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April 2, 1974

MEMO TO: J. R. Harbison
State Highway Engineer
Chairman, Research Committee

SUBJECT: Research Report No. 386; "Loads on Box Culverts under High Embankments"; KYHPR-72-68; HPR-1(9), Part II

The report submitted herewith records short-term progress in a long-term study of "imperfect trench" designs applied to box culverts under high fills. The "imperfect trench" has been employed expediently in a few instances to box culverts in Kentucky; but, heretofore, none had been instrumented. "Imperfect trench" designs have been applied routinely to pipe culverts since about 1957. The duration of the "imperfect trench" effect remains in question. Noticeable settlement or dips in the roadway directly over culverts leads to an abiding suspicion that the "cushion" is being collapsed. On the other hand, a hump in the roadway would be a tell-tale sign of undue load on a culvert if it were under a significant height of fill. If a culvert without a cushion above or below it cannot settle as fast as the earth beside it, the load becomes greater than the deadload of fill (WH). This expresses the folly of building box culverts on bearing piles.

To avoid a hump or a dip, the plane of equal settlement would have to be at the top of the embankment. The position of this plane can be estimated but not controlled; it depends on the size of the structure, fill height, and depth of soil underneath.

Although it is unlikely that a culvert structure having an "imperfect trench" cushion will ever bear as much overburden load as it would if it had been built without a cushion, the load might eventually approach WH. Assuming that a dip at the roadway is tolerable, the question becomes: Should large structures be designed for an eventual load of WH; or can it be confidently designed for a lesser load, and how much less?

Additional structures will be instrumented during the 1974 construction session -- on KY 627, between Winchester and Boonesboro.

Respectfully submitted,

Jas. H. Havens
Director of Research

JHH:gd
Attachment
CC's: Research Committee

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16. Abstract

The structural design of culverts requires a reliable estimate of the earth pressures which will come to bear on the structure during and after construction of the embankment. The actual bearing pressure at a given time and under various conditions of differential settlement may be greater or less than the deadload of the fill or embankment over the structure. This report describes the instrumentation and construction of three box culverts designed by the imperfect trench method. A total of 42 Carlson earth pressure cells were installed in conjunction with strain gages and settlement measuring devices, including inverted settlement plates and mercury settlement gages. Measurements made during the first few months indicate the imperfect trench has considerably reduced the overburden loads bearing on the structures.

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Research Report
386

LOADS ON BOX CULVERTS UNDER HIGH EMBANKMENTS

INTERIM REPORT
KYHPR-72-68; HPR-1(9), Part II

by

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Division of Research
Bureau of Highways
DEPARTMENT OF TRANSPORTATION
Commonwealth of Kentucky

April 1974

INTRODUCTION

The structural design of reinforced concrete box culverts is dependent upon a realistic estimate of loads to be supported. In years past, embankments over culverts seldom exceeded 40 ft (12.2 m). Now, embankment heights of 100 ft (30.5 m) or more are common *(1)*. The simplest but not necessarily most accurate estimate of loads or bearing pressures is the computation of the weight of earth above the structure. In the early 1900's, Anson Marston, Director of the Iowa State Engineering Experiment Station, encountered the problem of determining loads on large diameter pipes in deeper cuts than had previously been used. When failures occurred, he initiated experiments to determine why the recognized practices failed *(2)*. The theory which emerged *(3)* has been generally supported in subsequent investigations.

Theory

Marston theorized that loads bearing on underground structures are influenced by an arching effect wherein a portion of the soil weight above the structure is transferred through frictional forces to the side fill material. Depending upon the relative settlement of adjacent soil prisms, the arching effect could either increase or decrease the load to be borne by the structure. To control relative settlement and to decrease loads on underground structures, Marston devised the so-called imperfect trench *(2)* (Figure 1).

The imperfect trench insures that soil settlement directly above a structure will be greater than that on either side. Thus, frictional forces are mobilized in such a way that part of the material directly over the structure is supported by the side fill material. To accomplish this, the structure is first placed on its foundation and backfilling is initiated. When the fill has reached some predetermined height above the top of the structure, a trench is dug directly over the culvert the same width and depth as culvert dimensions. The trench is then filled with loose, compressible backfill and embankment construction is continued in the normal manner until grade elevation is reached.

The settlement differential caused by the trench is not continuous throughout the embankment height. At some elevation in the embankment, the side fill settlement equals that of the central soil prism. This is called the height of equal settlement and may be real or imaginary, depending on the height of fill. The existence of this plane has been shown in previous research, and Marston's theory of load reduction is widely accepted today *(2, 4, 5, 6, 7)*.

Present Design

Presently, the Kentucky Bureau of Highways uses AASHTO design formulas (Marston's) for underground structures *(8)*. For culverts in trenches on unyielding subgrades or untrenched on yielding foundations, the vertical design load *(2)* is computed as

$$P = WH.$$

For structures untrenched or on unyielding foundations, the vertical design load *(3)* is

$$P = W\,(1.92H - 0.87B) \text{ for } H > 1.7B,$$

where
- P = unit pressure, lb/ft^2,
- W = effective weight (lb) per cubic foot,
- H = height of fill over the culvert, ft, and
- B = overall width of the culvert if untrenched or width of the trench if trenched, ft.

Design pressure for sidewalls is calculated using the Rankine formula for equivalent fluid pressure *(4)*:

$$P_s = KV$$
$$K = [\sqrt{\mu^2 + 1} - \mu]/[\sqrt{\mu^2 + 1} + \mu]$$
$$= (1 - \sin\phi)/(1 + \sin\phi)$$
$$= \tan^2(45° - \phi/2) \text{ } (5),$$

where
- P_s = sidewall pressure, lb/ft^3,
- K = Rankine coefficient,
- μ = coefficient of internal friction, and
- ϕ = angle of internal friction.

A minimum value of P_s = 30 lb/ft^3 (481 kg/m^3) is used in design of underground structures *(8)*.

Application of formulas involves personal opinion of the designer or practice of the designing agency. An allowable weight reduction is permitted by AASHTO for trenched structures or those placed on yielding foundations. The Kentucky Bureau of Highways does not permit a weight reduction for trenched culverts on unyielding foundations *(9)*. The question of when to permit a weight reduction in design load was the original impetus to this research study. The imperfect trench has been observed as being effective over short periods of time under relatively low embankments but has been only partially observed under high embankments for long periods of time. Also, factors affecting loads on underground structures were of an empirical nature. More research was needed in the areas of load distribution and soil analysis *(10)*. Specifically, a method was needed for estimating relative settlement to be expected over a structure. Also, there was a need for review of the Rankine coefficient and its application for high embankments. The objectives of this project were therefore

1. to evaluate factors affecting load configurations under high fills and devise a method of predicting these loads,
2. to determine the height of equal settlement and to **determine whether that height is constant or changes with time**,
3. to devise a method to evaluate Rankine's coefficient for both positive projecting and imperfect conditions,
4. to **compare calculated values of load** by Marston's, Spangler's, and Costes' theories **to measured loads**,
5. to examine the adequacy and economy of design of those culverts under study and **make recommendations for future designs**, and
6. to **prepare standard drawings** such that design of culverts under high fills may be accomplished in a more economical manner.

Three box culverts were instrumented and monitored during and after completion of construction. This report describes the installations and presents data collected to date. Other sites which do not include the imperfect trench in the design will be instrumented to provide comparisons.

SITE DESCRIPTION

Location

The three test structures are located on a newly constructed section of US 27 in McCreary County between Whitley City and the Tennessee state line. Reasons for choice of the sites were: (1) fills to be placed over the culverts were designated as high fills, (2) the culverts were to be placed on solid rock foundations, and (3) the imperfect trench method of construction was specified. It was assumed that solid rock foundations would assure limited differential settlement and a more equal pressure distribution than could be expected with a soil foundation.

At Station 89 + 20, an 8 ft x 8 ft (2.4 m x 2.4 m) reinforced concrete box culvert was placed on a rock foundation under 48 ft (14.6 m) of fill. At Station 203 + 20, a 5 ft x 5 ft (1.5 m x 1.5 m) reinforced concrete box culvert was constructed on solid rock under a 72-ft (22-m) fill. At Station 210 + 50, a 5 ft x 6 ft (1.5 m x 1.8 m) reinforced concrete structure was placed on rock under a 96-ft (29.3-m) fill. At each site, backfill was completed to a height above the structure equal to the culvert height plus one ft (0.3 m). Then, a trench equal to the culvert height and width was excavated above each structure. The bottom third of each trench was filled with loose straw. The remaining two thirds was filled with material removed during trenching. Embankment construction was then continued in the normal manner.

Geology

The sites lie in an area where the Breathitt and Lee Formations outcrop. The area around Whitley City lies in the Corbin Sandstone, a member of the Lee Formation. This is characterized by coarse- to medium-grained sandstone, light gray to brown in color, occurring in massive formations around Stearns and Whitley City. Beds of shale and siltstone 10 to 90 ft (3.0 to 27.4 m) thick underly the Corbin Sandstone. Near Pine Knot, the massive sandstone of the Corbin member joins with carbonaceous shale and siltstone of the Lee Formation. This lies in the River Gem coal bed and is characterized by shale and siltstone with discontinuous beds of the sandstone.

The structures were to be placed on rock foundations. Those foundations were of shale that is common to the area.

Soil Classification

Soils used in the three embankments were sands with some shale intermixed. Particle-size analyses on samples from all three sites indicated the average sand content of the soils to be 74 percent, except at Station 203 + 50. There, the sand content increased with depth in the fill and ranged from 32 percent to 65 percent. Clay contents were low, ranging from 11 percent at Station 89 + 45 to 31 percent at Station 203 + 20. Results of particle-size analyses, Atterberg limits tests, and CIU (consolidated-isotropic-undrained) triaxial tests are listed in Table 1.

INSTRUMENTATION

Instrumentation was similar at all sites. Instrumentation included Carlson earth pressure cells, strain gages, settlement plates, and mercury-filled settlement gages.

Carlson Cells

At the intersection of each culvert and the centerline of the roadway, eleven Carlson earth pressure cells were installed *(11)*. Nine cells were placed on each culvert, and two were embedded in the foundation material. Three cells were placed on each side and three on the top slab of each culvert. Cells were placed diagonally across the faces of the culvert to reduce possibility of creating weakened planes (Figure 2). Cells in the sidewalls were cast in place. Prior to placement of concrete, cells were bolted to the outside wall form with three pieces of angle iron (Figure 3). Electrical cables were inserted through the inside wall form and tied to reinforcing bars for support. After the concrete had cured, bolts were removed, forms were stripped, and the Carlson cells remained embedded in the outside wall face.

Cells were placed in the top slab after the concrete had cured. This assured that no cell floatation would occur and that voids under the cell could be effectively eliminated. At the time of concrete placement, ports were cast in the top slab at points where cells were to be set. These were square ports with electrical conduit extending through the entire slab (Figure 4). Cables were inserted through the conduit to the inside of the structure after the cells were placed. 3M underground insulating resin was used to secure the units in their ports and to eliminate air voids under the cell base (Figure 5).

One pressure cell was embedded in the foundation material on each side of the culvert 4 to 8 ft (1.2 m to 2.4 m) from the culvert wall on the centerline of the roadway (Figure 2). These were set as closely as possible to the flowline elevation on a stable rock or shale base. In each case, a cavity was excavated in the foundation to hold the cell and the cell was placed face down on a layer of insulating resin. The resin provided a smooth and level surface for placing the cells. Cables to those cells were inserted through precast ports in the sidewalls. All cables were encased in downspout pipe for protection (Figure 6) and were run to a common collection point at the culvert inlet (Figure 7).

Between the centerline section and the inlet, three other pressure cells were installed -- two on the top slab and one on the sidewall (Figure 8).

Settlement Installations

Inverted settlement plates were installed at each test site (Figure 9). The 3-ft x 3-ft x 1/2-in. (0.9-m x 0.9-m x 1.3-cm) steel plates were placed at the top of the imperfect trench in order that settlement in the trench could be measured from inside the culvert barrel. Mercury-filled settlement gages *(12)* were installed at that elevation in order to obtain checks and additional settlement data for the trench and the material immediately surrounding it (Figure 10). Three other settlement gages, two of which were accidently destroyed, were installed at progressively higher elevations at Station 210 + 50 in order to check the total fill settlement (Figure 8). Remote observation sites were established on the inlet embankment slope at the elevation of each settlement gage installation. No settlement gages were installed at Station 203 + 20 due to several accidents, equipment malfunctions, and the pressing nature of the installations at the other two sites. At Station 89 + 10, three settlement gages were placed within the fill at varying elevations. All observation sites were referenced to benchmarks so that any relative settlements of the sites could be obtained and used for corrections.

Strain Gage Installations

Strain gages were mounted on the steel reinforcing bars (Figure 11) in the laboratory. These bars were installed within two separate sections in each culvert. One section was at the centerline installation in order that correlations could be made between pressure measured and strain in the steel. The other section was nearer the inlet where two pressure cells were installed (Figure 8). SR-4 strain gages were mounted with epoxy (EPY-150-BLH) on flat surfaces machined on the steel bars. The gages were then covered with a waterproofing material (Barrier E) and taped to prevent accidental destruction of the gage and leads during concrete placement. BLH 3-lead strain gage wire was used and inserted through downspout pipe (Figure 6) to the observation point. The gages were then connected to a switching unit (BLH Model 1220) and initial readings were taken at the time of installation. The strain measuring unit was a BLH Model 1200 Digital Strain Indicator.

DATA PRESENTATION AND DISCUSSION

Earth Pressures

Pressures recorded for each of the Carlson units have been plotted versus time in days (Figures 12 through 23). Expected pressures were calculated using P = WH. Pressures approaching expected pressures were recorded from bedrock units which were placed in such a way as to be outside the imperfect trench influence area. All other meters were in areas where the trench was expected to reduce pressures. The implication is that the imperfect trench is initially effective. At Station 203 + 20 in the centerline section, pressures exceeded the expected pressures. Meters 26, 27, and 28 (Figure 14B) all show extreme values, with 27 and 28 starting higher than expected pressures and 26 showing extremely low values. These values are questionable since all three meters are located in the same plane and at the same location on the culvert barrel. The section in question is under 72 ft (21.9 m) of fill, and all three units are located on the roof of the structure beneath the imperfect trench. The meters installed on bedrock were for measuring the fill pressure not affected by the imperfect trench. Meters 17 and 19 at Station 203 + 20 (Figure 13A) and 31 and 32 at Station 210 + 50 (Figure 17A) show pressures approaching the expected pressure. Meters 7 and 8 at Station 89 + 20 (Figure 23B) are exhibiting low pressures compared to the expected pressure. This would indicate the cells were not placed out of the imperfect trench influence area. If this were the case, Meters 7 and 8 should agree with Meters 1 and 3 (Figure 22A) which are located at approximately the same elevation. Examination of the curves indicates this to be the case with 7 and 8 showing about 10 psi (68.9 kPa), and 1 and 3 showing 6 psi (41.4 kPa).

