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16. Abstract

Field observations of the performance of an 11-ft (3.4-m) high cantilever wall on clay soil retaining a highway embankment in Houston, Texas were obtained. Measurements of wall translation and tilt, lateral earth pressure on the back face and bearing pressure on the footing were made periodically throughout a one-year period. Data acquisition began immediately after wall construction; data were obtained before, during and after placement of the select sand backfill. Measured lateral and bearing pressures were compared with calculated values obtained by published analytical methods, including the well-known Rankine and Coulomb theories of lateral earth pressure. Measured lateral pressures were integrated to obtain the resultant force on the wall. The resultant force was also computed by Culmann's graphical method for comparison with the value obtained by integration. Surcharge effects were studied by comparing the pressures and forces acting on the wall before and after placement of clay above the select sand backfill.

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FIELD MEASUREMENTS OF EARTH PRESSURE ON A CANTILEVER RETAINING WALL

by

Larry W. Schulze
Research Assistant

Harry M. Coyle
Research Engineer

and

Richard E. Bartoskewitz
Engineering Research Associate

Research Report No. 236-1

Determination of Earth Pressures for Use
in Cantilever Retaining Wall Design
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College Station, Texas

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ABSTRACT

Field observations of the performance of an 11-ft (3.4-m) high cantilever wall on clay soil retaining a highway embankment in Houston, Texas, were obtained. Measurements of wall translation and tilt, lateral earth pressure on the back face and bearing pressure on the footing were made periodically throughout a one-year period. Data acquisition began immediately after wall construction; data were obtained before, during and after placement of the select sand backfill. Measured lateral and bearing pressures were compared with calculated values obtained by published analytical methods, including the well-known Rankine and Coulomb theories of lateral earth pressure. Measured lateral pressures were integrated to obtain the resultant force on the wall. The resultant force was also computed by Culmann's graphical method for comparison with the value obtained by integration. Surcharge effects were studied by comparing the pressures and forces acting on the wall before and after placement of clay above the select sand backfill.

KEY WORDS: Backfill, Cantilever Retaining Walls, Deflection, Lateral Earth Pressure, Pressure Cells, Rotation

SUMMARY

The information presented in this report was developed during the third year of a five-year study on "Determination of Earth Pressures for Use in Cantilever Retaining Wall Design". The broad objective of this study is to develop a more economical design procedure for cantilever retaining walls.

The limited objective of the third year of this study was to measure the pressures acting on a typical cantilever retaining wall and to compare measured pressures with theoretical pressures. Twelve earth pressure cells were used to measure lateral earth pressures and bearing pressures. Measurements of wall movement were made during and after the backfilling operation. Data are presented in this report for measured pressures and movements obtained over a period of 385 days.

The total thrust of the measured lateral earth pressures was approximately 3.5 times greater than the thrust predicted from a Culmann graphical solution. The measured bearing pressures compared reasonably well with calculated bearing pressures. The wall movement data indicated that the wall moved toward the backfill during sand backfilling and away from the backfill during clay backfilling.

IMPLEMENTATION STATEMENT

Research Report 236-1 is a technical progress report which presents the results of the work accomplished during the third year of a five-year study on "Determination of Earth Pressures for Use in Cantilever Retaining Wall Design". Twelve Terra Tec pressure cells were installed in a standard cantilever retaining wall. Measurements of earth pressure and wall movements were made and will be continued during the fourth year of this study. Implementation of the results obtained thus far is not recommended because of the need to obtain additional field data in order that the long-term performance of the wall may be studied.

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INTRODUCTION

Present Status of the Question

At present, earth pressure calculations for retaining wall design are made according to the earth pressure theories proposed by Coulomb in 1776 and Rankine in 1857 (1). Both theories use limiting equilibrium mechanics with many simplifying assumptions, and large factors of safety are applied to account for uncertainties. According to these theories the lateral earth pressure is equal to the vertical stress at a point multiplied by a coefficient of lateral earth pressure. The coefficient is a function of wall movement, engineering properties of the backfill, and geometry of the wall and backfill.

The basic assumptions for the earth pressure theory proposed by C. A. Coulomb (1) are:

1. The soil is isotropic and homogeneous, and has the engineering properties of cohesion and internal shearing resistance,
2. The rupture surface is a plane,
3. The friction forces are distributed uniformly along the rupture surface,
4. The failure wedge is a rigid body,
5. Friction forces are developed between the wall and the soil, and

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Numbers in parentheses refer to the references listed in Appendix I.

6. Failure is a two-dimensional problem.

Rankine considered the soil to be in a state of plastic equilibrium and used essentially the same assumptions as Coulomb, except that wall friction was neglected, which greatly simplifies the problem.

Although many advances have been made during this century in various areas of soil mechanics and geotechnical engineering, earth pressure problems have received relatively little attention, especially where cantilever retaining walls are concerned. The research which has been conducted on problems related to earth pressure has served to illustrate the inaccuracies associated with the Rankine and Coulomb theories. These inaccuracies are to be expected because of the many simplifying assumptions that were made in order to solve the problem with the computational capability available at the time. Packshaw (9) recognized the error in Rankine's assumption of zero wall friction by stating that most walls are not frictionless, with the result that the friction between the wall and soil inclines the resultant pressure away from the normal to the wall. Consequently the direction of the resultant pressure bears no relation to the inclination of the ground surface. In 1920 Terzaghi (15) criticized the assumption that the soil is isotropic and acts as a homogeneous mass. Terzaghi stated: "The fundamental error was introduced by Coulomb, who purposely ignored the fact that sand consists of individual grains, and who dealt with the sand as if it were a homogeneous mass with certain mechanical properties." According to Bowles (1), the principal deficiencies in the Coulomb theory are in the assumptions of an ideal soil and a plane rupture surface.

Background on Present Research Program

A five-year research study was begun in 1977 at Texas A&M University to measure field lateral earth pressures on full-scale cantilever retaining walls with strip footings. The first year was devoted primarily to preparations for the field test. A literature review was conducted to determine the current state-of-the-art of cantilever retaining wall design, and to obtain information concerning recent research and new design concepts that have been developed (12). At the same time, a search was conducted to locate possible test sites where cantilever retaining walls were to be constructed.

During the second year of the study a standard cantilever retaining wall on a spread footing foundation with a protruding key was selected for instrumentation. Also during the second year the footing and key were instrumented with seven Terra Tec pneumatic pressure cells.

Due to extreme construction delays, the stem of the above mentioned retaining wall was not instrumented until the third year of the project. The stem was instrumented with five Terra Tec pneumatic pressure cells. In addition to the pressure cells, provisions were made to measure tilt, rotation, and lateral displacements of the wall. The collection of field data also was begun during the third year. During the present year (4th year) additional field data were collected, and the analysis of the data was begun.

Objective of the Study

The broad objectives of this research are: (1) to make long-term field measurements on full-scale cantilever retaining walls, and (2) to use these data to verify or modify present design criteria which are based on the classical earth pressure theories. The specific objectives of the study which are covered in this report are:

1. Obtain undisturbed soil samples at the wall site and perform laboratory tests to evaluate the engineering properties of the soil,
2. Measure and record earth pressures during backfilling operations and periodically thereafter for the duration of the study,
3. Measure and record wall displacements during backfilling operations and periodically thereafter for the duration of the study,
4. Obtain samples of the backfill soil and perform laboratory tests to determine the engineering properties, and
5. Compare observed measurements to those predicted by the classical earth pressure theories.

REVIEW OF PREVIOUS RESEARCH

Large Scale Retaining Wall Tests

The first major advancement in understanding the nature of earth pressure was made by Karl Terzaghi in 1934 (16) after conducting several large retaining wall tests with dry sand backfill. From these tests Terzaghi determined that the unit weight of the backfill and the amount of wall movement greatly affect the lateral pressure on a retaining wall. He also determined that the soil friction angle and the wall friction vary during wall movement and that the resultant pressure acts at a point higher than the lower third point as proposed by Coulomb. In 1936 Terzaghi (17) pointed out additional fallacies and limitations of Rankine's and Coulomb's earth pressure theories, particularly in relation to the assumption of a hydrostatic pressure distribution. Terzaghi went so far as to state that the use of Rankine's theory should be discontinued.

Modern construction methods of compacting the backfill with heavy machinery and various types of compaction equipment have been investigated by several researchers (3, 7, 13, 14) to determine the extent to which these variables affect the earth pressure on a retaining wall. Each of these studies provided evidence that compaction of backfill behind a rigid structure may produce a lateral earth pressure which exceeds that predicted by the classical earth pressure theories. The present level of knowledge about the mechanics of earth pressure indicates that the pressure produced by compaction depends on at least three factors: (1) the engineering properties of the soil,

(2) the dimensions of and the pressures produced by the compaction device, and (3) the deformation of the structure retaining the soil. Two new design procedures based on the compaction of the backfill behind retaining walls have been proposed by Broms (2) and Ingold (7). A more detailed discussion on the affects of compaction on lateral earth pressure is given later in this report.

In 1970 Sims and others (13) reported results concerning an investigation of the lateral pressure acting on a large retaining wall. The wall was instrumented with strain gages, and pressure cells were installed in the backfill to measure vertical and lateral earth pressures. The wall was backfilled with conditioned hopper ash. From these measurements, it was concluded that the lateral earth pressure was greater than expected, and the pressure distribution was different from that used in the design of the wall, which was based on the classical earth pressure theories.

In 1972 and 1973 researchers at Texas A&M University reported findings concerning earth pressure and wall movement measurements that were made on a precast panel retaining wall (11), and on a cantilever retaining wall (4) that was constructed on a footing supported by H-piles. In both studies the pressure measurements in the upper part of the wall agreed reasonably well with theoretical active pressures, but the measured pressures in the lower part of the wall were considerably higher than the theoretical pressures. The results of this study indicated that at rest lateral pressures are exerted in the lower part of retaining walls.

Aspects of Cantilever Retaining Walls That Have Not Been Studied

An extensive literature survey has revealed several aspects of cantilever retaining wall analysis, design, and construction that have received little, if any, consideration. Since many cantilever retaining walls rest on spread footing foundations, it would seem logical that the footings should be instrumented to measure bearing pressures. However, reports of any research that may have been conducted in this area were not found in the literature. Also, some of the modern retaining walls constructed on spread footing foundations have a key protruding from the base of the footing. It is logical to assume that the use of a key would help prevent sliding, especially when the key is very deep or embedded in stiff soil or rock. However, no documented studies have been found which relate to design criteria for keys.

Another interesting aspect of cantilever retaining walls that most researchers have overlooked is the fact that a triangular shaped "dead zone" exists above the footing and next to the back of the stem. Cantilever retaining walls are constructed on a heavy footing which extends beneath the backfill, as shown in Fig. 1. If such a wall yields by tilting or sliding until the backfill starts to fail, one part of the backfill adjoining the wall, represented by triangle bcd (the "dead zone"), remains practically undisturbed and acts as if it were part of the wall (18). Within the wedge shaped zone, abde, the soil is in the active Rankine state and no shearing stresses act along the vertical plane fd. The earth pressure against this plane is identical with that against a smooth vertical wall. In most cases retaining walls are instrumented with pressure cells mounted flush with the back face of the stem (19). Hence the pressures measured against the wall may be influenced by the dead zone and may not be identical to the pressures within the failure wedge. The measured earth pressure, if different than those proposed by earth pressure theories, may not invalidate the theories, but it could necessitate modifications in present design criteria.

Important Considerations in Retaining Wall Design

The design of retaining walls requires the careful consideration of several important variables. These variables include:

1. Engineering properties of the backfill and the subsurface foundation material,
2. Lateral earth pressures exerted on the wall,
3. Compaction of the backfill,
4. Stability of the wall against overturning, sliding, and settlement, and
5. Adequate provisions for drainage.

Each of these variables are considered separately in the design and analysis of retaining walls. However, it is important that the designer remember that all of the variables are interrelated and each influences the other when the construction phase is completed. A discussion of each of these variables follows.

Engineering properties of the backfill and subsurface material. - The loads imposed on a retaining wall depend on the engineering properties of the backfill. The unit weight of the backfill material is important because it directly affects the magnitude of the horizontal and vertical pressures within the retained mass of soil. Retaining walls are usually designed on the assumption of the active state of stress, which implies adequate yielding of the wall to develop the full shear strength of the backfill material. Thus, it is important to have an accurate measure of the shear strength of the backfill material. The stability of the wall is a function of the bearing capacity, compressibility, and shear strength of the subsurface material. Hence an extensive program of sampling and testing is necessary to determine those engineering properties of the subsurface soil necessary in evaluating the stability of the wall.

Lateral earth pressures exerted on the wall. - An important consideration in retaining wall design is the state of stress of the material behind the wall. The earth pressure which acts against an unyielding surface such as a basement wall or any type of rigid structure is referred to as the natural or at rest earth pressure. In situations where rigid retaining walls are founded on bed rock, the retained earth will impose pressures related in magnitude to the at rest values. The situation most commonly found in retaining wall design involves placement of retaining wall foundations on a soil capable of deforming slightly under load. This deformation allows the retained soil to extend laterally a small distance which may be sufficient to mobilize its full shearing resistance. When sufficient deformation has taken place, the soil pressure on the wall will reach its minimum value and is termed the active state of stress. Most retaining walls are designed to resist active earth pressure rather than the higher at rest pressure. The soil placed in front of the retaining wall undergoes compression as the retaining wall translates in the direction of applied loading. The resisting pressure of the soil will increasingly exceed the value of the at rest earth pressure. Ultimately, if the lateral movement of the wall is great enough, the full shearing resistance of the soil will be developed and the soil will have reached its passive state of stress. Deformation of the soil in excess of this condition will not significantly affect the magnitude of the resistive force.

One of the conditions of the Coulomb theory is that the wall tilts or moves forward a certain distance to develop the active case and that such movements take place after the completion of the backfilling. Since most retaining walls are designed on the basis that the soil will reach the active state of stress, it is important that the wall be designed to undergo the necessary movement. In 1936 Terzaghi's classical tests (17) established that the change in wall pressure from at rest to active or to passive is a function of wall movement. He showed that in compact cohesionless materials an average movement of 0.0005 times the wall height reduces the total horizontal load to the active value and that the pressure distribution becomes hydrostatic if the top of the wall moves 0.005 times its height. For loose materials the necessary movement is greater. According to Sims, et al. (13) there are several considerations that are often ignored in retaining wall design. One consideration is that if large factors of safety against overturning and sliding are provided, then wall movements will not be large enough to develop the full shear strength of the backfill. Other considerations are that if the wall stem is overdesigned or the wall is not braced during backfilling, pressure greater than the active is likely to occur.

Compaction of backfill. - Theoretical studies of lateral earth pressure have been concerned with the pressure developed by undisturbed soil or by soil dumped or loosely placed against a structure. In some situations, however, a loose backfill will gradually settle under its own weight or from loads imposed upon it. In order to eliminate this settlement, many designers require that the retaining wall backfill be compacted in the same way as other critical fills. This compaction can develop friction between the soil and the structure and cause a high residual lateral pressure.

A brief discussion of how this pressure is induced will help the reader understand what is meant by the term "residual lateral pressure". During the process of compaction the soil moves downward against the structure, developing friction. When the compacting pressure is released, the upward movement is restrained by friction, and full expansion cannot take place. This tends to maintain the lateral pressure at a higher level immediately adjacent to the structure. The lateral pressure remaining after the compaction pressure is removed is called "residual lateral pressure" and is often larger than the at rest earth pressure. Broms (2) has proposed that below a certain critical depth, the residual lateral pressure equals that induced during the compaction process.

Stability. - The purpose of any retaining wall is to provide the lateral resistance necessary to prevent horizontal movement of a mass of soil or rock (8). The tendency of a soil mass to displace and form a natural slope must be resisted in such a manner that the wall structure maintains a stable vertical condition. In order to design a suitable retaining structure, the following basic stability criteria must be evaluated.

1. Overturning: Calculation of the factor of safety with respect to overturning involves summing the forces tending to overturn the structure about its toe and comparing it to the summation of forces that resist overturning. These forces are usually summed as moments. However, it is most important to realize that the pressure at any point beneath the foundation should not exceed a predetermined allowable bearing pressure. If this occurs, excessive differential settlement or soil rupture could result. Several methods are used for determining an overturning factor of safety (10). However, there is no generally accepted procedure for computing this value for retaining walls (6).
2. Sliding: A more critical stability consideration involves sliding along the base, or failure due to shearing the soil adjacent to the bottom of the wall. A general definition of the factor of safety with respect to sliding is the ratio of the sum of the resisting forces to the sum of the driving forces. Where sliding resistance alone prevents displacement, the foundation soil shear strength will need to be great enough to offset the pressure exerted by the retained earth. It may not be possible to include passive earth pressure forces in sliding stability calculations, because

fully developed passive earth pressure requires much more displacement than is required to develop the active earth pressure for the same soil. When clay soils are involved, desiccation may open cracks which must first be closed prior to generation of passive resistance. This is one reason why the factor of safety against sliding is often computed without including the passive resistance. Other reasons include the possibility of soil removal during future construction or removal by scour action. Additional considerations include soil weakening due to freeze-thaw action or loosening due to root action. If these conditions could be eliminated, it would be desirable to include the passive resistance in the stability calculations.

Sliding stability may be marginally acceptable or even unacceptable when soil shear strength alone must offset the driving forces on retaining walls. One of the simplest and most common alternatives is the inclusion of a deep seated key beneath the footing. Even though the full development of passive pressure requires substantial displacement, other discouraging aspects of passive resistance can be avoided by proper design and placement of the key. Since the use of a key prevents failure from occurring along the base of the footing, failure must occur through some other surface. The location and depth of the key can have a significant influence on the location of the new failure surface. A detailed discussion of the suggested locations and resulting failure surfaces are beyond the scope of this report. However, this subject has been discussed in great detail in other references (1, 12).

3. Settlement: The design of any foundation requires an analysis of bearing pressure in order to define the minimum acceptable base dimension. Retaining walls not only balance lateral loads but must also have a foundation of sufficient size to support the vertical loads imposed by the backfill. Combining all of these loads into a resultant allows the designer to determine the distribution of bearing pressures. The peak pressure needs to be determined for comparison with the maximum allowable soil bearing capacity. If the peak pressure exceeds the soil capacity then there is a high probability of failure. Differential settlement may take place when localized overstress exists, just as total settlement failure may occur for overload conditions. When differential settlement occurs, it tends to aggravate the overall instability and may eventually result in overturning failure. For this reason, a settlement factor of safety of at least 2.0 or 3.0, depending on soil conditions, is recommended (19).

Drainage. - The design of a retaining wall is also greatly influenced by the type of backfill material used. Experience has shown that proper drainage of retaining wall backfill is one of the most important design requirements. Saturation of the backfill can produce a substantial increase in hydrostatic pressure on the back of the wall. Any buildup of hydrostatic pressure behind a retaining wall will reduce the margin of safety designed into the wall and could result in wall failure. A saturated soil will exert a pressure that is approximately twice as high as the pressure exerted by the same dry soil in the active state of stress. Wall failure could also be caused by improper design of filter material sizes in the drain. Proper design of the filter material sizes will provide protection of drains from clogging with soil fines. A detailed treatment of the types of drains and construction considerations can be found in several references (12, 19, 20).

