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Earth Pressures and Retaining Structures

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Earth Pressures and Retaining Structures

FIELD STUDY OF SHEAR TRANSFER IN STEEL SHEET PILE BULKHEAD

(Measuring Techniques and Summary of Test Data)

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This work has been carried out as part of an investigation sponsored by the American Iron and Steel Institute

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FIELD STUDY OF SHEAR TRANSFER IN STEEL SHEET PILE BULKHEAD

by

C. E. Brewer* and H. Y. Fang*

ABSTRACT

Economical design of sheet pile bulkheads requires an understanding of the action of the individual pile sections under field loading conditions. European arch web sheet pile design practice considers the entire wall cross-section to act as a single unit. American practice, on the other hand, considers each pile to act individually. Consequently, designs based on the European method are more economical than those based on the American method.

Which of the design auumptions most closely approximates actual field conditions depends primarily on the amount of shear transfer mobilized across the interlocks of the sheet piles. In order to investigate the degree of shear transfer actually mobilized, a full scale field test of a 30 foot steel arch web sheet pile wall (Type DP-2) was conducted. The piles, instrumented prior to driving with strain gages adequately protected against driving forces and ground-water intrusion, were driven 25 feet into an essentially cohesionless soil. Stress distributions within the

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wall and tie rod loads were monitored as the ground in front of the wall was periodically excavated in 5 ft. increments to within 5 feet of the toe of the sheeting.

It was concluded that, within the range of applied loads encountered in the investigation, shear transfer takes place across the interlocks of steel arch web sheet piles. Thus, it is suggested that the European practice of assuming that the piles act as a unit more closely approximates field conditions than the American practice of assuming individual pile action.

Composite action between the soil and the piling was observed to occur under certain loading conditions. However, further investigation of the soil-structure interaction is necessary in order to more clearly understand this phenomenon.

1. INTRODUCTION

Sheet piling is the term used to describe thin piles driven close together to form a wall. Sheet piling is used to resist lateral pressure caused by combinations of earth, water, and externally applied horizontal and vertical loads, and/or to prevent leakage of water and soil into an excavation. Sheet piles are made of wood, concrete, and steel and are used in waterfront structures, canal locks, dams, cofferdams, riverbank protection works, and retaining walls.

The section modulus of a structural member is a measure of its ability to resist bending. It is calculated by dividing the area moment of inertia of the member about its axis of bending (the neutral axis) by the distance from that axis to the outermost fiber of the member cross-section. The centroidal axis of an individual arch web sheet pile section is located between the axis of the interlocks and the web (Fig. 1). American engineers use this centroidal axis as the neutral axis for evaluating moment resistance. The location of the centroidal axis of a wall composed of several interlocked sheet piling sections is along the line of the interlocks (Fig. 1). European engineers use this axis, or an intermediate position, as the neutral axis to evaluate the moment resistance. As is evident from Fig. 1, the resistance of an individual section is about one half the resistance of the composite group. Consequently, designs based on the European method are more economical than those based on the American method.

The objective of this study was to evaluate the behavior of interlocked steel sheet piling in an actual field installation. More specifically, the strain distribution across a sheet pile was measured in order to experimentally locate the neutral axis of bending of the wall. This report presents a detailed description of the measuring technique used to study shear transfer in a sheet pile wall, includes a summary of the data obtained in the field test, and discusses the degree of shear transfer mobilized in arch web steel sheet piling interlocks.

2. REVIEW OF PREVIOUS TESTS

A review of literature concerning sheet pile structures was made by Krugmann, Boschuk, and Fang (1968). The reader is directed to this publication for more detailed information on sheet piling as the following review is limited to those papers most important to the present study.

Duke (1953) used stressmeters and transits to measure the stresses and deflections in a bulkhead that was constructed in Long Beach, California. His results for bending moments and deflections were considered so unreliable that they were discounted.

Hakman and Buser (1962) reported the results of a full scale field test on a bulkhead at the Port of Toledo, Ohio. They employed slope indicators, strain gages, and transits to measure the behavior of the bulkhead. Unfortunately, the strain gages failed to operate properly, thus eliminating the possibility of verifying the other data.

Tschebotarioff (1949,1964) reported the results of a long series of tests known as the Princeton Model Tests. He used pressure cells and strain gages to evaluate the performance of a large model bulkhead. Much valuable information on the lateral earth pressure distribution of different soils against a bulkhead was obtained.

Rowe (1952,1958) performed a series of laboratory tests on model bulkheads from which much data emerged, and from which design criteria were established.

None of the above investigations, however, touched upon the important design consideration mentioned previously, namely, the location of the neutral axis of bending for a sheet pile wall.

3. DESCRIPTION OF TEST SITE

One of the assumptions frequently made in the present methods of analyzing steel sheet pile walls is that the soil on both sides of the wall is cohesionless. Consequently, a major consideration in the selection of a test site was to locate a soil profile comparable to the so-called "ideal" design material.

Several sites were examined. The site finally chosen was located in Martins Creek, Pennsylvania (Fig. 2).

The soil profile was determined from the results of wash borings and from information supplied by the Pennsylvania Power and Light Company. Boring logs recorded the surface conditions, the strata changes and thicknesses, the standard penetration values for the soils, and the elevation of the groundwater table. Fig. 3 shows the results of this soil profile investigation. It can be seen that, in general, the test site consisted of a thin layer of sand and silt underlain by a thicker layer of sand, gravel, and boulders.

Soil classification tests, grain size analyses, density, water content determinations, and shear strength tests were performed on samples of soil taken from the site. The results of these tests are given in Table 3.

The test site had excellent accessibility for equipment and a low groundwater table, both desirable features. The soil was, however, slightly cohesive. This characteristic was not desirable as it would in most cases cause the actual loads on the bulkhead to be smaller than the calculated theoretical loads. However, it is believed that the reduced field loads would not significantly influence any conclusions to be drawn from the study.

4. DEVELOPMENT OF MEASURING TECHNIQUES

4.1 Selection of Instruments

A thorough review of previous investigations, and numerous communications with instrument manufacturers yielded the following conclusions:

1. The "stressmeter" is an outdated method of measurement.
2. The "slope-indicator" is an accurate instrument for measuring the slope of the piling, but if used alone, its readings are difficult to convert to strains. The budget would not allow a combination of instrumentation.
3. The strain gage is the simplest, and the most adaptable piece of equipment for measuring strain.

Consequently, the SR-4 strain gage was chosen as the primary means of measuring the strain conditions in the piling. However, strain gages must be protected during pile driving operations in order to remain operable. If subjected to force or to contact with foreign objects, damage may result.

It is believed that commercially produced heavy duty, weldable gages with protected lead wires would have been suitable, but their cost was prohibitive. Because of their low profile and reasonable cost, foil-type strain gages* were selected.

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*Manufactured by Dentronics, Inc., New Jersey

4.2 Evaluation of Instruments

4.2.1 Laboratory Evaluation

An evaluation of the strain gage system was undertaken in the laboratory to study methods for attaching, waterproofing, and protecting the gage. In addition, the behavior of the gage was observed under simulated field conditions.

In the laboratory, a gage was mounted on the outer portion of a 2" x 2" angle to simulate the actual mounting of a gage on the sheet piling. A protective epoxy covering* was placed over the gage, but no attempt was made to keep the covering from touching or adhering to the gage.

A test soil was prepared into which the angle could be driven. The soil was composed of equal amounts of coarse sand, obtained directly from the test site, and 3/4"-1" crushed stone. It was believed that the laboratory soil would be more abrasive to the gage, and its covering, than the soil at the field site. The soil mixture was placed in a 2' diameter cylinder of 4' height. A drain spout was tapped into the bottom and a manometer was attached so that the level of the water table could be controlled and measured. The sand and stone mixture was soaked, vibrated, and allowed to drain. This produced a very compact mixture for the test.

The angle was driven by a single acting 30 lb. hammer

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*Denseal #5, Dentronics, Inc., New Jersey into the laboratory test soil (Fig. 4). As the driving proceeded, frictional forces acting on the protective covering affected the strain readings. These readings stabilized when the pile was allowed to stand at any level for a period of time. It is believed that movement of the epoxy covering under the frictional forces was responsible for the observed fluctuations in readings. Apparently, the "creep" characteristic of the epoxy enabled the gage to recover. The pile was driven five times and the gage remained operable. The epoxy coating protected the gage against soil abrasion, but it was observed that the insulation on the lead wires could not withstand the frictional forces developed during driving. This was evidenced by the fact that the insulation was removed at several locations.

A second angle was gaged and the lead wires were coated with epoxy throughout their entire anticipated embedded length. The angle was driven five times into the test soil, and the gage performed satisfactorily throughout the driving operation. The lead wires that were coated with epoxy proved very durable and showed no signs of abrasive wear.

On a third gaged angle, in addition to coating the lead wires, a piece of teflon cloth was placed over the gage to prevent the gage from bonding to the epoxy covering. After completion of the fifth drive, the gage failed to zero. The angle was removed from the soil and the gage was allowed to dry and stabilize. After 4 hours, the gage balanced. The epoxy coating was cleaned by washing with water and the gage shorted again. It was observed that the epoxy coating had broken bond with the steel angle and water was penetrating into the gage. When the protective covering was removed, it was found that only about one-quarter of an inch surrounding the gage was providing a seal. Since some gages would be below the water table in the field, it was important to insure an adequate seal around the gage.

The laboratory testing of the foil-type strain gage proved that the gage could be successfully protected against abrasion under controlled laboratory conditions, and no trouble would arise as long as the epoxy covering remained unbroken and bonded to the steel. Therefore, extreme care was taken to properly bond the epoxy to the steel in all subsequent gage installations.

4.2.2 Field Evaluation

After it was shown that the gages could be protected under controlled laboratory conditions, it was decided to test them in the field at a nearby construction project.

Two gages were attached to a sheet pile in the field and driven 20' into a loose, silty sand having a high goundwater table. Both gages were protected by the epoxy covering. One gage was given additional protection by covering it with a steel shoe that was welded to the sheet pile (Fig. 5).

During and after driving, both gages performed satisfactorily, thereby substantiating the results of the laboratory evaluation.

The field testing of the foil-type strain gage showed that the gage could successfully withstand pile driving forces and a high groundwater table. It was decided to protect all gages with the steel shoe in order to offer protection against large boulders that might be enountered at the test site.

5. TEST PROCEDURES

5.1 Laboratory Test Procedure

The strain gages were installed on the sheet piling at Fritz Engineering Laboratory prior to delivering the piling to the test site. The piles were cleaned with high speed grinders to obtain a smooth surface for the gages. The ribbon wire was laid flat and clamped before the gage epoxy* was applied. After the wires were in place, the gages were attached and clamped while the epoxy was setting. Each gage was checked after installation to insure adhesion of the gage to the piling. This was accomplished by a "light bulb test" (Dally and Riley, 1965).

Because of the delicate nature of foil gages, it was necessary to use low temperature solder to install the wires. Terminal tabs were used to allow some play in the wires should they be accidentally pulled. After the wires were installed, gage readings were taken and the protective epoxy covering was applied (See Figs. 6 and 7).

