-303.	-146.
82	-1243.	-890.	-567.	-332.	-144.
84	-1311.	-1003.	-671.	-391.	-143.
86	-1348.	-1118.	-805.	-474.	-143.

TABLE 4 (CONTINUED)

STRESSES IN PSI (-COMPRESSION, + TENSION) FOR SEGMENTS 3, 5, 7, 9, 11

TIME	SEGMENT 3	SEGMENT 5	SEGMENT 7	SEGMENT 9	SEGMENT 11
88	-1367.	-1222.	-947.	-571.	-144.
90	-1378.	-1307.	-1076.	-667.	-146.
92	-1377.	-1369.	-1176.	-746.	-149.
94	-1290.	-1405.	-1238.	-798.	-153.
96	-1123.	-1379.	-1259.	-818.	-158.
98	-946.	-1270.	-1226.	-808.	-163.
100	-822.	-1098.	-1126.	-767.	-167.
102	-754.	-912.	-965.	-693.	-171.
104	-664.	-749.	-770.	-583.	-174.
106	-531.	-599.	-578.	-451.	-175.
108	-397.	-443.	-411.	-323.	-174.
110	-307.	-289.	-267.	-220.	-172.
112	-260.	-166.	-151.	-152.	-168.
114	-185.	-96.	-72.	-112.	-164.
116	-69.	-57.	-43.	-95.	-159.
118	-0.	-23.	-56.	-102.	-155.
120	-0.	-9.	-88.	-131.	-151.
122	-0.	-42.	-126.	-169.	-147.
124	-0.	-98.	-180.	-205.	-144.
126	-0.	-158.	-244.	-237.	-141.
128	-3.	-208.	-305.	-269.	-139.
130	-12.	-246.	-353.	-299.	-138.
132	-28.	-269.	-385.	-322.	-136.
134	-50.	-283.	-398.	-334.	-135.
136	-76.	-289.	-395.	-334.	-135.
138	-105.	-291.	-382.	-322.	-134.
140	-136.	-291.	-362.	-302.	-133.
142	-167.	-292.	-341.	-282.	-132.
144	-197.	-294.	-324.	-264.	-130.
146	-228.	-300.	-314.	-252.	-128.
148	-258.	-312.	-314.	-248.	-127.
150	-284.	-330.	-324.	-252.	-125.
152	-303.	-353.	-343.	-263.	-123.
154	-316.	-379.	-369.	-278.	-122.
156	-323.	-402.	-398.	-296.	-121.
158	-327.	-422.	-426.	-315.	-120.
160	-329.	-437.	-448.	-332.	-119.
162	-322.	-445.	-462.	-343.	-118.
164	-295.	-445.	-467.	-348.	-118.
166	-257.	-429.	-463.	-346.	-118.
168	-220.	-396.	-446.	-338.	-118.
170	-194.	-354.	-416.	-323.	-117.
172	-171.	-311.	-374.	-302.	-117.
174	-141.	-272.	-329.	-274.	-116.
176	-109.	-233.	-286.	-244.	-115.
178	-85.	-194.	-248.	-216.	-113.
180	-73.	-162.	-215.	-193.	-111.
182	-67.	-141.	-190.	-177.	-109.

TABLE 4 (CONTINUED)

STRESSES IN PSI (-COMPRESSION, +TENSION) FOR SEGMENTS 3, 5, 7, 9, 11

TIME	SEGMENT 3	SEGMENT 5	SEGMENT 7	SEGMENT 9	SEGMENT 11
184	-56.	-133.	-176.	-168.	-107.
186	-46.	-131.	-175.	-165.	-104.
188	-40.	-134.	-183.	-169.	-102.
190	-41.	-142.	-197.	-178.	-100.
192	-45.	-155.	-214.	-190.	-98.
194	-51.	-171.	-232.	-203.	-96.
196	-58.	-188.	-250.	-213.	-95.
198	-67.	-204.	-266.	-222.	-93.
200	-79.	-216.	-277.	-228.	-92.
202	-93.	-227.	-285.	-231.	-91.
204	-110.	-235.	-288.	-232.	-90.

TABLE 5

MAXIMUM COMPRESSIVE AND TENSILE STRESSES (PSI) IN THE SEGMENTS

SEGMENT	TIME	STRESS	TIME	STRESS
1	3	-7857.	204	-0.
2	7	-6788.	192	-0.
3	26	-3022.	125	-0.
4	27	-3001.	47	369.
5	28	-2935.	47	637.
6	28	-2763.	48	793.
7	28	-2483	48	804.
8	28	-2058.	49	681.
9	28	-1494.	49	454.
10	29	-800.	49	166.
11	40	-207.	0	-0.

PERMANENT SET OF PILE = 0.74024615 INCHES

NUMBER OF BLOWS PER INCH = 1.35090198

CONCLUSIONS

As a result of this field study of the dynamic stresses in and displacements of relatively short prestressed concrete piles driven in a firm clay, the following conclusions are drawn.

1. Maximum compressive stresses occurred at the head of the pile when firm resistance to penetration was encountered. Typical measured values ranged from 2000 to 3350 psi.
2. Maximum tensile stresses were found to occur at the mid-point and three-quarter point of the piles. Typical measured values ranged from 300 to 696 psi. The actual net tensile stress in the concrete is obtained by subtracting the prestressing force of about 713 psi from the measured values. It is apparent that the concrete in these short piles probably did not experience any net tensile stresses.
3. The computed stresses and displacements agreed fairly well with the measured data. It was indicated, however, that more effort needs to be directed toward determining the dynamic material properties involved.
4. The magnitude of the tensile stresses in these short piles was only about one-half of those measured previously in piles 92 and 95 ft in length. The compressive stresses were about the same magnitude.

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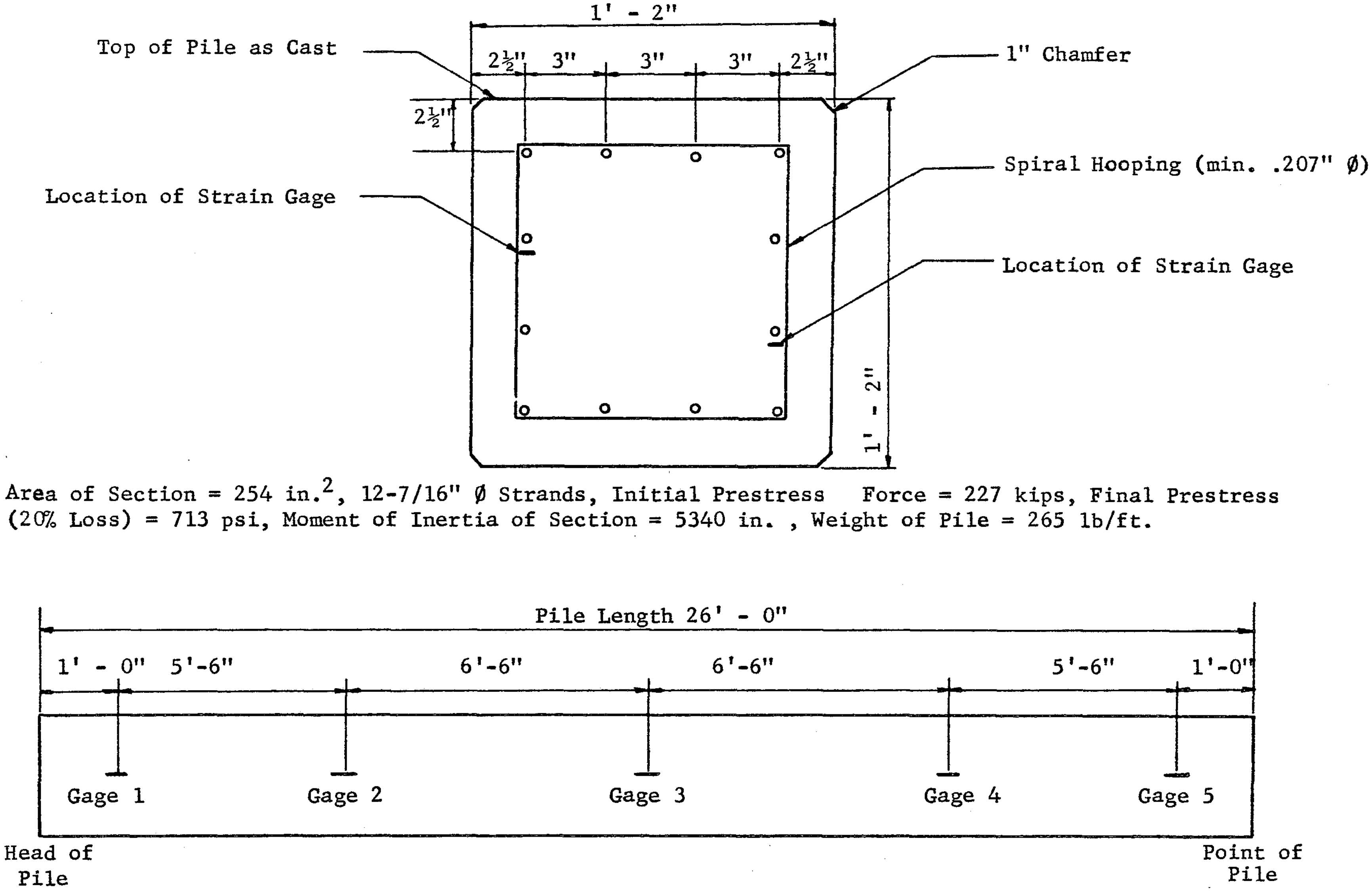