Settlement

Measured settlements (Figures 24 through 29) were typical of those encountered in sands. Most of the settlement occurred soon after completion of the embankments. At Station 210 + 50, the settlement gage values checked closely with settlement plate values (Table 2). At Station 203 + 20, no settlement gages were installed. The plates at that location show from 7 in. (17.8 cm) to 11 in. (27.9 cm) of settlement in the imperfect trench. At Station 89 + 20, the settlement plates are not in agreement with the settlement gage. Frictional resistance on the standpipe could cause an inverted settlement plate to "catch" and support the soil above it instead of allowing it to consolidate freely. The mercury gage at this location indicates a more logical value of settlement.

Strain Gages

In each structure, two cross sections of steel were instrumented with SR-4 strain gages. It was intented to compare the stress measured by the Carlson meters to that calculated from strain measurements. If this were successful, then much more instrumentation could be applied to future structures at a fraction of the cost of stress measuring devices. Each cross section had seven strain gages placed on three load-carrying steel bars (Figure 30). Section A was near the centerline Carlson cell installation and Section B near the secondary installation (Figure 8). Of the 42 total gages installed, only 16 were operational at the initial readings. All of these gages were at Stations 203 + 20 and 210 + 50. No gages were functional at Station 89 + 20. The maximum measured strain in each gage is recorded in Table 3.

Several problems occurred during construction which affected the success of the strain gage installations. The gaged steel was tied in place immediately before concrete was placed. Wires from each gage were run through the inside wall form at a common point and were numbered and identified throughout the cross section. The wires were tied with tape to the steel reinforcement to lessen the danger of damage due to concrete placement. On several occasions, the forms were removed without giving notice to or having the supervision of the study engineer. During form removal, many of the thin wires were broken by impatient construction workers

At Station 89 + 20, vandalism was suspected. Here the concrete forms were sucessfully removed without breaking the wiring, but all the identification tags had been torn from the wires. As a result, the gage installation at this site was abandoned.

In the future, the thin, low tensile strength wire will require some means of reinforcement throughout its total length to prevent accidental breakage. The long wiring distances involved splicing the strain gage wire several times. These soldered links were wrapped with electrical tape for protection, but an effective moisture barrier could not be guaranteed. This could account for some of the exceptionally high strains recorded at Station 203 + 20.

Although extreme care was exercised during concrete placement, some of the gages or wiring could have been torn from the mounts inside the wall forms. Likewise, if the barrier around a gage were punctured by large pieces of aggregate, possible chemical damage could have destroyed the gage. These physical problems plus the lack of experienced personnel made the strain gage installations less successful than had been anticipated.

Summary

Measurements from the settlement gages and Carlson pressure cells indicate that the imperfect trench is effective in reducing early-life loads on underground structures. Of course, the long-term effects of the trench will be studied further. Two mercury-filled settlement gages were accidentally destroyed at Station 210 + 50. The two key settlement gages and four inverted settlement plates are still operational at this site. Station 203 + 20 has three inverted settlement plates but no settlement gages. Three gages and three plates at Station 89 + 20 are all functional. From a total of 42 Carlson earth pressure cells, only one has malfunctioned. The strain gage installations suffered greater losses with only 16 of 42 still operating.

In the continuing study, data will be analyzed to find if pressure distributions at all three sites are similar. Adequacy of design can then be investigated and some conclusion drawn concerning the use of the imperfect trench and the present design formulas.

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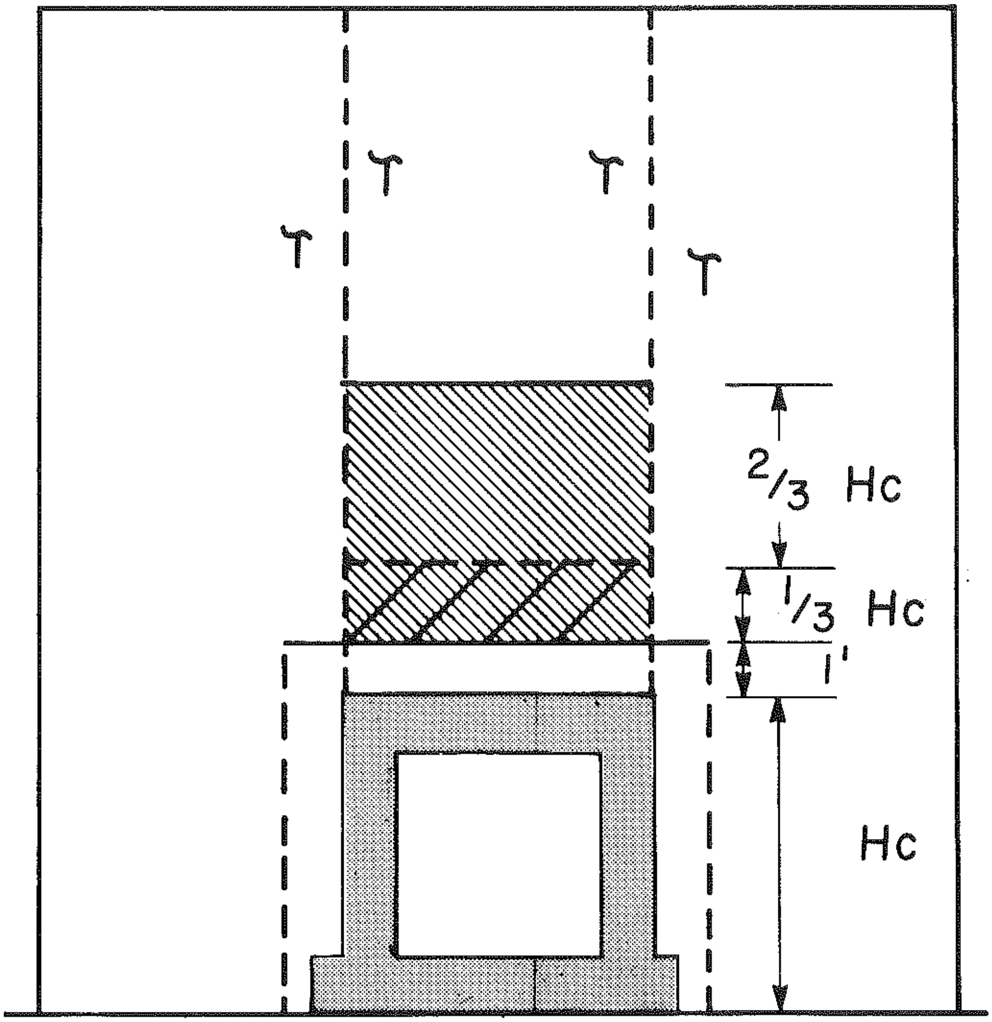


Figure 1. Imperfect Trench.

CARLSON CELL ARRANGEMENT

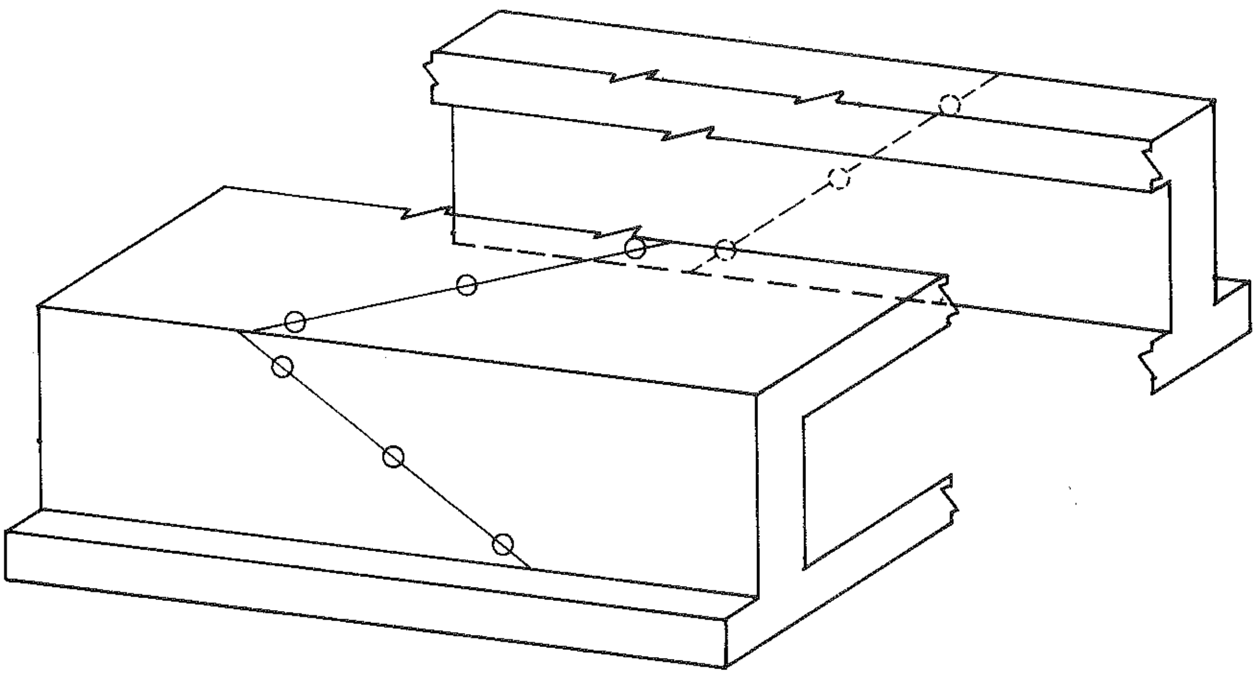


Figure 2. ℄ View of Carlson Meter Arrangement.

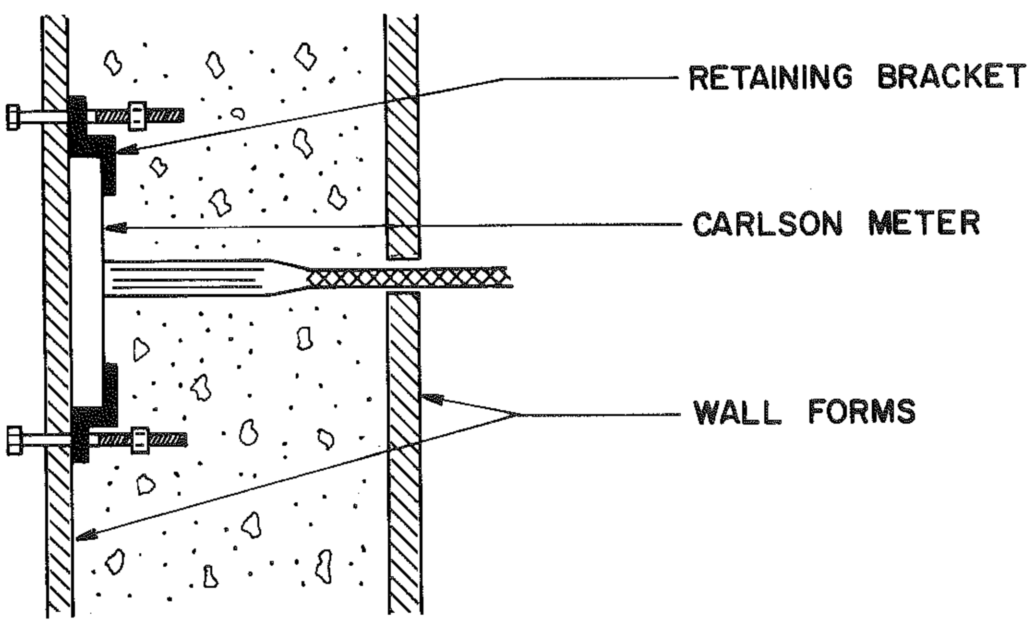
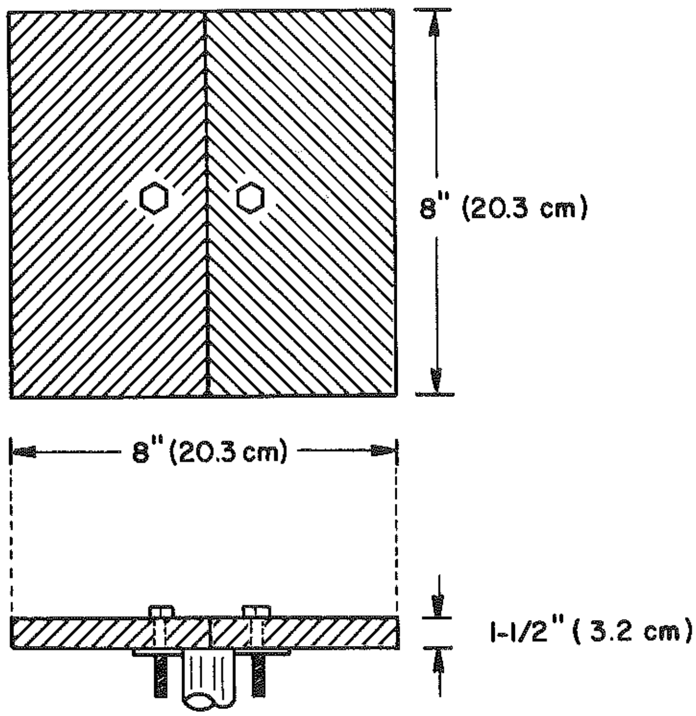
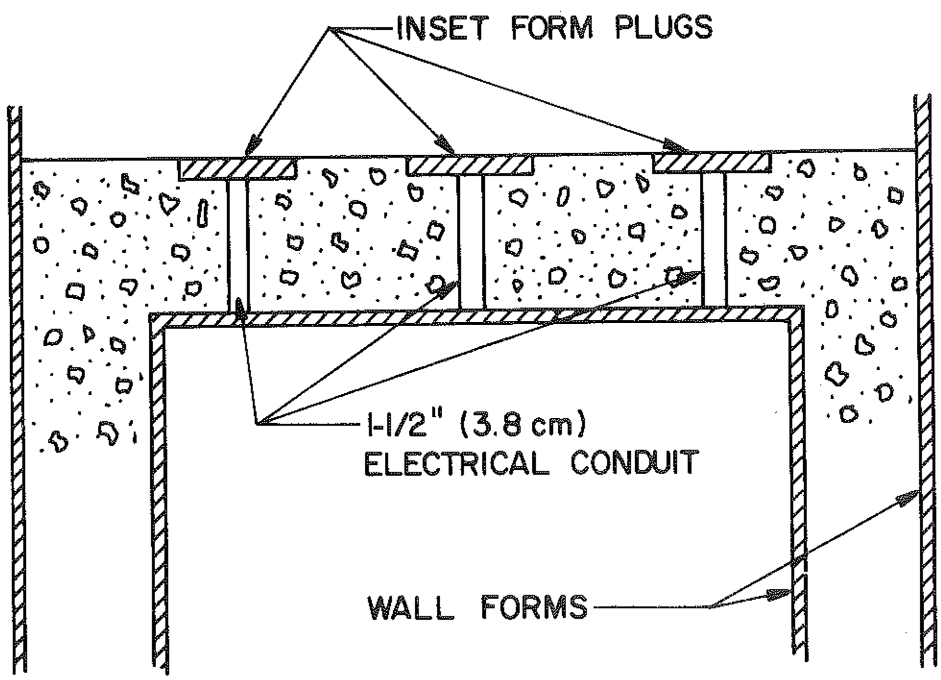


Figure 3. Carlson Meter Attached to Sidewall Form.