The strain gages were connected to the arch piles near the interlocks in order to evaluate the shear transfer across the joints. This information would, in turn, lead to the determination of the location of the neutral axis of bending for the sheet pile wall. The layout of the strain gages on the instrumented piles (piles 10, 11, 12, and 13) is shown in Fig. 8.

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*Denseal #5, Dentronics, Inc., New Jersey

After the gages were installed on the piling, they were "zeroed" at Fritz Engineering Laboratory with the piling hanging in a vertical position. The initial readings recorded in the laboratory were checked at random at the site before driving was started on each pile. The comparison between the random checks and the lab readings was good.

5.2 Field Test Procedure

The length of the sheet pile wall was 30 feet which was believed to be long enough to minimize undesirable end effects. The total length of the arch piles was 30 feet and they were driven to a depth of 25 feet. Consequently, once the arch piles were in place, approximately 5 feet of pile protruded from the ground surface. A standard driving rig with a low energy double action 9B3 steam hammer was used for the driving operations. A guide frame was used to insure plumbness of the wall. A transit and a six-foot level were used to aid in positioning the piles. The wall was anchored at its third-points by tie rods which were held back by H-piles driven 20 feet into the ground. The tie rods were attached to the wall by means of a wale welded to the wall at ground level. The wale and tie rods were located at ground level to facilitate instrumentation and test procedures. The entire test set-up is illustrated in Figs. 9, 10, and 11.

Throughout the wall, the measured out-of-plumbness during the driving never exceeded 1 in. in 30 feet in the

X-Y plane (Fig. 10). The deviation from the vertical in the Y-Z plane, however, did increase as the wall was driven (Fig. 11). This is attributed to the tendency of these piles to close on themselves if one side is pushed by an underground obstacle. The maximum deviation in the Y-Z plane was 5 in. in 30 feet. It should be noted that piles No. 10 and No. 13 met refusal and could not be driven to the required depth. For this reason they protrude 15 in. above the other piles (Fig. 12).

Initial readings were taken on all strain gages on the day excavation commenced (one week after completion of pile driving). In addition, wall deflections (measured by transit) were recorded for several wall locations. An initial load of 2000 lbs. was applied to each of the tie rods.

The first phase of testing to study the behavior of the sheet pile wall involved excavation in front of the wall in four stages. Initially, a 5 ft. excavation was made and all gages were read. Three hours after completion of the excavation, a collar broke that connected two sections of one of the tie rods together. The collar was quickly replaced and the test was continued. Strain gage readings were taken both before and after the tie rod failure. The excavation was left for one week, at which time, all the gages were read again. There was little difference between these readings and those taken after the tie rod was replaced. The next 5 ft. stage of excavation was then made and gage readings were taken one week after axcavation. This sequence of events was repeated until the axcavation reached the 20 ft. level. The wall was again left for one week before the final set of gage readings were taken. Fig. 12 shows field readings being taken following excavation. The sheet pile wall with excavation at the 10' level is shown in Fig. 13.

After completion of the excavation phase of the test, an attempt was made to subject the wall to different earth pressures. This was accomplished by increasing the load on each tie rod to 12,000 lbs. After one month, all gages were read and the tie rod loads released. Gage readings were taken one week after release of the tie rod loads, after which the piles were pulled.

6. SUMMARY OF TEST DATA

A brief explanation of the contents of each table of data obtained in this investigation is herein presented.

The results of soil analyses conducted both in the field and in the laboratory are presented in Table 1. The in-place density, taken after excavation to the 5' level was complete, was obtained by the sand cone method (ASTM 1556).

The type DP-2 arch web sheet piling was obtained from the Lackawanna Plant of the Bethlehem Steel Corporation. Excerpts from the mill report that accompanied the piling are presented in Table 2.

Table 3 summarizes the strain gage readings taken during the laboratory phase of the gage evaluation test. Included are strain gage readings taken after each test "drive and pull" as well as notes regarding the physical condition of the gages and their associated wiring. Similarly, Table 4 summarizes the strain gage readings taken during the field phase of the gage evaluation test.

Tables 5 through 10 contain the strain gage readings taken during excavation in front of the sheet pile wall, and during loading and unloading of the tie rods. All strain gages are numbered and their location may be determined by reference to Fig. 8.

Deflections of the top of the piles during excavation and during loading and unloading of the tie rods are recorded in Table 11.

Table 12 is the calibration data relating load to strain in the tie rods.

The variation of load in the tie rods with time just prior to and following the break in the left tie rod is contained in Table 13.

Table 14 lists the variation in tie rod load with depth of excavation.

A discussion of the data concerning shear transfer across the sheet pile interlocks is presented in the following sections of this report. No attempt has been made at this time to analyze, in detail, the data on tie rod loading.

7. DISCUSSION OF RESULTS

Figs. 13, 14, and 15 show the pertinent data from this investigation in graphical form. The distribution of vertical strain across the sheet pile interlocks is shown for all gage levels, and at all stages of excavation.

Although there is considerable scatter in the data and many of the gages failed completely, the graphs suggest that shear stress transfer across the interlocks between piles does occur. This is apparent because the vertical strain distribution across the interlocks may be reasonably approximated by a single continuous straight line at all stages of excavation at which there is sufficient data. If there was ony partial or no shear stress transfer across the interlocks, the vertical strain distribution across the piles would be shown by two discontinuous straigth lines.

Although interpretation of the vertical strain data for joint J-1 (Fig. 14) is not difficult, only gage level G1 at joint J-2 (Fig. 15) yields any useful information concerning shear transfer. Unfortunately, no more than two gages remained operable at each of the other J-2 gage levels. The apparent reversal of bending at gage levels G2 and G3 across joint J-3 (Fig. 16) may be attributed to either bending stresses induced during driving, or perhaps to excessive zero shift of the strain gages during and after driving.

It is of interest to note that the location of the neutral axis of bending for the sheet pile wall, given by the intersection of the vertical strain curve with the line of zero strain, does not always lie within the pile cross-section. For discussion purposes, consider the behavior of piles Nos. 10 and 11 at gage level G1 (Fig. 14). It can be seen that just prior to excavation, the neutral axis lies completely outside the pile cross-section toward the fill side of the wall. Thus, the piles are in tension due to bending induced during driving, and the compressive bending stresses are carried by the soil behind the piles. Such behavior may be considered composite action, with the wall and the soil acting as a unit.

As was noted earlier, following completion of the 5' excavation, one of the tie rods snapped causing the sheet pile wall to relax as it deflected in towards the excavation. The change in bending stress in the sheet pile resulting from this break (as reflected by the vertical strains) and the corresponding relocation of the neutral axis to a point within the pile cross-section, may be seen in Fig. 14. After the tie rod was replaced and as excavation proceeded, the neutral axis moved back towards its position just after driving.

8. CONCLUSIONS

From the results of this investigation, the following conclusions may be drawn:

1. Strain gage instrumentation installed on sheet piling prior to driving may be successfully protected against damage during driving and against groundwater corrosion.

2. Within the range of applied loads encountered in this investigation, the available data suggests that shear transfer takes place across the interlocks of arch web steel sheet piles. Thus it is believed that the European practice of assuming that the piles act as a unit more closely approximates the field conditions than the American practice of assuming individual pile action.

3. Composite action between the soil and the piling may occur under certain conditions. However, further investigation of the soil-structure interaction is necessary in order to more clearly understand this phenomenon.

9. ACKNOWLEDGEMENTS

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TABLE 1: SOIL PROPERTIES

Soil Classification (Unified Classification System) .... SM

In Place Wet Density (5' below original ground surface) .... 117 #/cu.ft.

In Place Water Content (5' below original ground surface) 10%

Unconfined Compression Strength (Laboratory Compacted Specimen- .... 1335 psf
Wet Density = 117 #/cu.ft., Water Content = 10%)

Grain Size Distribution - % Passing by Weight

Boring #1

Sieve Size	Depth	0-1.5'	5'-6.5'	10'-11.5'	15'-16.5'	20'-21.5'	25'-26.5'	28.5'-30'
# 4		98	97	100	97	40	60	96
# 60		10	55	68	15	15	25	19
#200		1	20	20	2	8	7	8

Boring #2

Sieve Size	Depth	0-1.5'	5'-6.5'	10'-11.5'	15'-16.5'	20'-21.5'	25'-26.5'
# 4		97	98	100	70	55	96
# 60		55	75	30	40	30	56
#200		20	20	10	10	7	30

TABLE 2: PROPERTIES OF DP2 SHEET PILING

STEEL TYPE - ASTM A328

PHYSICAL TEST RESULTS

HEAT NO.	DESCRIPTION	YIELD POINT STRESS (psi)	TENSILE STRENGTH (psi)	ELONGATION %	ELONGATION INCHES	BENDS
518V0031	DP2 PILING	44460	77750	29.5	8	OK
518V0035	DP2 PILING	44430	77460	21.5	8	OK

CHEMICAL ANALYSIS

HEAT NO.	CARBON	MANGANESE	PHOSPHORUS	SULPHUR
518V0031	.30	.80	.018	.023
518V0035	.32	.83	.014	.020

Data supplied by Bethlehem Steel Corporation Metallurgical Department, Shipment No. 504-14136.

TABLE 3: SUMMARY OF DATA, LABORATORY INSTRUMENTATION EVALUATION

TEST NO.		1	2	3	NOTES
GAGE PROTECTION		EPOXY	EPOXY	TEFLON AND EPOXY	
LEAD WIRE PROTECTION		NONE	EPOXY	EPOXY	
OPERATION	SIMULATED* DEPTH (ft.)	GAGE READINGS - MICRO-INCHES PER INCH x 10			
--	--	6030	7653	7650	Initial, before gage protection applied
--	--	6225	5990	5900	Initial, after gage protection applied
1st Drive	3	6220	5980	5900	------------------------------
--	--	6190	5943	5850	Following overnight soaking below "GWT"
1st Pull	--	6190	5955	5865	All tests: no visible signs of cover failure
2nd Drive	6	6190	5955	5865	------------------------------
2nd Pull	--	6190	5950	5860	All tests: no visible signs of cover failure
3rd Drive	9	6200	5955	5865	------------------------------
3rd Pull	--	6205	5970	5870	All tests: no visible signs of cover failure
4th Drive	12	6201	5970	5880	------------------------------
4th Pull	--	6200	5970	5875	All tests: no visible signs of cover failure
5th Drive	15	6240	5995	5600	Test 3: Difficult to balance gage Test 1: Lead wire insulation abraded
5th Pull	--	6260	6035	2020	Tests 1&2: no visible signs of cover failure
1 hr. Later		6240	6075	0700	Test 3: Cover cracked

*Depth simulated by the number of driving and pulling operations.