Area of Section = 254 in.^{2}, 12-7/16" Ø Strands, Initial Prestress Force = 227 kips, Final Prestress (20% Loss) = 713 psi, Moment of Inertia of Section = 5340 in. , Weight of Pile = 265 lb/ft.

Figure 1. Cross-section and Side View of Test Piles and Location of Strain Gages. (Test Piles 6 and 7)

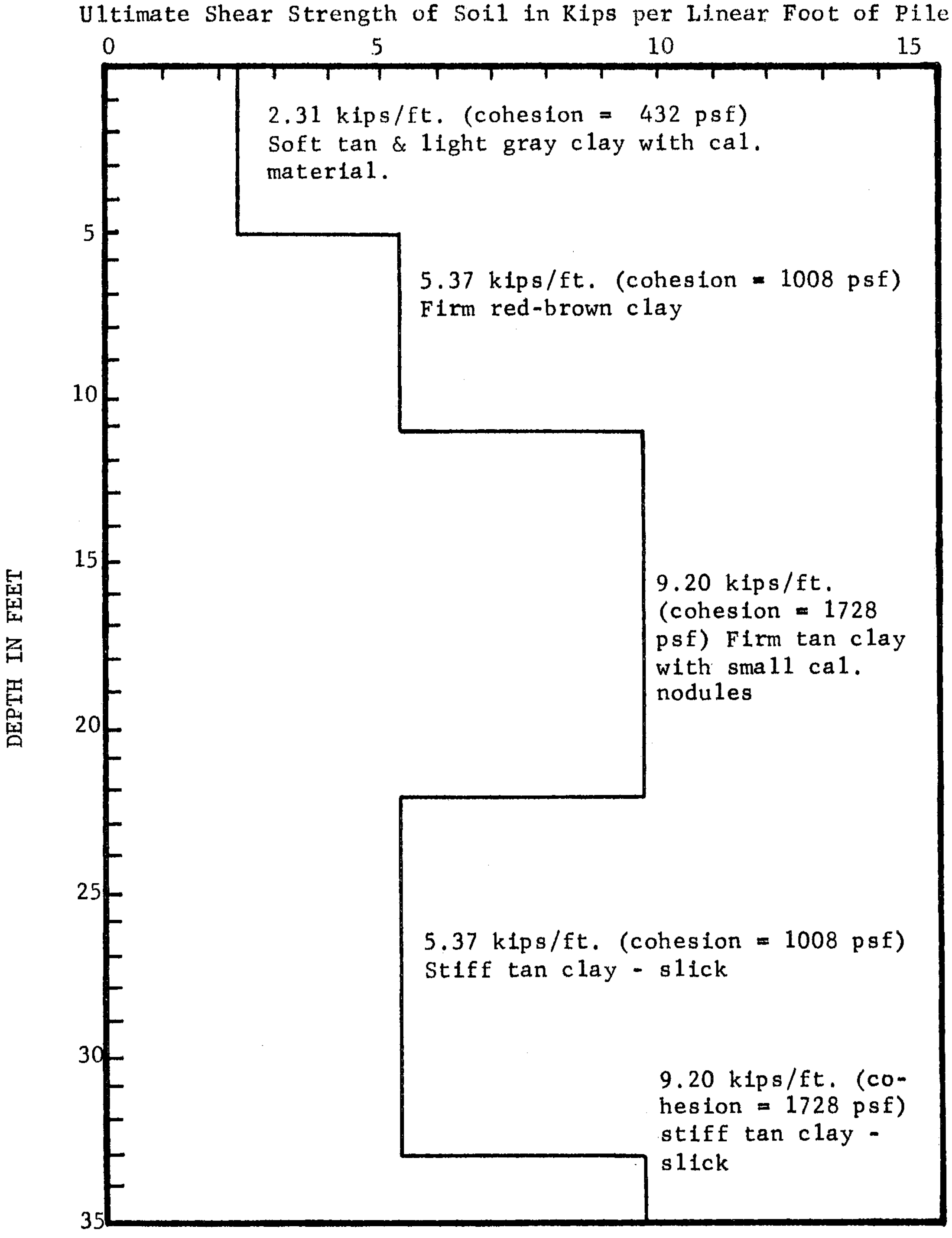


Figure 2. Ultimate Shear Strength and Description of Soil at Various Depths in the Ground. (Test Piles 6 and 7)

TECHNICAL DATA

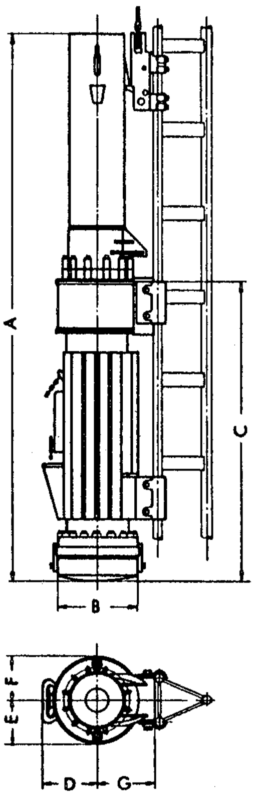
Piston weight, lbs. 4850

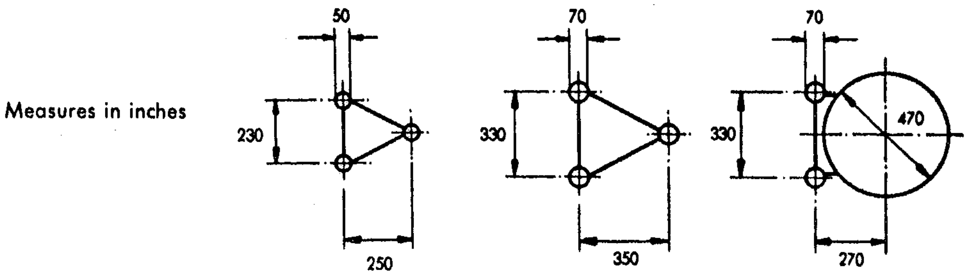
Measures in inches

A	154 1/2
B	22 53/64
C	83 17/64
D	14 3/8
E	11 13/16
F	12 33/64
G	15 5/8 (on GF-22 G-112)