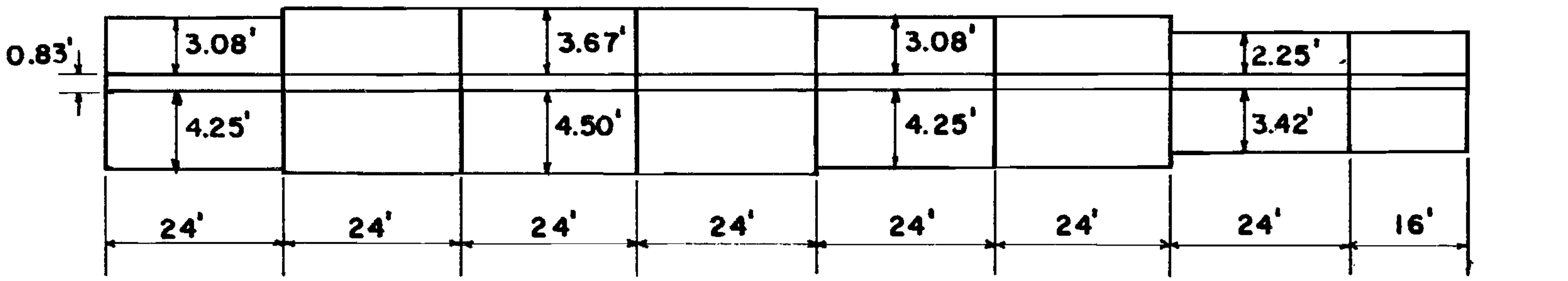
TEST WALL

Test Site

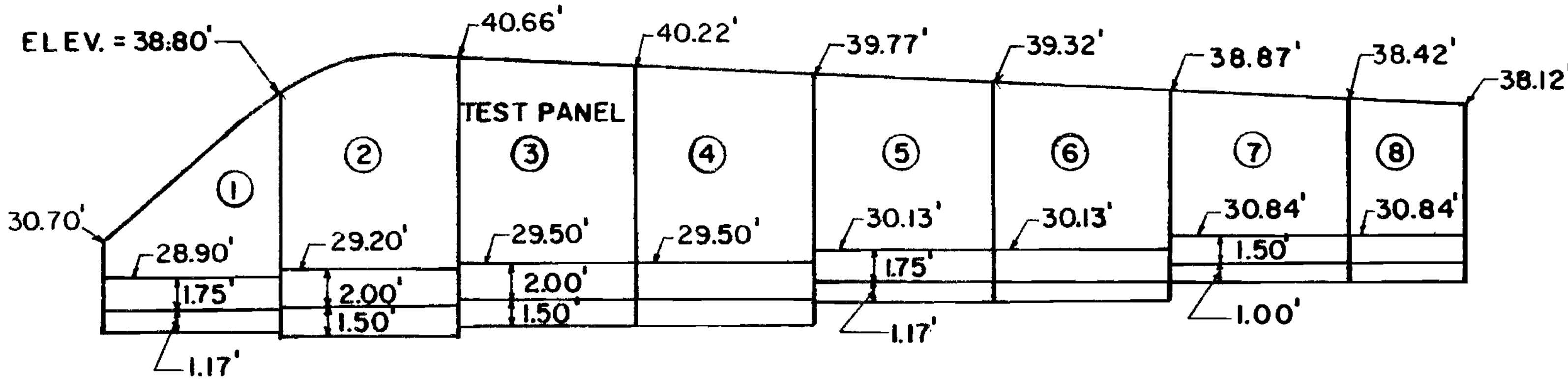
The test site selected is part of a State Department of Highways and Public Transportation (SDHPT) project located on the east side of Houston, Texas at the intersection of Interstate Highway 10 (IH-10) and Federal Road. IH-10 is being widened and elevated in this area. Four retaining walls are planned to be built at this intersection. The northeast and southeast retaining walls have been constructed. However, the northwest and southwest walls have not been constructed to date. One of the panels of the northeast wall was instrumented for this study.

Test Wall Description

The test wall is a standard cantilever retaining wall constructed on a spread footing foundation. The footing has a protruding key located approximately at midbase. The entire wall is 184 ft (56.1 m) long and is made up of 7 panels, 24 ft (7.3 m) long and 1 panel, 16 ft (4.9 m) long. An elevation and plan view of the wall is shown in Fig. 2. Panel No. 3 was selected for instrumentation because this panel had the largest stem height, and since it is an interior panel it should not experience any unusual deformation characteristics. The pressure cells are located approximately at the center of this panel. A cross-section of the instrumented section of Panel No. 3 and the locations of the pressure cells are shown in Figs. 3 and 4 respectively. The height of the stem where the pressure cells are located



(a) Plan View



(b) Elevation View

FIG. 2.— Test Wall (1 ft = 0.305 m)

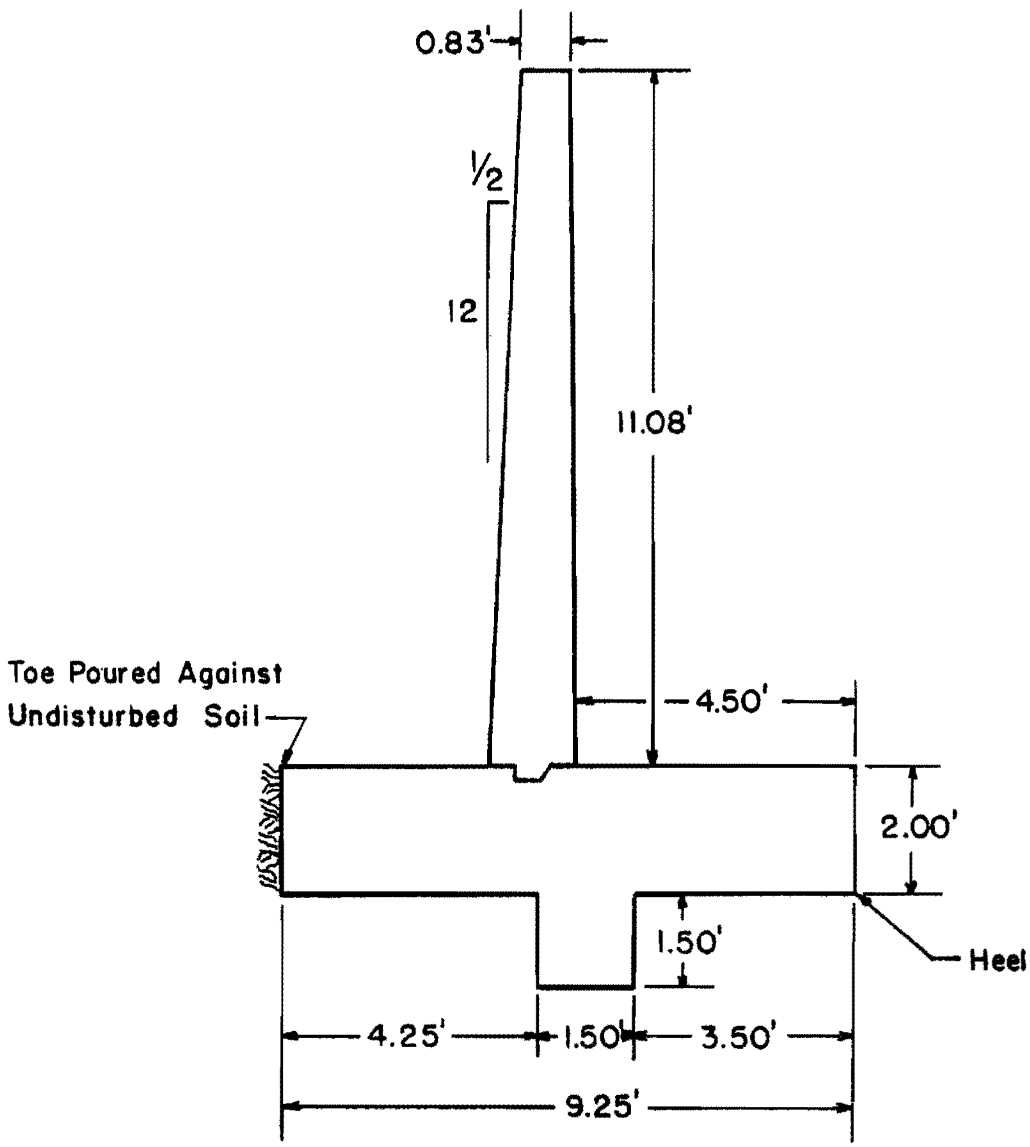


FIG. 3.— Cross Section of Instrumented Section of Panel No. 3 (1 ft = 0.305 m)

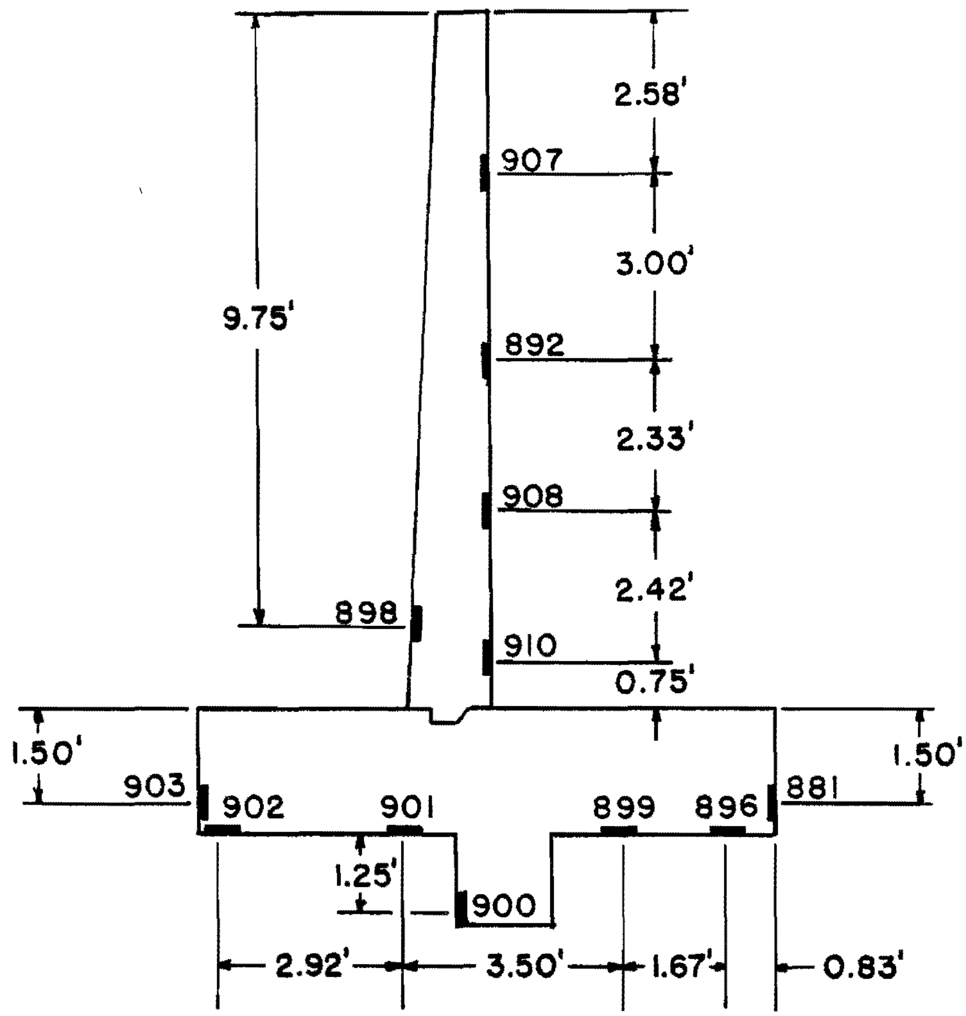


FIG. 4.— Locations of Pressure Cells
(1 ft = 0.305 m)

is 11.08 ft (3.38 m). The footing is approximately 9.25 ft (2.82 m) wide and 2 ft (0.61 m) deep with a 1.5 ft (0.46 m) by 1.5 ft (0.46 m) key located at approximately midbase.

The footing was constructed in such a manner which allowed the concrete to be placed against undisturbed soil on the toe side and wooden forms on the heel side. The front of the stem was constructed on a 12 to 1/2 batter. The front of the wall above final grade has an exposed aggregate finish. A graded filter with a 6 in. (152 mm) perforated pipe is provided to allow drainage and thus prevent hydrostatic pressure from developing behind the wall.

The construction at this site was extremely slow due to bad weather, delays in obtaining the water proofing material, and higher construction priorities at other sites. Construction of the footing was completed in July of 1978, and the stem was completed in January of 1979. The sand backfill was completed in July of 1979, and the clay backfill has not been completed to date. In May of 1980 steel sheet piling was driven approximately 60 ft (18.3 m) behind the wall. The sheet piling will support a new lane of IH-10 while the existing lane is being reconstructed. It is anticipated that the vehicular traffic on the new lane will also be acting as a surcharge load. However, this particular portion of the project has not been completed, and the vehicular aspect of surcharge has not been studied to date.

Instrumentation

Earth pressure. - The cantilever retaining wall was instrumented with twelve Terra Tec pneumatic pressure cells. The pressure cell locations are shown in Fig. 4.

The four pressure cells on the back of the stem and the cell on the vertical face of the heel were installed to gain a better understanding of the pressure distribution against the back of a cantilever retaining wall. Four pressure cells were installed at the base of the footing to examine the pressure distribution caused by the overturning moment and the selfweight of the structure and backfill material.

Three pressure cells were installed in the retaining wall to examine the pressure resisting sliding failure of the wall. One cell was installed on the front of the stem 16 in. (410 mm) above the top of the footing, and another was installed on the vertical face of the toe. The third cell was installed on the vertical face of the key. It is anticipated that the measured pressures from the pressure cell installed in the key will give information concerning the effectiveness of the key to increase the stability of the wall against sliding.

Pressure cell installation. - The four pressure cells at the base of the footing and the pressure cells on the vertical face of the toe and key were installed against undisturbed soil after excavation for the footing. Small steel pins were pushed into the soil to hold the cells secure. The four pressure cells on the back of the stem and the pressure cell on the vertical face of the heel were attached directly to the wooden forms. Several small holes were drilled through the forms on both sides of the proposed location of the pressure cell. Then the pressure cell was placed between the holes. Wire was placed around the cell and through the holes and tied on the backside of the form to hold the pressure cell in place. The concrete was then placed around the pressure cells making them part of the structure. The cell on the front of the stem could not be installed before the wall was constructed, therefore a wood pattern shaped like a pressure cell was nailed to the form. This made a cell-shaped depression on the face of the stem, into which the cell was grouted after construction. The pressure cell leads were routed through 2.0 in. (51 mm) PVC conduit pipe as shown in Fig. 5.

A short time after the forms on the stem were removed Cell No. 894, which was the second cell from the top of the wall, failed to give a pressure reading. Since the backfilling had not begun, Cell No. 894 was removed and replaced with cell No. 892.

The pressure cells on the back of the stem are not in direct contact with the sand backfill. A 1/8 in. (3.2 mm) rubber waterproofing material was placed on the backside of the wall before the sand backfill was placed. It is believed that the waterproofing material would have a negligible affect on the performance of the pressure cells.

Backfilling Procedure

The backfilling was accomplished in several stages. First an impervious clay layer was placed to a height of approximately 1.5 ft (0.46 m) above the top of the footing. The clay layer was compacted by a Bomag P-9 vibratory compactor. Then a 6 in. (152 mm) diameter perforated drain pipe was placed in the center of a 2 ft (0.61 m) high by 2 ft (0.61 m) wide layer of graded filter sand as shown in Fig. 6. The drain runs the entire length of the wall.