TABLE 4: SUMMARY OF DATA, FIELD INSTRUMENTATION EVALUATION

	GAGE READINGS: MICRO-INCHES PER INCH x 10	
PROTECTION	EPOXY ONLY	EPOXY AND METAL SHOE
Lab. Zero	6230	7840
Field Zero (Prior to Driving)	6245	7855
Immediately Following Driving	5820	7390
One Week Following Driving	5820	7390

TABLE 5: STRAIN GAGE READINGS -PILE NO. 10, ROWS 1 AND 2

GAGE READINGS - MICRO-INCHES PER INCH x 10

GAGE	LABORATORY HUNG ZERO	IMMEDIATELY FOLLOWING DRIVING	READINGS ONE WEEK AFTER EXCAVATION REACHED SPECIFIED LEVEL				ADDITIONAL READINGS AT 20' EXCAVATION LEVEL		
			5 ft.	10 ft.	15 ft.	20 ft.	1 DAY AFTER EXCAVATION	1 MONTH AFTER 12k TIE ROD LOAD APPLIED	1 WEEK AFTER TIE ROD LOAD RELEASED
1	8260	8785	7775	8210	7750	8490	7850	9030	6970
2	7445	9180	8070	8450	7950	8640	7080	9130	7020
3	8220	9010	7980	8360	7830	8570	8015	9240	7210
4	8290	9320	8240	8640	8090	8780	8265	9240	7150
5	8940	9920	8890	9280	8795	9365	9990	9830	7735
6	8460	9040	7965	8370	7880	8465	8090	8820	6740
7	8920	9605	8560	8990	8540	9125	8750	9455	7350
8	8070	8520	7450	7845	7380	7930	7580	8435	6320
9	8800	9290	8305	8775	8310	8900	8525	9270	7170
10	8450	9835	7820	8255	7800	8260	7970	8790	----
11	8270	8820	7760	8160	7670	8230	7890	8665	6650
12	8320	9185	7880	8380	7910	8495	8130	8910	6825
13	6930	7880	6760	7170	6665	7190	6865	7580	5520
14	7320	7880	6760	7170	6790	7230	6890	7690	5610
15	6260	6930	5820	6260	5740	6300	5940	6825	4760
16	5860	6565	5450	5885	5360	5930	5565	6730	4350
17	6030	6735	5600	5970	5580	6120	5770	6640	4560
18	5600	6440	5315	5695	5250	5780	5430	6300	4220
19	8110	8880	7740	8110	7615	8150	7805	8650	6560
20	7950	8470	7355	7750	7230	7805	7430	8320	6250
21	6180	6770	5680	6050	5525	6110	5710	----	4550
22	6870	----	----*	----	----	----	----	----	----
23	6810	----	----	----	----	----	----	----	----
24	7070	----	----	----	----	----	----	----	----

* ---- Denotes gage that would not balance

TABLE 6: STRAIN GAGE READINGS - PILE NO. 11, ROWS 1 AND 2

			GAGE READINGS - MICRO-INCHES PER INCH x 10						
GAGE NO.	LABORATORY HUNG ZERO	IMMEDIATELY FOLLOWING DRIVING	READINGS ONE WEEK AFTER EXCAVATION REACHED SPECIFIED LEVEL				ADDITIONAL READINGS AT 20' EXCAVATION LEVEL		
			5 ft.	10 ft.	15 ft.	20 ft.	1 DAY AFTER EXCAVATION	1 MONTH AFTER 12^k TIE ROD LOAD APPLIED	1 WEEK AFTER TIE ROD LOAD RELEASED
1	7120	6500	5430	6300	5840	6260	5830	7810	5760
2	6665	7070	6240	9790	5260	5600	5215	6370	3760
3	7080	6120	5380	5380	5220	5670	5220	6130	3515
4	10050	11160	10020	10310	9900	10060	9815	10920	8420
5	10610	5970	9900	9950	9115	10240	10150	11475	9003
6	10380	11505	10520	10720	10310	10980	10510	11122	8530
7	6600	----	----	----	----	----	----	----	----
8	7690	9600	8640	8660	8460	8800	8495	9670	7245
9	7280	8430	7540	7545	7270	7750	7455	8880	6680
10	7680	9085	8100	8305	7900	8130	8049	8980	6650
11	7880	8545	7550	7790	7440	7670	7450	8680	7410
12	8020	9770	9000	9420	9420	8430	8220	9350	7660
13	4520	7595	4945	4360	4670	4865	4620	5830	3540
14	5210	7000	5990	5990	5750	5990	5790	7030	4655
15	4400	6100	5090	4901	4880	5015	4920	6480	3860
16	----	----	----	----	----	----	----	----	----
17	6290	7500	6445	6680	6250	7610	6260	8040	5045
18	5290	6840	5795	6075	5530	5950	5550	7150	4280
19	4380	5700	4660	4420	4440	4910	4455	6340	3605
20	4520	6085	5030	5310	4780	5185	4800	6630	3695
21	4920	6585	5520	5775	5255	6290	5065	6240	3960
22	8420	9852	8745	8300	8330	9010	8390	10230	7950
23	9055	10445	9325	9245	9070	9360	9060	9990	7715
24	5230	5630	4540	4710	4350	4690	4410	5670	3240

* ---- Denotes gage that would not balance

TABLE 7: STRAIN GAGE READINGS - PILE NO. 11, ROWS 3 AND 4

GAGE NO.	GAGE READINGS - MICRO-INCHES PER INCH x 10								
	LABORATORY HUNG ZERO	IMMEDIATELY FOLLOWING DRIVING	READINGS ONE WEEK AFTER				ADDITIONAL READINGS AT 20' EXCAVATION LEVEL		
			EXCAVATION 5 ft.	REACHED 10 ft.	SPECIFIED 15 ft.	LEVEL 20 ft.	1 DAY AFTER EXCAVATION	1 MONTH AFTER 12k TIE ROD LOAD APPLIED	1 WEEK AFTER TIE ROD LOAD RELEASED
25	6930	----*	----	----	----	----	----	----	----
26	5890	6080	6580	6530	6210	6390	6225	8820	6400
27	----	----	----	----	----	----	----	----	----
28	6495	7650	6620	6940	6635	6690	5550	7710	4330
29	8030	6150	6160	6370	5955	5900	6865	6820	4440
30	7620	9360	8120	8460	8030	8240	8035	9180	6540
31	5410	6690	5550	5925	5465	5650	5460	6540	4190
32	6050	7590	6475	6880	6380	6600	6370	7900	5235
33	6860	7710	6665	7030	6690	7200	6980	9430	8480
34	7230	8975	6090	6290	6010	6330	7580	7530	5090
35	7340	7620	6575	7020	6475	6750	6480	7730	5020
36	7500	8935	7730	8060	7620	7810	6090	8710	6240
37	5300	6835	5140	5510	5110	5310	5070	6580	3850
38	5820	7555	6520	7010	6120	6420	6140	7470	4910
39	6350	7656	6100	6720	6030	6560	6130	8280	5100
40	5380	6885	5931	6680	5430	5730	5530	7440	4180
41	8680	----	----	----	2800	4000	2915	----	----
42	5320	7525	7080	6730	5880	6000	5820	5630	7145
43	4380	5960	4465	4830	4265	4840	4310	5700	3100
44	4360	5950	4750	5116	4690	4850	4650	6130	3290
45	5080	6450	5230	5650	5150	5360	5175	6450	3870
46	4010	5530	4970	5260	4200	6920	4215	----	3280
47	3690	5000	3880	4290	3730	4030	3730	4960	2560
48	4030	5415	4300	4730	4180	4410	6815	5530	3215

----* Denotes gage that would not balance

TABLE 8: STRAIN GAGE READINGS - PILE NO. 12, ROWS 1 AND 2

GAGE NO.	LABORATORY HUNG ZERO	IMMEDIATELY FOLLOWING DRIVING	READINGS ONE WEEK AFTER EXCAVATION 5 ft.	REACHED 10 ft.	SPECIFIED 15 ft.	LEVEL 20 ft.	ADDITIONAL READINGS AT 20' EXCAVATION LEVEL: 1 DAY AFTER EXCAVATION	1 MONTH AFTER 12^k TIE ROD LOAD APPLIED	1 WEEK AFTER TIE ROD LOAD RELEASED
	GAGE READINGS - MICRO-INCHES PER INCH x 10								
1	7030								
2	7050								
3	7020								
4	7310								
5	7030				*				
6	6780								
7	7640								
8	7430								
9	7600								
10	7080								
11	8120								
12	8460								
13									
14									
15									
16									
17									
18	5410	6850	5600	6060	5340	5500	5460	6520	4670
19	4660	5950	4980	5490	4770	4920	4775	5770	3865
20	3320	5000	3900	4530	3830	5100	3880	4620	2750
21	5255	6800	5810	6440	5820	6190	5935	6570	3650
22	5255	6860	5860	6315	5475	5700	5475	4420	2385
23	5180	6900	5950	6470	5840	6015	5870	4880	3785
24	5360	6875	6000	6865	6210	6830	6335	5450	4365

*Denotes gage that would not balance

TABLE 9: STRAIN GAGE READINGS - PILE NO. 12, ROWS 3 AND 4

GAGE NO.	LABORATORY HUNG ZERO	IMMEDIATELY FOLLOWING DRIVING	GAGE READINGS - MICRO-INCHES PER INCH x 10: READINGS ONE WEEK AFTER EXCAVATION REACHED SPECIFIED LEVEL 5 ft.	10 ft.	15 ft.	20 ft.	ADDITIONAL READINGS AT 20' EXCAVATION LEVEL: 1 DAY AFTER EXCAVATION	1 MONTH AFTER 12k TIE ROD LOAD APPLIED	1 WEEK AFTER TIE ROD LOAD RELEASED
25	----								
26	7020		*				*		
27	6545								
28	7345								
29	7550								
30	6685								
31	8680	10540	9590	10010	9290	9370	9335	10930	7850
32	8020	10122	8830	9400	8665	8450	8710	10880	8000
33	4200	5795	4760	5220	4530	4800	4565	5365	3340
34	9210	10915	9955	10580	9775	10050	9385	14540	10175
35	8010	9315	8495	9070	8390	8620	8420	9540	7580
36	7860	10240	10200	10155	9420	9060	9835	10111	8150
37	5210	9110	8350	9110	8110	10390	9920	16410	13300
38	6370	----*	----	----	----	8690	8000	9820	7755
39	7260	----	----	----	----	----	----	----	----
40	6700	7575	6765	7330	6880	7220	6910	6810	4870
41	4680	6150	5205	5820	5140	5570	5130	6040	3270
42	5460	6350	5695	6245	5830	6450	6260	6280	4665
43	3620	----	----	----	----	----	----	----	----
44	3825	5190	4200	4755	4030	4300	4000	5090	2920
45	3470	4950	3921	4475	3740	4040	3700	4790	2700
46	6370	7810	6785	7390	5650	7020	7650	8890	6705
47	5330	7070	6060	6655	5950	6240	5910	7100	4965
48	6905	8685	7710	8330	7670	7950	6610	8835	6760