FORM PLUG DETAIL

Figure 4. Top Slab Ports.



TOP SLAB FORMING LOCATIONS
CARLSON METER POSITIONS

Figure 5. Top Slab Meters in Place.

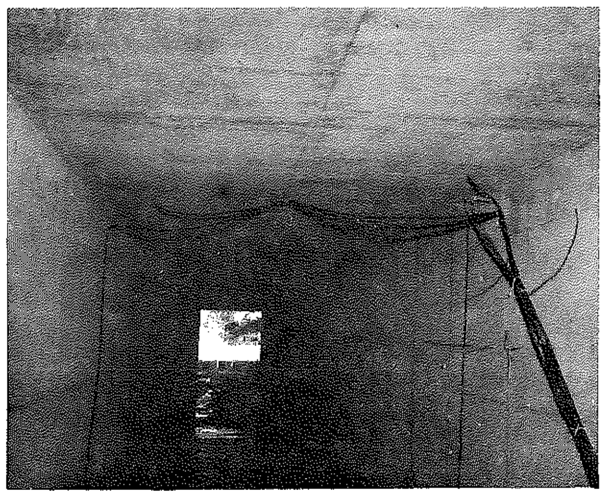


Figure 6. Cables inside Culvert Barrel.

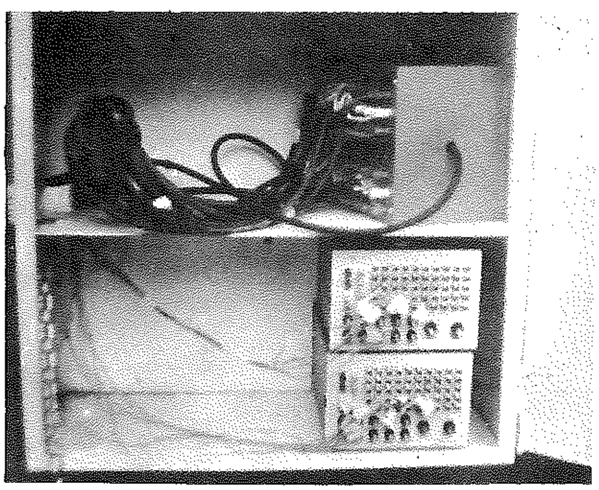


Figure 7. Cable Collection Box.

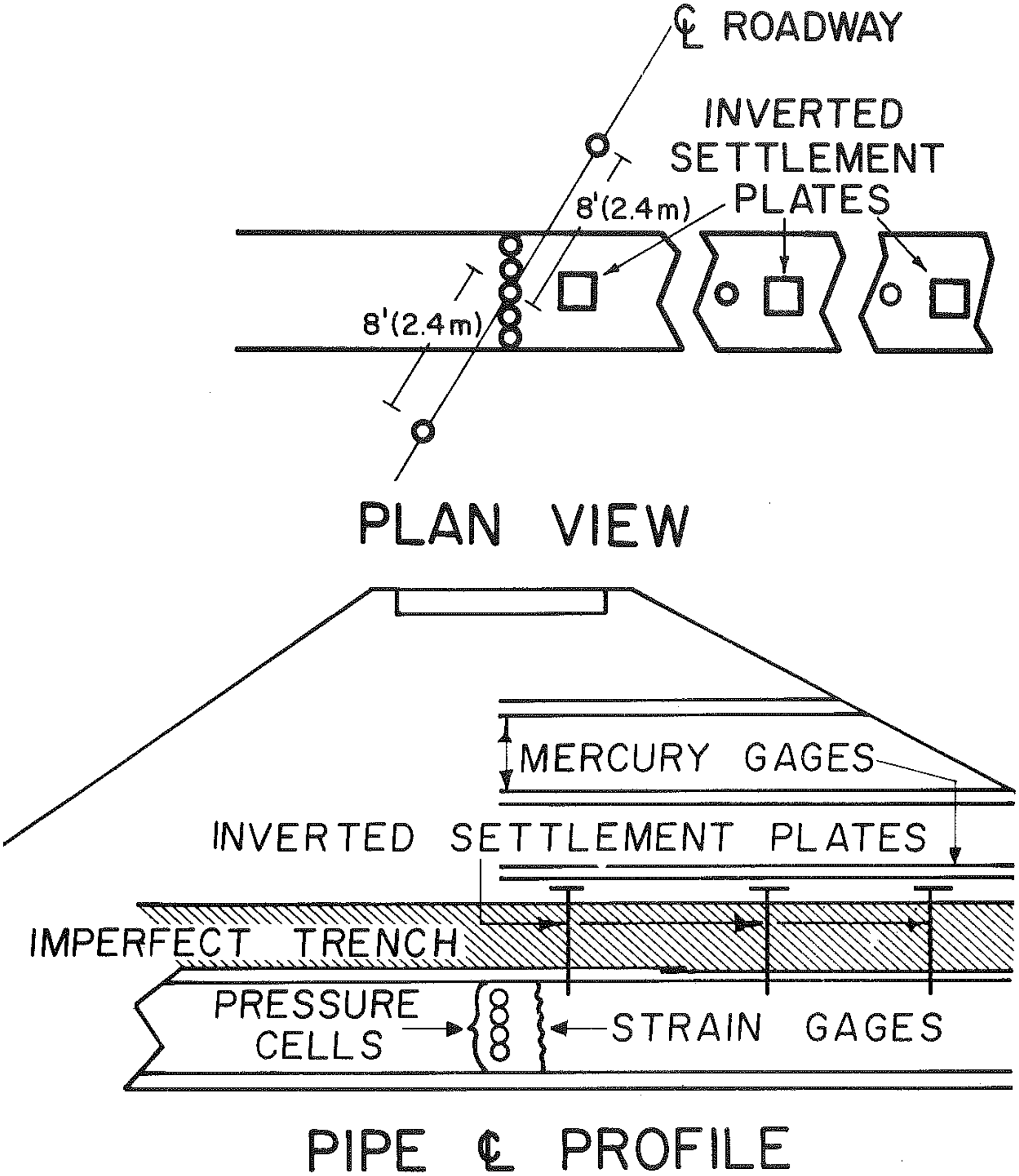


Figure 8. Plan and Profile Views of Station 210 + 50.

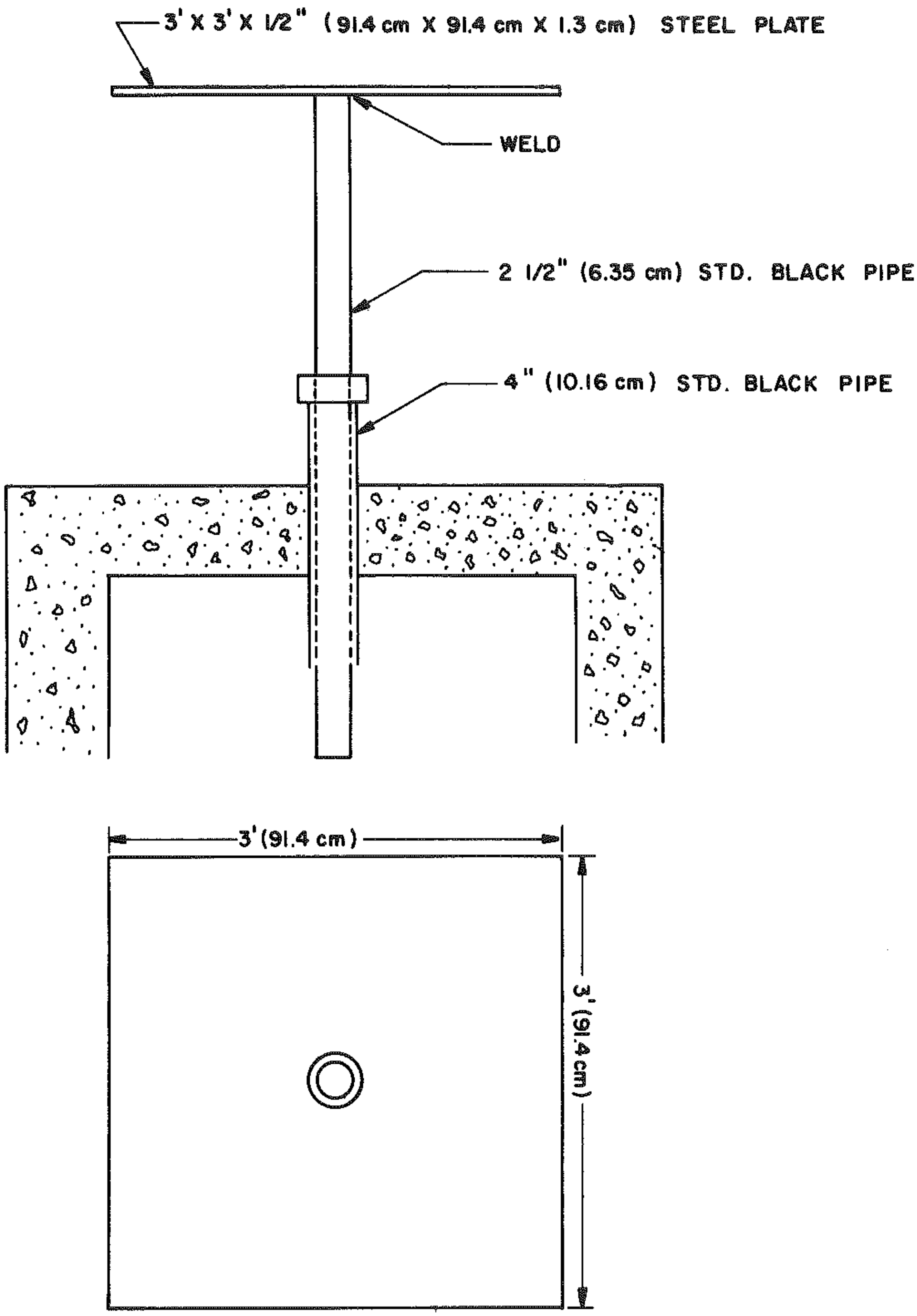


Figure 9. Inverted Settlement Plate.

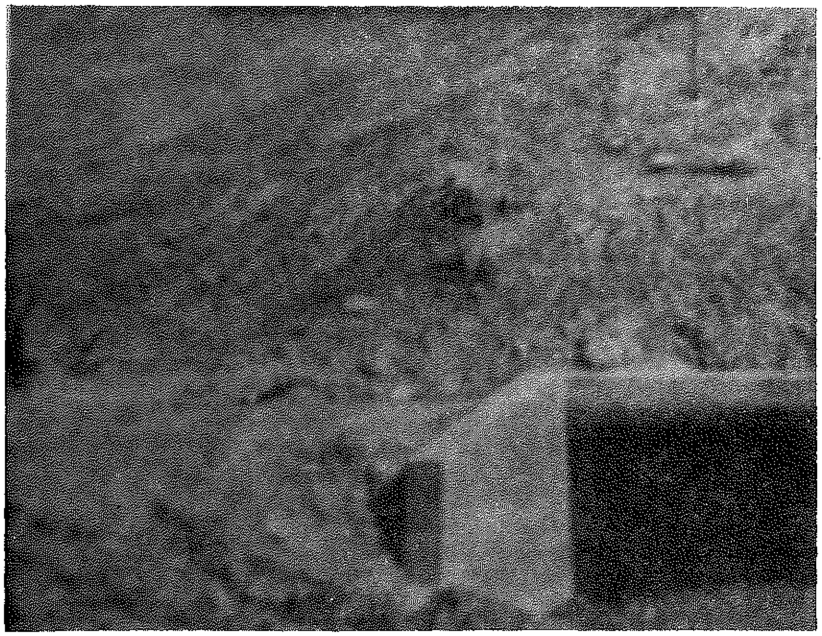


Figure 10. Mercury-Filled Settlement Gage Observation Site.

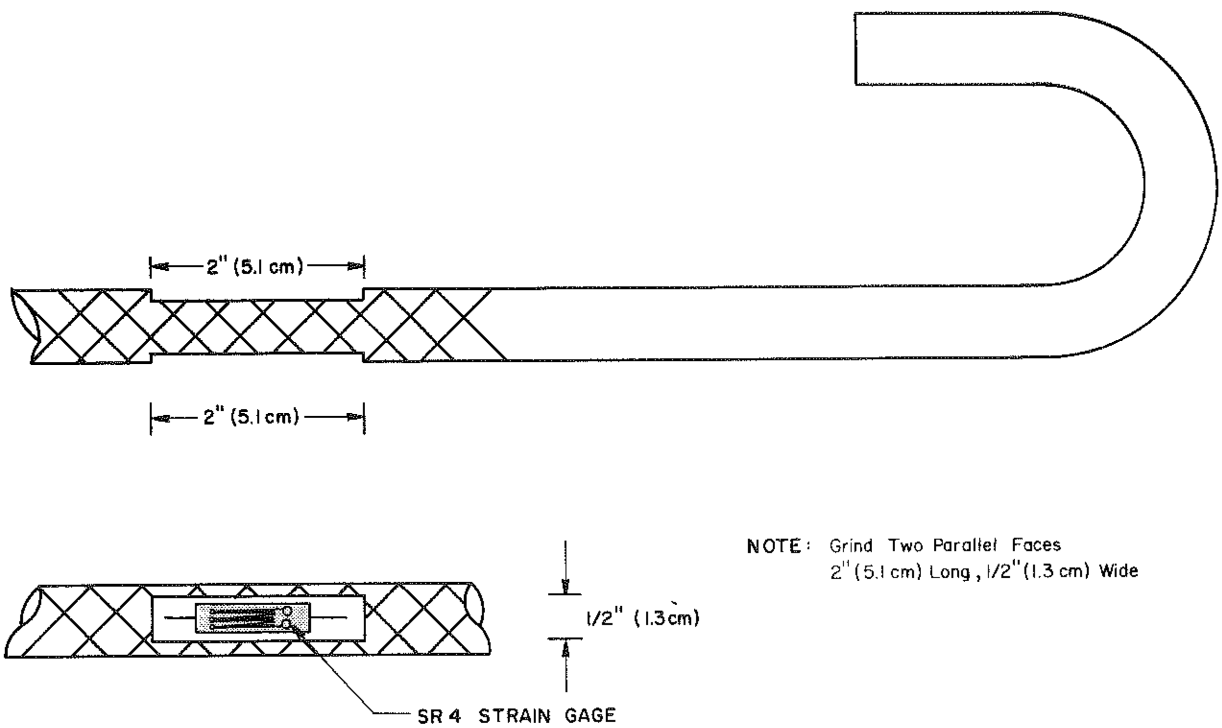


Figure 11. Steel Reinforcement Bar and Strain Gage.