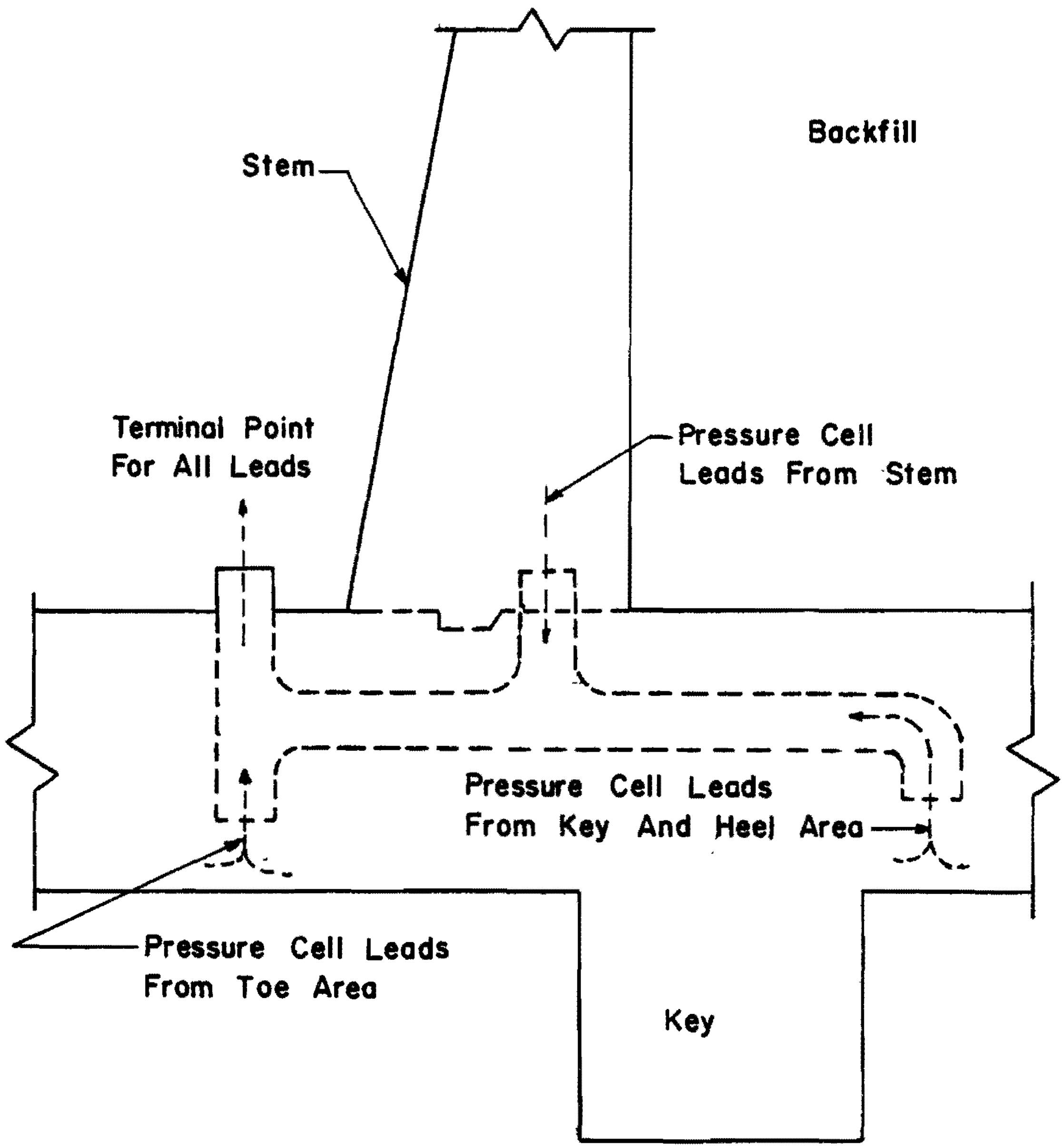


FIG. 5.—Configuration of Conduit for Pressure Cell Leads

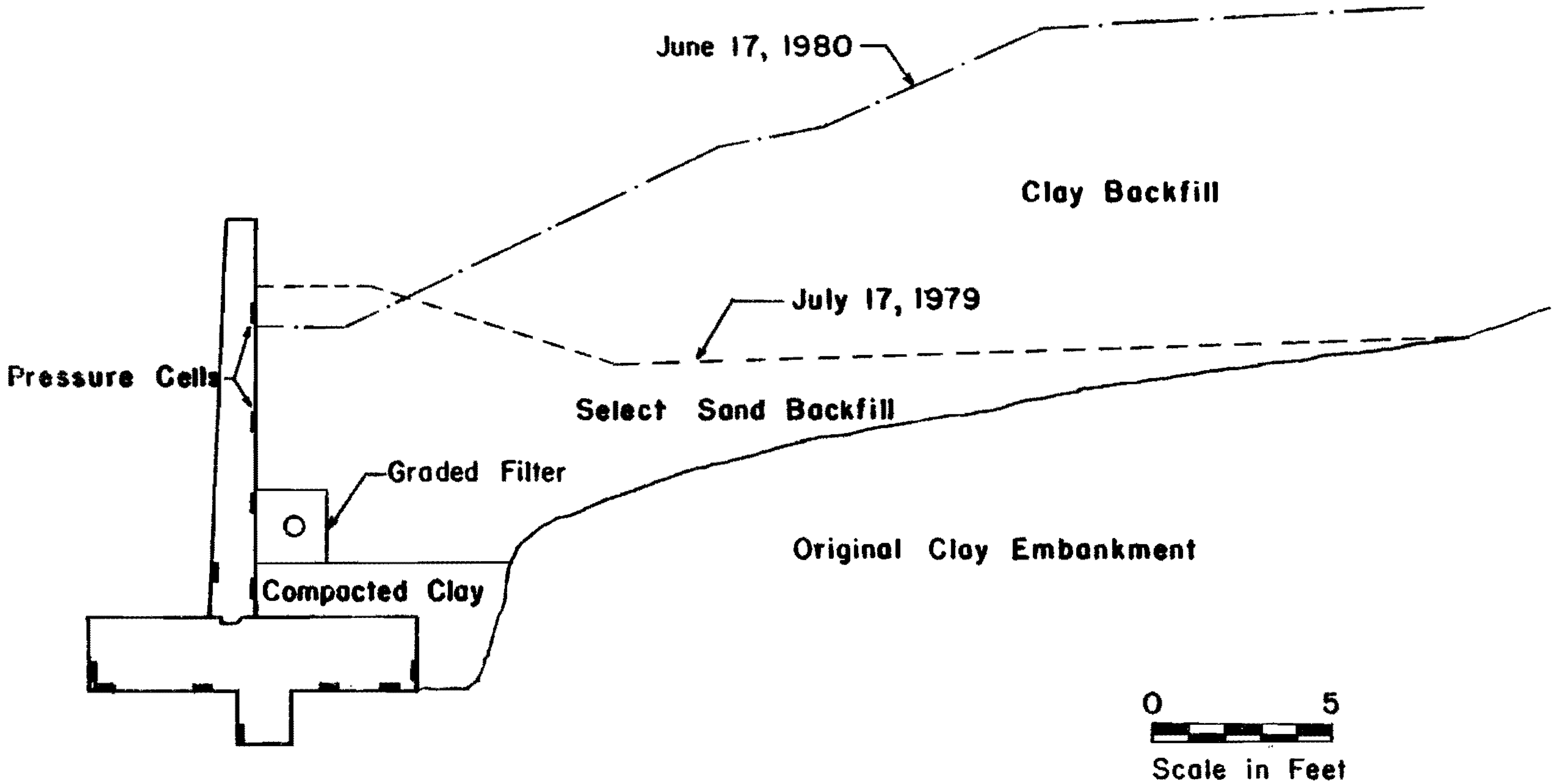


FIG. 6.—Profile of Backfill Material
(1 ft = 0.305 m)

After the drainage system was constructed, select sand backfill was loosely dumped behind the wall and spread by construction workers with shovels. This procedure was followed until the backfill was level with the drainage system. Then the backfill material was placed in a 3 ft (0.91 m) wide strip behind the wall in approximately 6 in. (152 mm) lifts. Each lift was compacted with a Bomag P-9 vibratory compactor. The remainder of the backfill beyond the 3 ft (0.91 m) wide strip was loosely dumped, and a bulldozer spread and compacted the material in approximately 6 in (152 mm) lifts. The backfilling operation was begun on June 26, 1979 and completed on July 17, 1979. A profile of the completed backfill on this date is shown in Fig. 6.

Properties of the Backfill Material

The backfill material is a fine-grained, subrounded to rounded, quartz sand. The grain size distribution of the backfill and filter material is given in Table 1. The backfill material has a coefficient of uniformity of 2.45 and a coefficient of curvature of 1.53. Tests were performed on the percent passing the No. 40 sieve to determine the Atterberg limits. However the soil exhibited no plasticity. Hence the backfill material was classified as a SP-SM according to the Unified Soil Classification System. The specific gravity of the material was determined to be 2.63.

Samples of the backfill material were taken during the backfilling operation to determine the wet and dry unit weights, the moisture content, and void ratio of the backfill material in place.

TABLE 1. - Sieve Analysis of
Backfill and Filter Material

Sieve No.	Percent Passing	
	Backfill Material	Filter Material
4	100.0	97.1
10	99.9	83.0
20	99.3	52.2
40	94.7	41.9
80	33.5	3.8
200	6.1	0.2

The samples were obtained by pushing a sampling tube into the backfill material and extracting the sample and sampling tube. Sample losses were minimal due to the presence of moisture in the sand. By determining the volume and weight of the sample in the tube, the unit weight of the material was calculated. The maximum, minimum, and average wet and dry unit weights, moisture content, and void ratio are given in Table 2. It should be noted that a wide range of unit weight, moisture content, and void ratio existed in the backfill material. Samples were prepared at approximately the maximum, minimum, and average void ratio as those determined from samples of the backfill and tested in the direct shear device. The effective angle of shearing resistance for the various void ratios are given in Table 3.

TABLE 2. - Unit Weight, Moisture Content and Void Ratio of Backfill Material

	Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Moisture Content (%)	Void Ratio
Maximum	124.5	105.4	18.3	0.760
Minimum	110.3	93.2	13.1	0.556
Average	116.8	100.4	16.3	0.644

TABLE 3. - Effective Angle of Internal Shearing Resistance (ϕ') for Maximum, Minimum, and Average Void Ratio of Backfill Material. (1 pcf = 0.157 kN/cu m)

Void Ratio	ϕ'
0.760	34^{o}
0.556	43^{o}
0.644	39^{o}

Placement of Clay Backfill

The placement of the clay backfill was begun in September of 1979. A profile of the clay backfill was taken June 17, 1980 and is shown in Fig. 6. Approximately 4 ft (1.22 m) of additional clay will be added before the backfilling is complete. After completion, the clay backfill will be used as the subgrade for construction of the new lane. The clay has a compacted average dry unit weight of 112 pcf (17.6 kN/m^3)

and average moisture content of 12.4%. The average total unit weight is 126 pcf (19.8 kN/m^3).

Soil Conditions

Soil conditions at the site were investigated with three soil borings. The soil borings were drilled on December 21, 1971, September 1, 1973, and June 17, 1980. The boring locations, designated as B-S1, B-S2, and B-S3 respectively,are shown in Fig. 7. Borings B-S1 and B-S2 were drilled by the SDHPT during the preliminary design stage, and boring B-S3 was drilled after the clay backfill was approximately 4 ft (1.22 m) below final grade. Soil borings B-S1 and B-S3 included Texas Cone Penetrometer (TCP) tests at various depths.

Laboratory tests on the undisturbed samples taken from borings B-S1 and B-S2 were performed by personnel at the SDHPT Laboratory in Houston, Texas, and undisturbed samples from boring B-S3 were tested at the laboratory at Texas A&M University. The tests performed on the undisturbed samples included Atterberg limits, moisture contents, and unit weights. The shear strength of the samples was determined by the Texas Triaxial Test (TAT) at the SDHPT Laboratory and by unconfined compression tests at the Texas A&M Laboratory. The results of the tests are plotted on the boring logs presented in Figs. 8, 9, and 10. Shear strengths were also estimated from TCP tests performed during boring B-S3. The shear strengths were estimated using correlations developed during an earlier research study at Texas A&M University (5). The estimated shear strengths agree reasonably well with those determined from unconfined compression tests and are plotted in Fig.

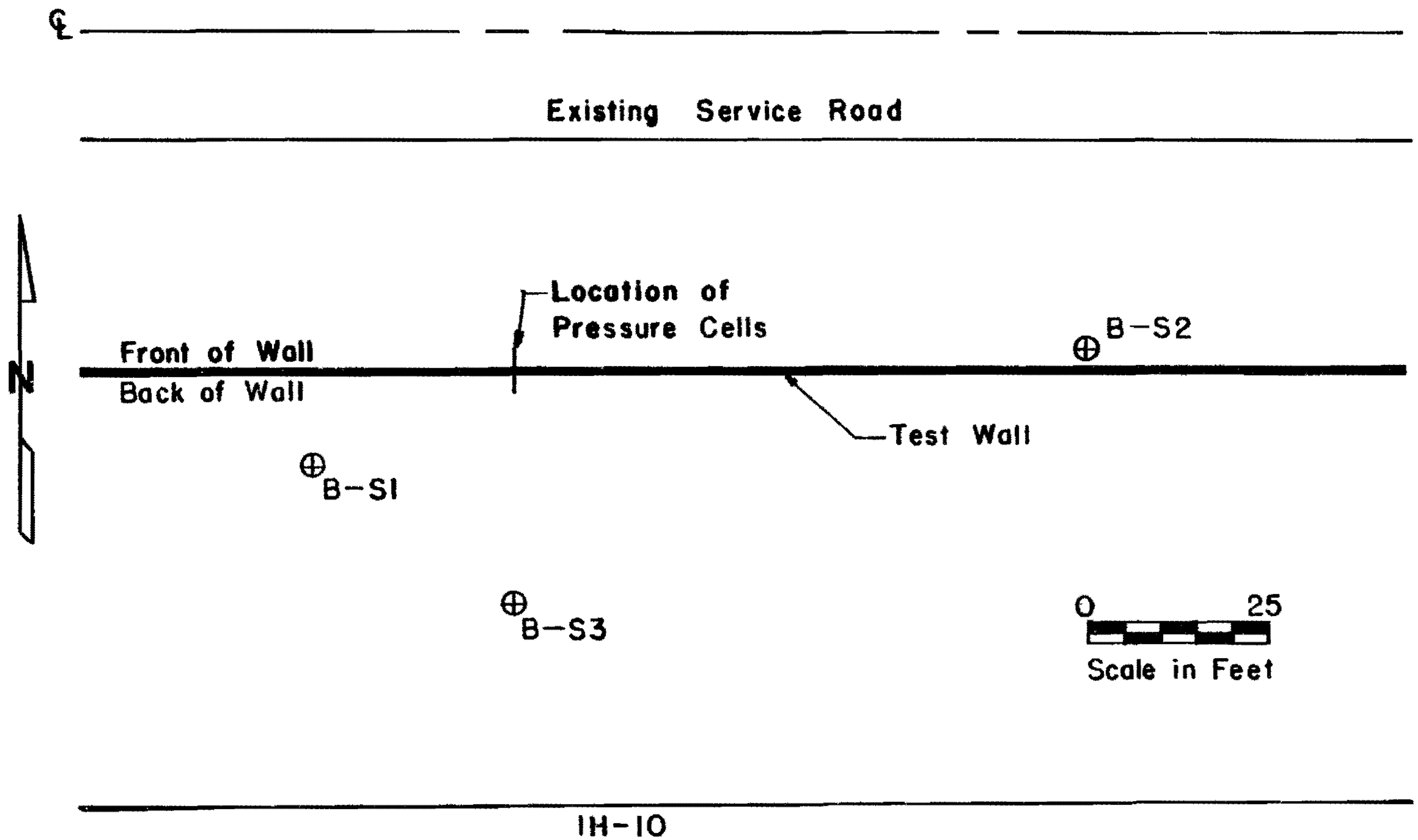


FIG. 7.— Location of Borings (1ft = 0.305 m)

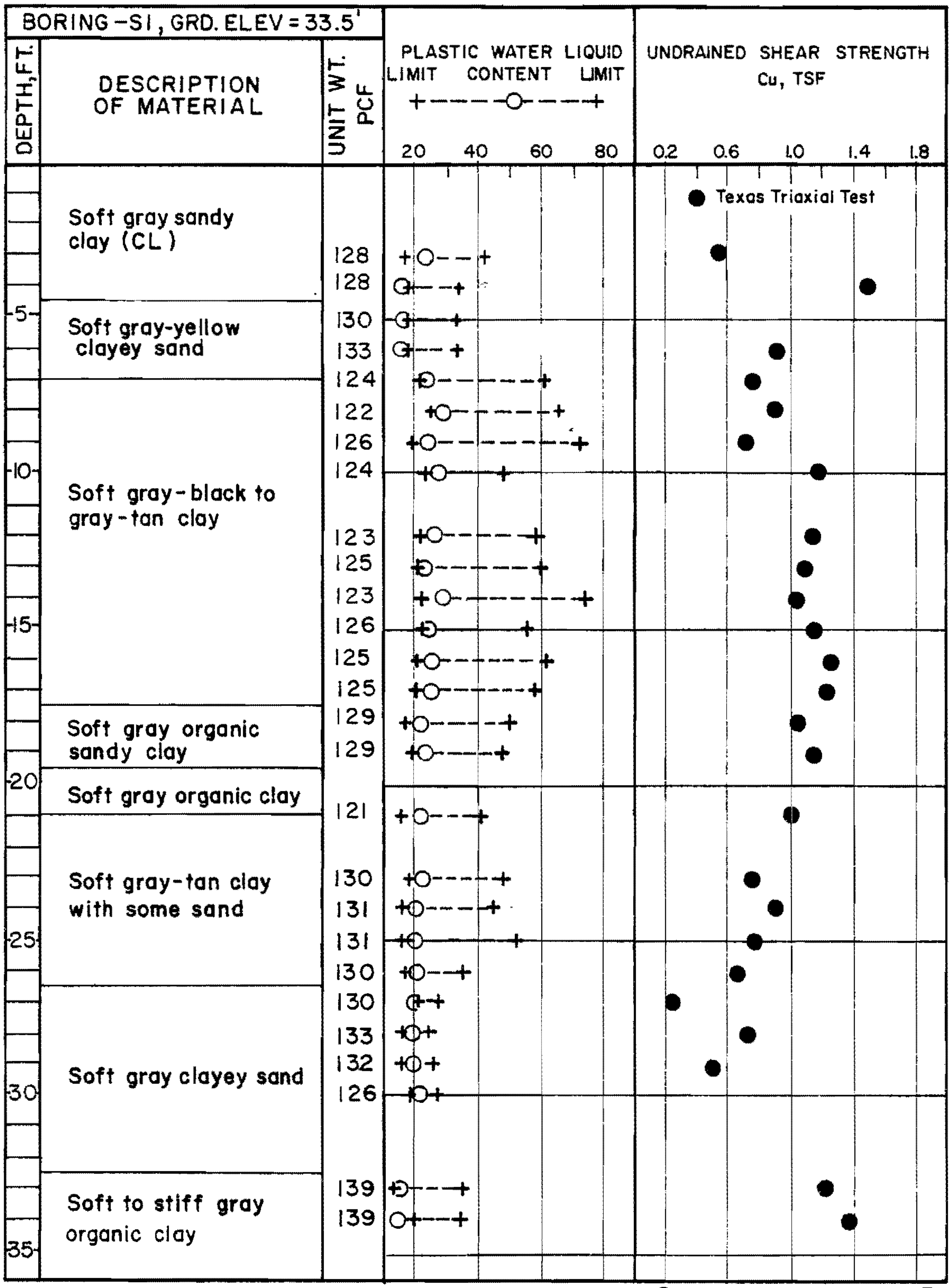


FIG. 8.– Boring-S1 (1 ft.=0.3048m, 1 tsf = 95.8 kN/m², 1 pcf = 0.157 kN/m³)

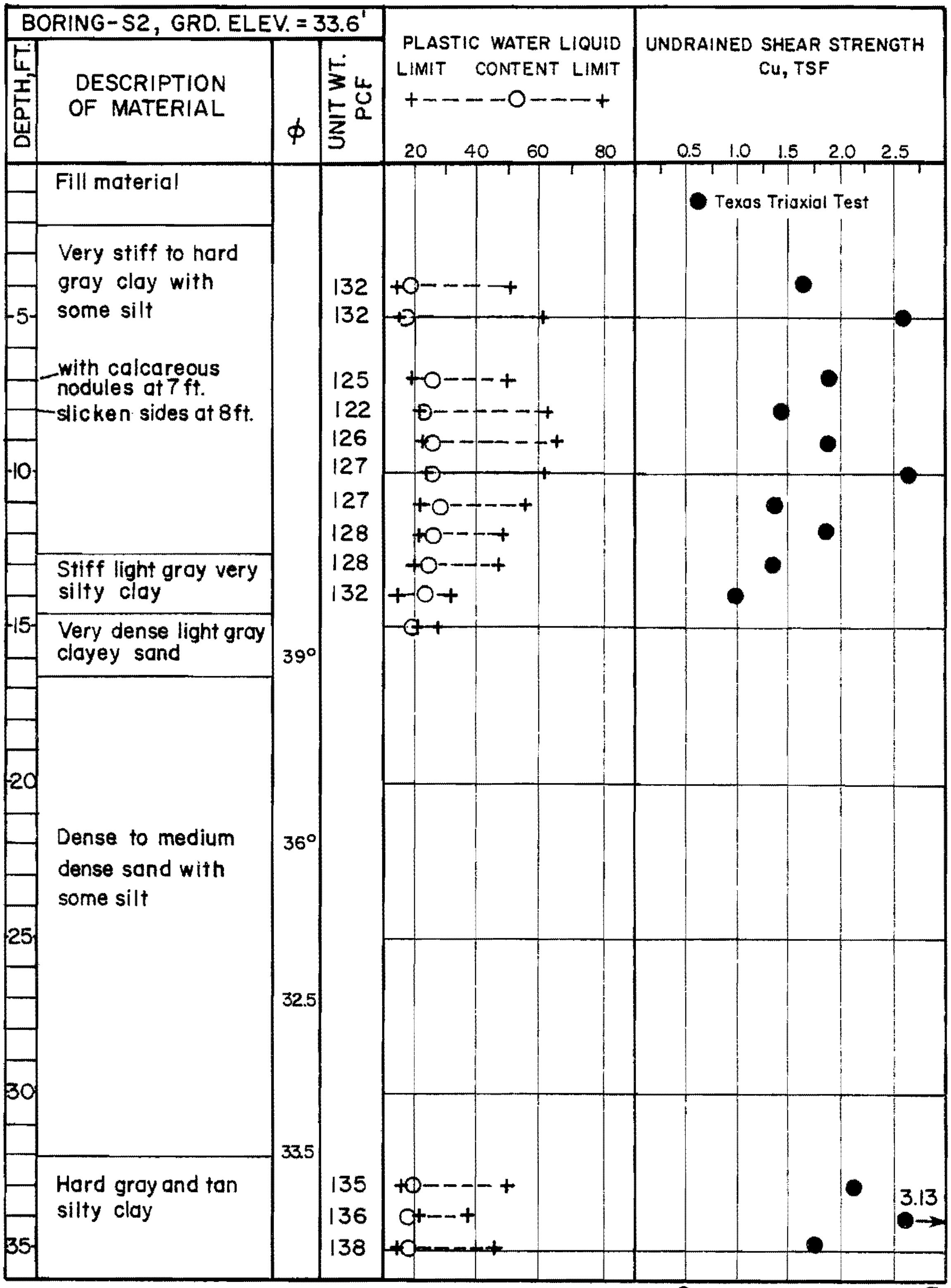


FIG. 9.– Boring-S2 (1 ft.=0.3048 m, 1 tsf=95.8 kN/m^2, 1 pcf=0.157 kN/m^3)