*---- Denotes gage that would not balance

TABLE 10: STRAIN GAGE READINGS - PILE NO. 13, ROWS 1 AND 2

GAGE NO.	LABORATORY HUNG ZERO	IMMEDIATELY FOLLOWING DRIVING	READINGS ONE WEEK AFTER EXCAVATION REACHED SPECIFIED LEVEL: 5 ft.	10 ft.	15 ft.	20 ft.	ADDITIONAL READINGS AT 20' EXCAVATION LEVEL: 1 DAY AFTER EXCAVATION	1 MONTH AFTER 12^k TIE ROD LOAD APPLIED	1 WEEK AFTER TIE ROD LOAD RELEASED
	GAGE READINGS - MICRO-INCHES PER INCH x 10								
1	8300	9115	7930	7885	7680	7225	7580	7600	6710
2	7500	8600	7405	6360	7125	6750	7050	7550	6590
3	8330	9500	8575	8560	8340	8015	8320	8490	7390
4	8020	8820	7815	7790	7605	7285	7590	7660	6540
5	7680	----*	----	----	----	----	----	----	----
6	8050	7960	6960	6760	6500	5615	5450	7260	6001
7	8400	7110	7660	6565	7410	7140	7430	7900	6630
8	8250	8805	8620	7490	8340	8060	8360	8600	7440
9	9070	8112	7425	7665	7450	7340	7460	7950	6810
10	8420	6650	8401	----	----	----	----	----	----
11	5790	9750	8740	8800	8530	8360	8550	8745	7580
12	6610	7950	7710	7700	7450	7330	7485	7930	6670
13	6530	6700	5720	5805	5475	5340	5485	5790	4500
14	6285	7600	6620	6725	6350	6270	6360	6680	5360
15	8285	7240	6280	6390	6060	5960	6070	6450	5180
16	7910	7020	6060	6170	5900	5810	5900	6520	5320
17	4190	6590	5650	5780	5460	5355	5455	5730	4450
18	5580	7480	6480	6600	6270	6150	6250	6540	5220
19	5400	5010	4055	4200	3820	3745	3830	4225	3065
20	5350	6695	5715	5640	5280	5275	5245	5750	4490
21	4410	6470	5489	5750	5250	5250	5260	5750	4350
22	8005	6170	5165	5330	4935	4960	4930	5240	4070
23	5785	5350	4370	4520	4150	4150	4150	4600	3210
24	6700	8870	7845	8030	7660	7660	7670	7111	6815

*---- Denotes gage that would not balance

TABLE 11: WALL DEFLECTIONS

PILE NO.	WALL DEFLECTION FROM TRANSIT LINE - INCHES							
	IMMEDIATELY FOLLOWING DRIVING	READINGS ONE WEEK AFTER EXCAVATION REACHED SPECIFIED LEVEL				ADDITIONAL READINGS AT 20' EXCAVATION LEVEL		
		5 ft.	10 ft.	15 ft.	20 ft.	1 DAY AFTER EXCAVATION	1 MONTH AFTER 12^k TIE ROD LOAD APPLIED	1 WEEK AFTER TIE ROD LOAD RELEASED
4	9.5	9.3	9.6	9.8	10.1	10.0	10.1	10.4
6	9.9	10.0	10.2	10.1	10.5	10.3	10.0	10.1
8	10.1	10.3	10.3	10.4	10.4	10.5	10.6	10.8
10	9.1	9.2	9.3	9.3	9.6	9.4	9.9	10.3
12	9.6	9.6	9.6	9.7	10.0	9.9	10.1	10.1
14	9.2	9.9	9.5	9.6	9.7	9.6	9.8	10.0
16	9.5	10.0	9.9	9.9	10.0	9.9	10.0	10.1

TABLE 12: CALIBRATION OF TIE RODS*

LOAD (kips)	STRAIN (MICRO-INCHES PER INCH x 10)	
	LEFT TIE ROD	RIGHT TIE ROD
0	11,900	12,735
1	12,025	12,845
2	12,145	12,955
3	12,240	13,060
4	12,357	13,170
5	12,470	13,280
6	12,580	13,390
7	12,690	13,500
8	12,800	13,610
9	12,910	13,720
10	13,020	13,830
11	13,130	13,950
12	13,242	14,060
13	13,356	14,180
14	13,470	14,290
15	13,580	14,400
16	13,690	14,510

*Calibrated in Fritz Engineering Laboratory

TABLE 13: VARIATION OF TIE ROD LOAD WITH TIME AFTER COMPLETION OF 5 FOOT EXCAVATION

TIME AFTER EXCAVATION (HOURS)	LEFT ROD		RIGHT ROD	
	STRAIN (MICRO-INCHES PER INCH x 10)	LOAD (kips)	STRAIN (MICRO-INCHES PER INCH x 10)	LOAD (kips)
1/2	12,390	4.2	13,280	5.1
1	12,420	4.5	13,290	5.1
1 1/2	12,480	5.0	13,330	5.2
2	12,510	5.2	13,500	7.0
LEFT TIE ROD BROKE AND WAS REPLACED				
2 1/2	12,500	5.1	13,290	5.1
3	12,480	5.0	13,200	4.1
3 1/2	12,480	5.0	13,190	3.9
4	12,490	5.0	13,190	3.9

TABLE 14: VARIATION OF TIE ROD LOAD WITH DEPTH OF EXCAVATION

DEPTH OF EXCAVATION (FEET)	TIE ROD LOAD (KIPS) LEFT ROD	RIGHT ROD
0	2.0	2.0
5	4.7	4.7
10	7.2	3.9
15	8.5	3.3
20	9.0	3.6

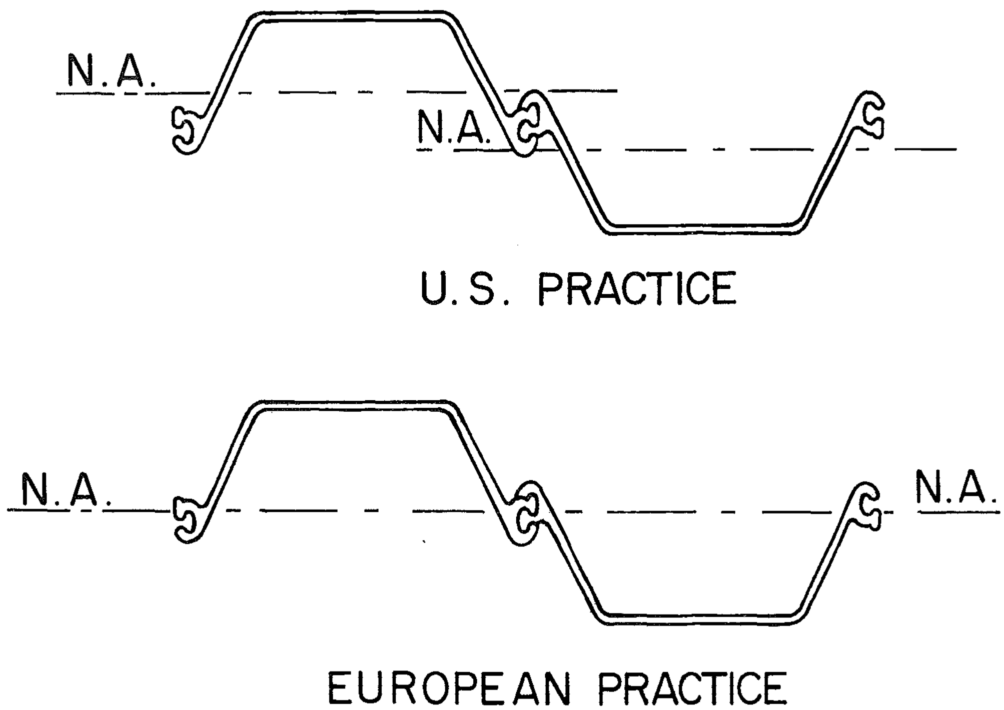


Fig. 1 Location of Neutral Axis (U. S. and European Practice)

