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Pressure-Deformation Measurements In Earth

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Lateral Pressures on Retaining Walls Due to Backfill Surface Loads

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For many decades the traditional method of evaluating the lateral pressure on a retaining wall due to a load applied at the surface of the soil backfill has been to substitute a uniformly distributed load for the actual load, and then calculate the pressure by either the Rankine or the Coulomb classical theory. This method of approach to the problem has several shortcomings and disadvantages. First, there is no logical or scientific basis for determining the magnitude of the uniformly distributed load in relation to the actual load. Judgement and intuition are the only guides for this substitution. Second, the lateral pressure on the wall resulting from the substitution is of uniform intensity throughout the entire height of the structure, whereas the intensity of pressure due to the actual load may vary considerably throughout the height of the wall.

The Iowa Engineering Experiment Station conducted experimental research during the decade from 1931 to 1941 to determine the lateral pressure on a wall due to concentrated loads applied at the backfill surface and to uniformly distributed line or strip loads parallel to the wall. These studies indicated that the surface loads produced lateral pressures which were closely related to the pressures calculated by the Boussinesq theory of stress distribution in a semi-infinite elastic medium and provided a basis for further study of the influence of loads applied at the backfill surface.

More recently, under the sponsorship of the Iowa Highway Research Board and the Iowa Highway Commission, further studies have been conducted in which lateral pressures due to uniformly distributed loads over finite areas on the soil backfill have been measured. This paper contains a resume of the earlier research and a detailed presentation of data obtained in the recent studies, together with a correlation with the Boussinesq theory.

● THE quantitative determination of the magnitude and distribution of lateral pressures on retaining walls, caused by an earth backfill and by loads superimposed upon the surface of the backfill, is the first necessary step in the structural design of earth restraining structures of this type. The engineering profession has available an abundance of information, both scientific and empirical, relative to lateral pressures caused by an earth backfill. Acceptable and widely used techniques have been evolved throughout modern engineering history, based largely upon the scientific principles enunciated by Coulomb and Rankine and refined by experimental and analytical research by Baker, Feld, Terzaghi and many others.

On the other hand, lateral pressures due to loads superimposed on the surface of a backfill have received relatively little attention from researchers. Designers of retaining walls subjected to loads in this latter category are forced to rely, to a much greater extent, upon rule-of-thumb procedures and individual judgement and intuition. One widely used and more or less traditional approach to this problem is to substitute a uniformly distributed load on the surface of the backfill which is assumed to produce the same lateral pressure effect on the retaining wall as the actual load. Then this uniformly distributed load is converted to an equivalent additional height of fill above the top of the wall, and pressure on the wall due to the augmented fill height is calculated by conventional methods. This traditional procedure is illustrated in Figure 1.

There are a number of shortcomings and unsatisfactory features associated with this method of handling the problem. First, there is no scientific basis or guide to aid the designer in selecting a quantitative value of uniformly distributed load which will produce the same effect on the wall as the actual load. A decision on this point must be based solely upon judgement without the aid of well established criteria. Second, the lateral pressure on a wall resulting from this substitution of a uniformly distributed surface load is of uniform intensity throughout the entire height of the wall, regardless of the position or the shape and degree of concentration of the actual surface load. Research in recent years has indicated rather definitely that this pattern of pressure distribution does not coincide with fact.

In 1932 the Iowa Engineering Experiment Station began a series of experimental stud-