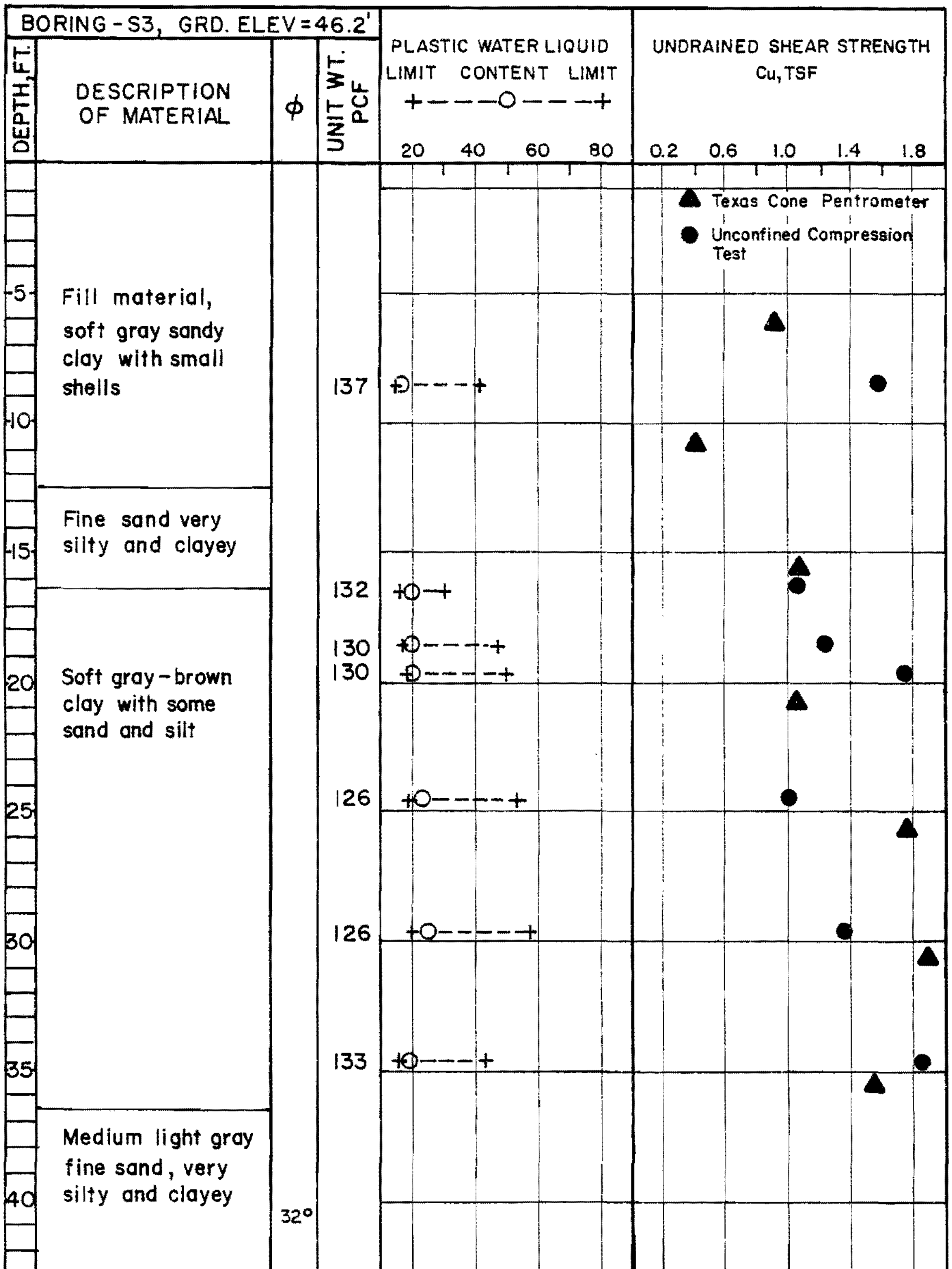


FIG. 10.– Boring-S3 (1 ft. = 0.3048 m, 1 tsf = 95.8 kN/m², 1 pcf = 0.157 kN/m³)

10. The effective angles of shearing resistance given in Figs. 9 and 10 were estimated from TCP tests performed during borings B-S2 and B-S3. These estimates are based on correlations from the same research study as referenced above.

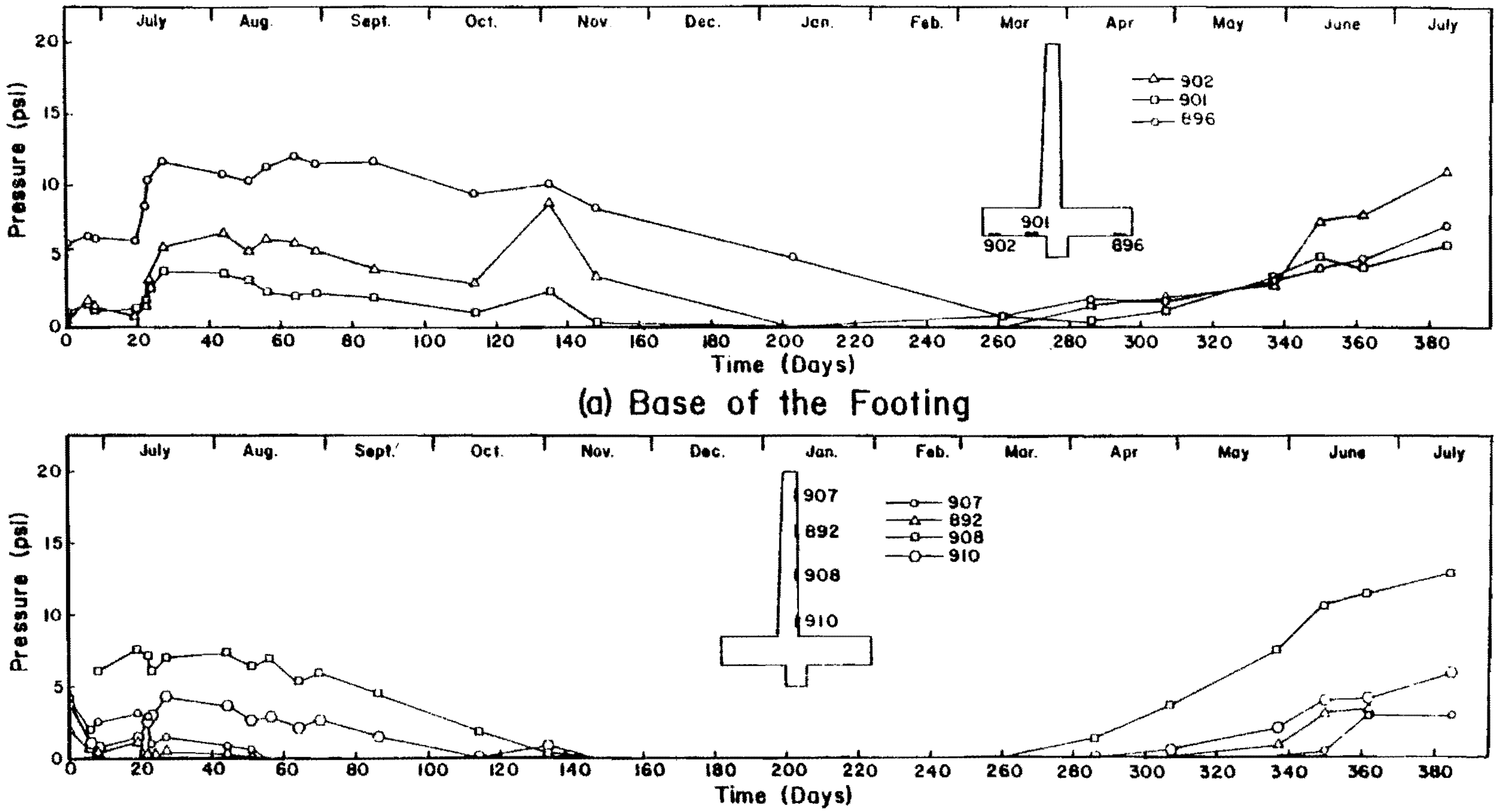
DATA COLLECTION

Pressure Cell Measurements

Each cell was calibrated in the laboratory before installation in the wall. During the calibration procedure, cell hysteresis, linearity, and calibration factors were established. From the calibration procedure, it was determined that none of the cells exhibited any problems with cell hysteresis, and all cells showed a linear increase and decrease between applied and measured pressures. It was also determined that the calibration factor for all the cells was essentially 1.00, i.e. 1 psi (6.9 kN/m^2) applied pressure equaled 1 psi (6.9 kN/m^2) measured pressure.

To determine the pressure measured by a particular cell, the field reading was recorded and a zero offset was subtracted. The zero offset was taken to be the field reading after the concrete was in place and the forms were removed. The pressures showed a seasonal variation which was attributed to temperature changes. Since thermocouples were not installed, a direct temperature correction could not be applied. However, based on previous experience with Terra Tec total earth pressure cells from three previously instrumented retaining walls, a method of temperature correction was developed. A discussion of how the temperature correction was developed and applied is given in Appendix III. The pressures acting on each Terra Tec cell through day 385 is plotted in Fig. 11.

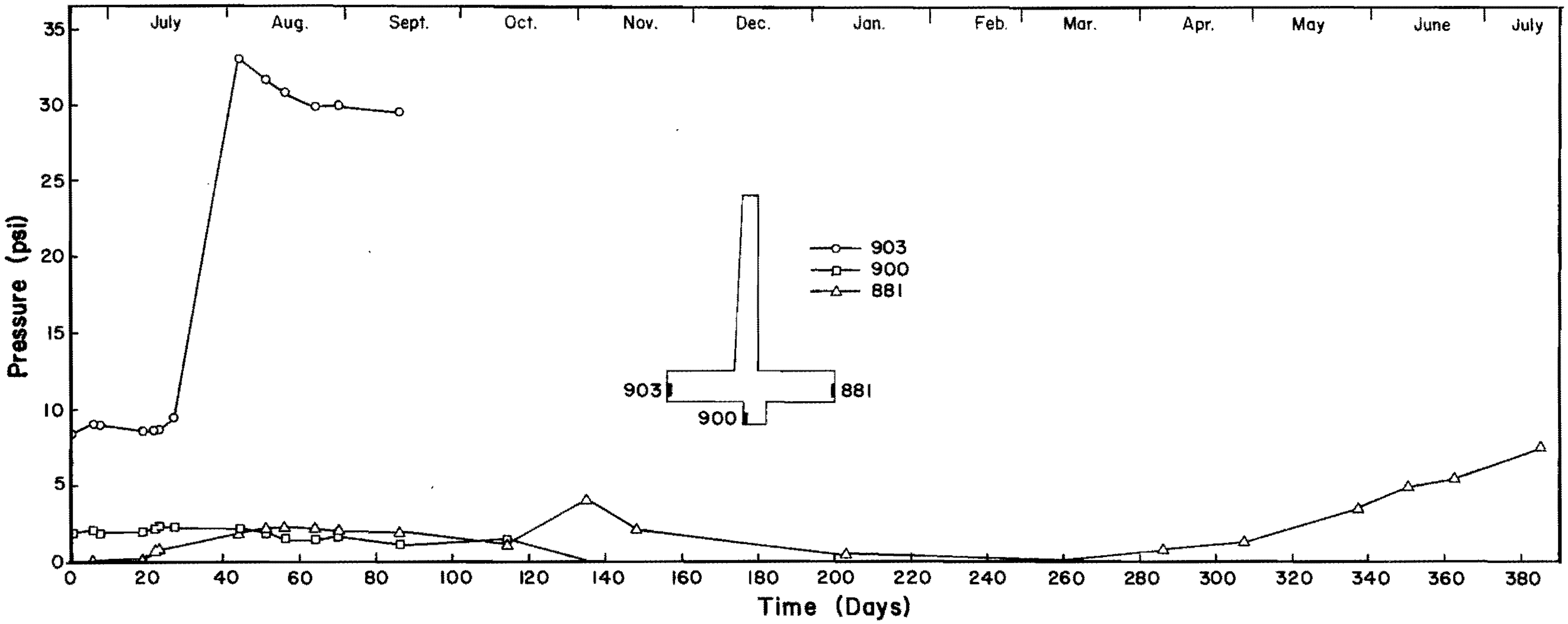
The temperature correction removed some of the pressure variation due to seasonal changes. However a variation is still present. The pressure variation is most evident for the upper two cells on the stem.



(a) Base of the Footing

(b) Back of the Stem

FIG. 11.— Measured Pressure Versus Time (1psi = 6.9 kN/m^2)



(c) Vertical Face of the Footing

FIG. 11.-(Continued)

This is most likely due to the fact that there is less backfill covering these pressure cells and temperature variations could be greater for these cells than can be accounted for in the temperature correction. This explanation is supported by the fact that enough backfill material was eroded away to expose the top Cell, No. 907. However, this cell began showing an increase in pressure after day 350. Since Cell No. 907 was exposed to the ambient temperature, it would show a greater increase in temperature during the summer and a greater decrease in temperature during the winter than can be accounted for in the developed temperature correction. The cells in the footing show less variation in pressure due to seasonal temperature changes, because they are buried approximately 4 to 5 ft (1.2 to 1.5 m) below the ground surface.

A cross-section of a pneumatic Terra Tec total earth pressure cell is shown in Fig. 12. As the temperature of the cell increases, the fluid in the cell expands causing a greater pressure to be measured than that pressure being exerted on the exterior of the cell. Likewise, if the temperature decreases, the fluid contracts, and a lesser pressure than the applied pressure will be measured. According to the manufacturer, these temperature affects are greatest when the cell is exposed to ambient conditions.

Cell No. 903, which is located on the vertical face of the toe, showed a large increase in pressure on day 44. The increase in pressure was caused by the stockpiling of steel sheet piling in front of the instrumented section of the retaining wall. Since the pressure cells measure total pressure, the total pressure increased. The total pressure appeared to be reaching a constant value when the cell became

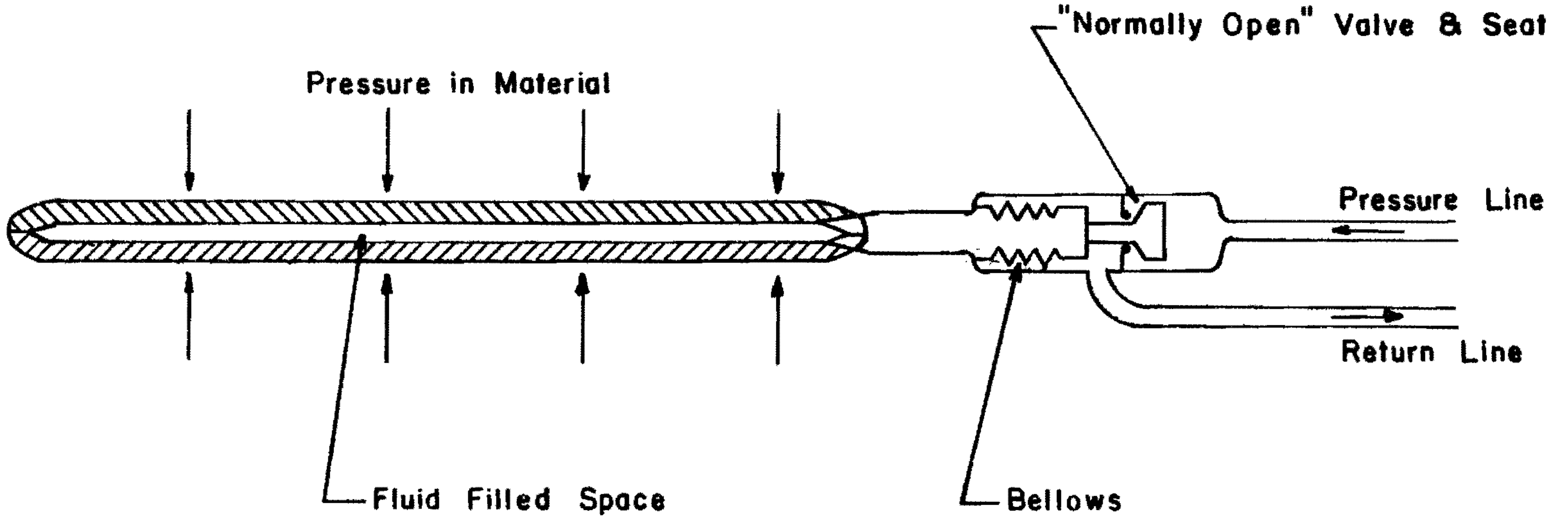


FIG. 12.—Cross Section of a Terra Tec Pressure Cell

inoperative after day 86. Cell No. 900, located on the vertical face of the key, did not show any increase in pressure due to the stockpiling of the steel sheet piling. A possible explanation for this condition is that the location of the key, as shown in Fig. 4, is such that no significant stress increase was imposed on this cell.

Cell No. 899, which is located on the bottom of the footing in the heel area nearest the key, became inoperative several weeks after the concrete was placed. Cell No. 881, which is located on the vertical face of the heel, and Cell No. 892, located on the back of the stem, became inoperative after day 203 and 362 respectively. The reason for these cells becoming inoperative is that the "normally open" valve, as shown in Fig. 12, is closed and will not open. According to the manufacturer it is possible to reverse the flow through the pressure cell leads and open the valve; however, this has been done repeatedly to no avail. The location of these pressure cells prevented their removal and replacement.

No pressures are reported for Cell No. 898 which is located on the front of the stem. The contractor did not backfill in front of the wall where this pressure cell was located so that a box could be installed to store the readout connectors of the pressure cell leads. The contractor has not backfilled this area to date.

Wall Movement

Since the amount and type of wall movement greatly affects the pressures exerted on the wall and foundation, provisions were made to measure lateral translation and tilting or rotation of the wall. A reference point was established in front of the wall across the service road. This reference point was referenced to several other points so that it could be re-established in the event of disturbance. A small hook was bolted to the front of the wall so that a steel tape could be attached to the hook and readings were taken to the reference point. The location of the hook is shown in Fig. 13.

Eight metal plates were epoxied to the front of the wall to make measurements for tilt or rotation. The location and spacing of the plates are shown in Fig. 13. The metal plates provided a flat smooth surface so that an inclinometer could be placed against the plates to measure the tilt or rotation. The inclinometer measurements were made to the nearest minute, and the changes in readings are given in Table 4. It can be seen from the values in Table 4 that there is a lack of consistency in the changes in readings on a particular day, especially during the early part of the study. These readings give the impression that the wall was experiencing some very odd deformations. However it is believed that this is not due to wall movement, but rather to a warping or rotation of the plates from temperature affects on the plates themselves or the epoxy. In the latter part of the study, after September 1979, when the clay backfill was being placed the changes in readings became more consistent, showing a slight rotation of wall away from the backfill. A plot of the measured displacements for day 385 is shown in Fig. 14.