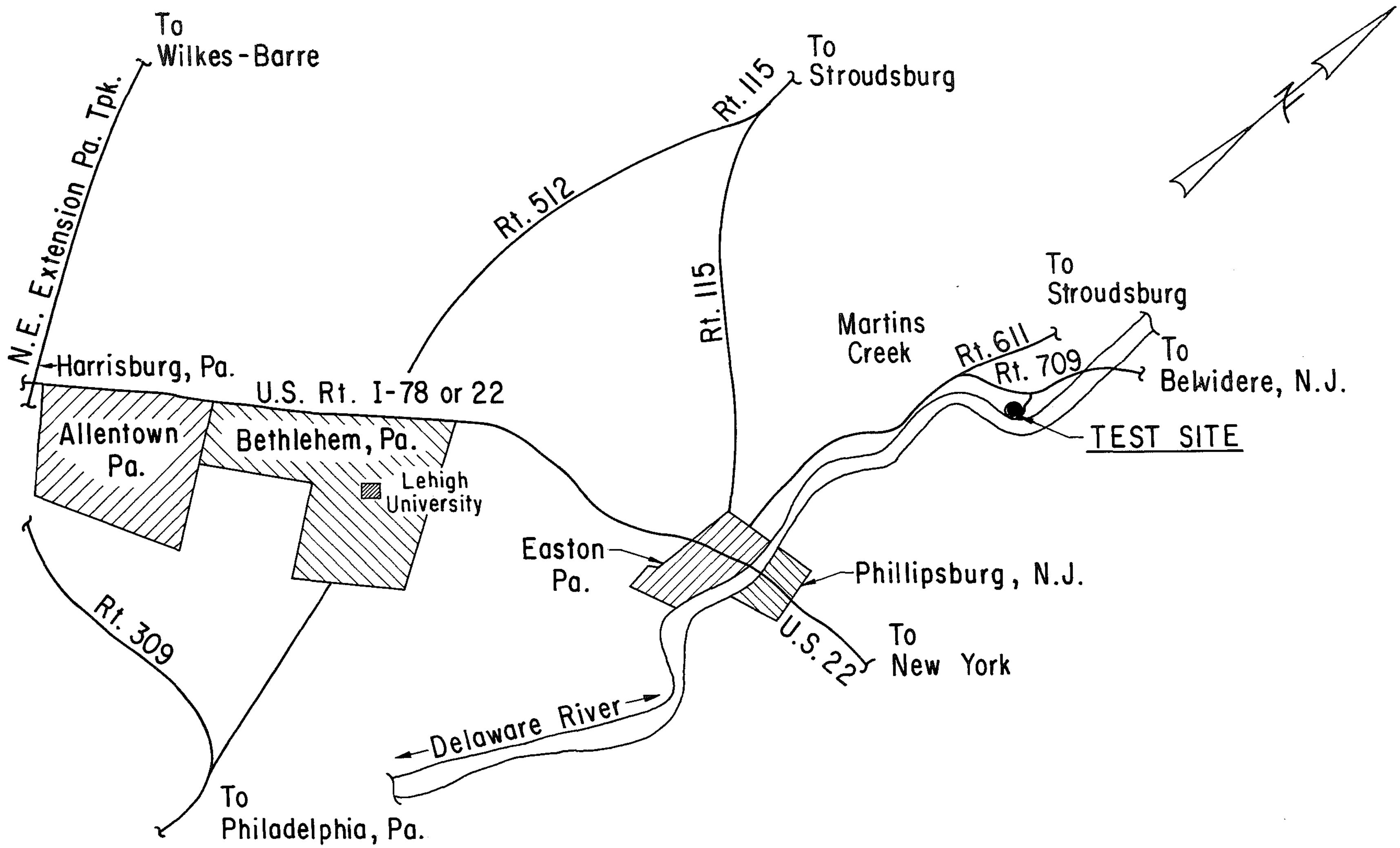


Fig. 2 Test Site - Martins Creek, Pennsylvania

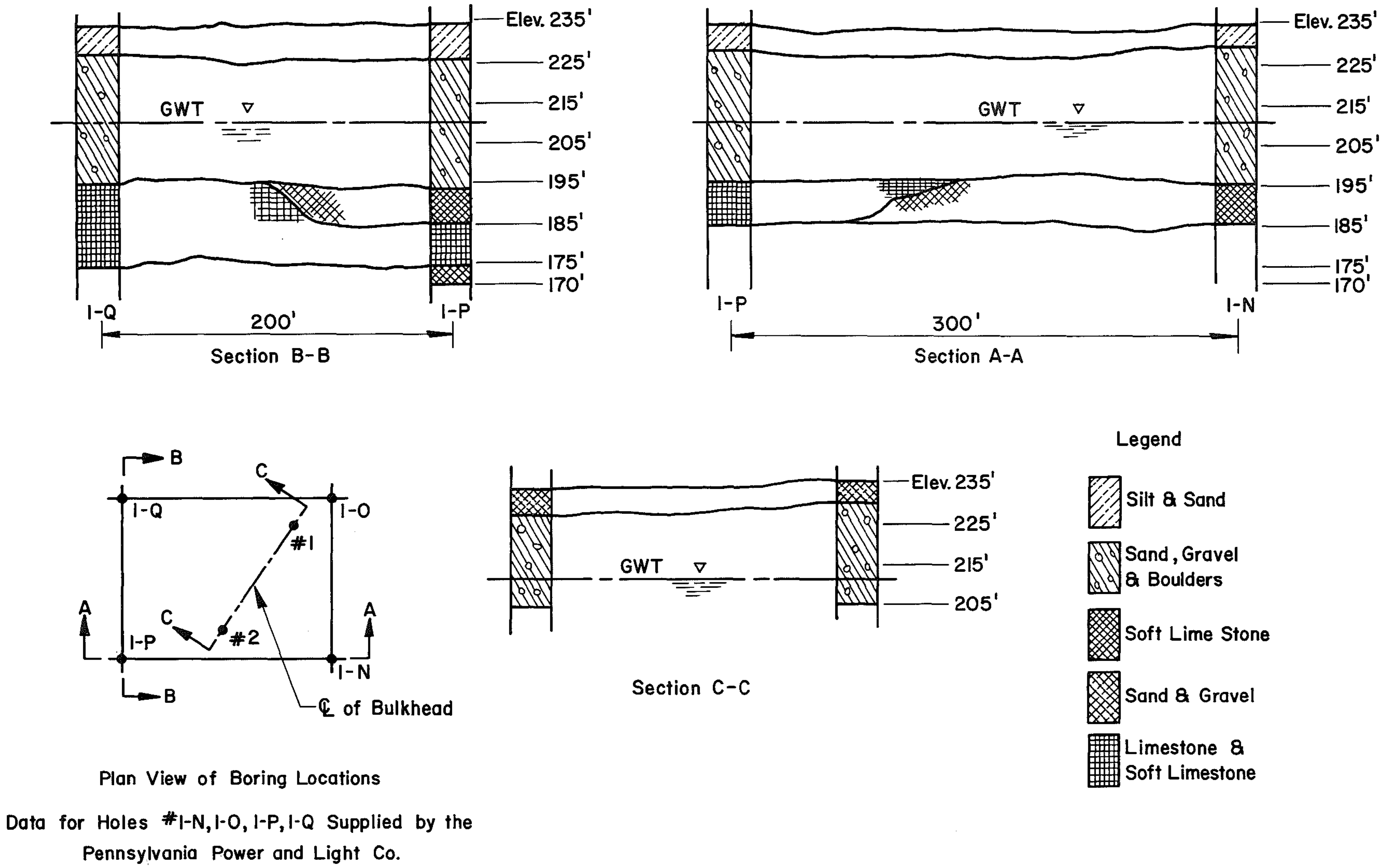


Fig. 3 Soil Profiles

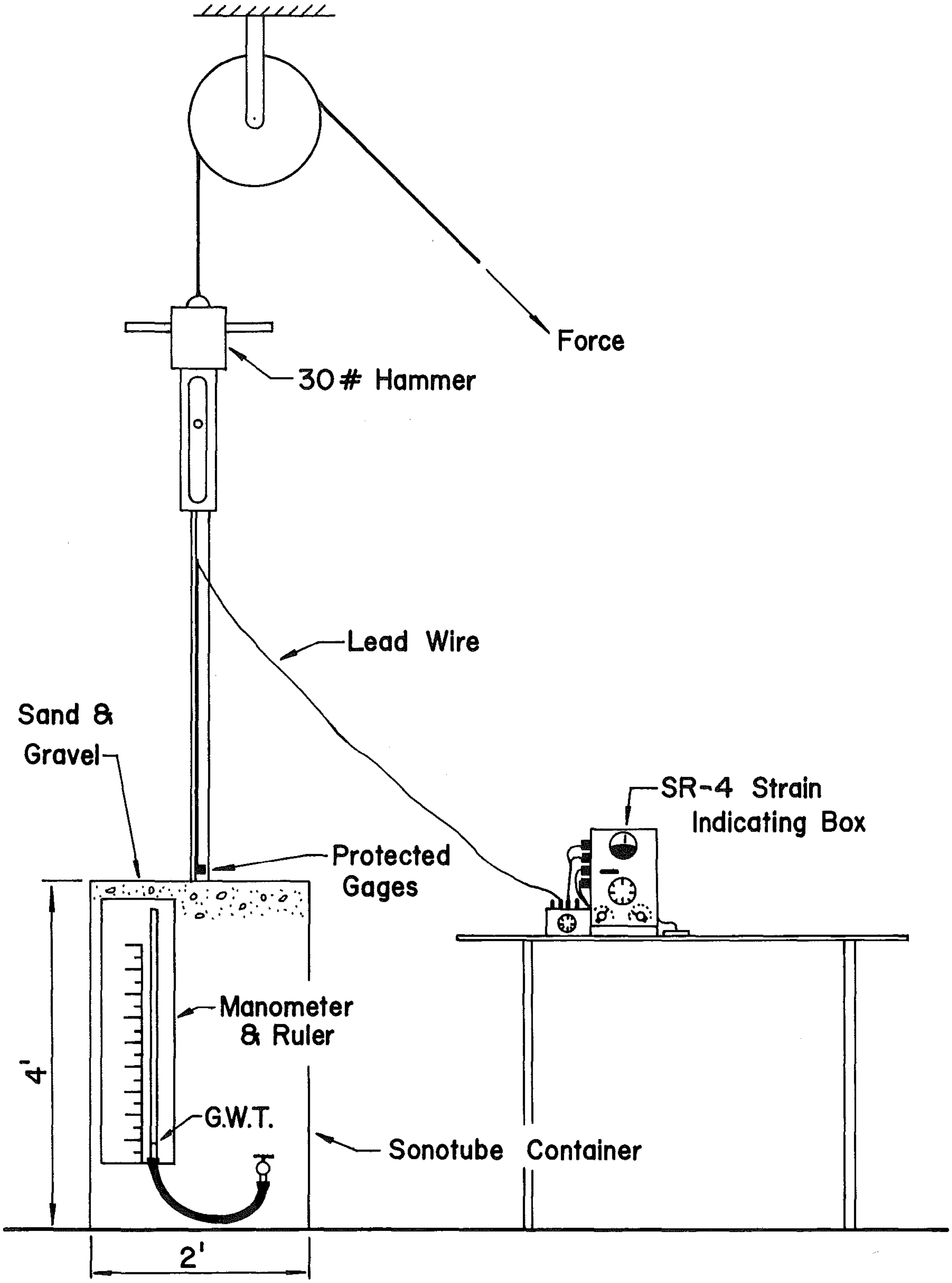


Fig. 4 Laboratory Apparatus for Evaluation of Instrumentation

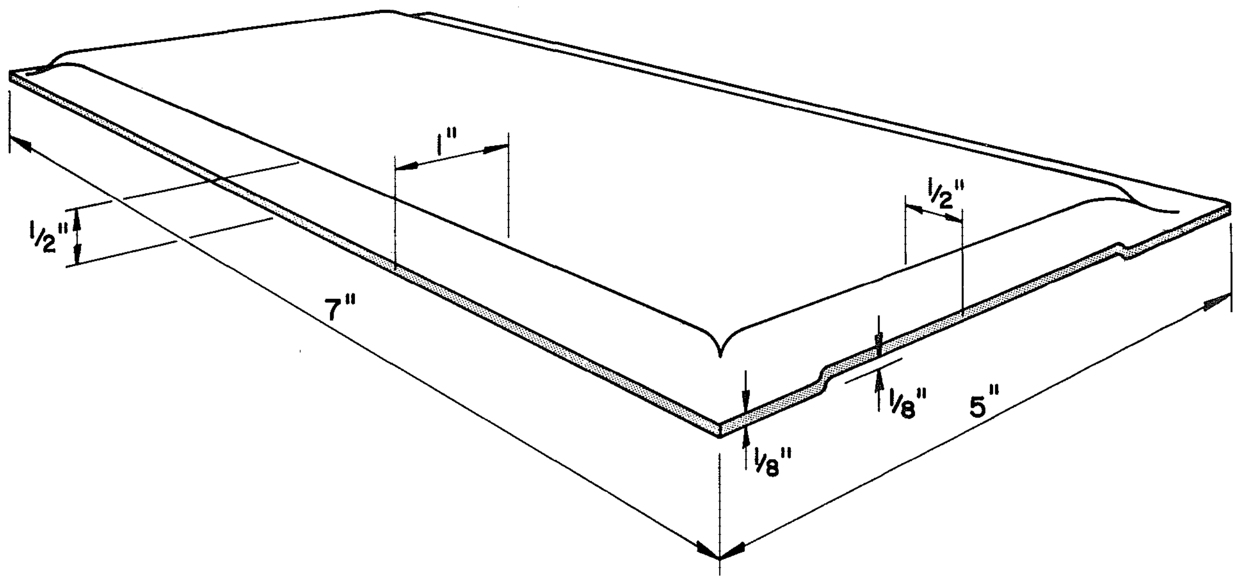


Fig. 5 Steel Protective Shoe

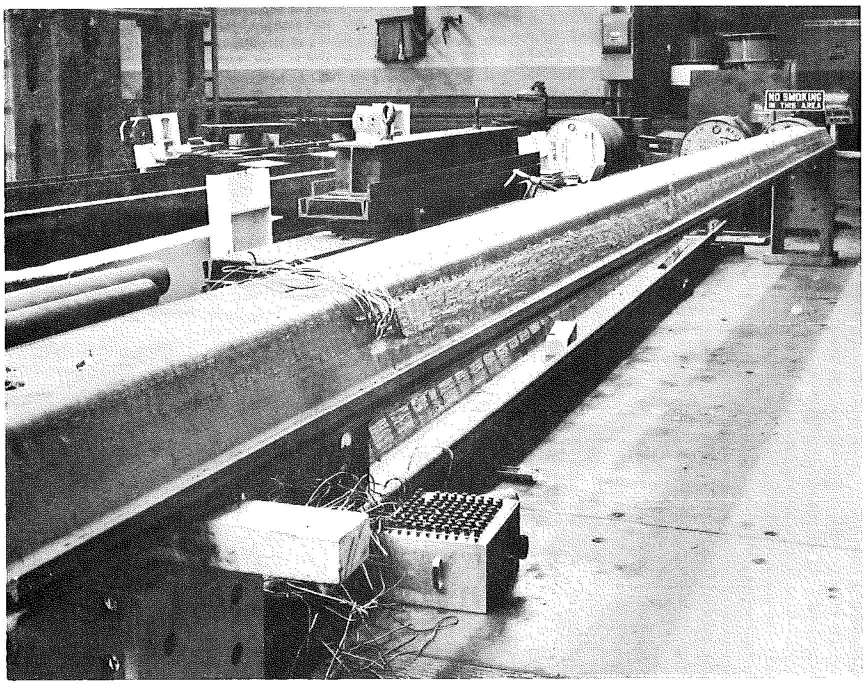