Example of detail measurements for Hammer Lead

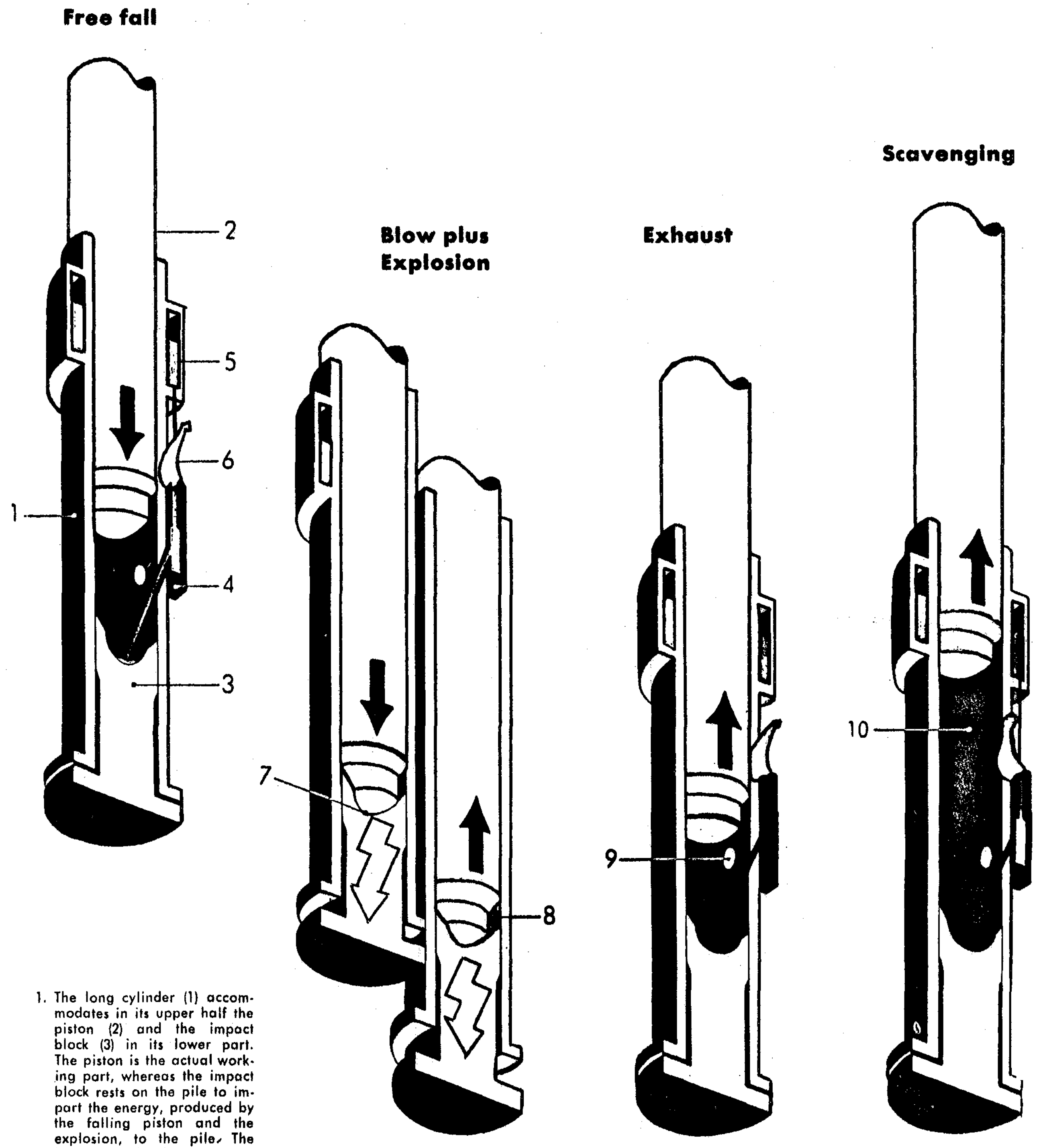
g	2 3/4
h	13
i	10 5/8
k	18 1/2





Piston weight	4,850	lbs.
Weight of hammer (without accessories)	9,768	lbs.
Accessories: tripping device	286	lbs.
transport slide	375	lbs.
tool-kit	326	lbs.
Shipping weight net (hammer + accessories	10,755	lbs.
Shipping weight gross	11,964	lbs.
Storage space	230	cu. ft.
Weight of anvil	1,147	lbs.
Number of blows	42-60	per min.
Energy output per blow	39,800	ft. lbs.
Maximum explosion pressure on pile	158,700	lbs.
Fuel consumption, continuous working	3.44	U.S. gal. per hour
Oil consumption, continuous working	0.39	U.S. gal.
Fuel tank capacity	10.2	U.S. gal.
Oil chamber capacity	7.0	U.S. qts.

Figure 3 Technical Data for Delmag Diesel Hammer Model D22



Free fall

Blow plus Explosion

Exhaust

Scavenging

1. The long cylinder (1) accommodates in its upper half the piston (2) and the impact block (3) in its lower part. The piston is the actual working part, whereas the impact block rests on the pile to impart the energy, produced by the falling piston and the explosion, to the pile. The impact block also serves to seal off the combustion chamber at the lower end. The fuel pump (4) and the fuel tank (5) are attached to the cylinder. To start the hammer the piston (2) is lifted and, when reaching a certain height, is automatically released. During the downward fall of the piston (2) a pump lever (6) on the fuel pump (4) is activated injecting a fixed amount of Diesel Fuel into the combustion chamber at a pressure of 1,5 atmospheres.

2. As the piston (2) continues to fall it closes the exhaust ports, compresses the remaining air in the cylinder and hits the concave ball pan of the impact block. The impact of the falling piston (2) atomizes the Diesel fuel lying in the ball pan, and the highly compressed air causes these atomized fuel particles to ignite. The combustion pressure thus created exerts an additional force onto the pile, which is already travelling downward under the compression force developed by the falling piston, and the blow from the piston further serves to throw the piston (2) up for the next working cycle.

3. As the piston clears the exhaust ports (9) in its upward motion the internal and external pressures are equalized.

4. As the piston continues its upward motion fresh air is drawn into the cylinder (10), which is thus being scavenged. The pump lever (6) is now released, allowing a new charge of Diesel fuel to enter the fuel pump (4).