A plumb bob apparatus was used to establish a vertical reference line from which additional tilting or rotation of the wall was measured. The plumb bob apparatus is shown in Fig. 13. A 15 lb (67 N)

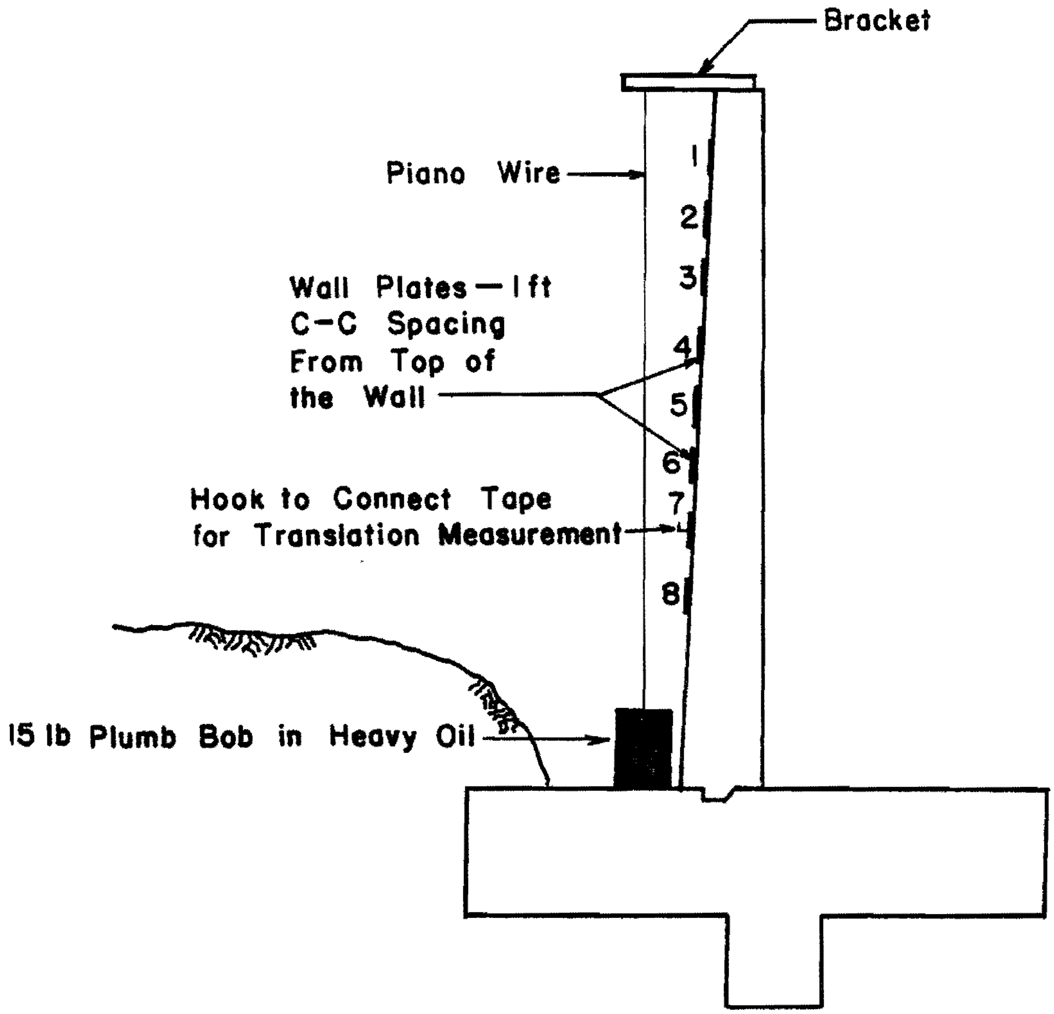


FIG. 13.— Movement Measuring System
(1 lb_m = 0.453 kg ; 1 ft = 0.305 m)

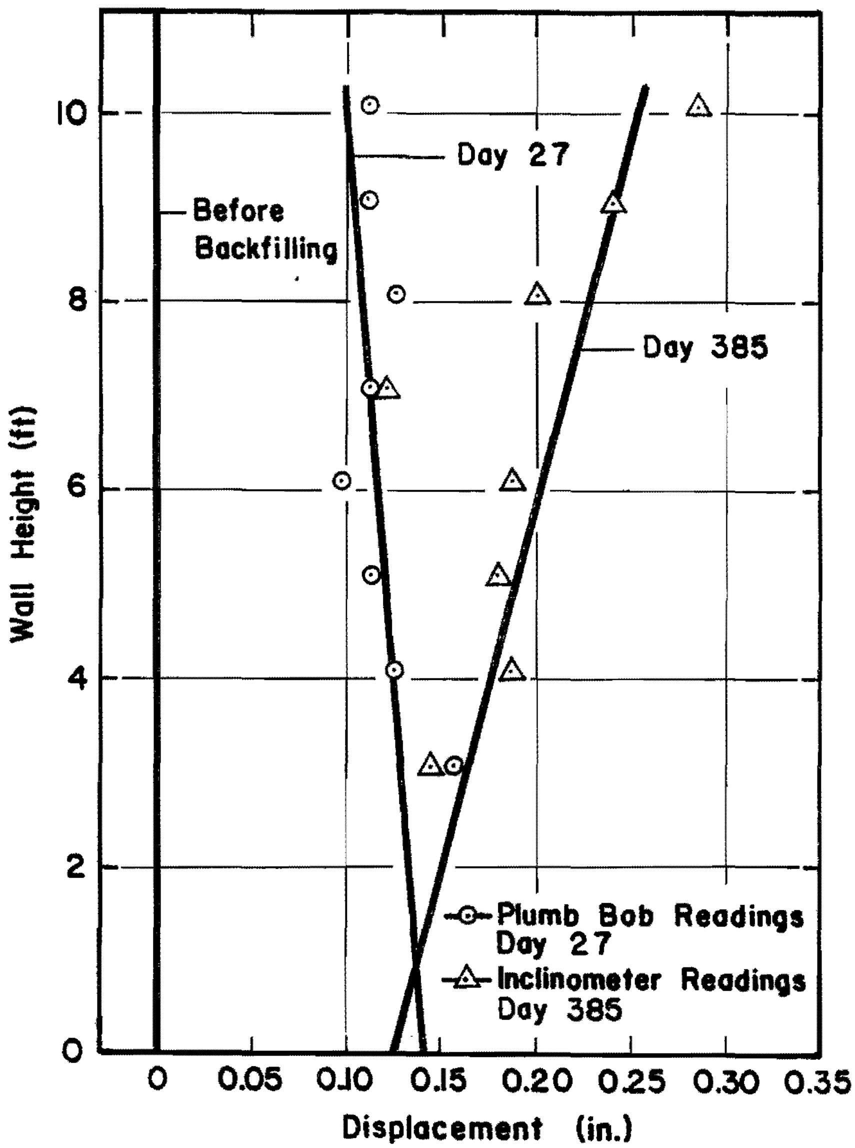


FIG. 14.—Wall Displacement (1 ft = 0.305 m; 1 in = 25.4 mm)

TABLE 4. - Wall Movement Data, Inclinometer Readings

Date	Day	Tilt or Rotation (Minutes)							
		Plate 1	Plate 2	Plate 3	Plate 4	Plate 5	Plate 6	Plate 7	Plate 8
6/25/79	5	87^{o} 15'	86^{o} 58'	87^{o} 29'	87^{o} 03'	87^{o} 26'	87^{o} 37'	88^{o} 13'	87^{o} 38'
6/28/79	8	0	0	+ 1	+ 2	- 2	0	+ 3	+ 2
7/09/79	19	0	+ 1	- 2	+ 2	0	- 1	- 1	- 2
7/12/79	22	+ 1	- 2	+ 1	+ 1	- 6	- 2	0	- 1
7/13/79	23	0	+ 1	- 1	- 1	- 3	- 1	- 2	- 2
7/17/79	27	- 2	0	- 1	+ 2	- 3	+ 1	- 1	- 1
8/03/79	44	0	- 4	- 2	0	0	0	0	a
8/10/79	51	- 2	+ 2	- 4	0	- 2	- 1	+ 1	- 4
8/15/79	56	+ 2	+ 2	- 3	- 3	- 1	0	+ 1	- 2
8/23/79	64	- 2	+ 2	- 3	+ 2	- 1	+ 1	- 1	a
8/29/79	70	+ 4	+ 1	- 3	0	0	0	+ 1	a
9/14/79	86	+ 5	+ 5	- 1	0	- 1	+ 1	- 1	a
10/12/79	114	+ 2	+ 5	- 1	0	0	0	+ 2	- 1
11/02/79	135	+ 2	+ 6	+ 1	+ 2	+ 1	+ 2	+ 1	0
11/15/79	148	+ 7	+ 9	+ 5	+ 4	+ 5	+ 7	+ 4	+ 6
1/09/80	203	+ 5	+ 1	- 1	+ 2	0	0	+ 2	- 5
3/07/80	261	+ 5	+ 4	+ 3	- 1	- 1	+ 5	+ 6	a
4/22/80	307	+ 5	+ 2	- 2	0	+ 1	+ 1	a	a

TABLE 4. - (Continued)

Date	Day	Tilt or Rotation (Minutes)							
		Plate 1	Plate 2	Plate 3	Plate 4	Plate 5	Plate 6	Plate 7	Plate 8
6/25/79	5	87° 15'	86° 58'	87° 29'	87° 03'	87° 26'	87° 37'	88° 13'	87° 38'
5/22/80*	337	+ 8	+ 5	+ 5	- 1	+ 7	+ 9	+ 7	+ 12
6/04/80*	350	+ 6	+ 6	+ 7	+ 4	+ 8	+ 9	+ 3	+ 10
6/16/80	362	+ 1	+ 3	- 2	+ 4	- 5	+ 1	+ 3	0
7/09/80	385	+ 4	+ 3	+ 2	- 1	+ 2	+ 2	+ 3	0

* A different inclinometer was used to make the measurements.

a - Unable to make reading due to muddy site conditions.

NOTE: Negative sign indicates movement toward backfill.

Positive sign indicates movement away from backfill.

plumb bob was suspended by a piano wire from a brace rigidly mounted on the top of the wall into a bucket of oil to prevent wind oscillation. A metal scale was used to measure the horizontal distance from a mark on each plate to the piano wire. A level bubble was mounted on the scale so that the scale could be maintained in a horizontal position. Measurements were made to the nearest 1/64 in. (0.397 mm). The changes in plumb bob readings are given in Table 5. Also, a plot of the measured displacements for day 27 is shown in Fig. 14.

As the wall rotates or tilts, the vertical reference line established by the plumb bob apparatus shifts with the movement of the wall. Thus it is necessary to measure the movement of some point near one of the plates relative to some fixed datum. Then the changes in the plumb bob readings can be related to this point to determine the movement of the wall. The measurement from the hook to the reference point across the service road would locate plate No. 7, because the hook is mounted next to plate No. 7. This measurement could not be made from day 27 to day 385 because the stockpiled sheet piling obstructed the measurement. Therefore, the plumb bob readings taken after day 27 do not give the true location of the wall, but they do give the direction of tilt or rotation. As the wall rotates or tilts away from the backfill the horizontal distance between the plates and the piano wire will increase, and as the wall rotates or tilts toward the backfill the horizontal distance between the plates and the piano wire will decrease.

The data reported in Table 5 indicate a slight rotation or tilt toward the backfill while the sand backfill was being placed and for a while after the sand backfill was complete. This can also be seen

TABLE 5. - Wall Movement Data, Plumb Bob Readings

Date	Day	Deflection (1/64 inch)							
		Plate 1	Plate 2	Plate 3	Plate 4	Plate 5	Plate 6	Plate 7	Plate 8
6/28/79	8	$11\frac{5}{64}$	$10\frac{32}{64}$	$9\frac{62}{64}$	$9\frac{23}{64}$	$8\frac{48}{64}$	$8\frac{11}{64}$	$7\frac{42}{64}$	$6\frac{58}{64}$
7/09/79	19	0	- 1	- 2	- 1	0	0	- 1	+ 1
7/12/79	22	0	0	0	0	- 1	- 1	- 1	+ 1
7/13/79	23	0	0	0	0	- 1	0	- 1	+ 1
7/17/79	27	- 3	- 3	- 2	- 3	- 4	- 3	- 2	0
8/03/79	44	- 3	- 2	- 2	- 1	- 2	- 3	a	a
8/10/79	51	- 3	- 2	- 2	- 3	- 4	- 3	- 2	0
8/15/79	56	- 1	- 2	- 2	- 1	- 2	- 1	- 2	0
8/23/79	64	- 3	- 2	0	- 1	- 2	- 1	0	a
8/29/79	70	- 1	0	+ 2	+ 3	+ 2	+ 5	+ 6	a
9/14/79	86	- 1	- 2	0	- 1	- 2	- 1	0	a
10/12/79	114	- 1	- 2	0	- 1	0	+ 1	0	+ 4
11/02/79	135	- 1	- 2	0	+ 1	0	+ 3	+ 2	+ 6
11/15/79	148	- 1	- 1	+ 1	+ 1	0	+ 1	+ 2	+ 4

a - Unable to make reading due to muddy site conditions.

NOTE: Negative sign indicates wall movement toward backfill.

Positive sign indicates wall movement away from backfill. 1 in. = 25.4 mm from the measured displacements plotted in Fig. 14 for day 27. The plumb bob readings could not be obtained after day 148 because the hole in front of the wall became filled with too much mud and sludge to place the bucket of oil in the hole.

ANALYSIS OF RESULTS

Theoretical Lateral Pressure

The primary objective of this research program is to develop a more economical cantilever retaining wall design. In order to accomplish this objective, it is necessary to determine whether or not the predicted earth pressure compares favorably with the measured pressure on the real structure. The predicted pressure is usually obtained from an equation which has been derived from a theoretical analysis, as opposed to an equation resulting from empirical correlations. Due to the irregular shape of the backfill, a pressure distribution using either Rankine's or Coulomb's theory could not be calculated. However, a total thrust and point of application was obtained by the Culmann solution. The Culmann solution is a graphical solution of the Coulomb theory.

The Culmann solution considers wall friction, δ, irregularity of the backfill, any surcharges (either concentrated or distributed loads), and the angle of internal shearing resistance (1). A rigid, plane rupture surface is assumed. Essentially, the solution is a graphical determination of the maximum value of soil pressure. The procedure for the Culmann graphical solution can be found in several texts on Soil Mechanics and Foundation Engineering (1, 6, 19). An example of the Culmann graphical solution is presented in Appendix IV.

Since a wide range in void ratios and unit weights were determined to exist in the cohesionless backfill material, a Culmann solution was performed using the maximum, minimum, and average values. These three cases were also solved using the clay backfill as a surcharge. The parameters used in the Culmann solution are given in Table 6.

TABLE 6. - Parameters Used for Culmann Solution

Parameters	Without Clay Surcharge			With Clay Surcharge		
	Case 1	Case 2	Case 3	Case 1-S	Case 2-S	Case 3-S
ϕ (Degrees)	43	39	34	43	39	34
γ Sand (pcf)	124.5	116.8	110.3	124.5	116.8	110.3
α (Degrees)	90	90	90	90	90	90
δ (Degrees)	0	0	0	0	0	0
γ Clay (pcf)	-	-	-	126.0	126.0	126.0

NOTE: 1 pcf = 0.157 kN/m^3

The wall friction angle, δ, is a difficult parameter to evaluate. Approximate values of δ for various types of wall surfaces and finishes may be found in some texts on soil mechanics and foundations. It is generally acceptable to apply Rankine's active conditions to cantilever retaining walls (6, 19). Rankine's theory considers zero wall friction and assumes that the resultant acts at the same angle as the slope of the backfill. It can be seen from the profiles of the backfill shown in Fig. 6 that the backfill is essentially horizontal next to the wall. Also, the surface of the water proofing material is approximately frictionless. Thus δ was assumed to be equal to zero for all cases listed in Table 6.

The total thrust, point of application, and angle of the failure surface as determined from the Culmann solution are given in Table 7. The Culmann solution presented in Appendix IV is a solution for Case 2, without the clay surcharge.

TABLE 7. - Results from Culmann Solution

	Without Clay Surcharge			With Clay Surcharge		
	Case 1	Case 2	Case 3	Case 1-S	Case 2-S	Case 3-S
Total Thrust (k/ft)	1.23	1.33	1.53	1.85	2.22	2.84
Point of Application* (ft)	3.59	3.57	3.57	3.72	3.80	3.72
Angle of Failure Surface (Degrees)	68	67	63	63	60	56

*Point of application is measured from bottom of the heel.
NOTE: 1 k/ft = 14.59 kN/m; 1 ft = 0.305 m

Comparison with Measured Lateral Pressures

Without the clay surcharge. - The data presented in Fig. 15 represents the measured lateral earth pressure at each depth on a given day without the clay surcharge. The earth pressure distributions have been plotted for twenty-seven, forty-four, and fifty-one days following the start of backfilling. These pressure distributions correspond to the profile taken on July 17, 1979 (see Fig. 6) when the clay backfill was not in place.

Each pressure distribution shown in Fig. 15 was integrated over

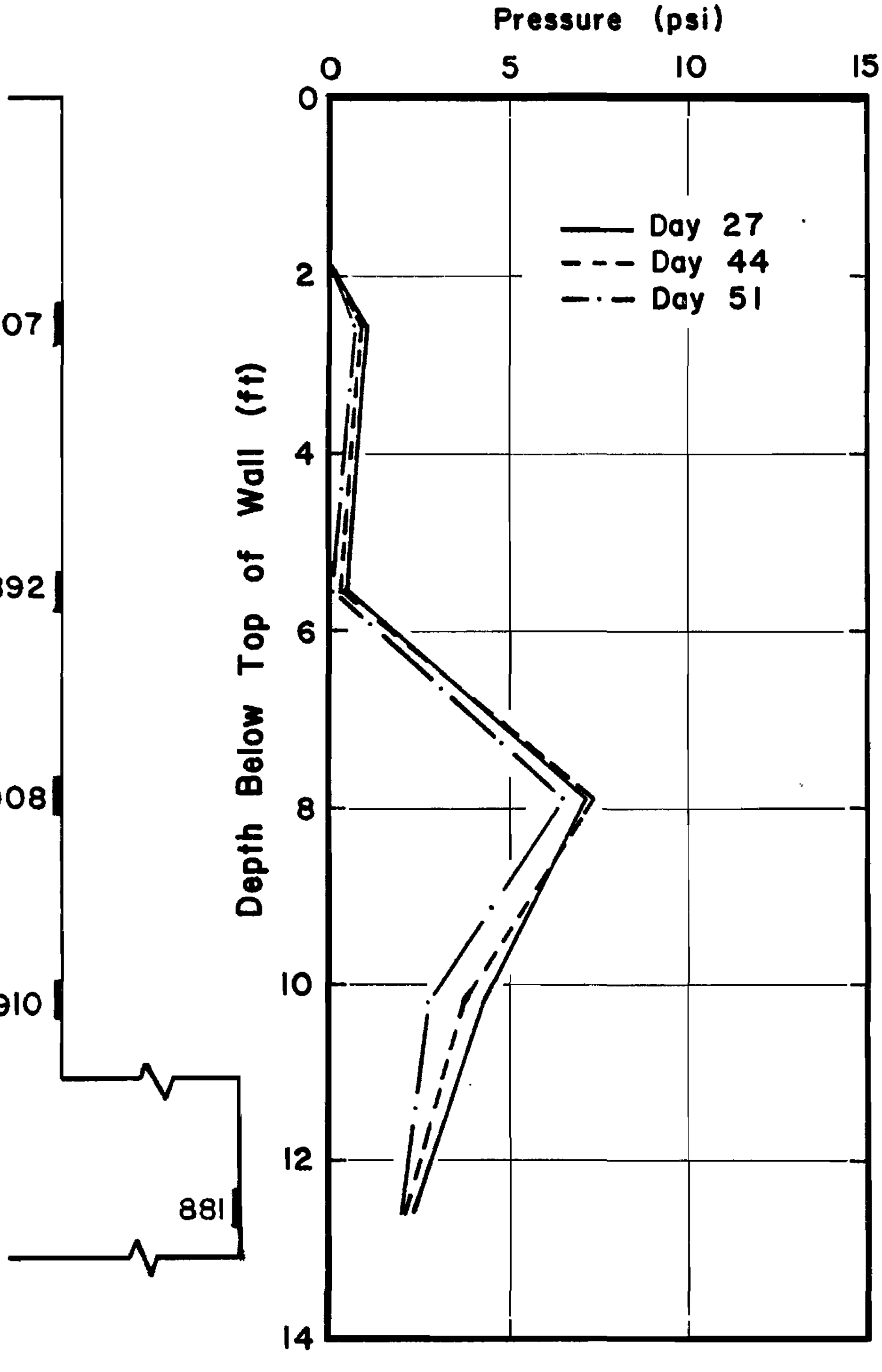


FIG. 15.— Distribution of Lateral Pressures, without the Clay Surcharge (1 ft = 0.305 m; 1 psi = 6.9 kN/m^2)

the height of the backfill against the stem to determine the resultant thrust and point of application. In order to perform the integration and locate the point of application several assumptions were made. These assumptions are:

1. Plane strain conditions exist,
2. The pressure at the top of the backfill is zero,
3. The pressure is distributed linearly between pressure cells, and
4. The resultant thrust acts through the centroid of the pressure distribution diagram.