Fig. 6 A Steel Sheet Pile being Equipped with Strain Rosettes

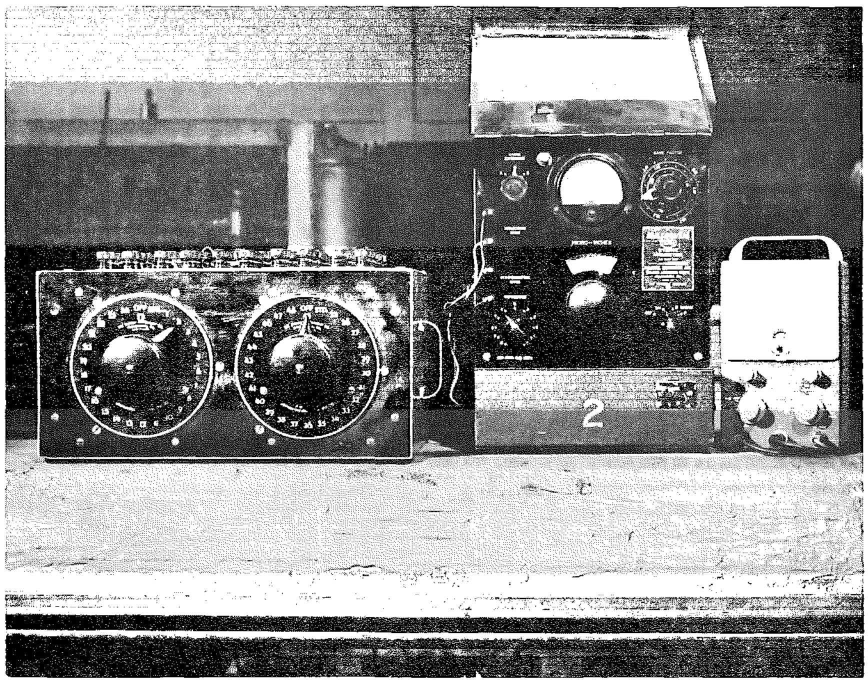


Fig. 7 Laboratory Instrumentation

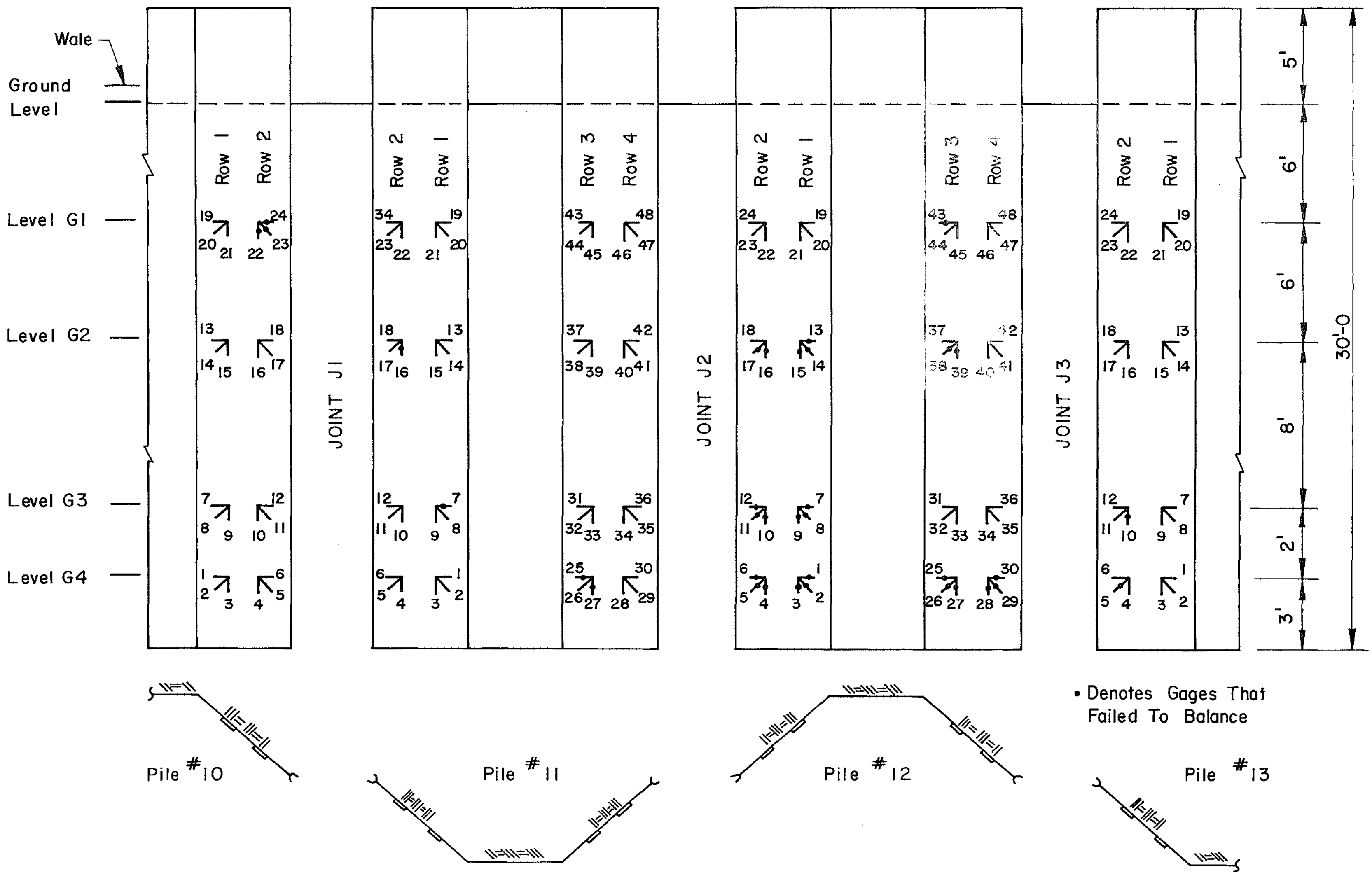


Fig. 8 Location of Strain Gages

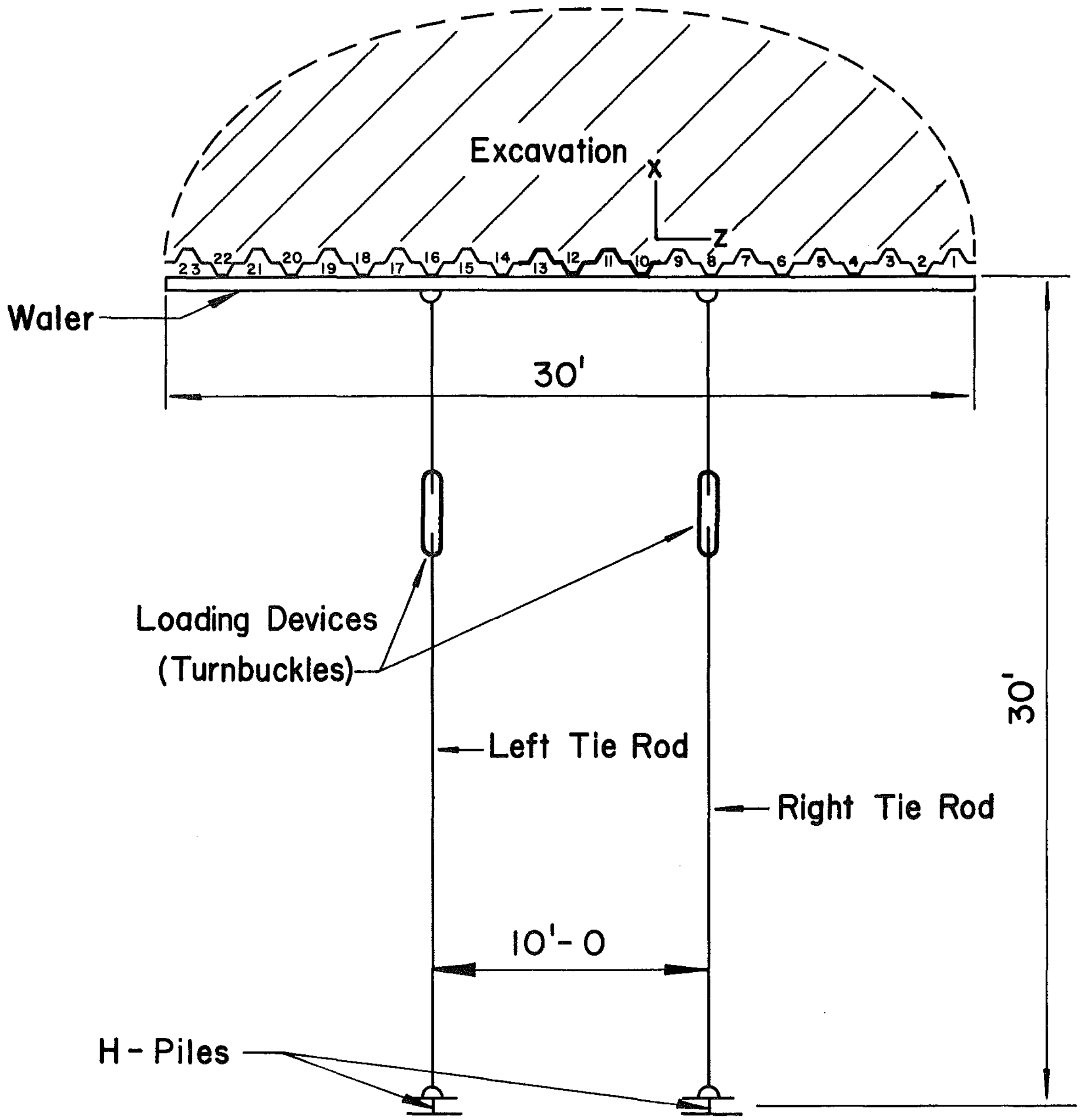


Fig. 9 Plan View of Sheet Pile Wall

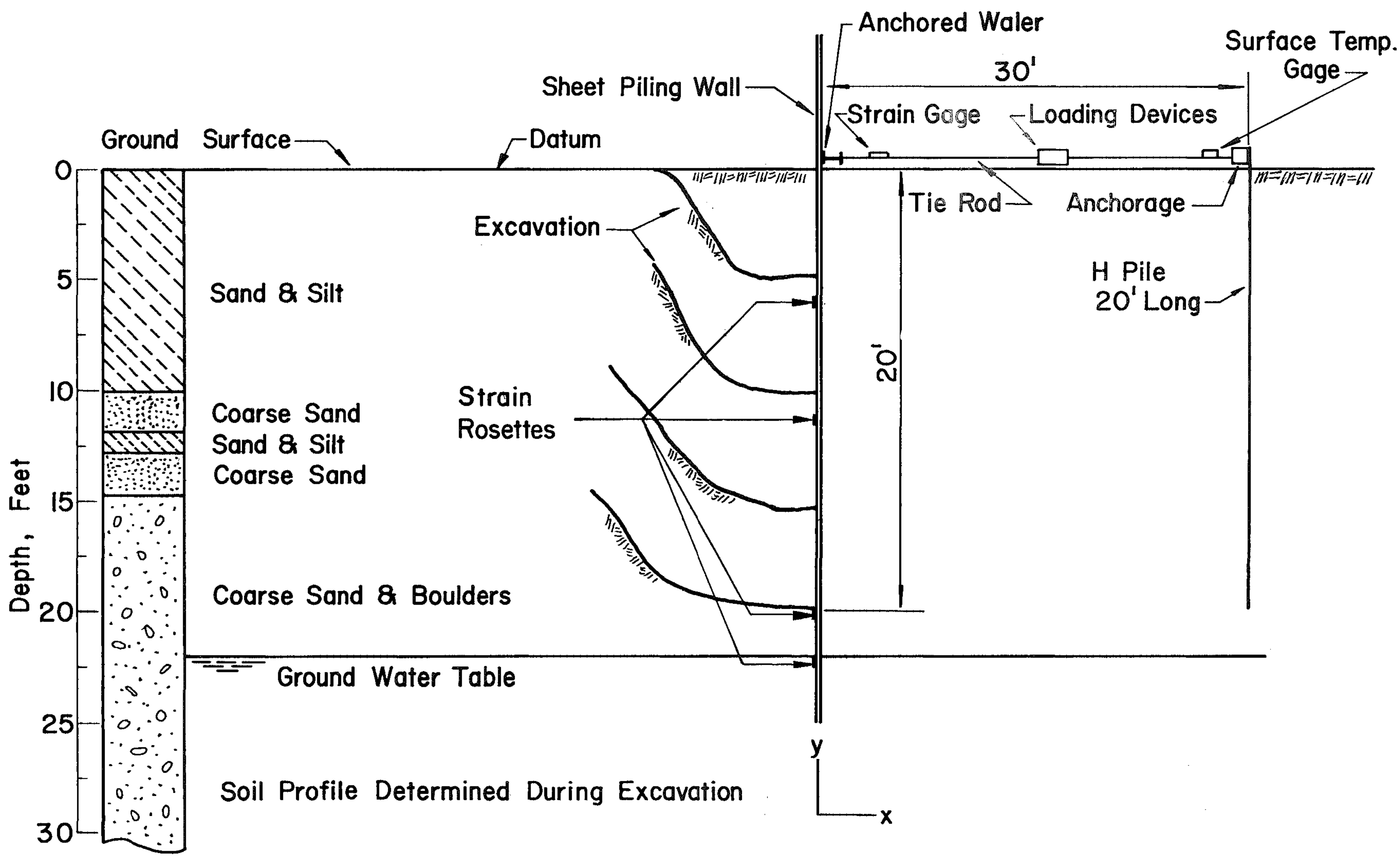


Fig. 10 Cross-Section of Sheet Pile Wall