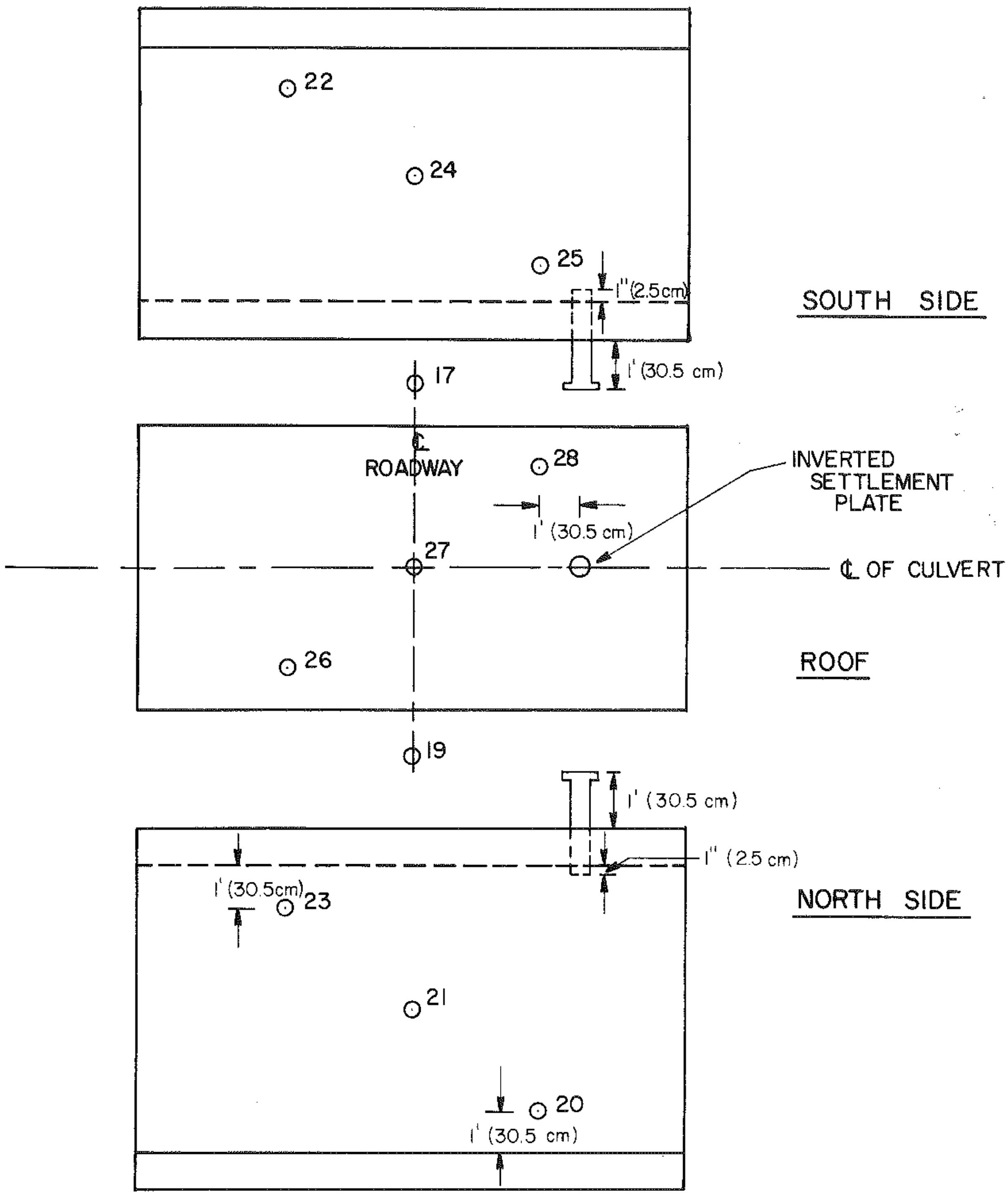


Figure 12. Carlson Meter Locations at Station 203 + 20.

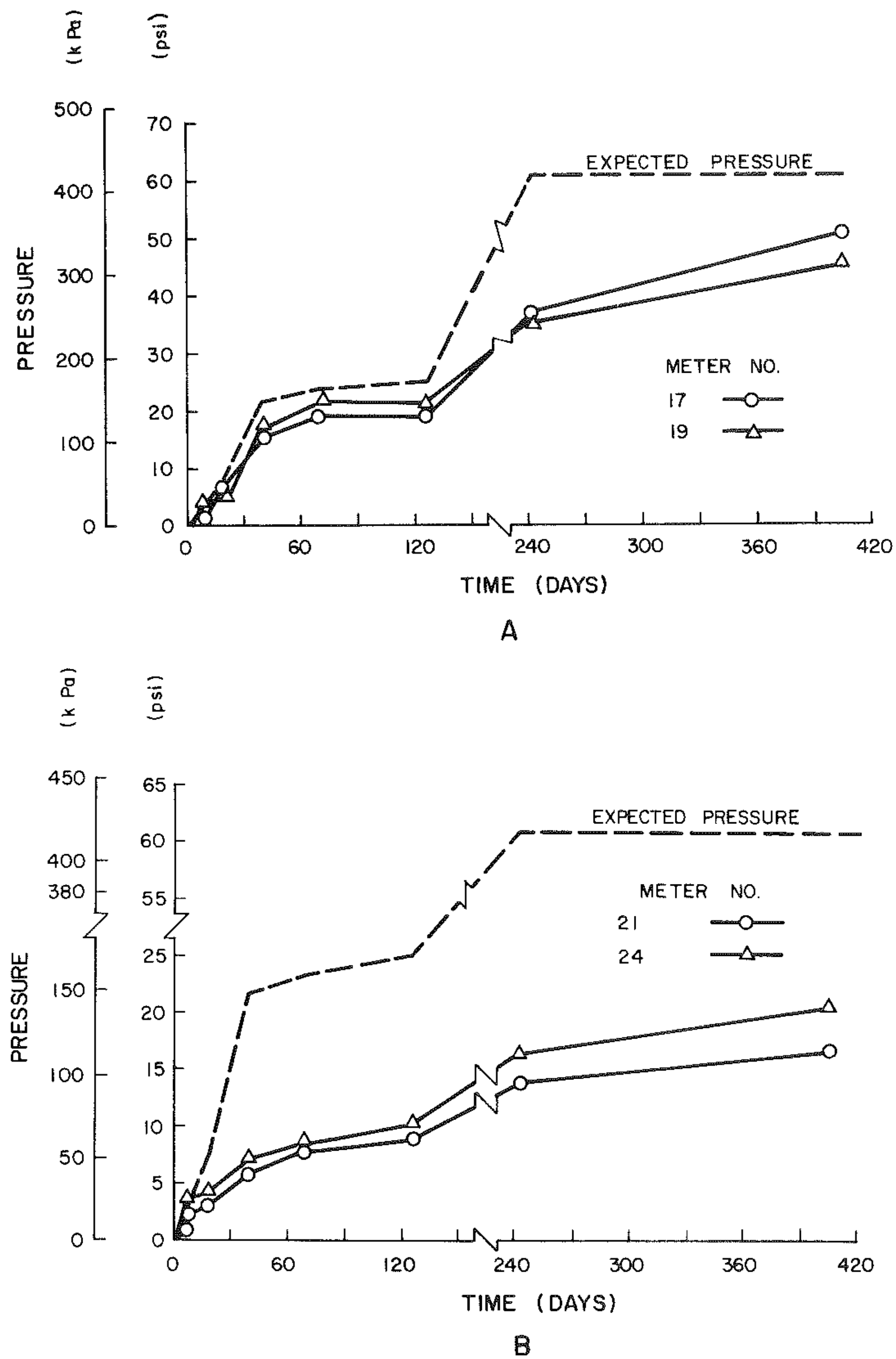


Figure 13. Load versus Time for (A) Meter Nos. 17 and 19 and (B) Meter Nos. 21 and 24.

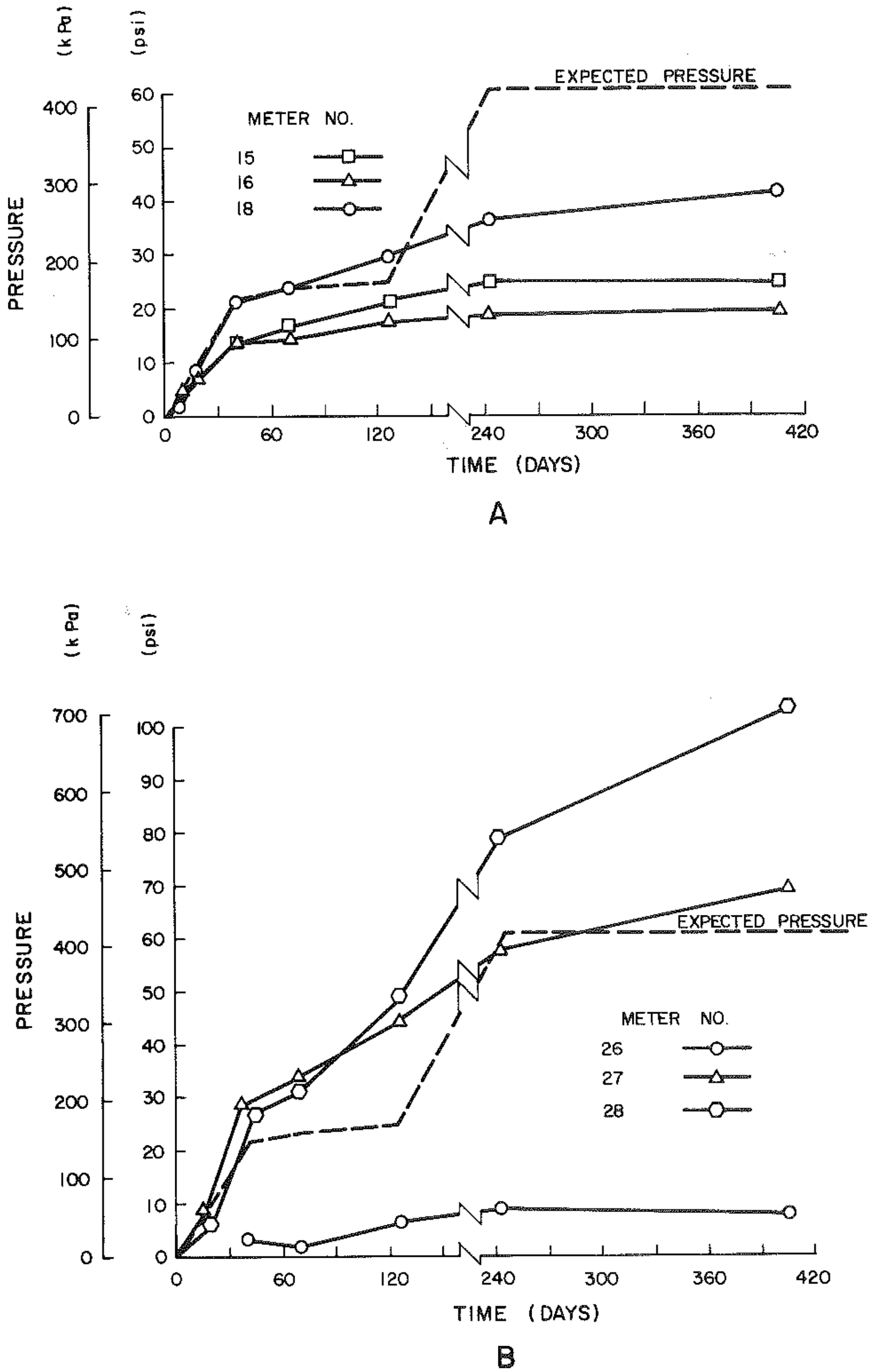
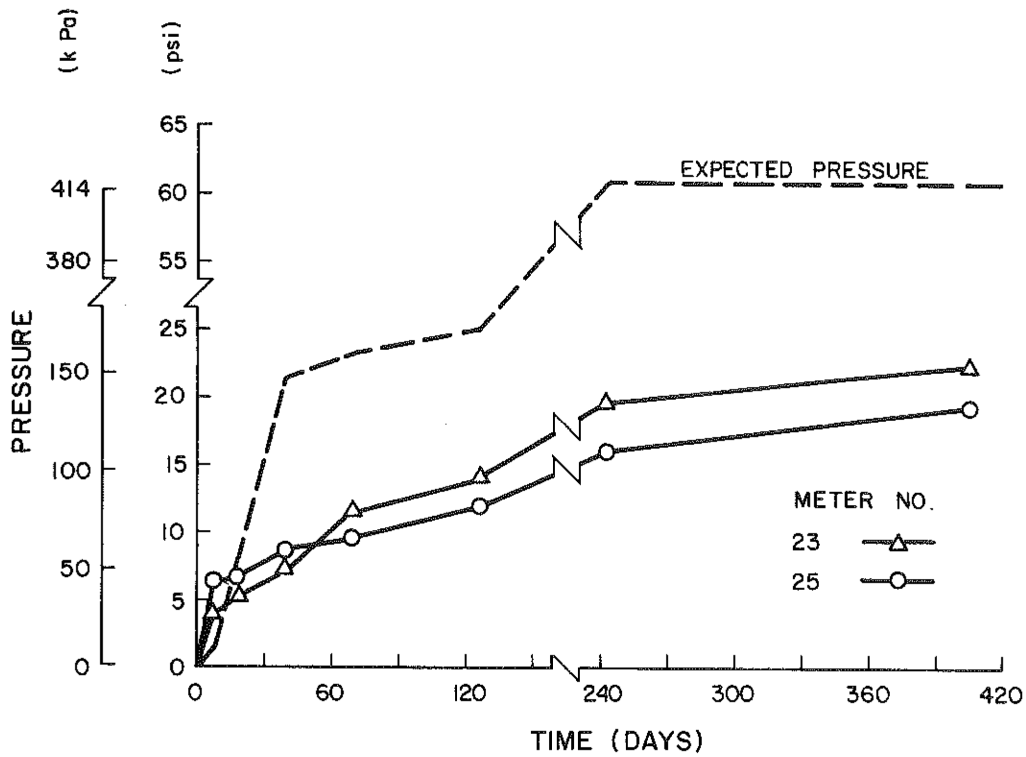
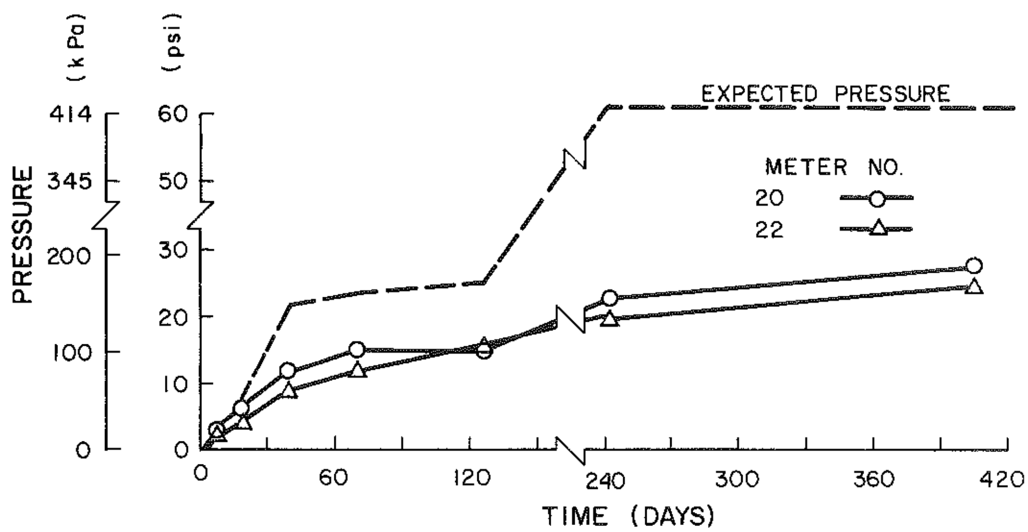


Figure 14. Load versus Time for (A) Meter Nos. 15, 16, and 18 and (B) Meter Nos. 26, 27, and 28.