The results of this integration are given in Table 8.

TABLE 8. - Results of Integration of Measured Lateral Earth Pressures, without the Clay Surcharge

	Day 27	Day 44	Day 51
Total Thrust (k/ft)	4.99	4.62	3.79
Point of Application* (ft)	4.46	4.40	4.25

* Point of application is measured from the bottom of the heel.

NOTE: 1 k/ft = 14.59 kN/m; 1 ft = 0.305 m

Comparing Table 8 with Table 7 (without the clay surcharge), it can be seen that the calculated total thrust from the measured pressures is considerably greater than the total thrust determined from the Culmann solution. The average of the thrusts determined from the measured pressures is 4.47 k/ft (65.2 kN/m). The ratio of this average thrust from the measured pressures to the thrust determined in Case 2 from the Culmann solution, which involves the average engineering properties of the sand backfill, is 3.4.

A possible explanation of why the thrust from the measured pressures is greater than the theoretical thrust can be obtained from a consideration of the wall movement. As stated earlier and shown in Fig. 14, when the sand backfill was placed the plumb bob readings indicated that the wall was tending to rotate or tilt toward the backfill. This type of movement is indicative of the passive condition. Thus, higher pressures than those predicted for the Rankine active condition would be expected. Another explanation could be that the compaction of the 3 ft (0.91 m) wide strip next to the back of the wall induced residual lateral pressures that caused higher pressures to be exerted against the back of the wall than earth pressure theory can predict. Still another explanation can be obtained by consideration of the dead zone (see Fig. 1). When considering a cantilever retaining wall the Rankine active pressure is assumed to act on a vertical plane through the heel. However, the self weight of the dead zone may be adding some additional pressure to the back of the wall that is not taken into account when calculating the lateral earth pressure according to the Culmann method. Also, since the material within the dead zone moves with the wall as if it were part of the wall (18), pressures closer to the at-rest condition could be measured against the lower part of the wall.

With the clay surcharge. - The data presented in Fig. 16 represent the measured lateral earth pressures at each depth on a given day with the clay backfill in place. The lateral earth pressure

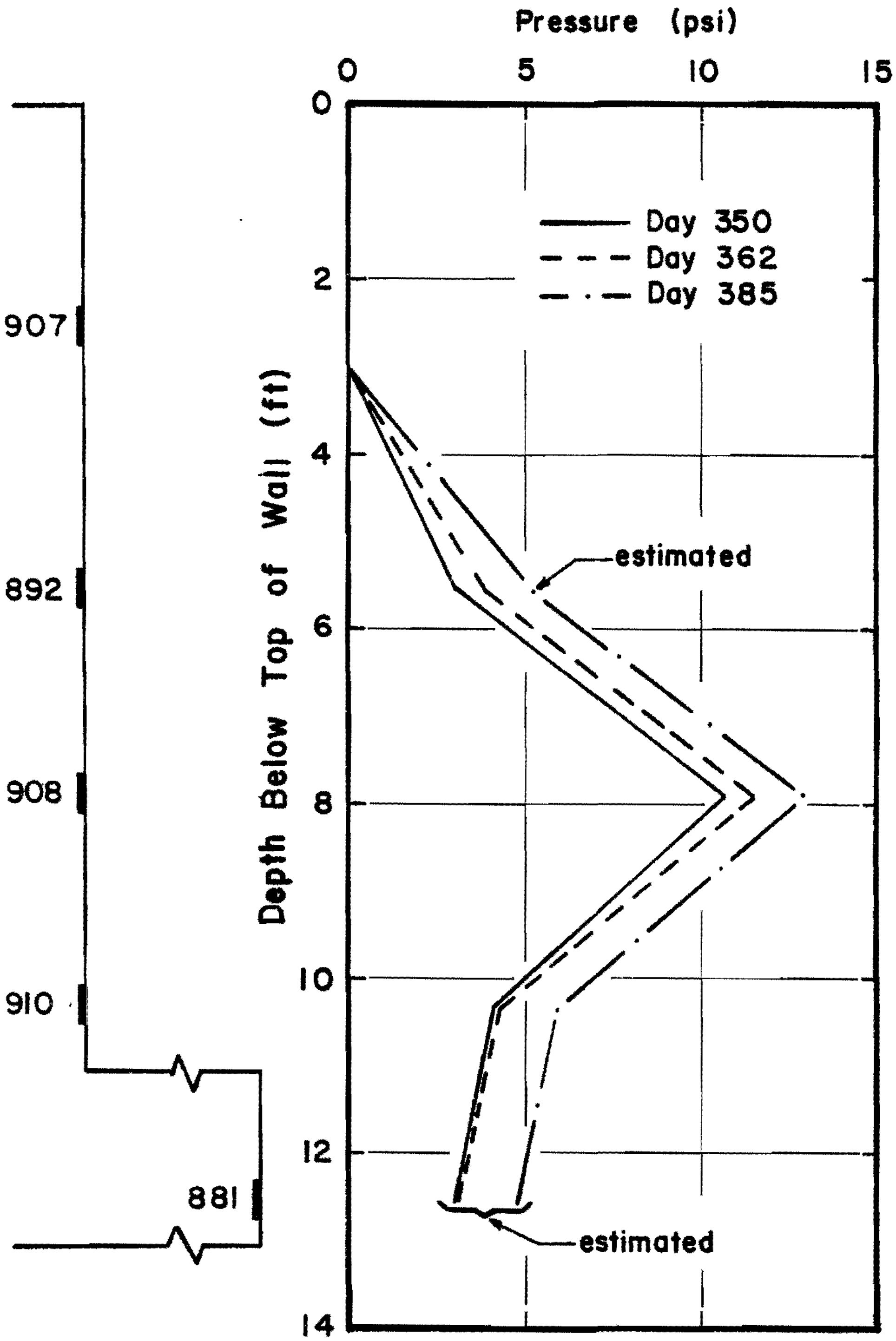


FIG. 16.— Distribution of Lateral Pressures, with the Clay Surcharge (1ft = 0.305 m; 1 psi = 6.9 kN/m^2)

distributions have been plotted for 350, 362 and 385 days after the start of backfilling. These distributions correspond to the profile taken June 17, 1980 (see Fig. 6). Since Cell No. 881, which is located on the vertical face of the heel, became inoperative after day 203 the pressures for this particular cell were estimated based on the trend of previous pressure distributions when the cell was operating properly. Also, after day 362 Cell No. 892 became inoperative, so the pressure for this cell on day 385 was estimated from trends of previous pressure distributions. The top of the backfill next to the back of the stem is below Cell No. 907. However, this cell indicated an increase in pressure on day 350. As stated earlier this increase in pressure is believed to be due to temperature, so this pressure increase was neglected when integrating the pressure distributions.

Each pressure distribution shown in Fig. 16 was integrated over the height of the backfill against the stem to determine the resultant thrust and point of application. The same assumptions for integration as stated previously were used to perform the integration of these pressure distributions. The results of the integration are given in Table 9.

TABLE 9. - Results of Integration of Measured Lateral Earth Pressures, with the Clay Surcharge

	Day 350	Day 362	Day 385
Total Thrust (k/ft)	6.76	7.42	9.36
Point of Application* (ft)	4.66	4.74	4.62

* Point of application is measured from the bottom of the heel.
NOTE: 1 k/ft = 14.59 kN/m; 1 ft = 0.305 m

Comparing the results shown in Table 7 (with the clay surcharge) and Table 9 indicates that the total thrust of the measured pressures is much greater than the thrust determined from the Culmann solution. The average thrust from the measured pressures is 7.85 kips/ft (114.5 kN/m). The ratio of this average thrust to the thrust determined in Case 2-S from the Culmann solution, which involves the average engineering properties of the sand backfill, is 3.5. Note that the ratio determined in the previous section was 3.4. Thus it can be concluded that the total thrust from the measured lateral pressures is approximately 3.5 times larger than the predicted total thrust with and without the clay surcharge. It can also be seen from Figs. 15 and 16 that all the pressure distributions have essentially the same shape. Hence it appears that the pressure cells are functioning properly.

A possible explanation of why the resultant thrust from the measured pressures is greater than the thrust determined from the Culmann solution can be obtained from a consideration of the wall movement. Referring to the changes in inclinometer readings (see Table 4), it can be seen that the changes in inclinometer readings indicate that the wall has rotated or tilted away from the backfill. There is very little movement in the lower part of the wall which is where the greatest pressures exist. It should be noted that any translation that might have occurred during the time span from day 27 to day 385 could not be measured. The dead zone phenomenon as discussed earlier also could be contributing to the higher pressures in the lower part of the wall.

Calculated Bearing Pressures

The loads from any structure must be transferred to the supporting soil by some type of foundation. The load transferred to the supporting soil by spread or strip footing foundation is termed the bearing or foundation pressure. It is important that the bearing pressure be less than the allowable bearing capacity of the soil or shear failure will result. The most favorable condition is for the resultant of all loads to intersect within the middle third of the base of the foundation. If this condition is satisfied the entire area beneath the base is subjected to compression which insures stability of the structure.

The resultant, its location, and the bearing pressure were computed using the maximum, minimum and average engineering properties of the sand backfill, with and without the clay surcharge. The resultant and its location were computed by summing moments about the heel. After the resultant and its location were determined, the bearing pressure was computed assuming a rigid footing and a linear pressure distribution using the following equation:

$$q = \frac{V}{b}\left(1 \pm \frac{6e}{b}\right) \qquad (1)$$

where q = bearing pressure,

V = resultant vertical force,

b = width of footing, and

e = eccentricity of the resultant vertical force.

The results of the bearing pressure calculations are given in Table 10.

TABLE 10. - Results of Bearing Pressure Calculations

	Without Clay Surcharge			With Clay Surcharge		
	Case 1	Case 2	Case 3	Case 1-S	Case 2-S	Case 3-S
Resultant Force (k/ft)	10.52	10.20	9.94	10.06	9.78	9.53
Point of Application* (ft)	4.08	4.17	4.29	4.42	4.64	4.70
q Toe (psi)	5.88	6.11	6.60	7.37	8.23	8.31
q Heel (psi)	10.40	9.58	8.75	8.15	6.87	6.37

* Point of application is measured from the back of the heel.

NOTE: 1 ft = 0.305 m; 1 psi = 6.9 kN/m^2; 1 k/ft = 14.59 kN/m

Referring to Table 10, the resultant forces for the cases with the clay surcharge are less than those without the clay surcharge because some of the backfill was eroded, thereby reducing the weight of the soil above the heel. The points of application of all the resultants are within the middle third of the base. Note that points of application for the resultant forces of the cases without the clay surcharge are located to the right of the center of the footing. Thus the bearing pressure is greater at the heel than at the toe. However, when the clay surcharge was considered the resultant force shifted to the left of the center of gravity, except for Case 1-S.

Comparison with Measured Bearing Pressures

Without the clay surcharge. - The bearing pressure distributions for twenty-seven, forty-four, and fifty-one days following the start of backfilling are plotted in Fig. 17. These data represent the measured bearing pressures at each pressure cell location on a given day before the clay backfill was added.

Each bearing pressure distribution shown in Fig. 17 was integrated over the width of the footing to determine the resultant vertical force and its location relative to the back of the heel. The pressure was assumed to be distributed linearly between pressure cells. The pressure at the edges of the toe and heel were extrapolated by extending the assumed linear pressure distribution until it intersected a vertical plane through the edge of the toe and heel respectively. The point of application was assumed to be located at the centroid of the pressure distribution. The results of the integration are given in Table 11.

TABLE 11. - Results of Integration of Measured Bearing Pressures, without the Clay Surcharge

	Day 27	Day 44	Day 51
Resultant Force (k/ft)	9.36	9.01	8.20
Point of Application* (ft)	3.57	3.73	3.59

* Point of application is measured from the back of the heel.

NOTE: 1 k/ft = 14.59 kN/m; 1 ft = 0.305 m

Comparing the results given in Table 11 and Table 10 shows that the resultant force of the measured bearing pressures is slightly less than the calculated resultant forces. The average ratio of the

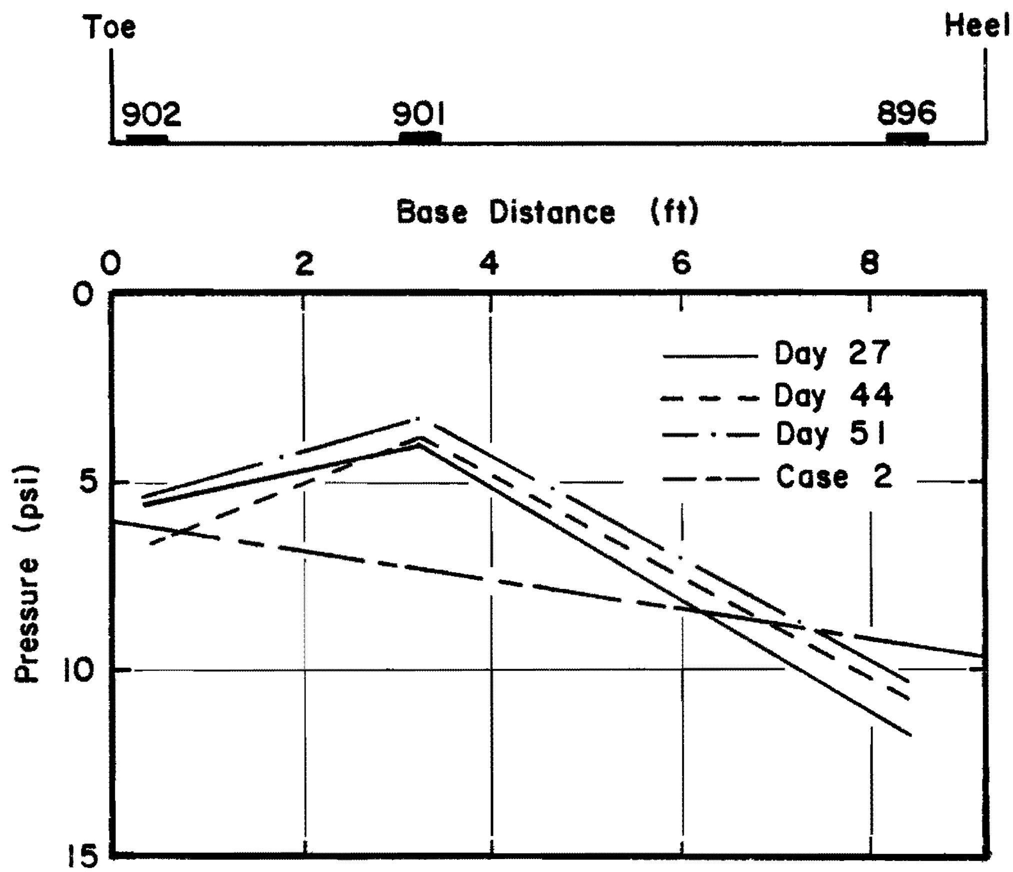


FIG. 17.— Bearing Pressure Distributions, without the Clay Surcharge (1 ft = 0.305 m; 1 psi = 6.9 kN/m^2)

resultants of the measured bearing pressures to the calculated pressures for Case 2 is 0.87. The points of application of the resultant forces from the measured pressures are within the middle third and compare reasonably well with the calculated values. It should be noted that the measured bearing pressures as well as the calculated bearing pressures show a higher intensity at the heel than at the toe. This is in agreement with the plumb bob readings which indicate that the wall tended to rotate or tilt toward the backfill during the sand backfilling operation and after completion.

A possible explanation of why the resultant forces of the measured pressures are less than the calculated resultant can be obtained by examining the measured bearing pressures shown in Fig. 17. The calculated pressure distribution for Case 2 is plotted in Fig. 17 since it is a result of using the average engineering properties of the sand backfill. The pressures measured by Cell Nos. 902 and 896 compare reasonably well with the calculated pressures. However, the pressure measured by Cell No. 901 is considerably less than the calculated pressure. Due to the wall movement, the bearing pressure could be concentrated on the bottom of the key causing less bearing pressure on the base of the footing adjacent to the key. Since Cell No. 901 is located near the key, this could result in lower pressure measurements for this cell.

<u>With the clay surcharge</u>. - The data presented in Fig. 18 represent the measured bearing pressure at each pressure cell location on a given day after the clay surcharge was added. The bearing pressure distributions have been plotted for 350, 362, and 385 days after the

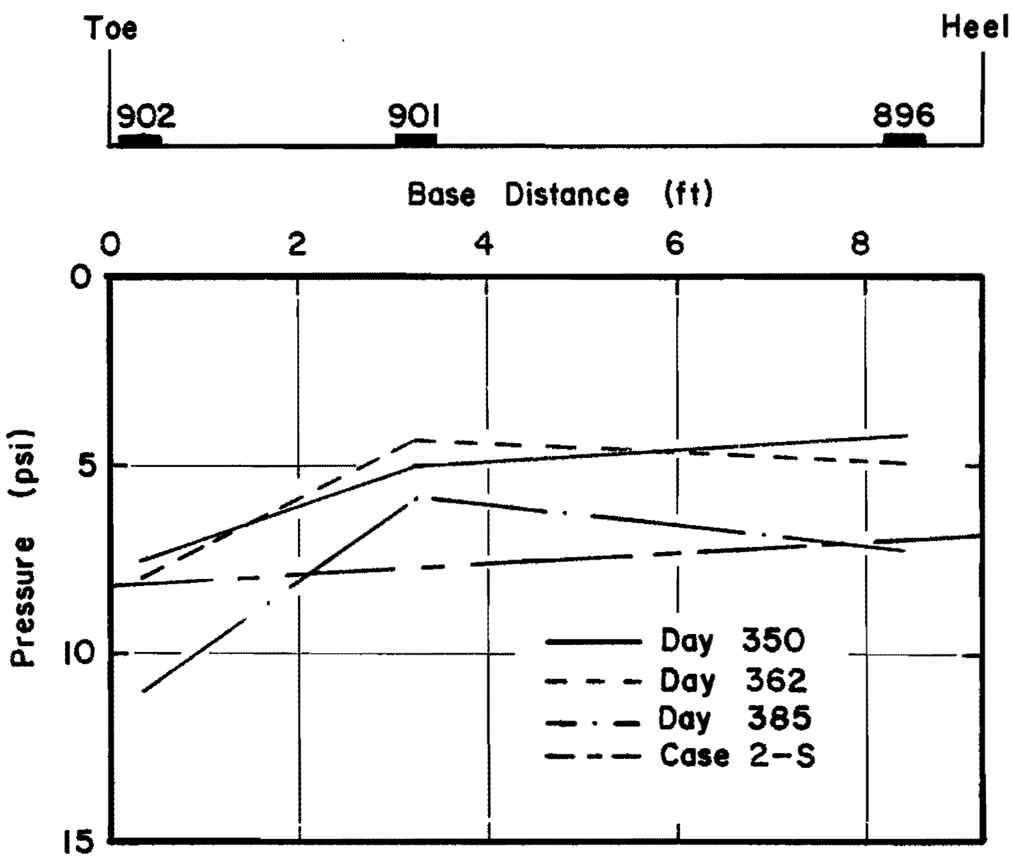


FIG. 18.— Bearing Pressure Distributions, with the Clay Surcharge (1 ft = 0.305 m; 1 psi = 6.9 kN/m^2)

start of backfilling.

Each bearing pressure distribution shown in Fig. 18 was integrated over the width of the footing to determine the resultant force and its point of application. The same assumptions and procedure for extrapolating the bearing pressures at the edges of the toe and heel as previously stated were used to perform the integration. The results of the integration are given in Table 12.

TABLE 12. - Results of Integration of Measured Bearing Pressures, with the Clay Surcharge

	Day 350	Day 362	Day 385
Resultant Force (k/ft)	6.93	6.99	9.77
Point of Application (ft)	5.11	5.00	4.93

* Point of application is measured from the back of the heel.