Figure 4 Working Principle of Delmag Diesel Pile Hammers

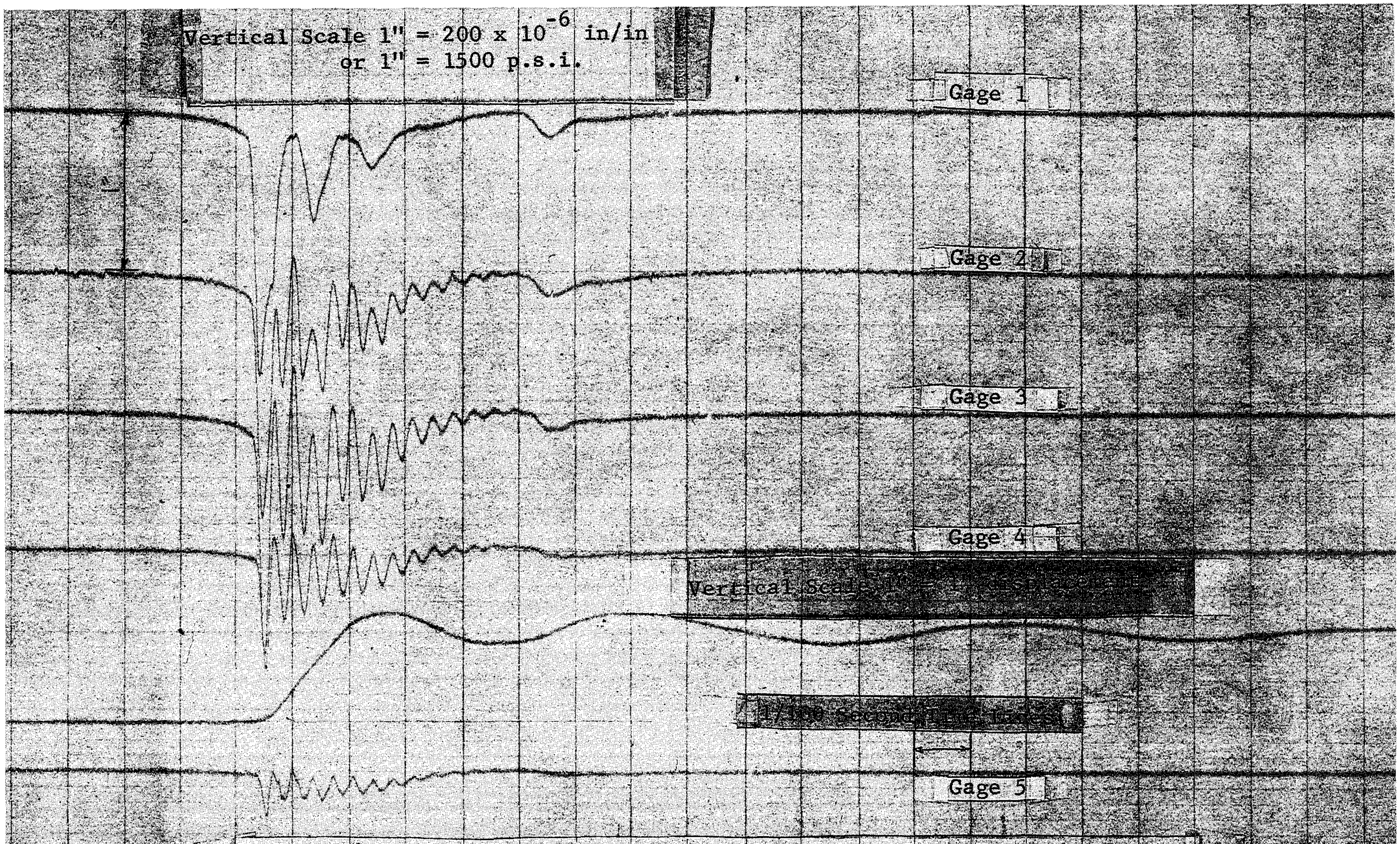
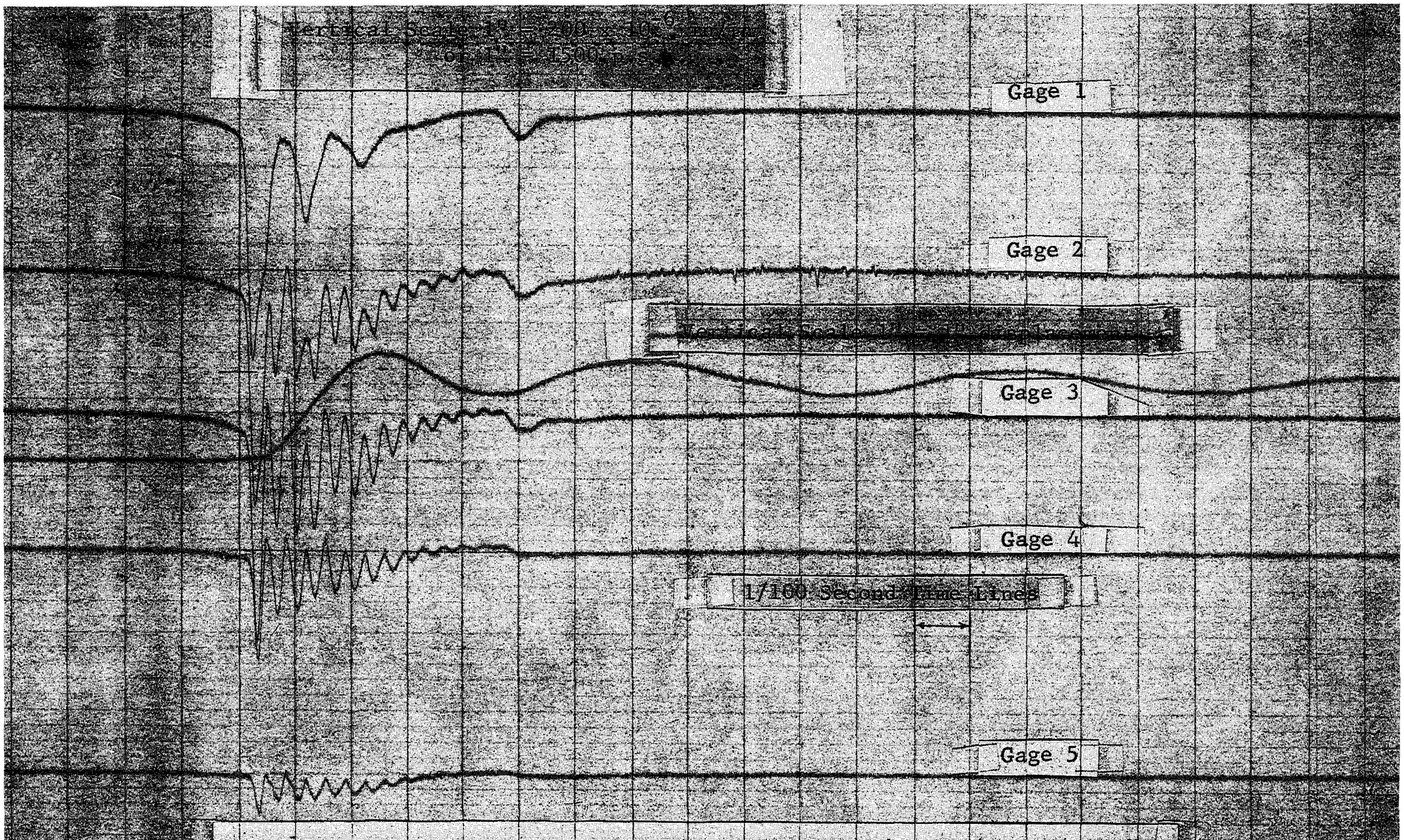


Figure 5. 16" Square Prestressed Concrete Pile 26' long (Test Pile 7) Penetration in Ground 19'



~~Figure~~ 6. 16" Square Prestressed Concrete Pile 26' long (Test Pile 7)
Penetration in Ground 19'