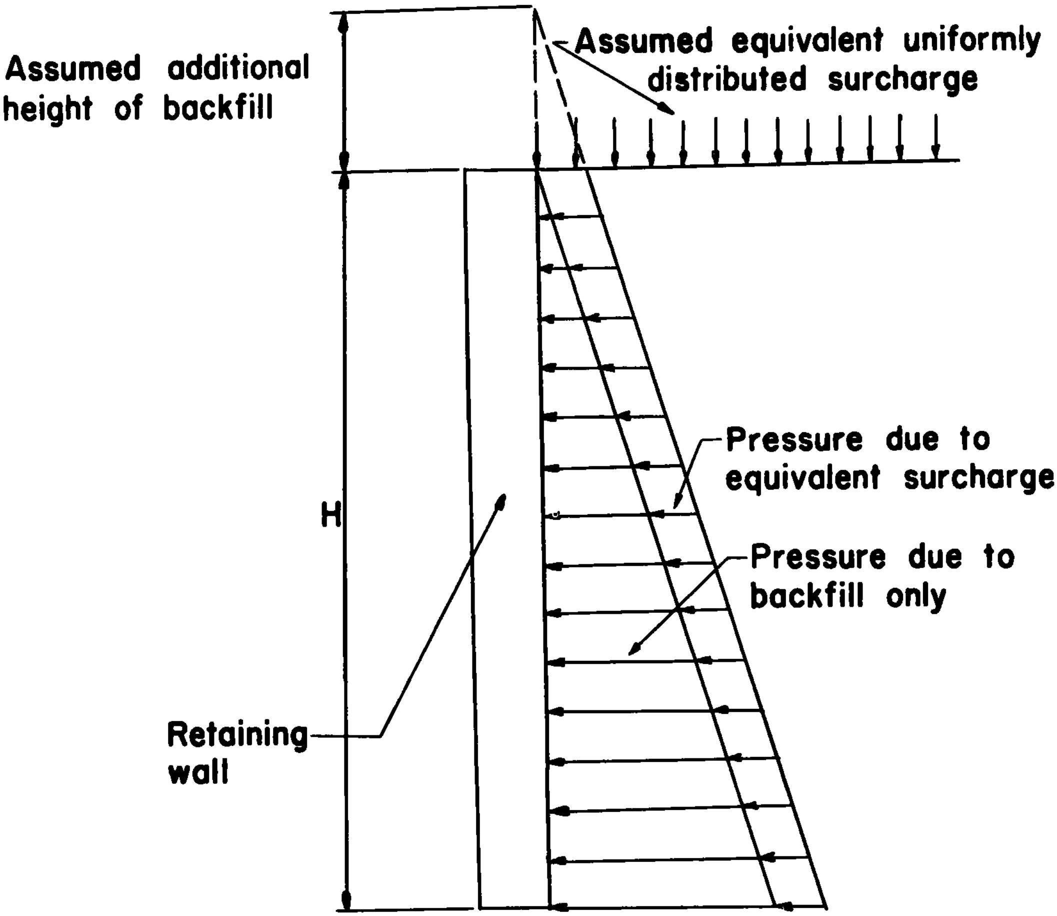

Figure 1. Traditional method of estimating lateral pressure due to surcharge load.

ies in which the magnitude and distribution of lateral pressures on a retaining wall caused by the application of a concentrated load on the surface of the backfill were measured. Later, this work was extended to include the pressures caused by a uniformly distributed line or strip load applied on the backfill surface and oriented parallel to the wall. The results of these studies were published in 1936 (3) and 1938 (4).

Still later, in 1939, a project was established in cooperation with the U. S. Bureau of Public Roads in which the lateral pressures on a retaining wall caused by a uniformly distributed load applied over a finite area on the surface of the backfill were to be measured. This project was scarcely started when shortages of labor and materials occasioned by the defense build-up prior to World War II began to be effective and little progress was made. Finally after the outbreak of the war, the project was suspended.

In 1951 the project was re-established in cooperation with the Iowa Highway Commission upon recommendation of the Iowa Highway Research Board. The purpose of this