NOTE: 1 k/ft = 14.59 kN/m; 1 ft = 0.305 m

Comparing the results given in Table 12 and Table 10 shows that the resultant of the measured bearing pressures is less than the calculated resultants. The average ratio of the resultant of the measured bearing pressures to the calculated pressures for Case 2-S is 0.81. This ratio compares closely with the ratio of 0.87 for the case without the clay surcharge.

The point of application of the resultant force calculated from the measured pressures is located within the middle third of the base and compares closely with the calculated values. It should be noted that the measured bearing pressures have shifted and indicate a larger bearing pressure at the toe than at the heel. This agrees well with the calculated bearing pressure distribution. This shift in bearing pressure is also in agreement with the inclinometer readings which indicate that the wall is tending to rotate or tilt away from the backfill since the clay surcharge has been added (see Fig. 14).

A possible explanation of why the resultant force of the measured pressures is less than the calculated resultant can be obtained by examining the measured bearing pressures shown in Fig. 18. The bearing pressures plotted for days 350 and 362 show that the pressures measured by Cell No. 902 agree very closely with the calculated bearing pressures. However, the pressures measured by Cell Nos. 901 and 896 are considerably less than the calculated bearing pressures. However, all the measured bearing pressures on day 385 agree reasonably well with the calculated bearing pressure. The fact that these pressures agree reasonably well could be an indication that the wall has stabilized and no more movement will occur under the present loading. Additional measurement will be necessary for verification.

The Effect of the Key

One of the purposes of this research program is to determine if a spread footing foundation with a protruding key could be used as effectively in some conditions where current SDHPT criteria require a foundation supported on drilled shafts or piling. The SDHPT has developed a complete set of cantilever retaining wall designs which are capable of being adapted to the majority of wall requirements

(12). Depending on the wall height requirements for a particular job, a standard wall design type is chosen from one of ten standardized designs. Then the type of foundation (spread footing with a protruding key, piling, or drilled shafts) is selected based on four classifications for soil bearing capacity requirements. The use of the spread footing foundation with a protruding key would be more economical than a foundation on drilled shafts or piling.

The data collected to date are not sufficient to make any definite conclusion or recommendation as to the effectiveness of the key. However, some observations can be made from the data thus far collected. The measured pressures for Cell No. 900, located on the vertical face of the key, are given in Table 13 for the days previously discussed.

TABLE 13. - Measured Pressures for the Pressure Cell Located on the Key.

	Without Clay Surcharge			With Clay Surcharge		
Day	27	44	51	350	362	385
Measured Pressure (psi)	N.R.*	1.9	2.2	4.9	5.5	7.5

* No reading obtained due to a leak in one of the pressure lead connectors.

NOTE: 1 psi = 6.9 kN/m^2

The data presented in Table 13 show that there was a substantial increase in pressure after the clay surcharge was added. Thus it appears movement of the key against the soil caused an increase in the passive resistance to sliding. Further observations can be made by examining Fig. 11c. The data show that the pressure exerted on Cell No. 900 increased as the sand backfill was placed. This reaction is expected, since the other data discussed indicate that the retaining wall rotated toward the backfill. If the wall is assumed to be a totally rigid structure, a rotation toward the backfill would cause the front of the key to push against the soil thereby increasing the pressure exerted against it. After the completion of the sand backfilling, the pressure remained reasonably constant until the clay backfilling was started. The pressure increased slightly, then gradually decreased, and is presently increasing at a fairly rapid rate. These data, as well as the other data previously discussed, indicate that the wall rotated away from the backfill as the clay backfill was added which would explain the gradual decrease in pressure. The increase in pressure that the cell is currently exhibiting indicates that the wall could be translating away from the backfill. This cannot be substantiated because site conditions prevented translation measurements during this time.

The key's true effectiveness cannot be fully evaluated at this time, since data cannot be reported for the two other pressure cells installed on the front of the wall and footing. As stated previously, Cell No. 898 (located on the front of the stem) had no pressure applied to it because no backfill has been placed in front of the wall where this pressure cell is located. Also, Cell No. 903 (located on the vertical face of the toe) became inoperative after day 86, and before that day it was subjected to very high pressures from the stockpiled sheet piling. When the front of the wall is backfilled, and Cell No. 898 begins to respond to pressures exerted on it, a more conclusive analysis can be performed on the effectiveness of the key to increase passive resistance to sliding.

It should be noted that the bearing pressure measurements indicate that the key could be affecting the distribution of bearing pressures. Future research involving a spread footing foundation with a protruding key should investigate the key's effect on the distribution of bearing pressures. This investigation could be conducted by installing a pressure cell horizontally in the base of the key.

SUMMARY AND RECOMMENDATIONS

Summary

The following is a summary of the significant aspects of the study concerning long-term field measurements of earth pressures on a cantilever retaining wall at the end of the third year.

1. A standard cantilever retaining wall on a spread footing foundation with a protruding key was instrumented with twelve Terra Tec pneumatic earth pressure cells. The measured pressures showed some seasonal variation which was attributed to temperature changes. A temperature correction was applied which removed some of the seasonal variation. Eight of the original twelve pressure cells are currently operating properly.
2. Lateral earth pressure measurements were made for 385 days after the start of backfilling. The average ratio of the resultant thrust as determined from the measured lateral pressures to the thrust determined from a Culmann graphical solution is approximately 3.5, both before and after the clay backfill was added. The higher measured lateral pressures can be explained by considering the movement of the wall. During the sand backfilling, the wall rotated or tilted toward the backfill which should create higher pressures. During the clay backfilling, the rotation was away from the backfill; but, there was very little movement in the lower part of the wall where the higher pressures

were measured. Another possible explanation for the higher measured pressures in the lower part of the wall could be the dead zone phenomenon. The self weight of dead zone material may be adding some additional pressures to the back of the wall. Also, since the material within the dead zone moves with the wall as if it were part of the wall, pressures closer to the at-rest condition could be measured against the lower part of the wall.

3. Bearing pressure measurements were made for 385 days after the start of backfilling. The average ratio of the resultant as determined from the measured bearing pressures to the calculated resultant was approximately 0.84, both before and after the clay backfill was added. The points of application of the resultant forces of all the measured bearing pressures were located within the middle third of the base of the footing. The calculated and measured bearing pressures showed a higher intensity at the heel than at the toe after the sand backfill was placed. However, after the clay backfill was added, the calculated and measured bearing pressures showed a higher intensity at the toe than at the heel. The measured bearing pressure distributions agreed reasonably well with the calculated bearing pressure distributions.

4. Measurements for tilt or rotation of the wall were made. Provisions were made to measure translation movement; however, after day 27 sheet piling stockpiled in front of the

wall prevented this measurement from being made. The plumb bob readings indicate that the wall tended to rotate toward the backfill during the placement of the sand backfill. This movement explains the measured bearing pressure distribution when only the sand backfill was considered. After day 148 site conditions prevented the measurements with the plumb bob apparatus. The inclinometer readings indicate that the wall rotated or tilted away from the backfill after the clay backfill was placed. This movement explains the shift in the higher pressure intensity from the heel to the toe.

5. There are not sufficient data to fully evaluate the effectiveness of the key. However, the data collected thus far indicate that the key caused an increase in passive resistance after the clay backfill was added. Also, the measured bearing pressures indicate that the key could be affecting the distribution of bearing pressures along the base of the footing.

6. The engineering properties of the backfill material were determined. A wide range in unit weights and void ratio existed in the backfill. The backfill is a fine-grained, subrounded to rounded, quartz sand. The average total unit weight is approximately 117 pcf (18.4 kN/m^3). Its gradation and plasticity were such that it was classified as a SP-SM according to the Unified Soil Classification System. The effective angle of internal shearing resistance for the average void ratio was 39^0.

7. The soil conditions at the site were investigated by taking three borings. Two borings were performed by the SDHPT and tests on undisturbed samples were performed at the SDHPT laboratory. Another boring was drilled by the SDHPT drilling rig and the undisturbed samples were tested at the Texas A&M University laboratory. Two of the borings were supplemented with TCP tests at various depths.

Recommendations

The following recommendations are made concerning the research accomplished thus far and continued research in this program.

1. Continue measuring the earth pressures and wall movement. The effects of vehicular traffic on the measured pressures should be studied when this section of the highway is open.
2. The plumb bob and hook-to-reference-point measurements should be continued as soon as site conditions permit.
3. The comparison between measured field pressures and theoretical pressures should continue to be made so that the overall objective of verifying or modifying the existing retaining wall design procedures can be accomplished.
4. Any future instrumentation of retaining walls using Terra Tec pneumatic pressure cells should include the installation of thermocouples adjacent to the pressure cells.
5. Thicker plates should be mounted on the front of the wall for inclinometer measurements to try and eliminate warping of the plates. Also, instead of using epoxy to mount the plates on

the wall, they should be bolted to the wall for the purpose of eliminating any temperature effects on the epoxy.

6. Provisions should be made to measure settlement of the retaining wall.
7. Alternative provisions should be made for obtaining translation measurements in case obstructions prevent the use of the prime method.
8. Any future instrumentation of retaining walls with a protruding key should include a pressure cell mounted horizontally in the base of the key to examine the key's effect on the distribution of bearing pressures.
9. The feasibility of installing several additional pressure cells in the backfill material, within the Rankine active zone, should be investigated.

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APPENDIX II. - NOTATION

b = width of footing (ft),

C_T = temperature correction factor (psi/^oF),

e = eccentricity of the resultant vertical force (ft),

P_c = corrected pressure (psi),

P_i = gage pressure measured on a particular day (psi),

P_o = field-zero pressure (psi),

q = bearing pressure (psi),

T_i = estimated temperature for the day P_i was measured (^oF),

T_o = estimated temperature for the day P_o was measured (^oF),

and V = resultant vertical force (lb)

APPENDIX III. - TEMPERATURE CORRECTION

As stated in the section on data collection, the measured pressures showed a great deal of variation which was attributed to seasonal changes in temperature. During an earlier research study, three retaining walls in Houston, Texas, were instrumented with both Terra Tec pneumatic pressure cells and thermocouples (4, 11). These three retaining walls were instrumented with a total of fifteen Terra Tec pressure cells. The temperature and pressure data collected during this research study were used to develop a temperature correction. Note that only the pressure and temperature measurements obtained after the retaining walls were completely backfilled and no significant wall movement or activity behind the wall occurred were considered for the temperature correction.

A plot of measured pressure versus measured temperature was made for each cell. A typical plot is shown in Fig. 19. The slope of a best fit line using a least squares analysis was determined for each cell. The average slope was determined to be 0.072 psi/^{o}F (0.89 kN m^{-2}/^{o}C). This value is termed the temperature correction factor.

In order to apply the temperature correction factor a reference temperature for the field zero values and each pressure measurement is needed. A curve with temperature plotted versus the days of the year was developed from the temperature data collected at the three retaining walls over a four year period (July 1, 1971 through June 25, 1975). All the temperature readings obtained at a particular site on a particular day were averaged. These average temperatures were plotted versus the day on which they were obtained. The data

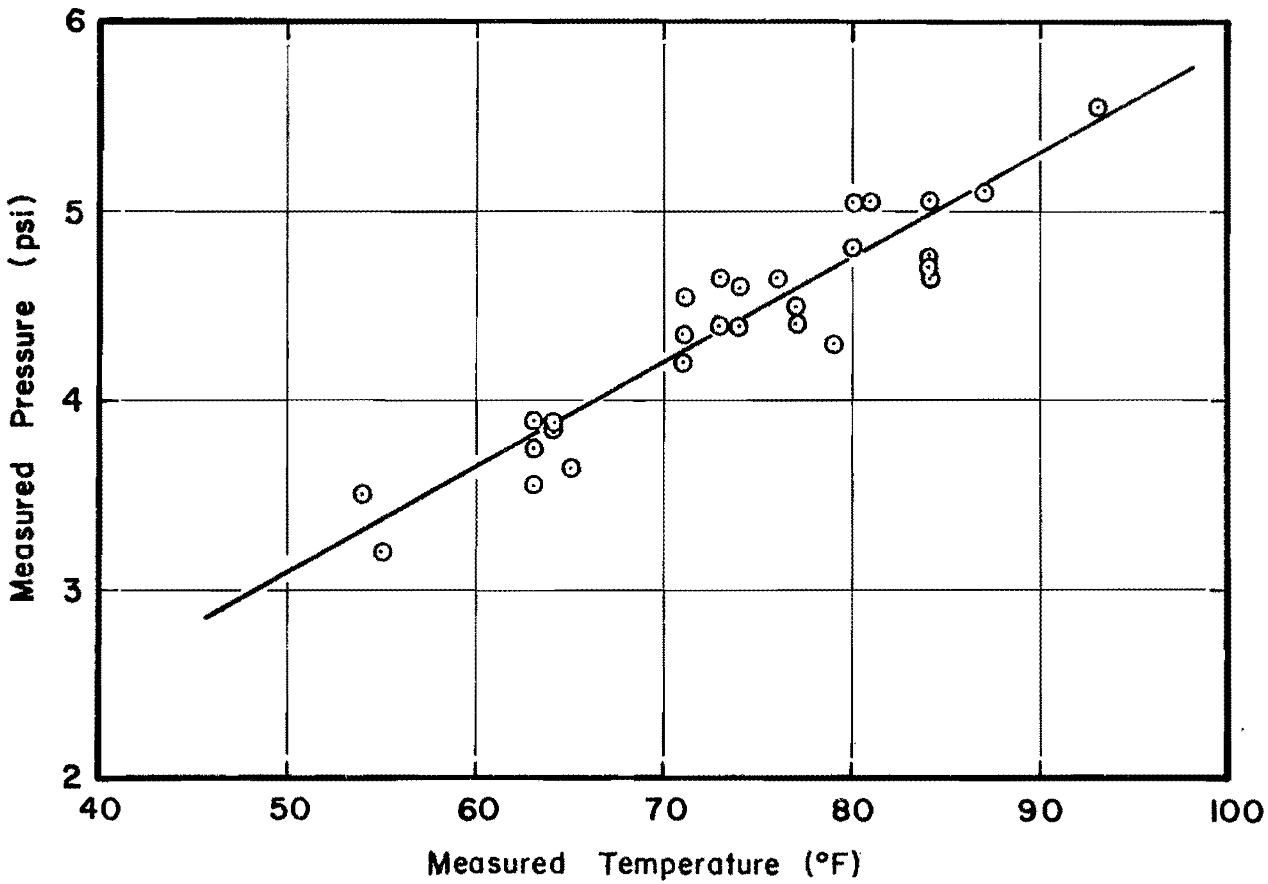


FIG. 19.— Measured Temperature Versus Measured Pressure ($°C = 0.5556(°F-32)$; $1 \text{ psi} = 6.9 \text{ kN/m}^2$)

were plotted as shown in Fig. 20. The reference temperatures for the field zero values were established by entering Fig. 20 with the date when the field zero value was measured and reading the estimated temperature for that day. Then for each day pressure measurements were made a temperature was estimated using the curve in Fig. 20. Then the corrected pressure was computed using the following equation:

$$P_c = P_i - P_o - C_T(T_i - T_o) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

where P_c = corrected pressure (psi),

P_i = gage pressure measured on a particular day (psi),

P_o = field-zero pressure (psi),

T_i = estimated temperature for the day P_i was measured (^oF),

T_o = estimated reference temperature for the day P_o was measured (^oF), and

C_T = temperature correction factor (0.072 psi/^oF).

The corrected pressures for each cell are given in Tables 14 through 23. It should be noted that most of the pressure cells showed negative pressures even after the temperature correction was applied. A possible explanation for the pressure cells on the back of the stem showing negative pressures can be obtained by considering when the zero values for these cells were measured. The field zero values for the cells on the back of the stem were obtained during the summer with no backfill present. Since these cells were exposed to the ambient temperature at this time, the reference temperature could be considerably higher than the estimated temperature from Fig. 20.

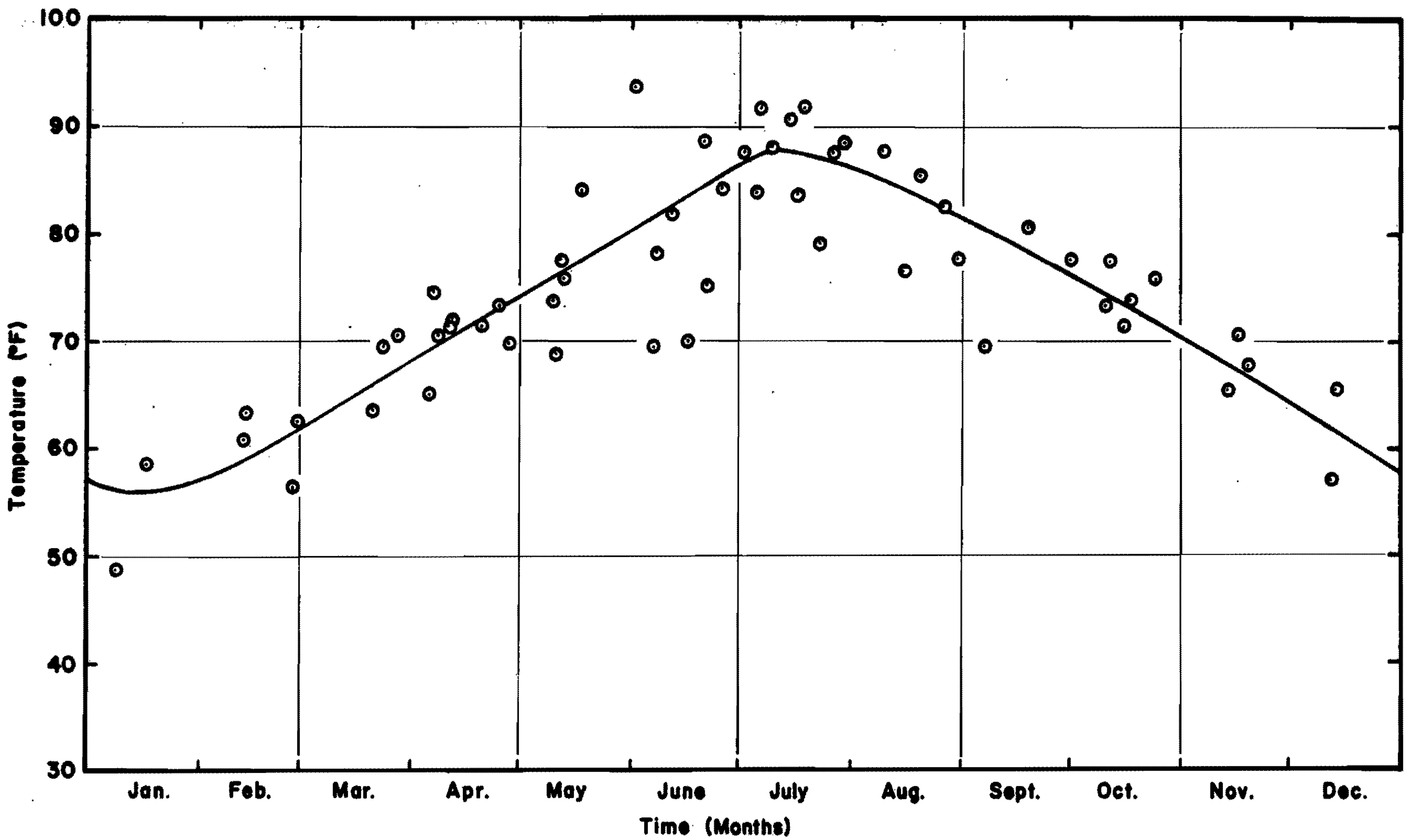


FIG. 20.— Time Versus Temperature (°C = 0.5556(°F−32))

TABLE 14. - Corrected Pressures for Cell No. 896 (T_o = 83°F)