A



B

Figure 15. Load versus Time for (A) Meter Nos. 23 and 25 and (B) Meter Nos. 20 and 22.

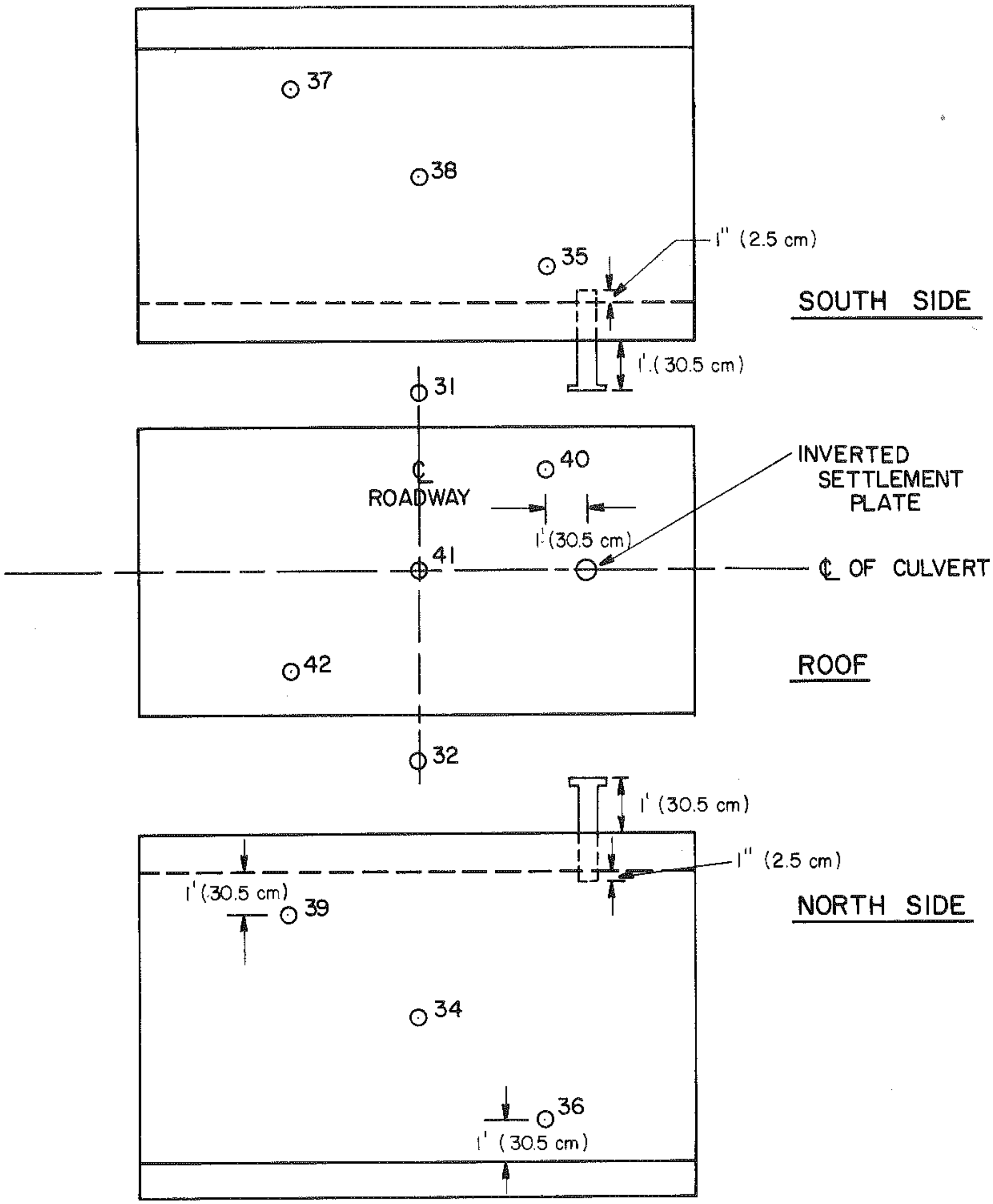


Figure 16. Carlson Meter Locations at Station 210 + 50.

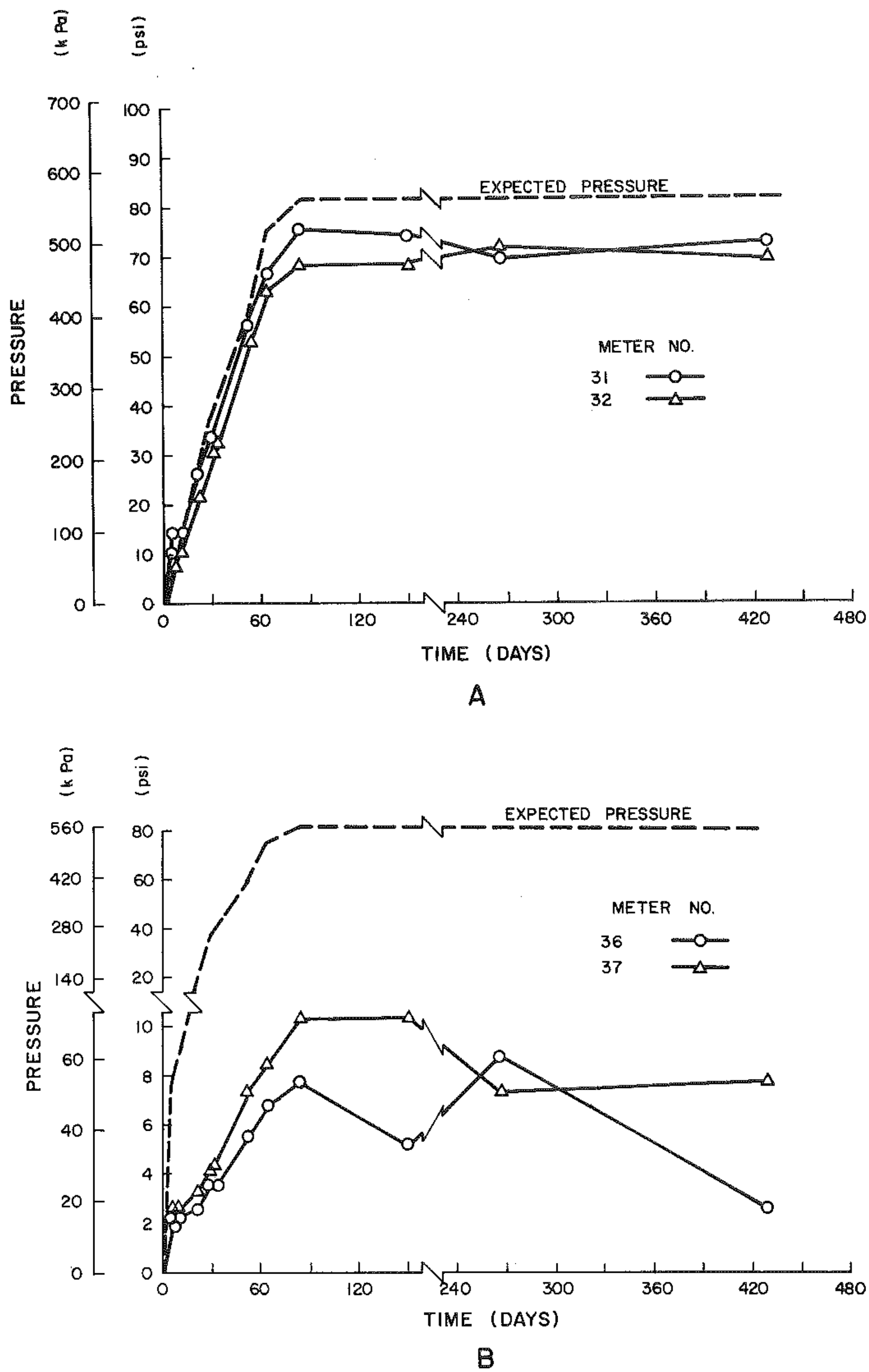


Figure 17. Load versus Time for (A) Meter Nos. 31 and 32 and (B) Meter Nos. 36 and 37.

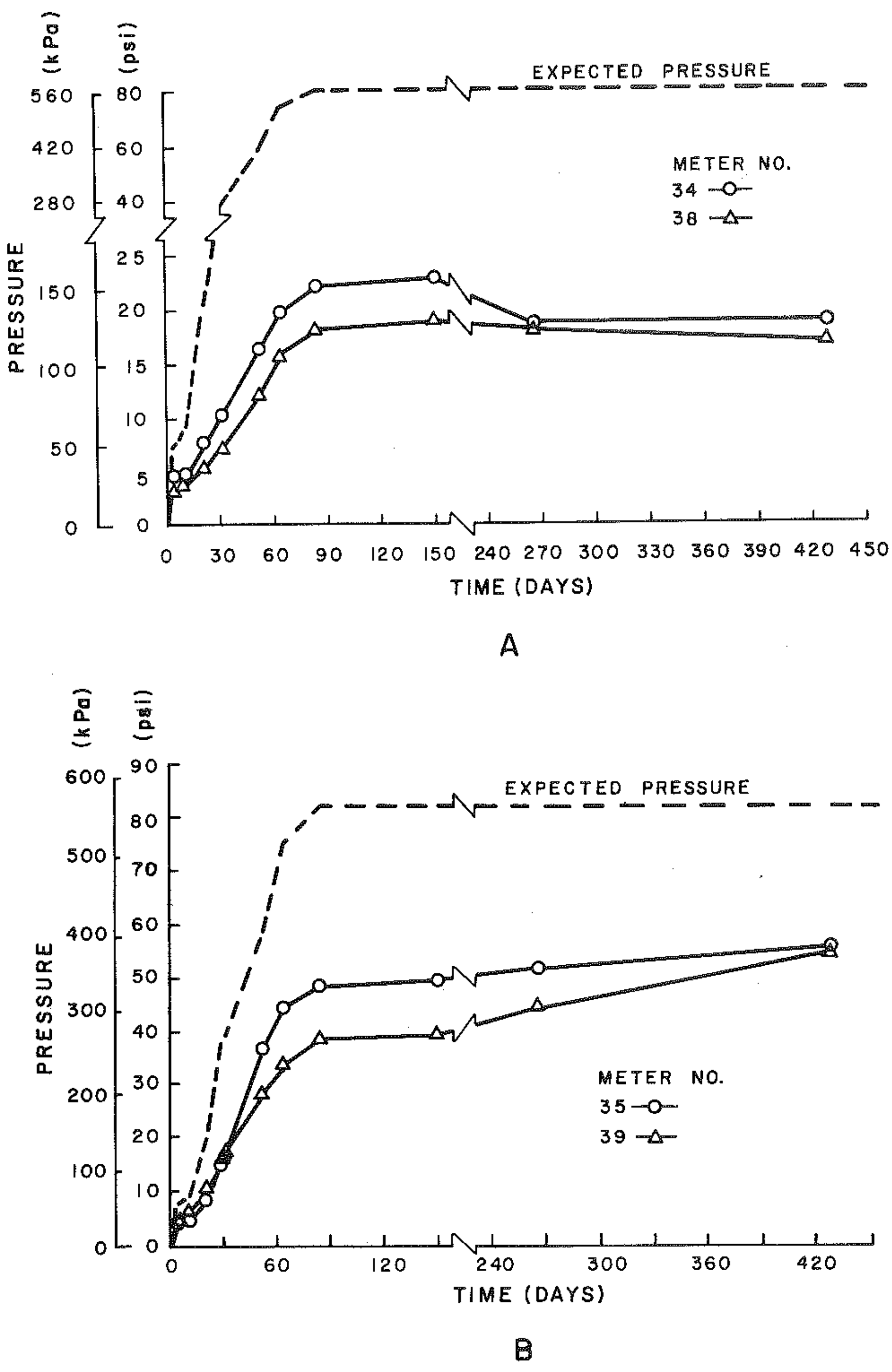


Figure 18. Load versus Time for (A) Meter Nos. 34 and 38 and (B) Meter Nos. 35 and 39.

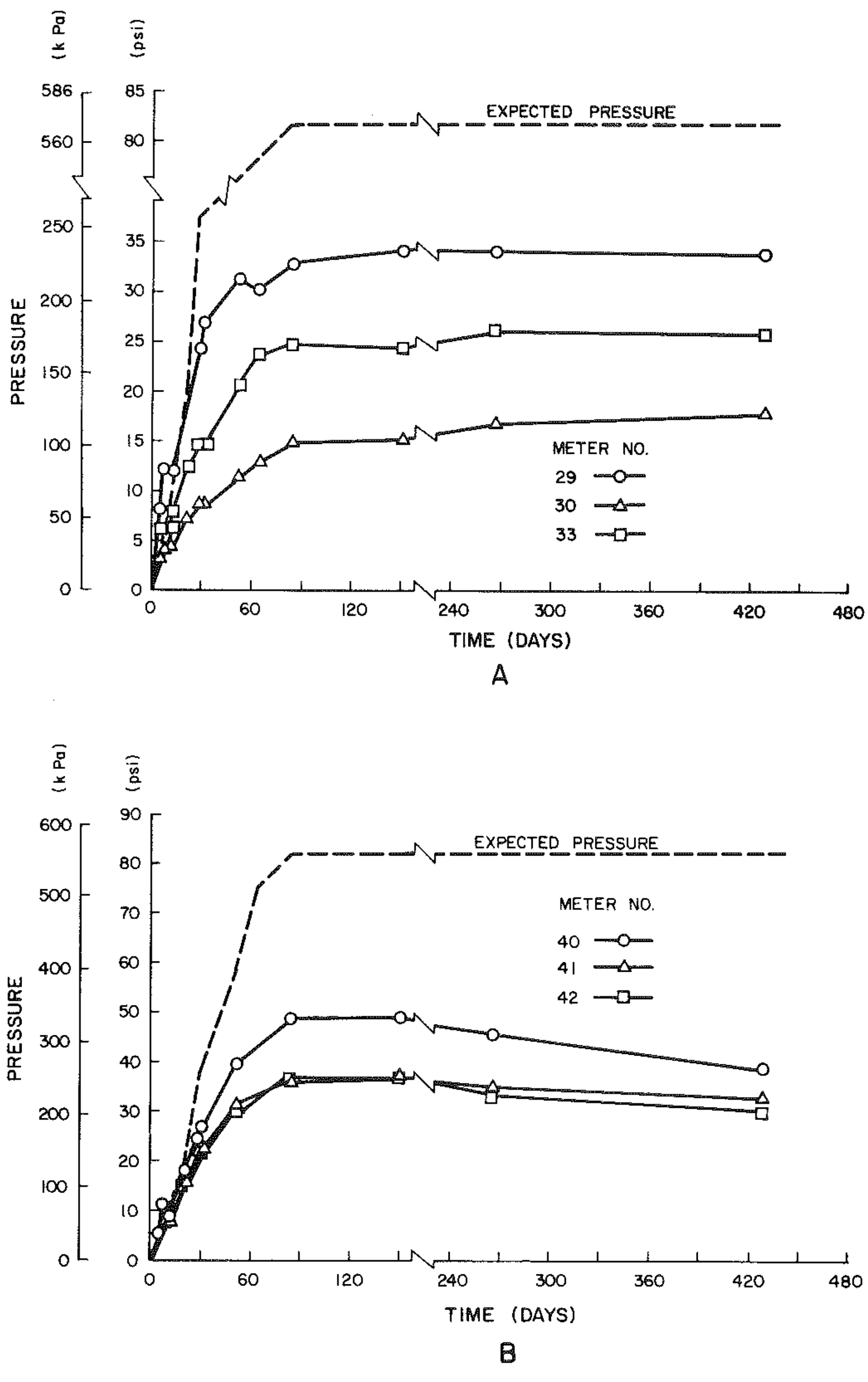


Figure 19. Load versus Time for (A) Meter Nos. 29, 30 and 33 and (B) Meter Nos. 40, 41, and 42.