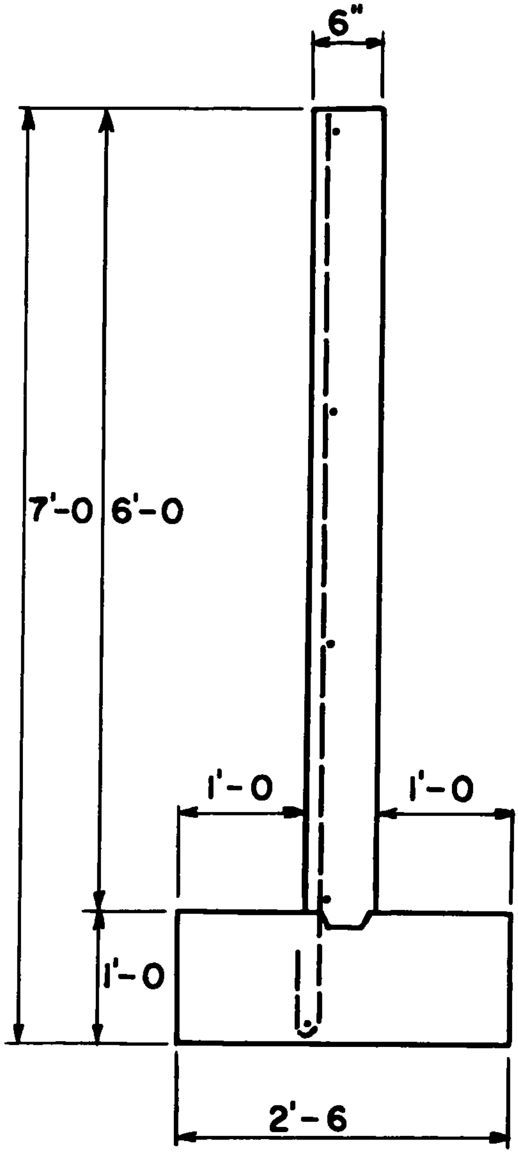

Figure 2. Cross section of experimental wall - 1932.

paper is to review the results of the measurements of lateral pressures caused by concentrated loads and uniformly distributed line loads on the backfill surface and to present in detail the data obtained in the more recent studies with uniformly distributed area loads.

All of these pressure measurements have been conducted on reinforced concrete retaining walls of cantilever design and T-shaped cross section. The experimental walls have been relatively rigid in character, that is, the yield of the wall has been relatively small in comparison with the deflection of the vartical plane in the backfill at the back face of the wall, if the soil mass had been continous without interruption by the restraining structure. The first experiments with concentrated loads and uniformly distributed line loads were conducted with retaining walls 6 feet high above the base, as shown in Figure 2.

The pressure measuring devices in the earlier experiments were of two types; stainless steel friction ribbons and Goldbeck pressure cells. They were installed flush with the vertical back face of the walls and were calibrated prior to placement of the soil backfill by means of an air pressure apparatus clamped on the wall successively over each pressure measuring unit.

The pressure caused by the soil backfill was first measured. Then concentrated loads of various magnitudes were placed on the backfill surface at various positions back of the wall and the total pressure measured. The difference between the total pressure and that due to the backfill alone was considered to be the pressure caused by the applied surface load.

The backfill material was a pit run gravelly sand consisting of about 40 percent gravel, 48 percent sand, 8 percent silt and 4 percent of 5 micron clay. The liquid limit was 17 and the plasticity index, 4. It was placed behind the experimental retaining walls by hand methods without special compaction. The concentrated loads applied to the surface of the backfill consisted of one rear wheel of a heavily loaded truck. The wheel was equipped with dual hard rubber tires and was considered to transmit a concentrated load to the backfill surface, although of course, there was a finite area of contact of about one square foot. This truck wheel load was positioned at various distances behind the back face in a vertical plane at right angles to the wall and passing through the vertical element in which the pressure measuring devices were installed. The applied wheel loads were of several different magnitudes in the several series of tests which were run, ranging from 6,800 lb to 10,650 lb.

The results of Series V of these early tests with concentrated loads are shown in Figures 3, 4 and 5. The wheel load in this series was 10,450 lb, but the data are shown in terms of unit lateral pressure per 1,000 lb of load. Although the data points are scat-