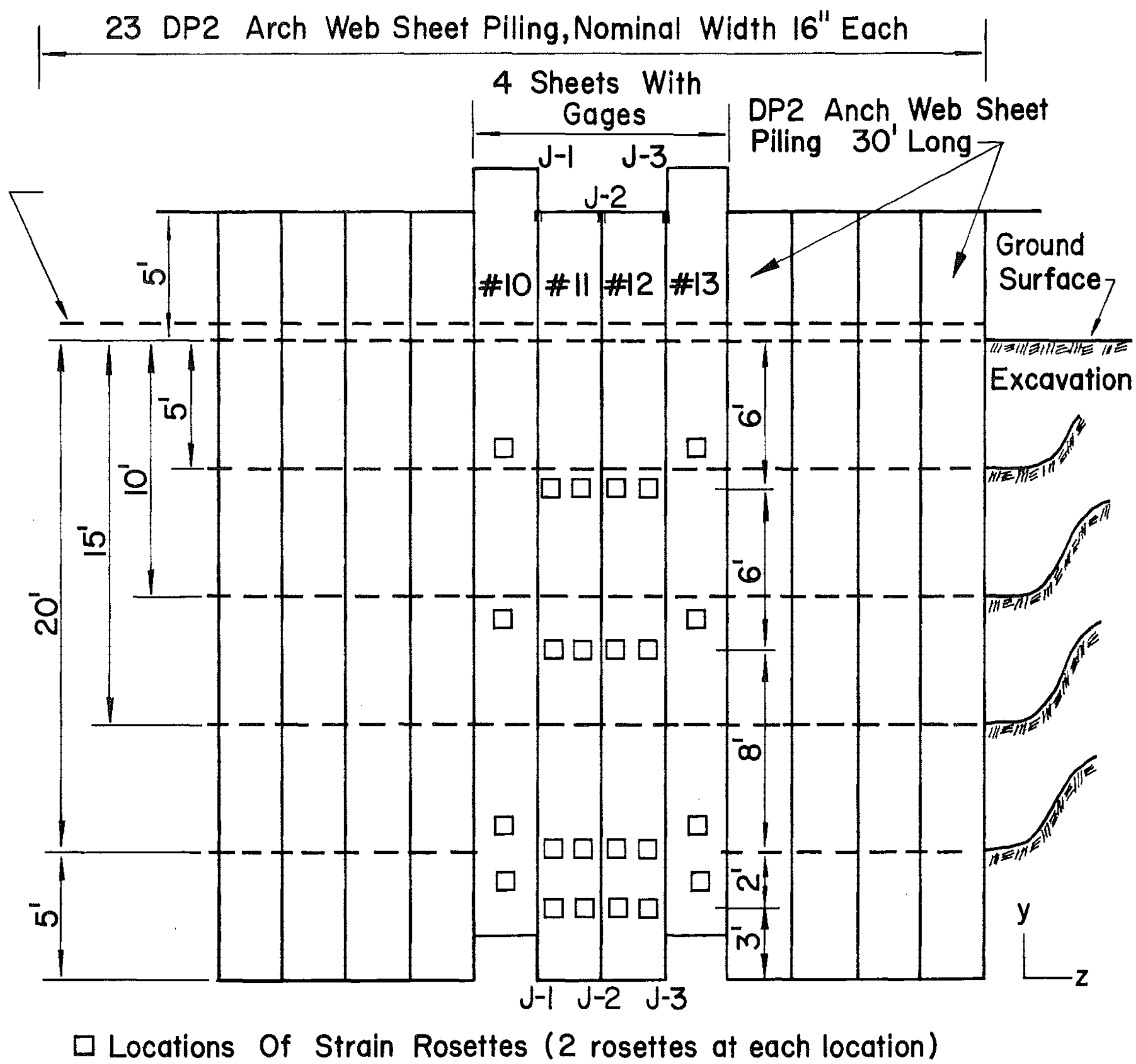


Fig. 11 Front of Sheet Pile Wall Showing Location of Strain Gages

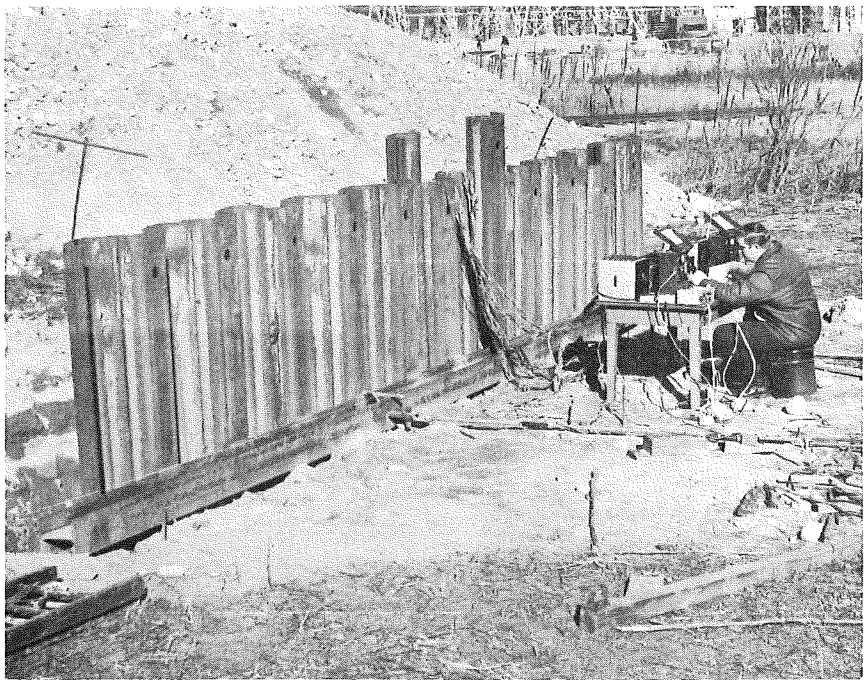


Fig. 12 Photograph Showing Field Test in Progress

Fig. 13 Photograph of Excavation at the 10' Level

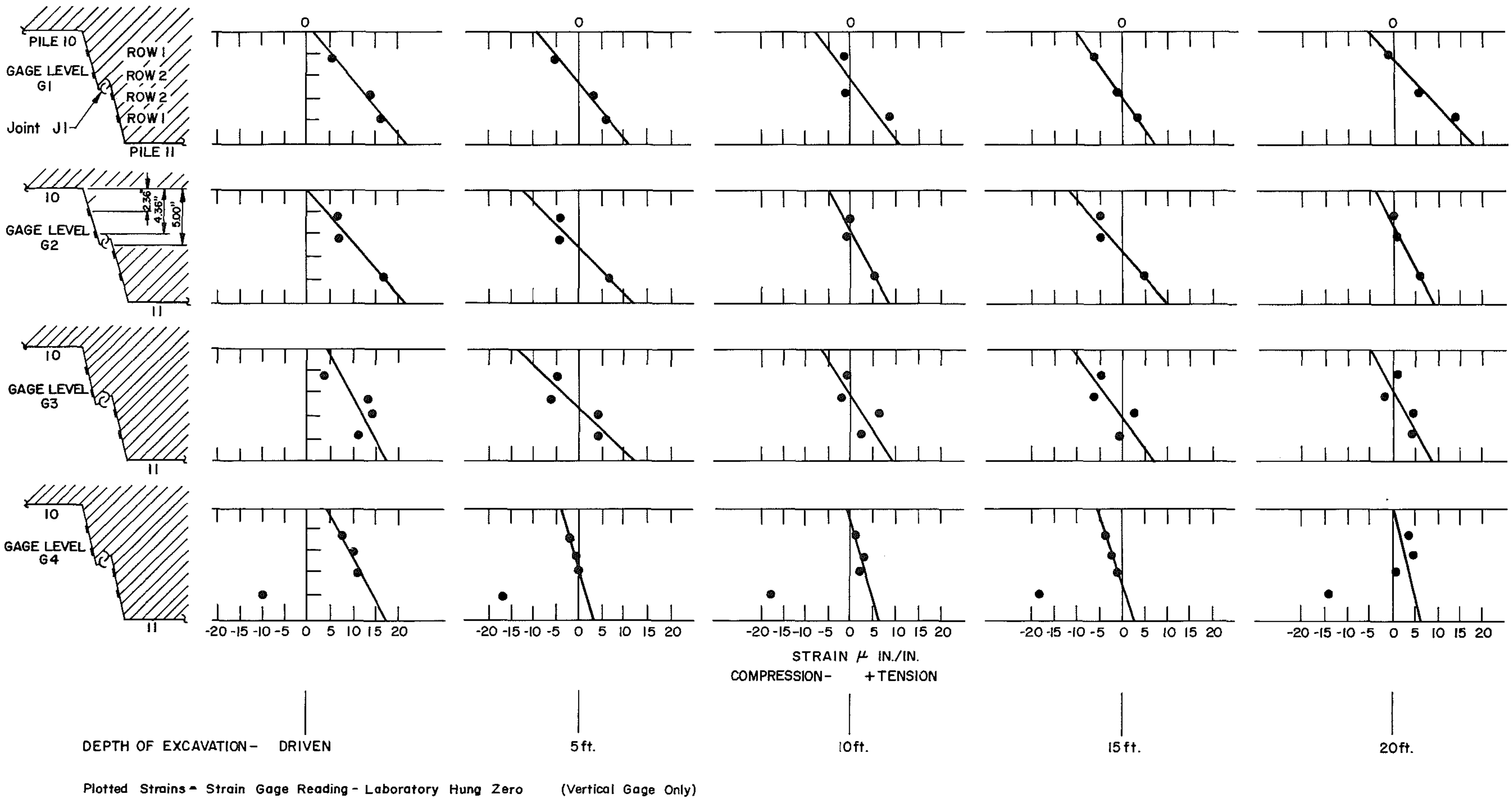
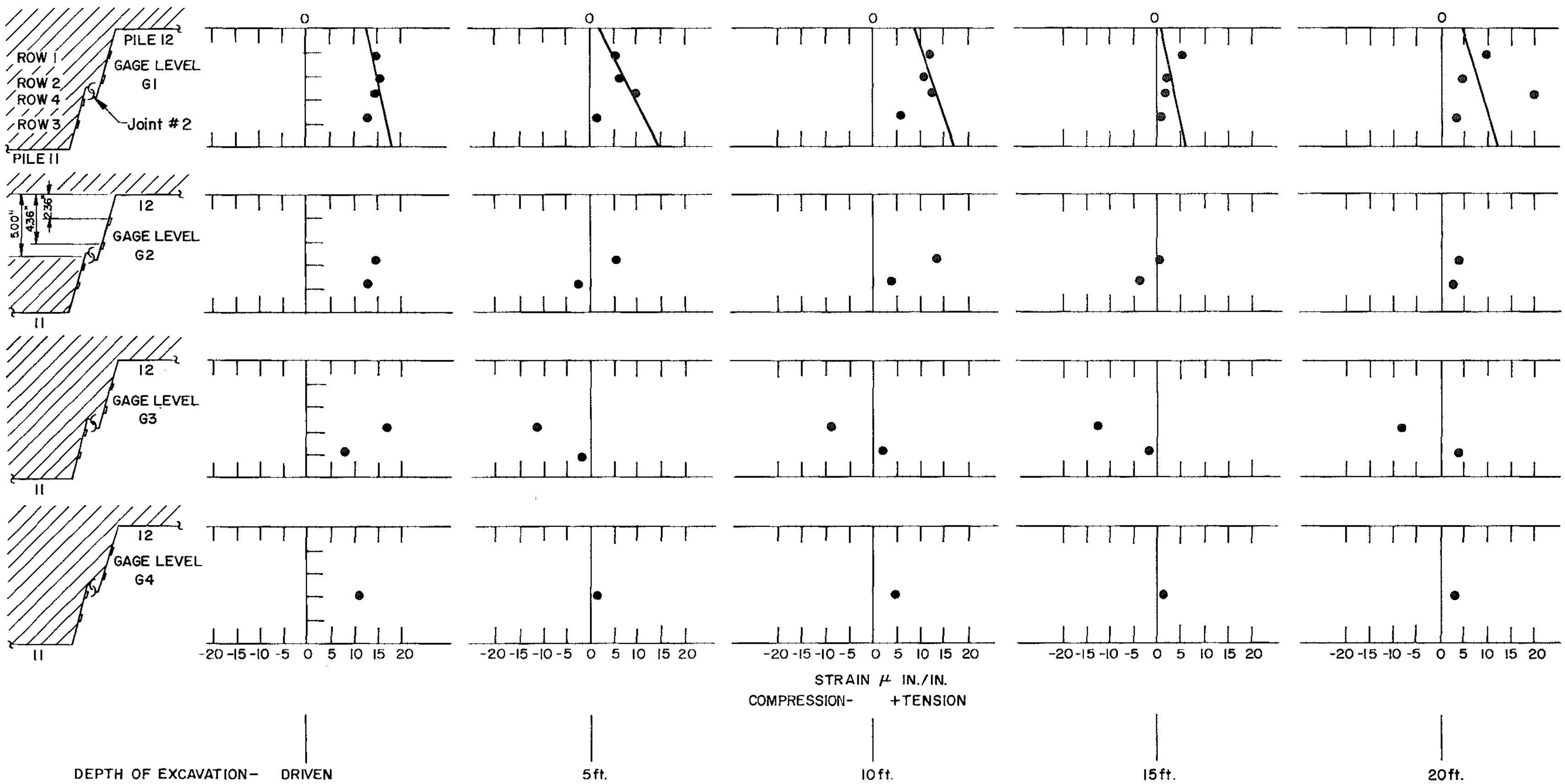


Fig. 14 Vertical Strain Distribution Interlock (Joint 1, Piles 10 and 11)