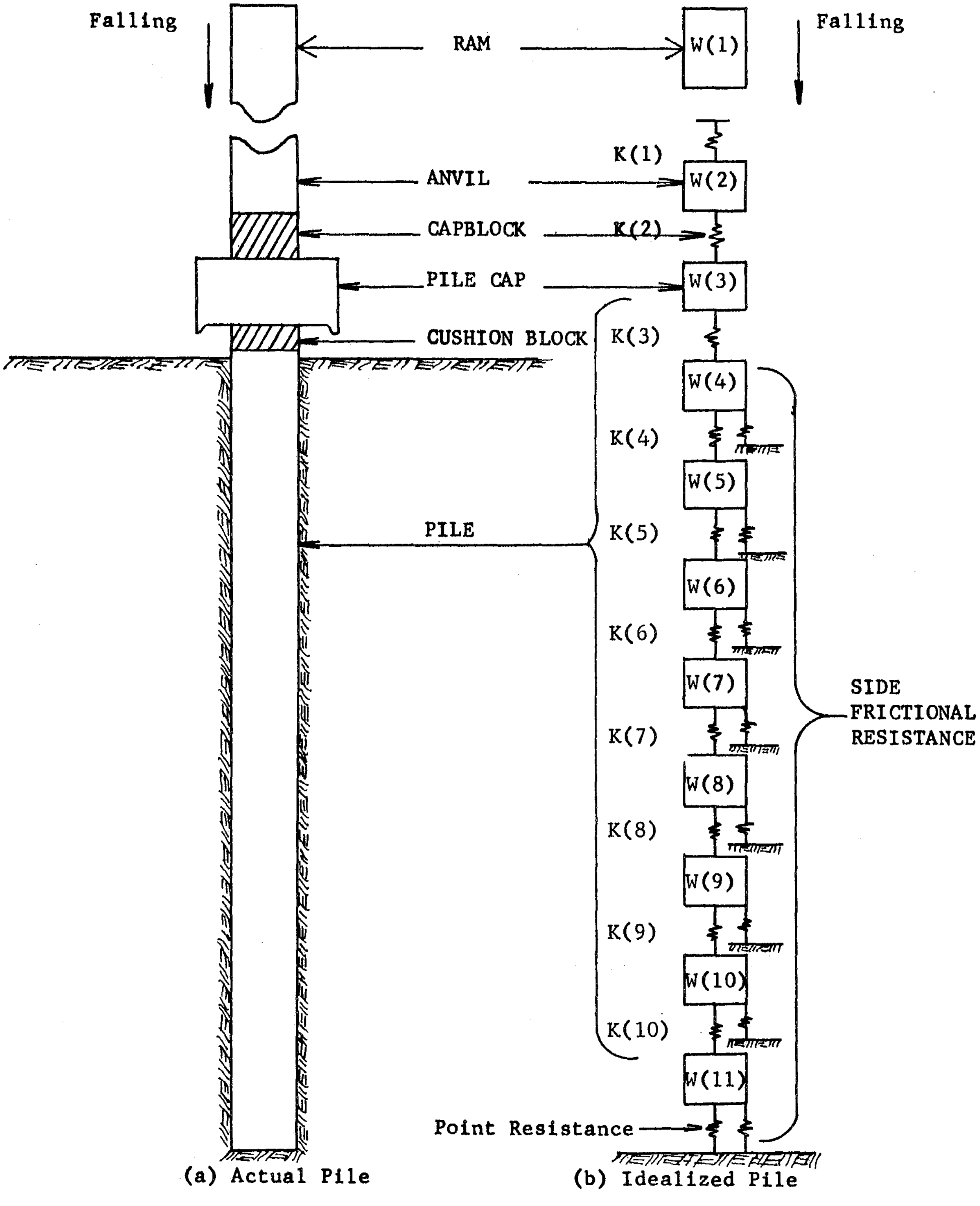


Figure 7. Method of Idealizing a Pile for Purpose of Analysis. This pile was divided into eight segments of equal lengths. Segment 1 is the ram, 2 is the anvil, 3 is the helmet, and 4 is the first segment of the pile.