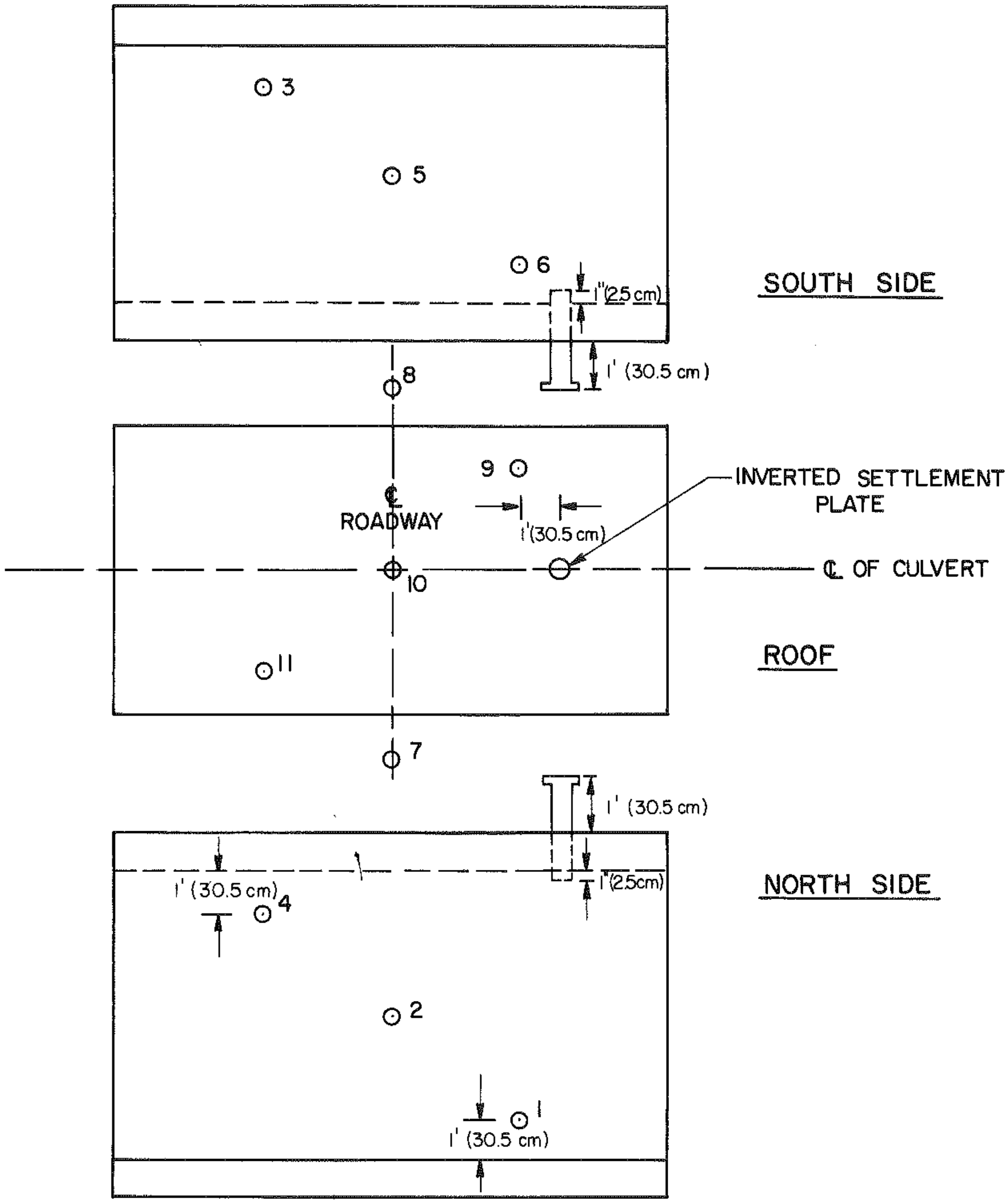


Figure 20. Carlson Meter Locations at Station 89 + 20.

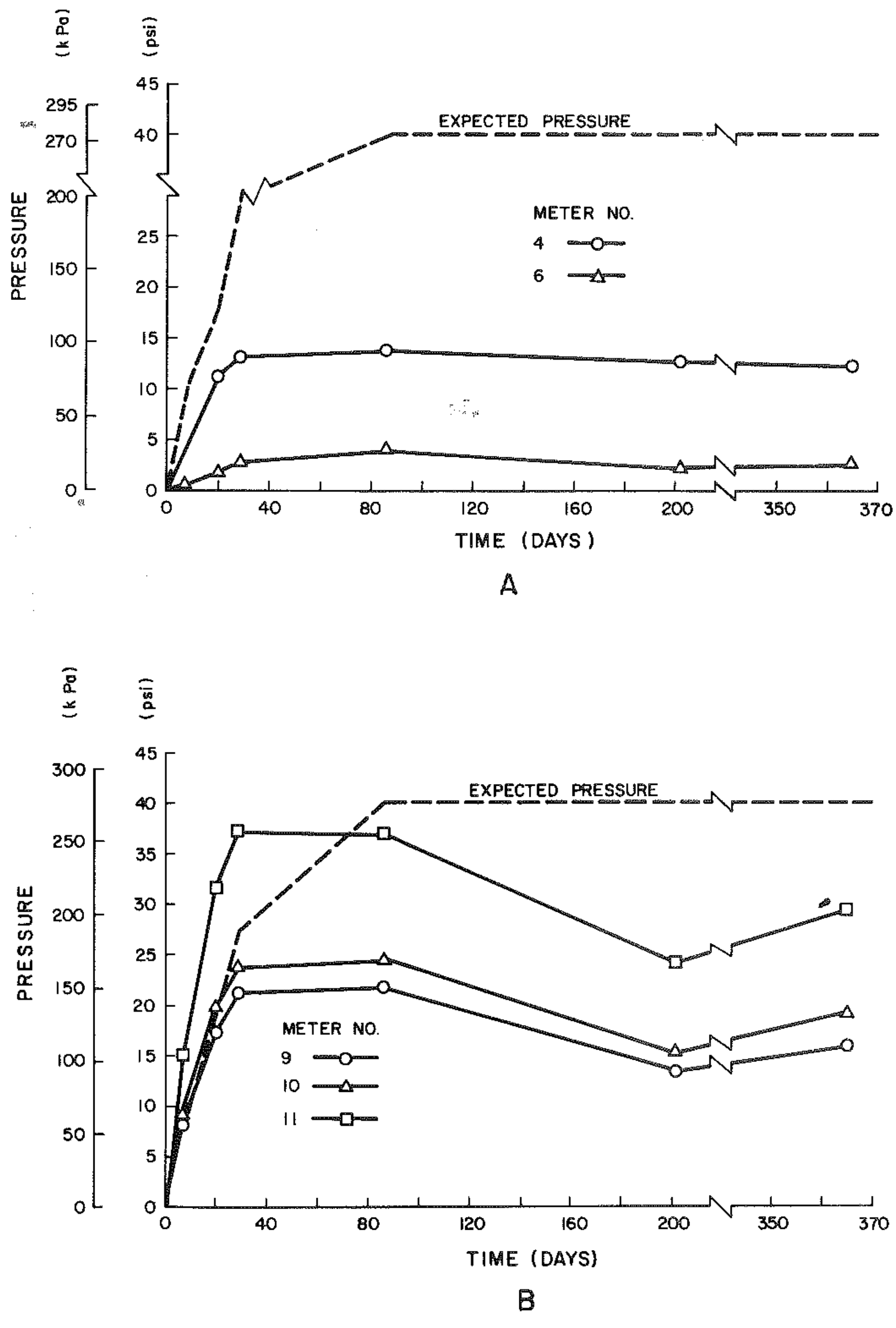


Figure 21. Load versus Time for (A) Meter Nos. 4 and 6 and (B) Meter Nos. 9, 10, and 11.

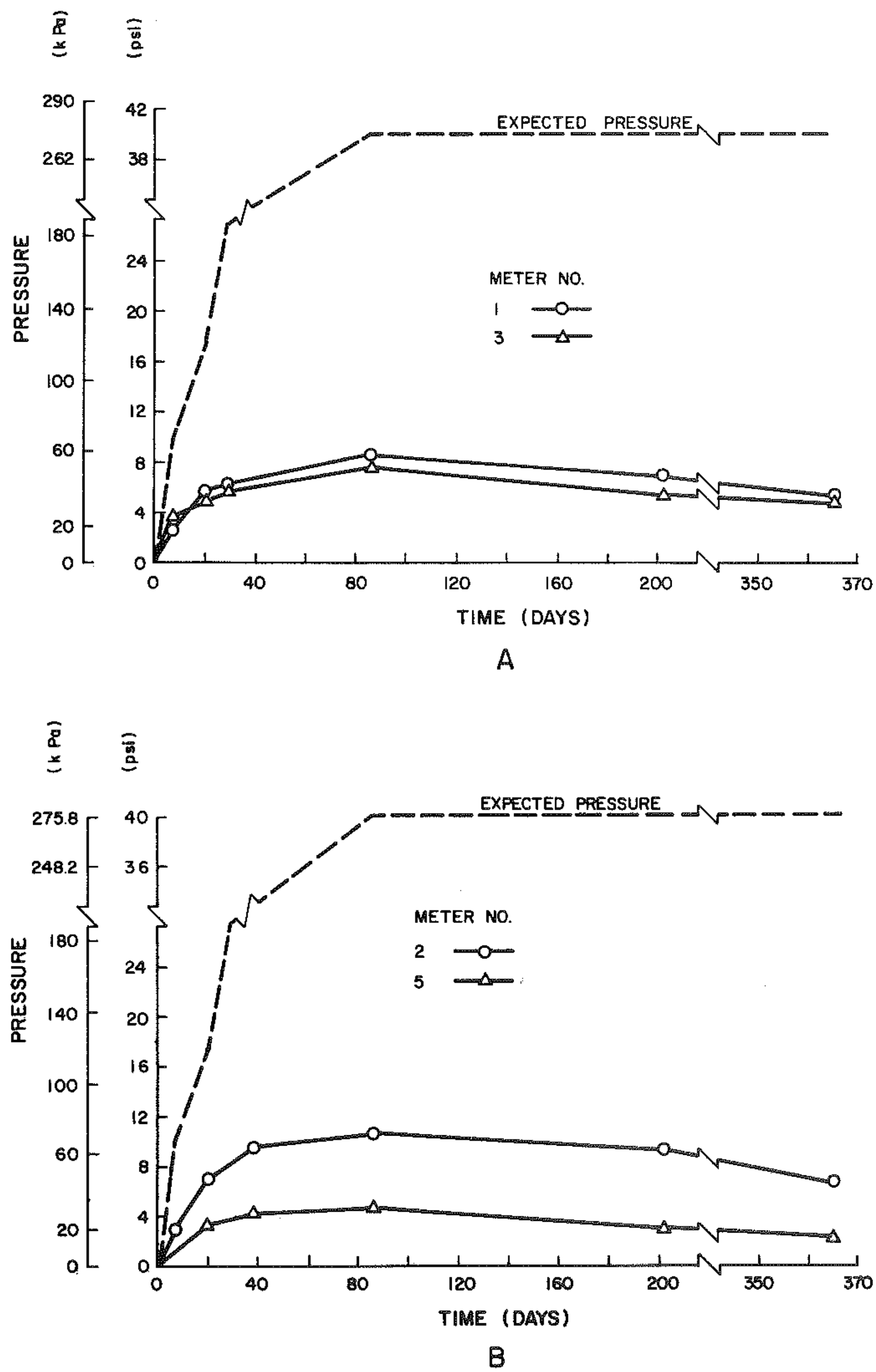
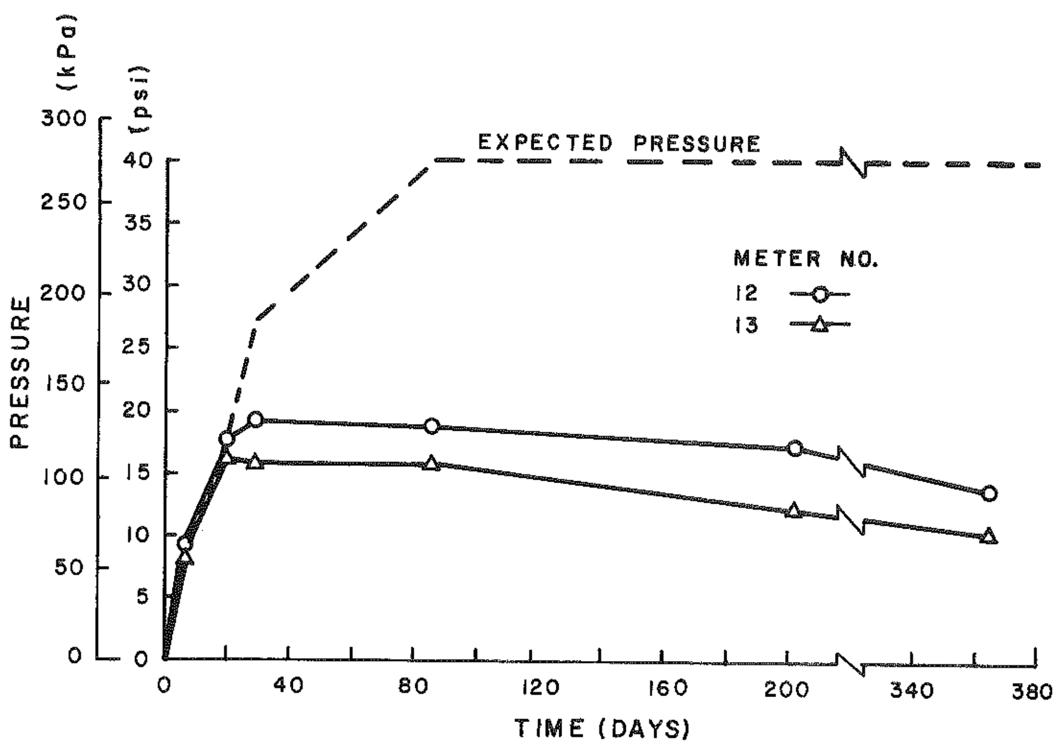


Figure 22. Load versus Time for (A) Meter Nos. 1 and 3 and (B) Meter Nos. 2 and 5.