Plotted Strains = Strain Gage Reading - Laboratory Hung Zero (Vertical Gage Only)

Fig. 15 Vertical Strain Distribution Across Interlock (Joint 2, Piles 11 and 12)

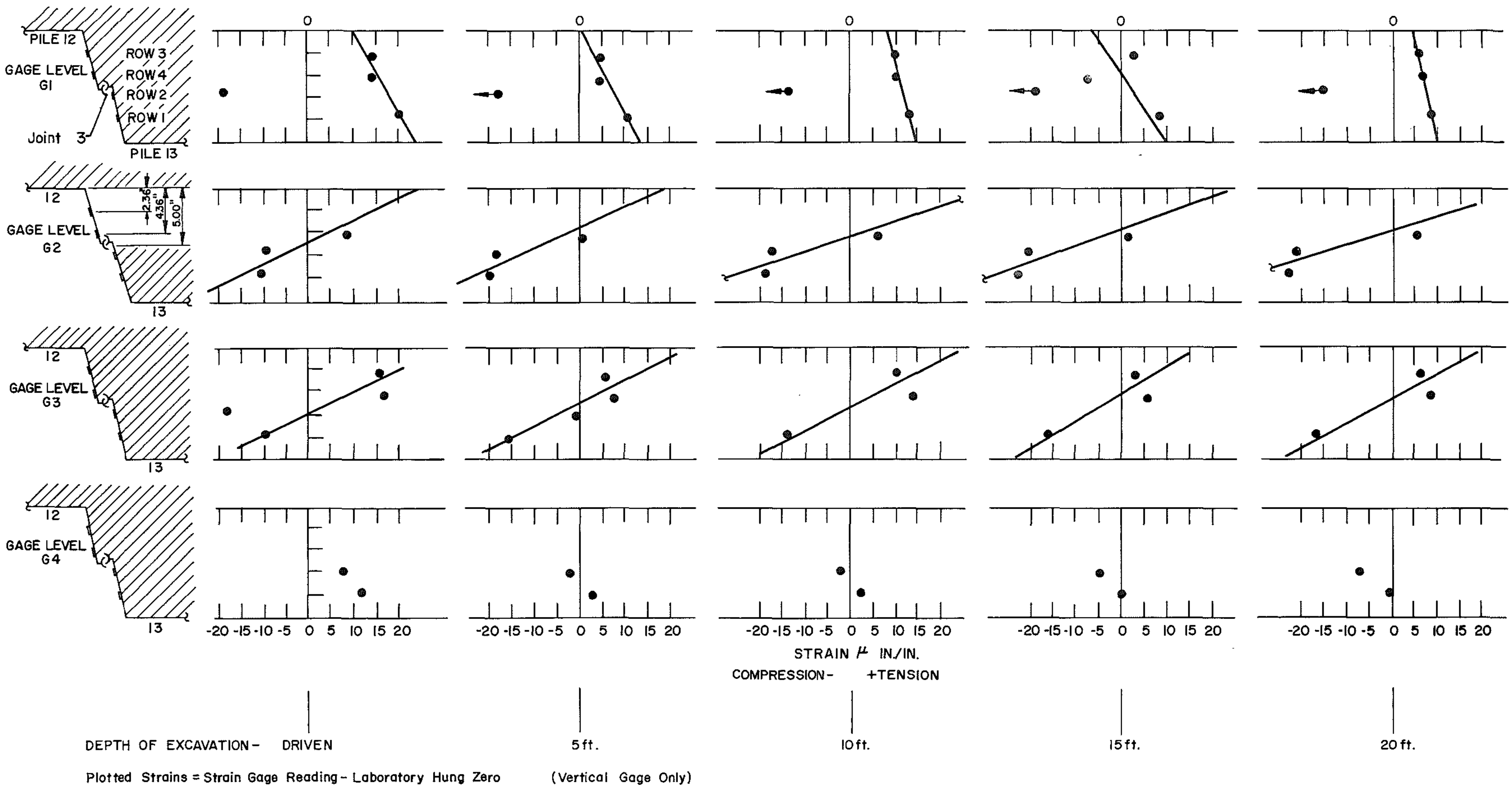


Fig. 16 Vertical Strain Distribution Across Interlock (Joint 3, Piles 12 and 13)

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