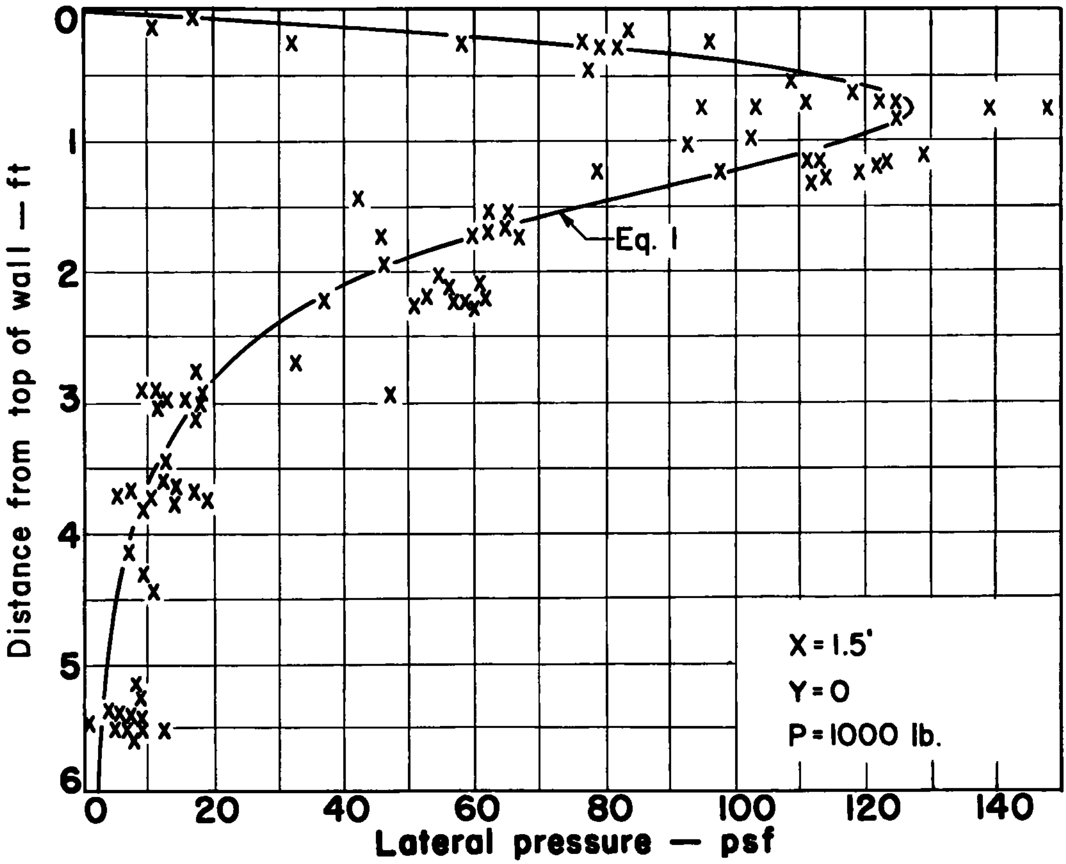

Figure 3. Lateral pressure due to concentrated load - 1934.

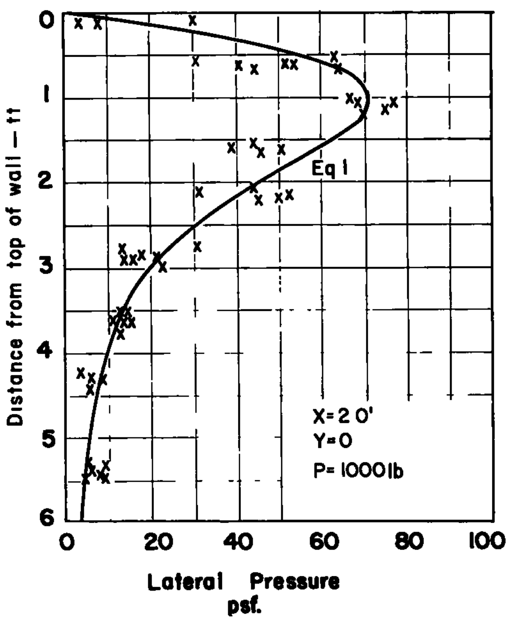

Figure 4. Lateral pressure due to concentrated load - 1934.

tered over a wide area, the general pattern of lateral pressure is unmistakably similar to that indicated by the Boussinesq equation for normal stress on a vertical plane in a semi-infinite elastic medium, due to a point load acting at the surface, with the exception that the magnitude of the measured pressures is roughly about double that indicated by the Boussinesq formula when Poisson's Ratio is assumed to be 0.5. Thus on the basis of these experiments we may write the equation

$$h_c = P\frac{x^2 z}{R^5} \tag{1}$$

in which

h_c = horizontal unit pressure at any point on the wall due to a concentrated surface load;

P = concentrated load applied at surface of backfill;

x, y and z = coordinates of any point on the wall;

R = radial distance from load to any point $= \sqrt{x^2 + y^2 + z^2}$.