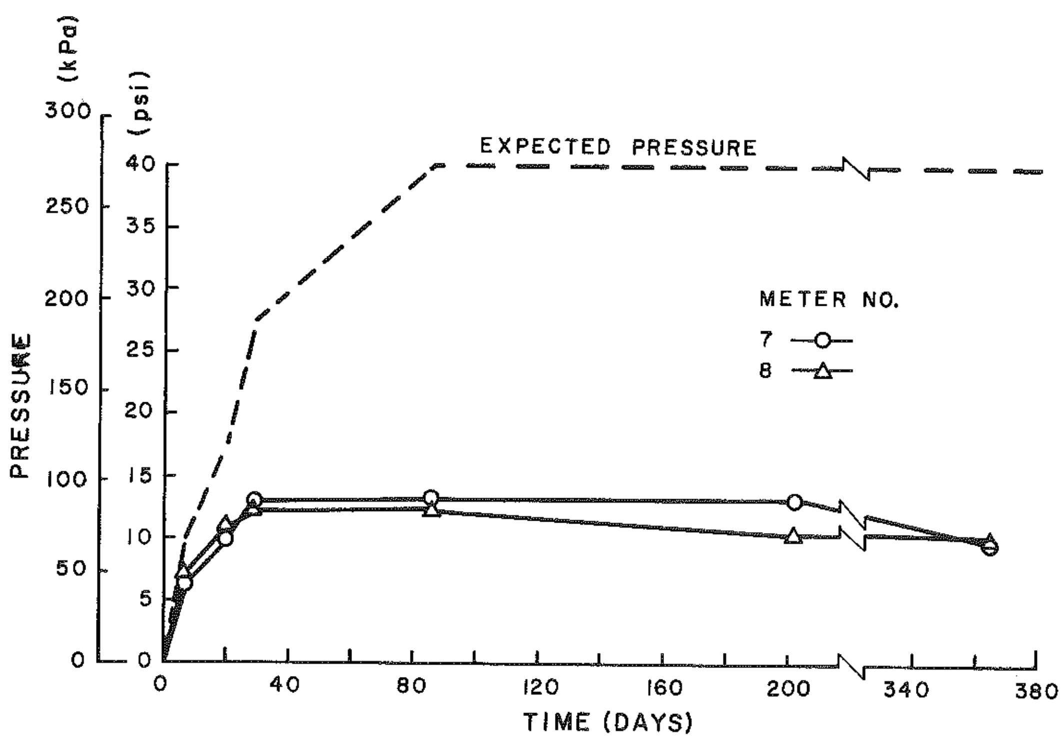


Figure 23. Load versus Time for (A) Meter Nos. 12 and 13 and (B) Meter Nos. 7 and 8.

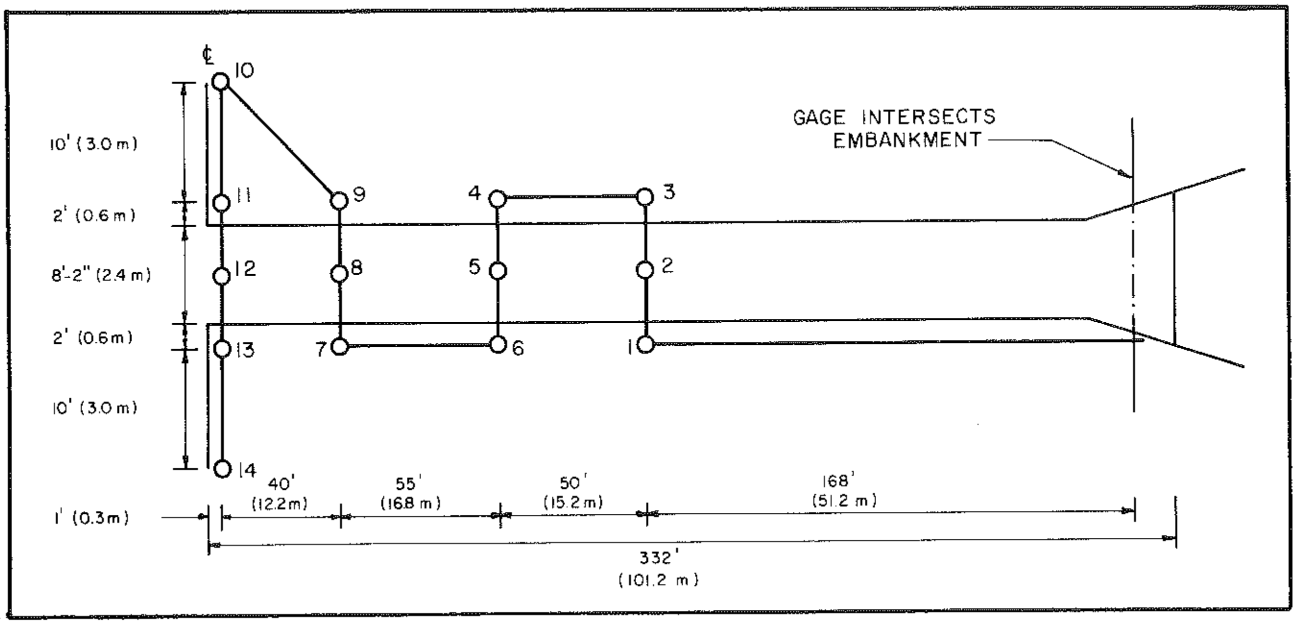


Figure 24. Plan View of Mercury-Filled Settlement Gage No. 1 at Station 210 + 50.

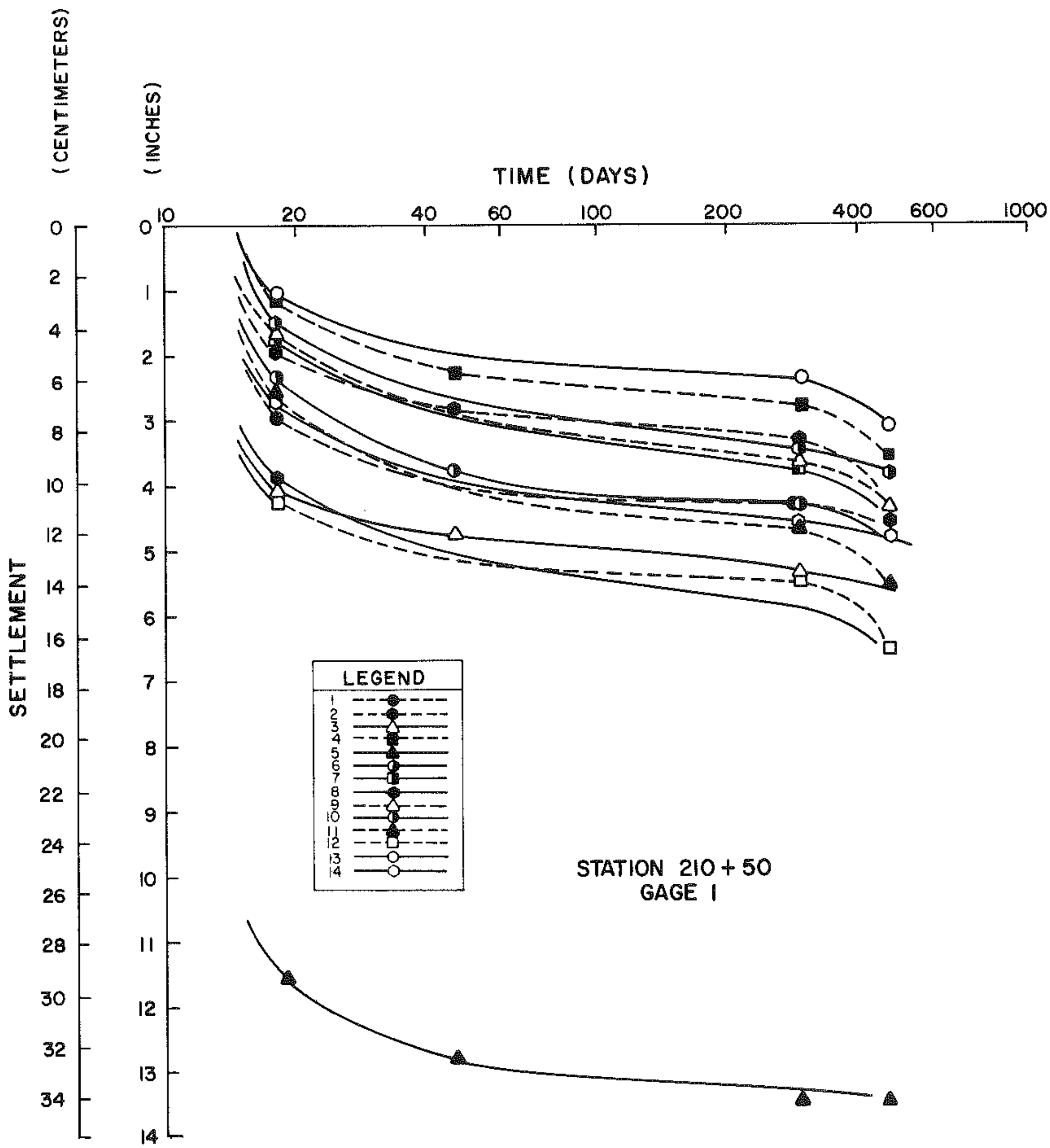


Figure 25. Settlement versus Time for Gage No. 1 at Station 210 + 50.

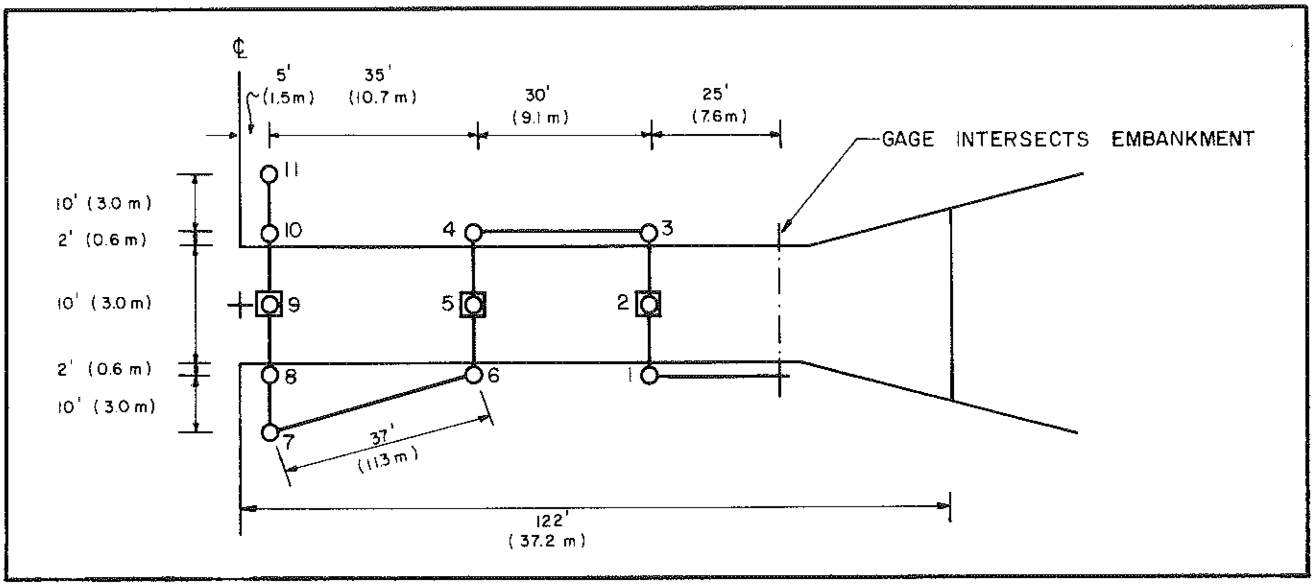


Figure 26. Plan View of Mercury-Filled Settlement Gage No. 1 at Station 89 + 20.

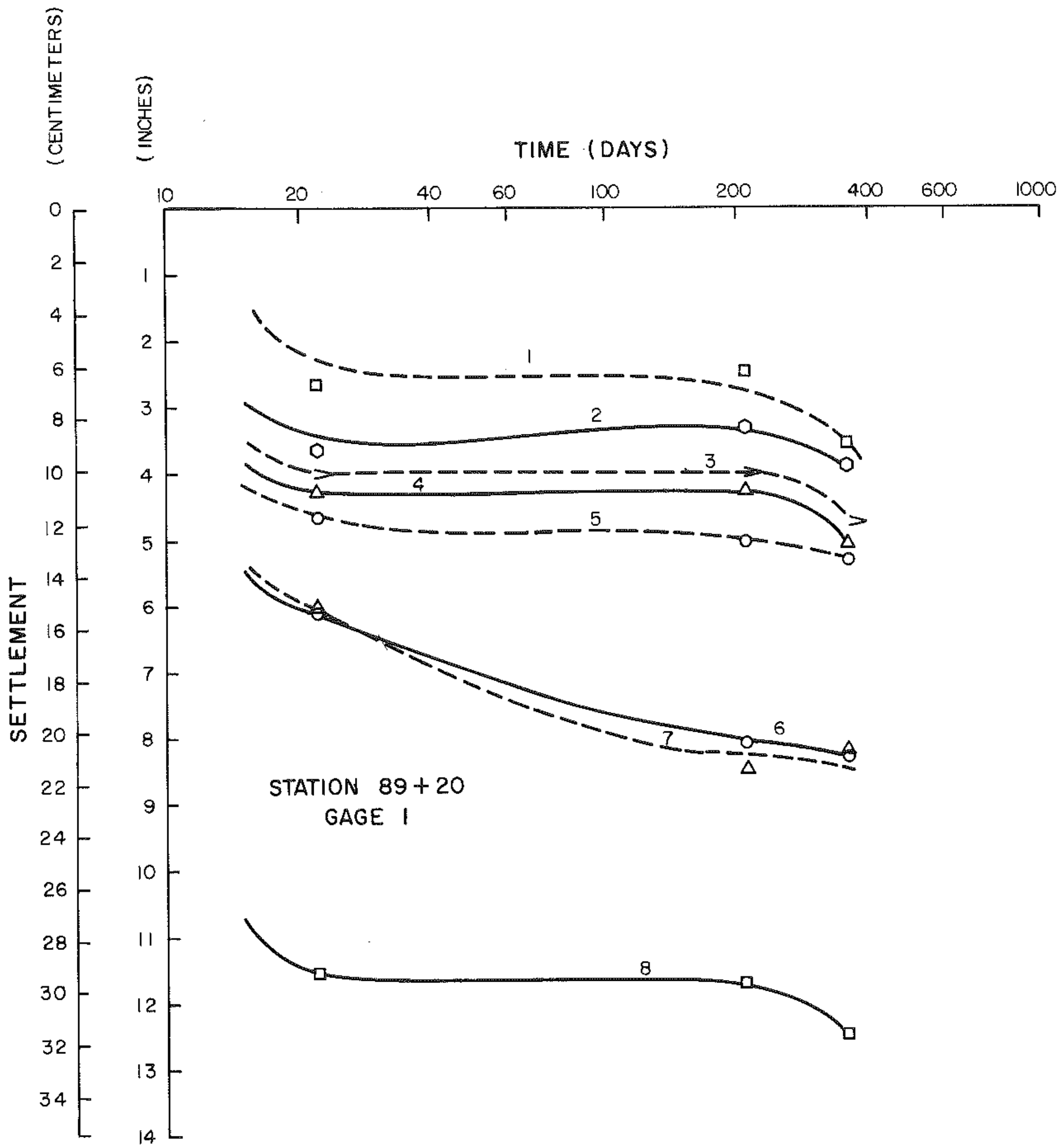


Figure 27. Settlement versus Time for Gage No. 1 at Station 89 + 20.