Date	Day	Measured Pressure (psi)	Estimated Temperature (°F)	Field Zero (psi)	Temperature Correction (psi)	Corrected Pressure (psi)
6/20/79	0	16.0	84	10.0	- 0.07	+ 5.9
6/26/79	6	16.7	86	10.0	- 0.22	+ 6.5
6/28/79	8	16.5	86	10.0	- 0.22	+ 6.3
7/09/79	19	16.6	88	10.0	- 0.36	+ 6.2
7/12/79	22	18.9	88	10.0	- 0.36	+ 8.5
7/13/79	23	20.8	88	10.0	- 0.36	+ 10.4
7/17/79	27	22.0	87.5	10.0	- 0.32	+ 11.7
8/03/79	44	21.0	85.5	10.0	- 0.18	+ 10.8
8/10/79	51	20.4	85	10.0	- 0.14	+ 10.3
8/15/79	56	21.4	84	10.0	- 0.07	+ 11.3
8/23/79	64	22.2	84	10.0	- 0.07	+ 12.1
8/29/79	70	21.5	82	10.0	+ 0.07	+ 11.6
9/04/79	86	21.5	80	10.0	+ 0.22	+ 11.7
10/12/79	114	18.8	74	10.0	+ 0.65	+ 9.4
11/02/79	135	19.2	70	10.0	+ 0.94	+ 10.1
11/15/79	148	17.3	67.5	10.0	+ 1.12	+ 8.4
1/09/80	203	13.0	56.5	10.0	+ 1.91	+ 4.9
3/07/80	261	9.4	64	10.0	+ 1.37	+ 0.8
4/01/80	286	10.9	68	10.0	+ 1.08	+ 2.0
4/22/80	307	11.1	72.5	10.0	+ 0.76	+ 1.9
5/22/80	337	13.0	78.5	10.0	+ 0.32	+ 3.3
6/04/80	350	14.1	81	10.0	+ 0.14	+ 4.2
6/16/80	362	14.9	83.5	10.0	- 0.04	+ 4.9
7/09/80	385	17.5	88	10.0	- 0.35	+ 7.2

TABLE 15. - Corrected Pressures for Cell No. 881 (T_o = 83°F)

Date	Day	Measured Pressure (psi)	Estimated Temperature (°F)	Field Zero (psi)	Temperature Correction (psi)	Corrected Pressure (psi)
6/20/79	0	12.9	84	10.9	- 0.07	+ 1.9
6/26/79	6	13.2	86	10.9	- 0.22	+ 2.1
6/28/79	8	13.0	86	10.9	- 0.22	+ 1.9
7/09/79	19	13.3	88	10.9	- 0.36	+ 2.0
7/12/79	22	13.5	88	10.9	- 0.36	+ 2.2
7/13/79	23	13.6	88	10.9	- 0.36	+ 2.3
7/17/79	27	13.5	87.5	10.9	- 0.32	+ 2.3
8/03/79	44	13.1	85.5	10.9	- 0.18	+ 2.0
8/10/79	51	13.0	85	10.9	- 0.14	+ 2.0
8/15/79	56	12.5	84	10.9	- 0.07	+ 1.5
8/23/79	64	12.5	84	10.9	- 0.07	+ 1.5
8/29/79	70	12.5	82	10.9	+ 0.07	+ 1.7
9/04/79	86	11.9	80	10.9	+ 0.22	+ 1.2
10/12/79	114	10.6	74	10.9	+ 0.65	+ 0.3
11/02/79	135	10.0	70	10.9	+ 0.94	+ 0.0
11/15/79	148	8.9	67.5	10.9	+ 1.12	- 0.9
1/09/80	203	7.8	56.5	10.9	+ 1.91	- 1.2

TABLE 16. - Corrected Pressures for Cell No. 892 (T_o = 82°F)

Date	Day	Measured Pressure (psi)	Estimated Temperature (°F)	Field Zero (psi)	Temperature Correction (psi)	Corrected Pressure (psi)
6/20/79	0	11.4	84	9.4	- 0.14	+ 1.9
6/26/79	6	9.0	86	9.4	- 0.29	- 0.7
6/28/79	8	10.9	86	9.4	- 0.29	+ 0.3
7/09/79	19	10.9	88	9.4	- 0.43	+ 1.1
7/12/79	22	10.0	88	9.4	- 0.43	+ 0.2
7/13/79	23	8.7	88	9.4	- 0.43	- 1.1
7/17/79	27	10.2	87.5	9.4	- 0.40	+ 0.4
8/03/79	44	10.0	85.5	9.4	- 0.25	+ 0.3
8/10/79	51	9.7	85	9.4	- 0.22	+ 0.1
8/15/79	56	9.5	84	9.4	- 0.14	0.0
8/23/79	64	8.4	84	9.4	- 0.14	- 1.1
8/29/79	70	8.6	82	9.4	0.00	- 0.8
9/04/79	86	7.6	80	9.4	+ 0.14	- 1.7
10/12/79	114	5.0	74	9.4	+ 0.58	- 3.8
11/02/79	135	2.1	70	9.4	+ 0.86	- 6.4
11/15/79	148	N.R.	67.5	9.4	-	N.R.
1/09/80	203	N.R.	56.5	9.4	-	N.R.
3/07/80	261	3.0	64	9.4	+ 1.30	- 5.1
4/01/80	286	3.9	68	9.4	+ 1.01	- 4.5
4/22/80	307	6.9	72.5	9.4	+ 0.68	- 1.8
5/22/80	337	10.0	78.5	9.4	+ 0.25	+ 0.9
6/04/80	350	12.4	81	9.4	+ 0.07	+ 3.1
6/16/80	362	13.4	83.5	9.4	- 0.11	+ 3.9

TABLE 17. - Corrected Pressures for Cell No. 900 (T_o = 83°F)

Date	Day	Gage Press (psi)	Estimated Temperature (°F)	Field Zero (psi)	Temperature Corrected (psi)	Indicated Press (psi)
6/20/79	0	3.0	84	6.6	- 0.07	- 3.7
6/26/79	6	6.8	86	6.6	- 0.22	0.0
6/28/79	8	6.5	86	6.6	- 0.22	- 0.3
7/09/79	19	7.1	88	6.6	- 0.36	+ 0.1
7/12/79	22	7.7	88	6.6	- 0.36	+ 0.7
7/13/79	23	7.8	88	6.6	- 0.36	+ 0.8
7/17/79	27	N.R.	87.5	6.6	- 0.32	N.R.
8/03/79	44	8.7	85.5	6.6	- 0.18	+ 1.9
8/10/79	51	8.9	85	6.6	- 0.14	+ 2.2
8/15/79	56	9.0	84	6.6	- 0.07	+ 2.3
8/23/79	64	8.9	84	6.6	- 0.07	+ 2.2
8/29/79	70	3.6	82	6.6	- 0.07	+ 2.1
9/04/79	86	8.3	80	6.6	+ 0.22	+ 1.9
10/12/79	114	7.2	74	6.6	+ 0.65	+ 1.2
11/02/79	135	9.8	70	6.6	+ 0.94	+ 4.1
11/15/79	148	8.0	67.5	6.6	+ 1.12	+ 2.5
1/09/80	203	5.1	56.5	6.6	+ 1.91	+ 0.4
3/07/80	261	4.5	64	6.6	+ 1.37	- 0.7
4/01/80	286	6.3	68	6.6	+ 1.08	+ 0.8
4/22/80	307	7.1	72.5	6.6	+ 0.76	+ 1.3
5/22/80	337	9.8	78.5	6.6	+ 0.32	+ 3.5
6/04/80	350	11.4	81	6.6	+ 0.14	+ 4.9
6/16/80	362	12.1	83.5	6.6	- 0.04	+ 5.5
7/09/80	385	14.4	88	6.6	- 0.35	+ 7.5

TABLE 18. - Corrected Pressures for Cell No. 901 (T_o = 83°F)

Date	Day	Measured Pressure (psi)	Estimated Temperature (°F)	Field Zero (psi)	Temperature Correction (psi)	Corrected Pressure (psi)
6/20/79	0	12.3	84	11.1	- 0.07	+ 1.1
6/26/79	6	13.1	86	11.1	- 0.22	+ 1.8
6/28/79	8	12.6	86	11.1	- 0.22	+ 1.3
7/09/79	19	12.6	88	11.1	- 0.36	+ 1.3
7/12/79	22	13.4	88	11.1	- 0.36	+ 1.9
7/13/79	23	14.2	88	11.1	- 0.36	+ 2.7
7/17/79	27	15.4	87.5	11.1	- 0.32	+ 4.0
8/03/79	44	15.1	85.5	11.1	- 0.18	+ 3.8
8/10/79	51	14.5	85	11.1	- 0.14	+ 3.3
8/15/79	56	13.8	84	11.1	- 0.07	+ 2.6
8/23/79	64	13.5	84	11.1	- 0.07	+ 2.3
8/29/79	70	13.5	82	11.1	+ 0.07	+ 2.5
9/14/79	86	13.1	80	11.1	+ 0.22	+ 2.2
10/12/79	114	11.6	74	11.1	+ 0.65	+ 1.1
11/02/79	135	12.8	70	11.1	+ 0.94	+ 2.6
11/15/79	148	10.3	67.5	11.1	+ 1.12	+ 0.3
1/09/80	203	7.1	56.5	11.1	+ 1.91	- 2.1
3/07/80	261	10.5	64	11.1	+ 1.37	+ 0.8
4/01/80	286	10.5	68	11.1	+ 1.08	+ 0.5
4/22/80	307	11.5	72.5	11.1	+ 0.76	+ 1.2
5/22/80	337	14.3	78.5	11.1	+ 0.32	+ 3.5
6/04/80	350	16.0	81	11.1	+ 0.14	+ 5.0
6/16/80	362	15.4	83.5	11.1	- 0.04	+ 4.3
7/09/80	385	17.2	88	11.1	- 0.35	+ 5.8

TABLE 19. - Corrected Pressures for Cell No. 902 (T_o = 83°F)

Date	Day	Measured Pressure (psi)	Estimated Temperature (°F)	Field Zero (psi)	Temperature Correction (psi)	Corrected Temperature (psi)
6/20/79	0	11.7	84	11.2	- 0.07	+ 0.4
6/26/79	6	13.3	86	11.2	- 0.22	+ 1.9
6/28/79	8	12.9	86	11.2	- 0.22	+ 1.5
7/09/79	19	12.4	88	11.2	- 0.36	+ 0.8
7/12/79	22	13.2	88	11.2	- 0.36	+ 1.6
7/13/79	23	15.0	88	11.2	- 0.36	+ 3.4
7/17/79	27	17.2	87.5	11.2	- 0.32	+ 5.7
8/03/79	44	18.1	85.5	11.2	- 0.18	+ 6.7
8/10/79	51	16.7	85	11.2	- 0.14	+ 5.4
8/15/79	56	17.6	84	11.2	- 0.07	+ 6.3
8/23/79	64	17.4	84	11.2	- 0.07	+ 6.1
8/29/79	70	16.6	82	11.2	+ 0.07	+ 5.5
9/04/79	86	15.2	80	11.2	+ 0.22	+ 4.2
10/12/79	114	13.8	74	11.2	+ 0.65	+ 3.2
11/02/79	135	19.1	70	11.2	+ 0.94	+ 8.8
11/15/79	148	13.7	67.5	11.2	+ 1.12	+ 3.6
1/09/80	203	8.7	56.5	11.2	+ 1.91	- 0.6
3/07/80	261	7.9	64	11.2	+ 1.37	- 1.9
4/01/80	286	11.7	68	11.2	+ 1.08	+ 1.6
4/22/80	307	12.5	72.5	11.2	+ 0.76	+ 2.1
5/22/80	337	17.0	78.5	11.2	+ 0.32	+ 6.1
6/04/80	350	18.6	81	11.2	+ 0.14	+ 7.5
6/16/80	362	19.2	83.5	11.2	- 0.04	+ 8.0
7/09/80	385	22.5	88	11.2	- 0.35	+11.0

TABLE 20. - Corrected Pressures for Cell No. 903 (T_o = 83°F)

Date	Day	Measured Pressure (psi)	Estimated Temperature (°F)	Field Zero (psi)	Temperature Correction (psi)	Corrected Pressure (psi)
6/20/79	0	18.0	84	9.5	- 0.07	+ 8.4
6/26/79	6	18.8	86	9.5	- 0.22	+ 9.1
6/28/79	8	18.6	86	9.5	- 0.22	+ 8.9
7/09/79	19	18.5	88	9.5	- 0.36	+ 8.6
7/12/79	22	18.5	88	9.5	- 0.36	+ 8.6
7/13/79	23	18.6	88	9.5	- 0.36	+ 8.7
7/17/79	27	19.2	87.5	9.5	- 0.32	+ 9.4
8/03/79	44	42.8	85.5	9.5	- 0.18	+ 33.1
8/10/79	51	41.3	85	9.5	- 0.14	+ 31.7
8/15/79	56	40.4	84	9.5	- 0.07	+ 30.8
8/23/79	64	39.5	84	9.5	- 0.07	+ 29.9
8/29/79	70	39.4	82	9.5	+ 0.07	+ 30.0
9/04/79	86	38.9	80	9.5	+ 0.22	+ 29.6

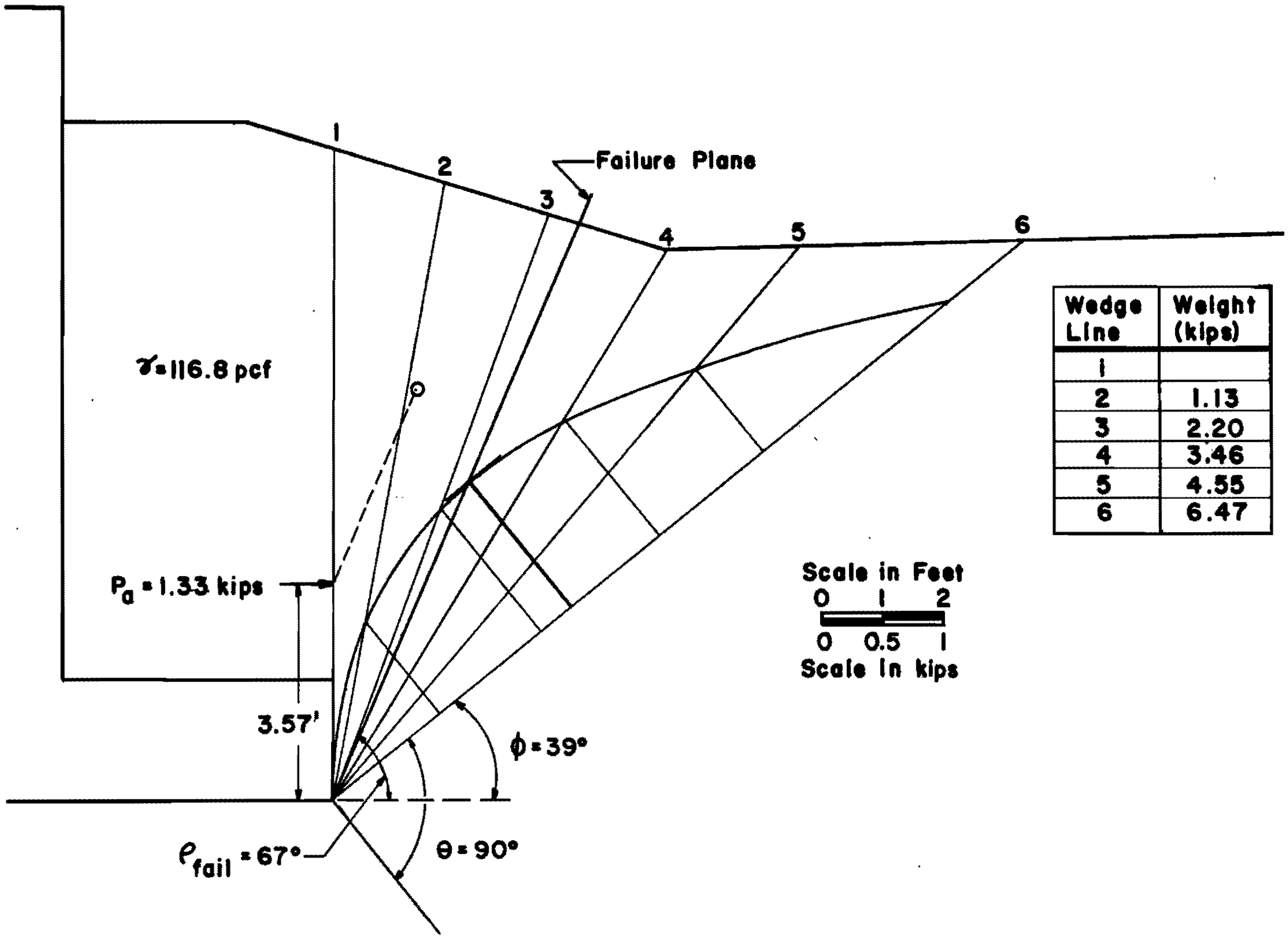
TABLE 21. - Corrected Pressures for Cell No. 907 (T_o = 82°F)

Date	Day	Measured Pressure (psi)	Estimated Temperature (°F)	Field Zero (psi)	Temperature Correction (psi)	Corrected Pressure (psi)
6/20/79	0	13.9	84	9.2	- 0.14	+ 4.6
6/26/79	6	11.4	86	9.2	- 0.29	+ 1.9
6/28/79	8	12.0	86	9.2	- 0.29	+ 2.5
7/09/79	19	12.7	88	9.2	- 0.43	+ 3.1
7/12/79	22	12.4	88	9.2	- 0.43	+ 2.8
7/13/79	23	10.5	88	9.2	- 0.43	+ 0.9
7/17/79	27	11.1	87.5	9.2	- 0.40	+ 1.5
8/03/79	44	10.3	85.5	9.2	- 0.25	+ 0.8
8/10/79	51	10.0	85	9.2	- 0.22	+ 0.6
8/15/79	56	9.2	84	9.2	- 0.14	- 0.1
8/23/79	64	7.3	84	9.2	- 0.14	- 2.0
8/29/79	70	7.8	82	9.2	0.00	- 1.4
9/04/79	86	8.0	80	9.2	+ 0.14	- 1.1
10/12/79	114	5.6	74	9.2	+ 0.58	- 3.0
11/02/79	135	2.0	70	9.2	+ 0.86	- 6.3
11/15/79	148	N.R.	67.5	9.2	0.00	N.R.
1/09/80	203	2.8	56.5	9.2	+ 1.84	- 4.6
3/07/80	261	5.2	64	9.2	+ 1.30	- 2.7
4/01/80	286	6.1	68	9.2	+ 1.01	- 2.1
4/22/80	307	7.3	72.5	9.2	+ 0.68	- 1.2
5/22/80	337	7.2	78.5	9.2	+ 0.25	- 1.7
6/04/80	350	9.5	81	9.2	+ 0.07	+ 0.4
6/16/80	362	12.2	83.5	9.2	- 0.11	+ 2.9
7/09/80	385	12.5	88	9.2	- 0.42	+ 2.9

However, the negative pressures reported for the cells in the footing cannot be explained at this time. All negative pressures have been plotted as zero pressure which is the lowest pressure that realistically could have been measured.

APPENDIX IV. - EXAMPLE OF CULMANN SOLUTION

An example of Culmann graphical solution is shown in Fig. 21. The example presented is the solution of Case 2 which involves the average engineering properties of the sand backfill. This case is one of the solutions without the clay surcharge. All other Culmann solutions were performed in the same manner.



Wedge Line	Weight (kips)
1	
2	1.13
3	2.20
4	3.46
5	4.55
6	6.47

FIG. 21.— Culmann Solution for Case 2 (1 ft = 0.305 m; 1 pcf = 0.157 kN/m^3; 1 kip = 4.45 kN)

NOTES