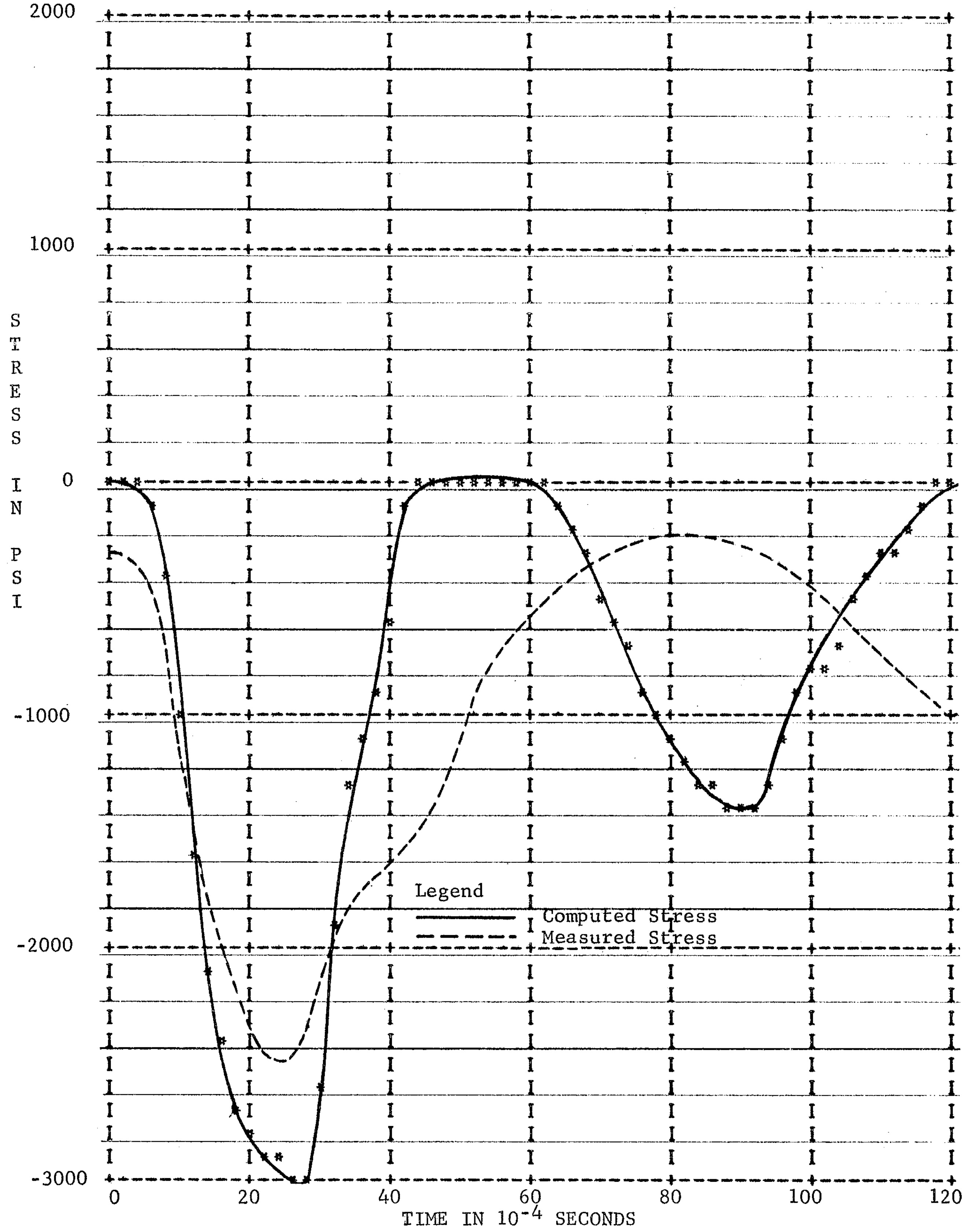


FIGURE 8. Stress at Pile Head vs Time
Test Pile 7, 19' Penetration in Ground

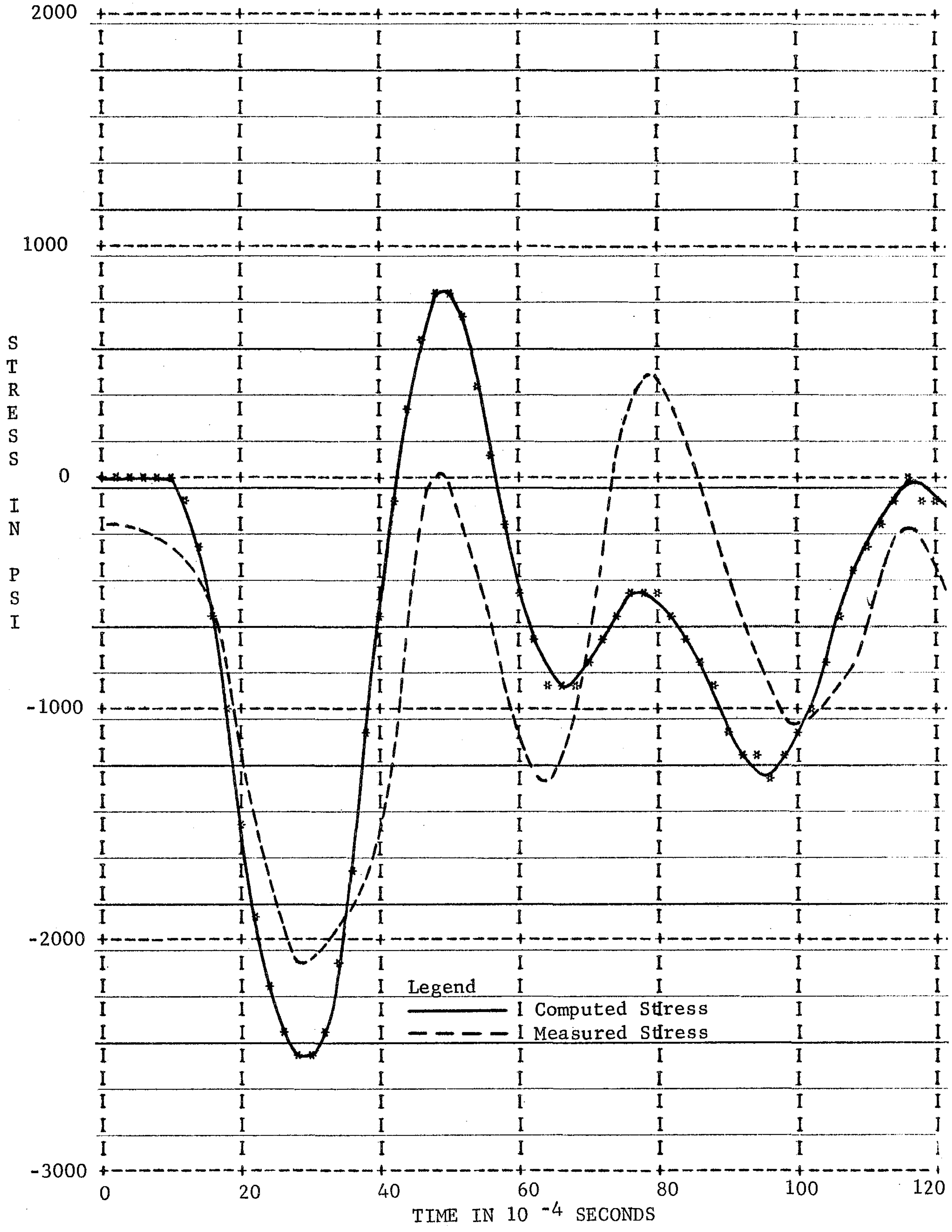


FIGURE 9. Stress at Mid-Length of Pile vs Time
Test Pile 7, 19' Penetration in Ground

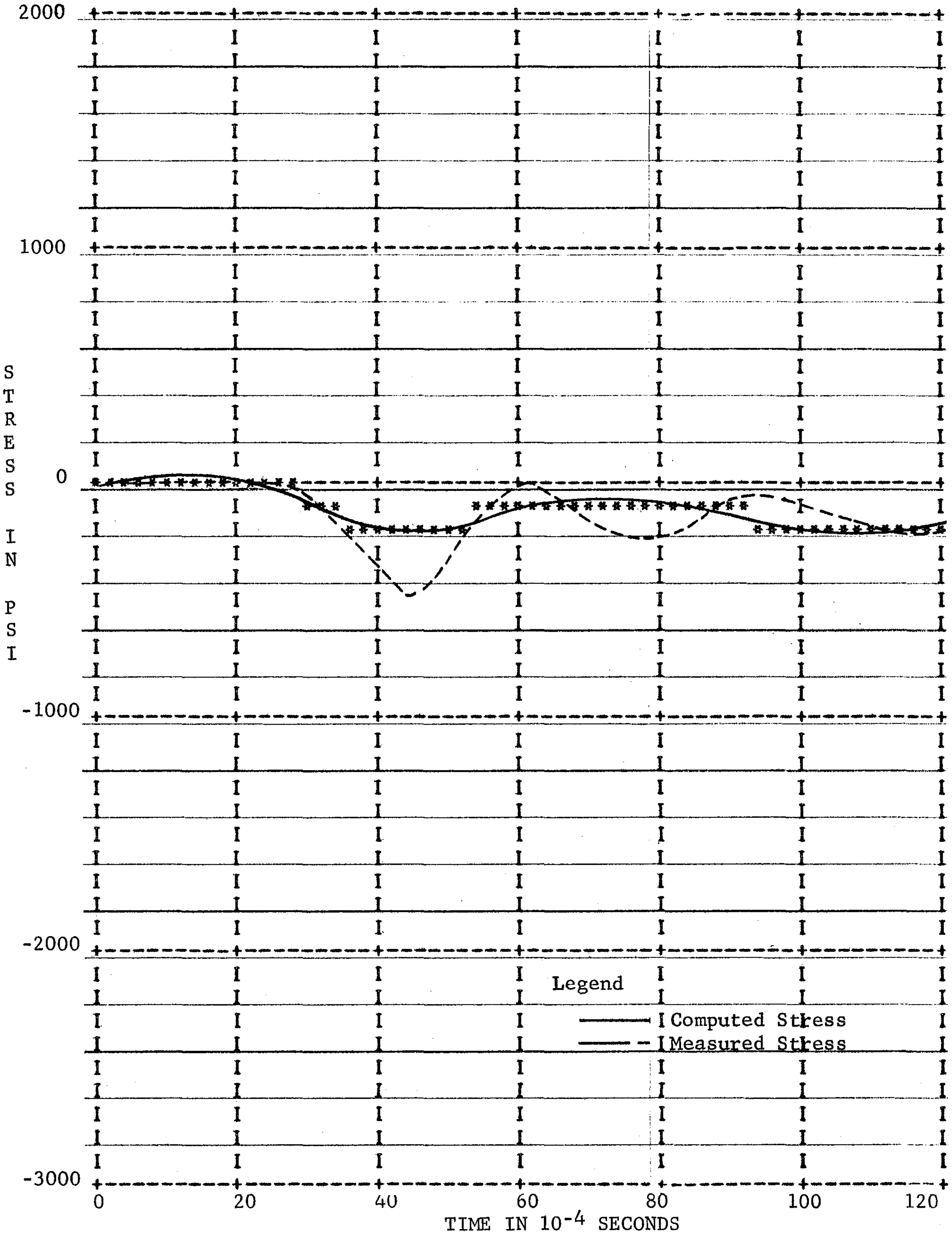


FIGURE 10. Stress at Point of Pile vs Time
Test Pile 7, 19' Penetration in Ground