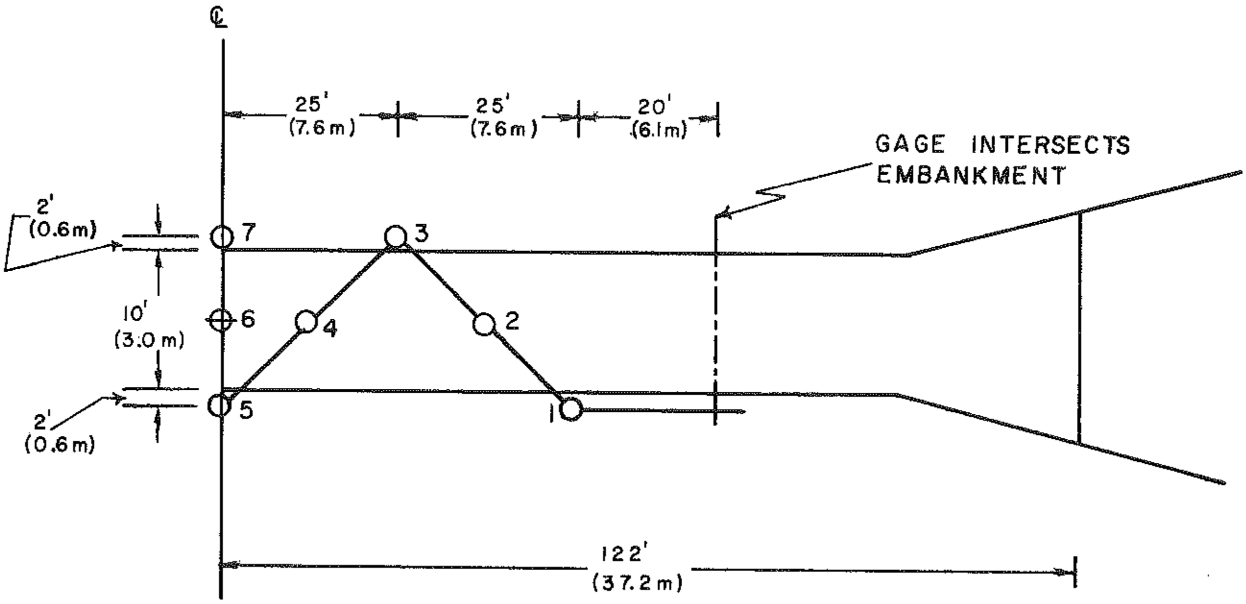


Figure 28. Plan View of Mercury-Filled Settlement Gage No. 2 at Station 89 + 20.

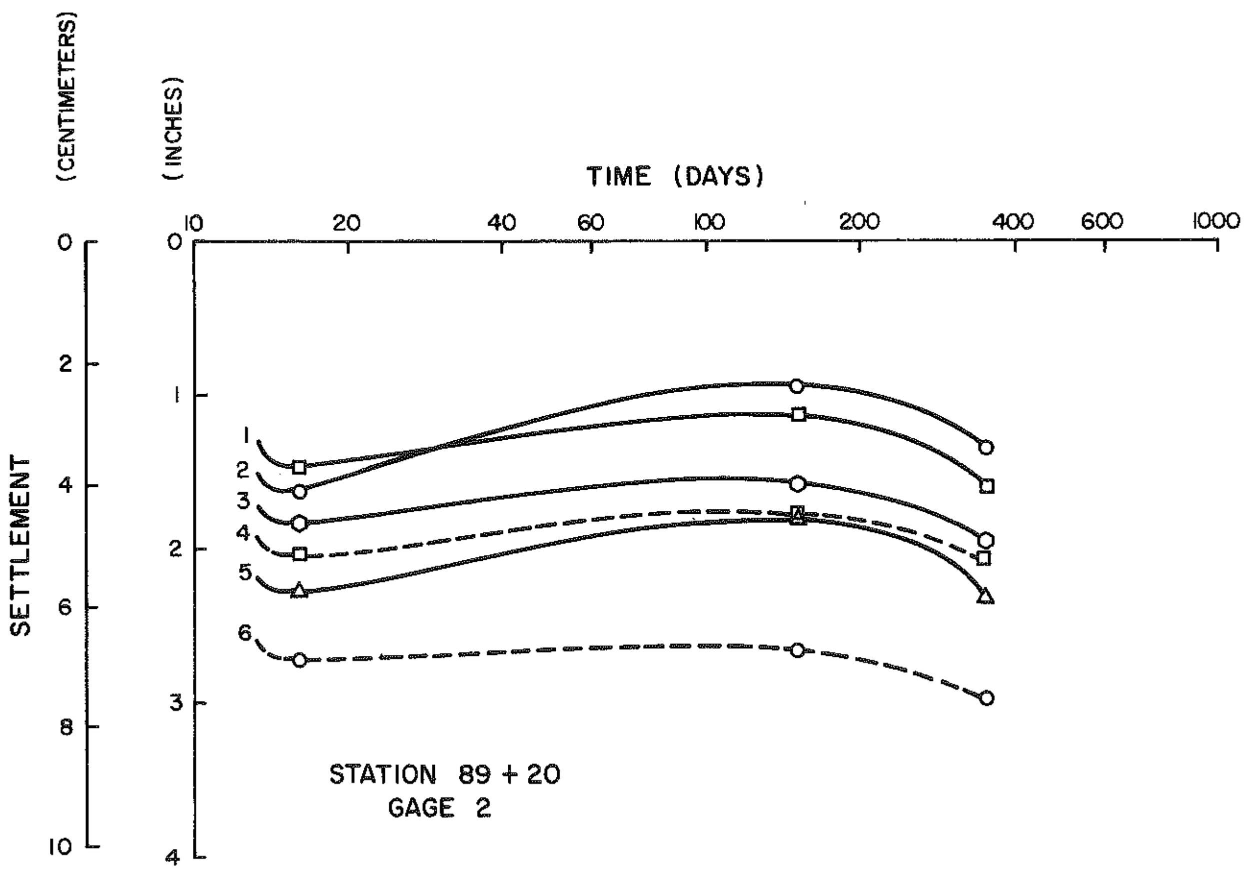


Figure 29. Settlement versus Time for Gage No. 2 at Station 89 + 20.

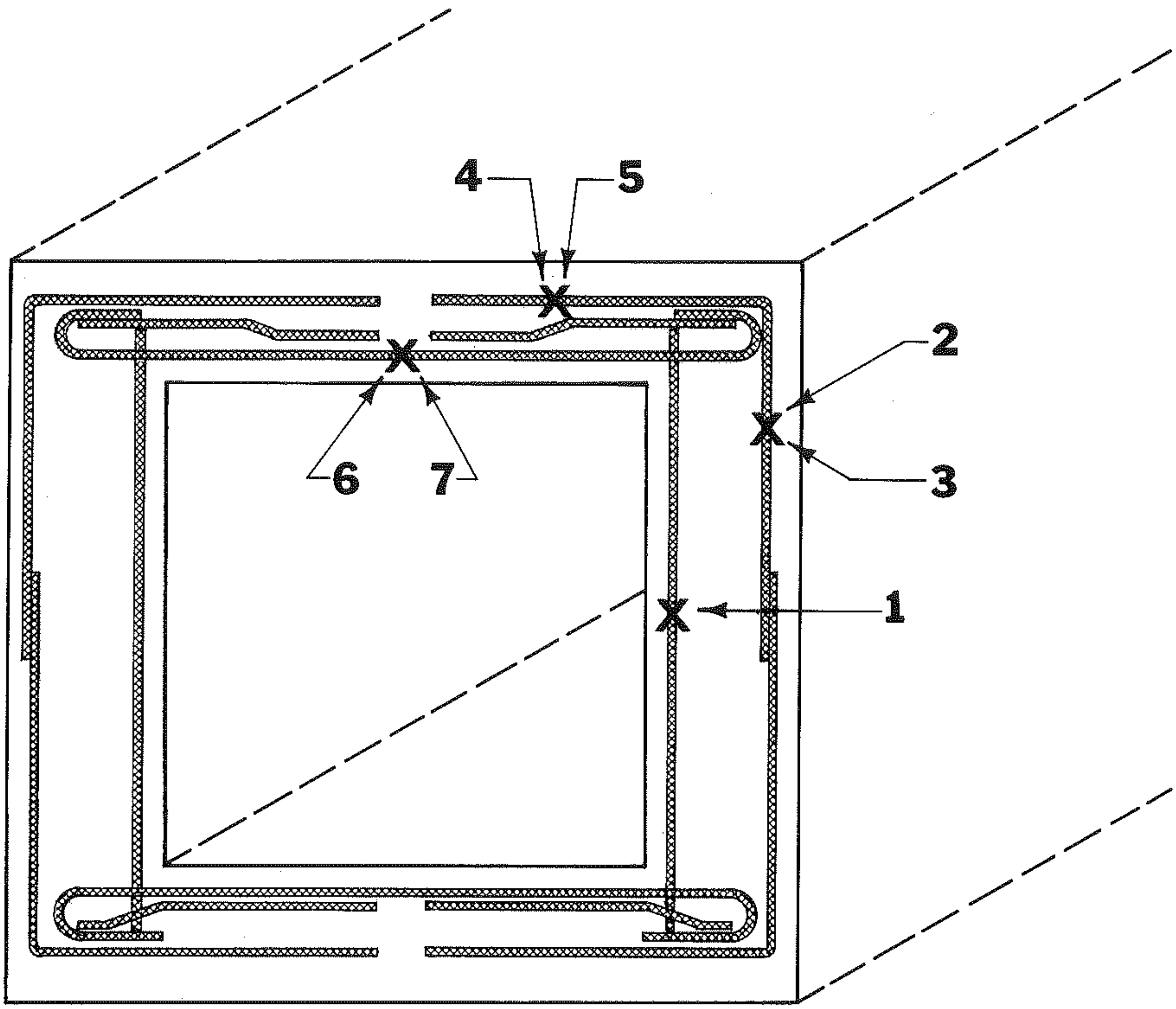


Figure 30. Strain Gage Locations.

TABLE 1

SUMMARY OF SOIL TEST DATA

	STATION 89 + 45		STATION 203 + 50				STATION 210 + 50	
	SAMPLE 1	SAMPLE 2	SAMPLE 2	SAMPLE 3	SAMPLE 4	SAMPLE 6	SAMPLE 1	SAMPLE 2
Classification								
AASHO	A-2-4	A-4	A-4	A-4	A-4	--	A-2-4	A-2-4
Unified	SM	SM	ML	SM	SM	--	SM	SM
Particle-Size Distribution								
% Sand (4.76 mm - 74μ)	76	67	32	51	65	--	77	76
% Silt (74μ - 5μ)	13	18	37	26	18	--	11	12
% Clay (< 5μ)	11	15	31	23	17	--	12	12
Liquid Limit (%)	NP	NP	27.5	19.3	NP	NP	NP	NP
Plasticity Index (%)	--	--	8.5	1.2	--	--	--	--
Unit Weight (lb/ft^3)	115.1	--	120.2	--	--	117.3	115.0	--
(kg/m^3)	1844	--	1925	--	--	1879	1842	--
Moisture Content (%)	12.6	--	14.4	14.7	9.0	11.4	6.0	--
Triaxial Tests (CIU)								
ϕ' (degrees)	38.3	--	30.5	27.4	33.5	32.7	--	40.6
c'	0	--	0	0	0	0	--	0
Comments: Material Description	Brown Sand	Brown Sand	Red Sand w/ Inorganic Silt	Pink Silty Sand	Brown Sand	Gray Sand	Red Sand	Red Sand
Comments: At Elevation (ft)	1216	1211	1234	1224	1214	1184	1265	1255
(m)	370.6	368.1	376.1	373.1	370.0	360.9	385.6	382.5
Comments: Depth Below Grade (ft)	5-7	10-12	20-22	30-32	40-42	70-72	10-12	20-22
(m)	1.5-2.1	3.0-3.7	6.1-6.7	9.1-9.8	12.2-12.8	21.3-21.9	3.0-3.7	6.1-6.7

TABLE 2

COMPARISON OF SETTLEMENTS BY PLATES AND MERCURY GAGES

		SETTLEMENT PLATES			MERCURY GAGES		
STATION	POINT	PLATE NO.	SETTLEMENT (IN.)	SETTLEMENT (CM)	MERCURY UNIT	SETTLEMENT (IN.)	SETTLEMENT (CM)
89 + 50	1	1	2.40	6.10	2	8.31	21.11
	2	2	1.68	4.27	5	12.53	31.83
	3	3	2.40	6.10	8	8.24	20.93
203 + 20	4	1	10.75	27.31			
	5	2	9.82	24.94			
	6	3	7.30	18.54			
210 + 50	7	1	4.65	11.81	2	4.85	12.32
	8	2	6.79	17.25	5	13.48	34.24
	9	3	7.11	18.06	8	6.09	15.47
	10	4	6.90	17.53	12	6.55	16.64

TABLE 3
MAXIMUM MEASURED STRAINS

SITE	GAGE NO.[b]	SECTION A	SECTION B
Station 203 + 20	1	- 714[c]	
	2	- 703	
	3	- 714	- 675
	4	3522	- 680
	5	- 15960	
	6	- 19650	
	7	17740	
Station 210 + 50	1		-550
	2	- 570	
	3	- 535	
	4	- 364	-78
	5		- 550
	6		
	7	- 213	

[a]Gage Base Lengths = 0.5 in. (1.25 cm)

[b]SR-4 FAE-50-12-S6 ET Strain Gages

[c]Plus Sign Indicates Tension, Negative Sign Indicates Compression