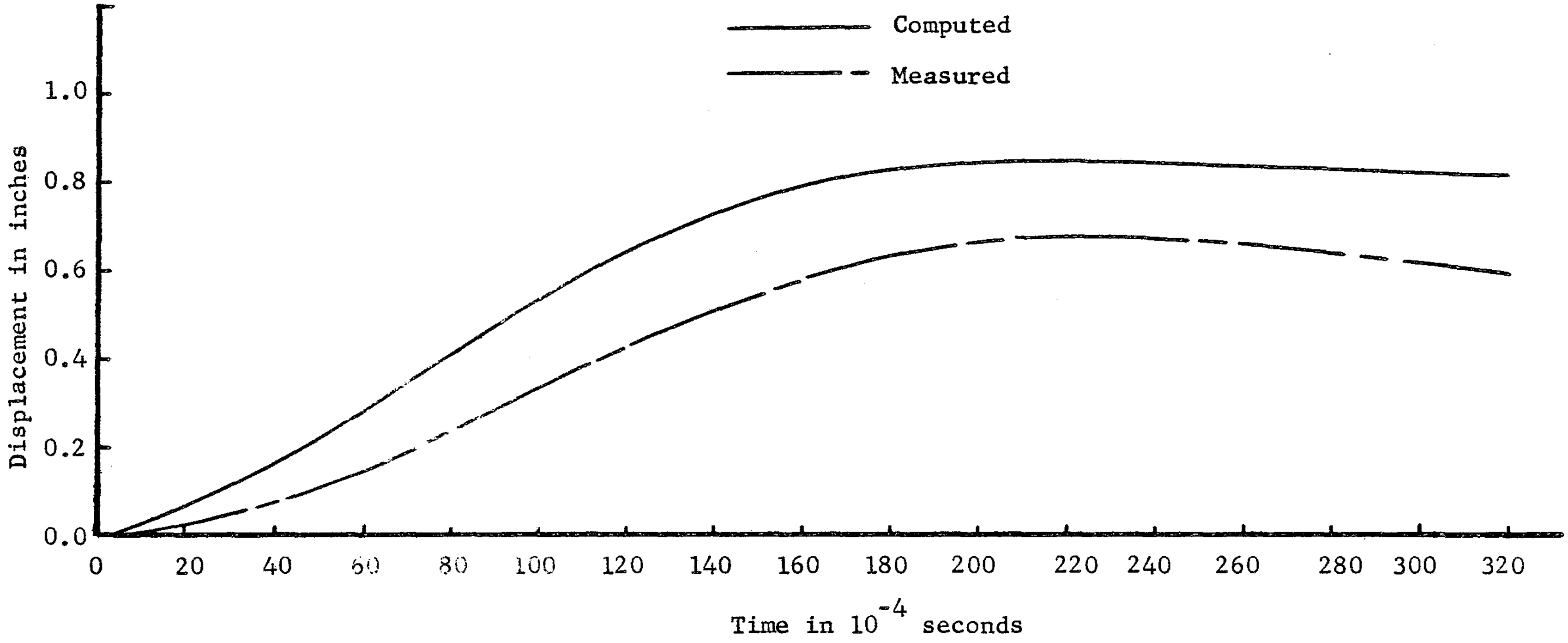


Figure 11. Displacement of pile head <u>vs</u> time
Test Pile 7, 19' penetration in the ground.

APPENDIX

Test Pile No. 6

Depth of Pile in Ground Feet	Computed Hammer Drop Feet	Penetration Per Blow In Inches Set	Penetration Per Blow In Inches Quake	(-) Compression and (+) Tension Stress in Concrete Pile Gage 1	Gage 2	Gage 3	Gage 4	Gage 5
9				-1218	-1104	-1036	- 814	-251
				+ 30	+ 158	+ 278	+ 179	+ 63
				-1355	-1151	-1113	- 830	-282
	3.56			+ 30	+ 158	+ 247	+ 325	+ 31
				-1462	-1325	-1268	- 976	-282
	3.77			+ 30	+ 142	+ 247	+ 358	+ 31
				-1508	-1388	-1268	-1009	-267
	3.83			+ 30	+ 110	+ 294	+ 293	+ 31
				-1477	-1514	-1237	-1003	-267
	3.77			+ 30	+ 173	+ 294	+ 309	+ 31
	3.73 avg.			-1404*	-1296*	-1184*	- 927*	-270*
				+ 30*	+ 148*	+ 272*	+ 293*	+ 37*
3				-2422	-2145	-2087	-1399	-424
				+ 30	+ 205	+ 464	+ 472	+ 47
				-2452	-2192	-2180	-1757	-502
	5.13			+ 15	+ 221	+ 464	+ 407	+ 0
				-2437	-2145	-2118	-1741	-408
	5.07			+ 15	+ 221	+ 433	+ 390	+ 16
				-2422	-2160	-2118	-1708	-392
	5.03			+ 15	+ 31	+ 371	+ 390	+ 0
	5.07 avg.			-2433*	-2160*	-2126*	-1651*	-431*
				+ 18*	+ 170*	+ 433*	+ 414*	+ 31*

---

*Average Values

Test Pile No. 6 (Continued)

Depth of Pile in Ground Feet	Computed Hammer Drop Feet	Penetration Per Blow In Inches		(-) Compression and (+) Tension Stress in Concrete Pile				
		Set	Quake	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5
12				-2071 + 30	-1877 + 173	-1732 + 464	-1464 + 309	-392 + 0
	4.87			-2208 + 15	-2050 + 205	-1917 + 464	-1611 + 374	-408 + 16
	4.51			-2041 + 15	-1892 + 158	-1778 + 433	-1464 + 358	-424 + 16
	4.69			-2147 + 15	-1892 + 189	-1948 + 417	-1464 + 488	-408 + 0
	4.60			-2132 + 15	-1892 + 173	-1840 + 402	-1562 + 325	-408 + 16
	4.12			-1965 + 15	-1734 + 173	-1700 + 371	-1399 + 244	-361 + 0
	4.44			-2163 + 15	-1814 + 220	-1732 + 417	-1432 + 293	-392 + 16
	4.54 avg.			-2104* + 17*	-1879* + 184*	-1807* + 342*	-1485* + 342*	-399* + 9*
15				-2543 + 15	-2302 + 79	-2242 + 263	-1806 + 357	-439 + 0
				-2482 + 15	-2255 + 142	-2226 + 309	-1757 + 358	-439 + 0
	5.22 avg			-2512* + 15*	-2325* + 100*	-2234* + 286*	-1781* + 358*	-439* + 0*
19				-2589 + 0	-2366 + 95	-2288 + 340	-1806 + 325	-408 + 0
	5.43			-2665 + 0	-2366 + 158	-2288 + 356	-1871 + 407	-439 + 47

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* Average Values

Test Pile No. 6 (Continued)

Depth of Pile in Ground Feet	Computed Hammer Drop Feet	Penetration Per Blow In Inches Set	Quake	(-) Compression and (+) Tension Stress in Concrete Pile Gage 1	Gage 2	Gage 3	Gage 4	Gage 5
19				-2772	-2444	-2396	-1985	-470
	5.68			+ 0	+ 237	+ 356	+ 325	+314
				-2711	-2460	-2412	-1952	-470
	5.65			+ 0	+ 126	+ 294	+ 358	+ 0
	5.58 avg.			-2681*	-2409*	-2346*	-1904*	-447*
				+ 0*	+ 154*	+ 336*	+ 354*	+ 90*
20				-2772	-2460	-2396	-1936	-470
				+ 0	+ 16	+ 309	+ 358	+ 0
				-2741	-2444	-2381	-1920	-470
	5.53			+ 0	+ 79	+ 278	+ 325	+ 0
				-2818	-2523	-2412	-2050	-470
	5.70			+ 0	+ 63	+ 309	+ 342	+ 0
				-2711	-2460	-2319	-1871	-470
	5.49			+ 0	+ 0	+ 247	+ 358	+ 0
	5.57 avg.			-2761*	-2472*	-2377*	-1944*	-470*
				+ 0*	+ 40*	+ 286*	+ 346*	+ 0*
22.5		0.77	0.14	-3350	-2870	-2891	-2196	-580
				+ 0	+ 158	+ 417	+ 374	+ 16
		0.65	0.17	-3046	-2570	-2550	-1985	-580
	6.34			+ 0	+ 16	+ 356	+ 325	+ 31
		0.66	0.11	-2863	-2507	-2443	-1952	-455
	6.00			+ 0	+ 0	+ 340	+ 342	+ 31
		0.66	0.17	-2893	-2523	-2396	-1871	-471
	5.95			+ 0	+ 126	+ 495	+ 407	+ 0
		0.69*	0.15*	-3038*	-2617*	-2570*	-2004*	-522*
	6.10 avg.			+ 0*	+ 75*	+ 402*	+ 362*	+ 20*

---

* Average Values

Test Pile No. 6 (Continued)

Depth of Pile in Ground Feet	Computed Hammer Drop Feet	Penetration Per Blow In Inches		(-) Compression and (+) Tension Stress in Concrete Pile				
		Set	Quake	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5
24				-2924	-2523	-2505	-1952	-392
				+ 0	+ 0	+ 387	+ 325	+ 0
				-2893	-2602	-2474	-1871	-439
	5.90			+ 0	+ 158	+ 433	+ 374	+ 0
				-2894	-2634	-2470	-1952	-439
	5.92			+ 0	+ 32	+ 464	+ 390	+ 0
				-2818	-2444	-2365	-1920	-392
	5.79			+ 0	+ 126	+ 464	+ 325	+ 31
				-2970	-2602	-2551	-2083	-408
	6.06			+ 0	+ 0	+ 371	+ 374	+ 16
				-2863	-2523	-2396	-1952	-439
	5.89			+ 0	+ 0	+ 325	+ 456	+ 0
	5.91 avg.			-2894*	-2555*	-2460*	-1955*	-418*
				+ 0*	+ 53*	+ 407*	+ 374*	+ 8*

---

* Average Values

Test Pile No. 7

Depth of Pile in Ground Feet	Computed Hammer Drop Feet	Penetration Per Blow In Inches		(-) Compression and (+) Tension Stress in Concrete Pile				
		Set	Quake	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5
2				-1818	-1680	-1623	- 838	-320
				+ 0	+ 386	+ 618	+ 296	+ 0
				-1363	-1294	-1237	- 608	-240
	3.45			+ 0	+ 202	+ 464	+ 247	+ 32
				-1182	-1142	-1051	- 510	-208
	3.16			+ 0	+ 84	+ 387	+ 197	+ 0
				-1212	-1176	-1113	- 543	-192
	3.23			+ 0	+ 840	+ 340	+ 164	+ 16
	3.28 avg.			-1394*	-1323*	-1256*	- 625*	-240*
				+ 0*	+ 378*	+ 452*	+ 226*	+ 12*
11				-2030	-1982	-1933	- 986	-352
				+ 0	+ 202	+ 155	+ 197	+ 0
				-2000	-1865	-1933	- 986	-352
	4.69			+ 0	+ 218	+ 464	+ 197	+ 0
				-2121	-2016	-2056	-1035	-417
	4.79			+ 0	+ 218	+ 464	+ 181	+ 0
	4.74 avg.			-2050*	-1954*	-1974*	-1002*	-373*
				+ 0*	+ 212*	+ 361*	+ 192*	+ 0*
15				-2348	-2352	-2319	-1184	-465
				+ 0	+ 168	+ 464	+ 197	+ 0
				-2273	-2218	-2195	-1118	-417
	5.22			+ 0	+ 168	+ 402	+ 230	+ 0
				-2379	-2318	-2304	- 773	-449
	5.27			+ 0	+ 286	+ 464	+ 197	+ 32

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*Average Values

Test Pile No. 7 (Continued)

Depth of Pile In Ground Feet	Computed Hammer Drop Feet	Penetration Per Blow In Inches Set	Penetration Per Blow In Inches Quake	(-) Compression and (+) Tension Stress in Concrete Pile Gage 1	Gage 2	Gage 3	Gage 4	Gage 5
15				-2333	-2268	-2242	-1151	-417
	5.22			+ 0	+ 235	+ 433	+ 164	+ 32
	———			———	———	———	———	———
	5.23 avg.			-2333*	-2289*	-2265*	-1056*	-437*
				+ 0*	+ 214*	+ 441*	+ 197*	+ 16*
18		0.50	0.07	-1636	-1680	- 155	- 658	-288
				+ 0	+ 0	+ 46	+ 33	+ 0
		0.98	0.12	-2848	-2906	-2675	-1414	-513
	6.56			+ 0	+ 185	+ 541	+ 378	+ 0
		0.72	0.10	-2545	-2604	-2427	-1200	-417
	4.98			+ 0	+ 168	+ 464	+ 230	+ 0
		0.56	0.25	-2439	-2554	-2396	-1233	-481
	5.48			+ 0	+ 151	+ 464	+ 164	+ 0
		0.62	0.18	-2500	-2638	-2350	-1184	-449
	5.23			+ 0	+ 252	+ 510	+ 247	+ 48
		0.58	0.22	-2469	-2436	-2242	-1151	-449
	5.15			+ 0	+ 252	+ 464	+ 164	+ 0
	———	———	———	———	———	———	———	———
		0.66*	0.16*	-2406*	-2470*	-2041*	-1140*	-433*
	5.48 avg.			+ 0*	+ 201*	+ 415*	+ 203*	+ 8*
19		0.30	0.22	-1666	-1680	-1484	- 740	-256
				+ 0	+ 0	+ 77	+ 33	+ 0
		0.82	0.13	-2879	-2906	-2628	-1447	-513
	6.17			+ 0	+ 0	+ 649	+ 164	+ 0
		0.58	0.17	-2500	-2554	-2288	-1151	-449
	5.15			+ 0	+ 134	+ 464	+ 164	+ 0
		0.60	0.10	-2576	-2654	-2396	-1233	-481
	5.33			+ 0	+ 168	+ 464	+ 164	+ 0

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* Average Values

Test Pile No. 7 (Continued)

Depth of Pile in Ground Feet	Computed Hammer Drop Feet	Penetration Per Blow In Inches		(-) Compression and (+) Tension Stress in Concrete Pile				
		Set	Quake	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5
19		0.50	0.15	-2454	-2453	-2226	-1151	-417
	5.00			+ 0	+ 134	+ 417	+ 181	+ 0
		0.60	0.12	-2576	-2604	-2350	-1282	-449
	5.28			+ 0	+ 168	+ 464	+ 164	+ 0
		0.56	0.12	-2454	-2520	-2242	-1151	-288
	4.95			+ 0	+ 17	+ 309	+ 99	+ 0
		0.28	0.16	-2435	-2520	-2242	-1151	-449
	5.03			+ 0	+ 470	+ 696	+ 263	+ 0
		0.53*	0.15*	-2449*	-2486*	-2233*	-1163*	-413*
	5.27avg.			+ 0*	+ 11*	+ 442*	+ 154*	+ 0*
20				-2394	-2486	-2242	-1134	-433
				+ 0	+ 0	+ 340	+ 164	+ 0
				-2666	-2738	-2412	-1282	-449
	5.68			+ 0	+ 286	+ 557	+ 263	+ 0
				-2500	-2554	-2319	-1200	-417
	5.32			+ 0	+ 101	+ 387	+ 132	+ 0
				-2520*	-2592*	-2324*	-1205*	-433*
	5.50 avg.			+ 0*	+ 129*	+ 428*	+ 186*	+ 0*
25				-2651	-2722	-2319	-1233	-417
				+ 0	+ 134	+ 464	+ 230	+ 0
				-2621	-2688	-2396	-1233	-449
	5.68			+ 0	+ 252	+ 541	+ 263	+ 32

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*Average Values

Test Pile No. 7 (Continued)

Depth of Pile in Ground Feet	Computed Hammer Drop Feet	Penetration Per Blow In Inches		(-) Compression and (+) Tension Stress in Concrete Pile				
		Set	Quake	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5
25				-2788	-2755	-2474	-1315	-481
	5.27			+ 0	+ 218	+ 387	+ 296	+ 16
				-2697	-2688	-2474	-1233	-465
	5.68			+ 0	+ 168	+ 402	+ 247	+ 0
				-2697	-2654	-2443	-1184	-481
	5.37			+ 0	+ 218	+ 433	+ 296	+ 0
				-2576	-2570	-2319	-1184	-449
	5.52			+ 0	+ 202	+ 464	+ 247	+ 0
	5.50 avg.			-2672*	-2680*	-2404*	-1230*	-457*
				+ 0*	+ 199*	+ 449*	+ 263*	+ 8*

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*Average Values