The results of these experiments were first presented at the International Conference on Soil Mechanics and Foundation Engineering at Harvard University in 1936. In a discussion of the paper, Dr. R. D. Mindlin (2) of Columbia University pointed out that it can be shown theoretically by the method of images that the pressure on a smooth, rigid wall is exactly double that indicated by the Boussinesq formula. The retaining walls used in these experiments were relatively rigid in character, but they were not smooth to the extent that no shearing stresses existed on the back face, which may partially account for the fact that a number of the experimental points indicate pressures greater than double the Boussinesq pressures. Nevertheless, this suggestion by Mindlin lends confidence to the pressure measurements.

If Equation 1 is integrated in the direction parallel to the wall, between the limits ∞ and $-\infty$ the following expression is obtained

$$h_1 = 1.33\ P_1 \frac{x^2 z}{R_1^4} \tag{2}$$

in which

h_1 = horizontal unit pressure at any point on the wall due to a uniformly distributed parallel line load of infinite length;
P_1 = load per unit length;
R_1 = slant distance from line load to any point,
$= \sqrt{x^2 + z^2}$.

The validity of this integration was investigated experimentally by placing a narrow strip load on the backfill and measuring the lateral pressure on the retaining wall. The

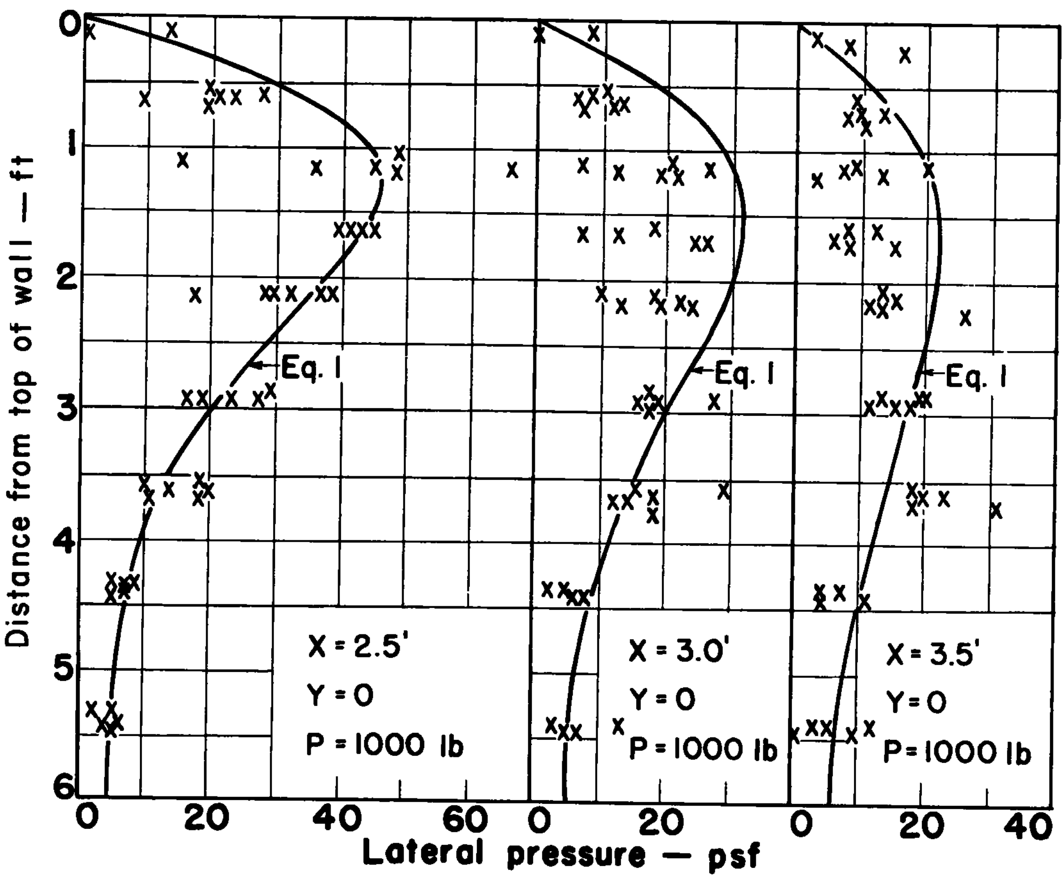

Figure 5. Lateral pressure due to concentrated load - 1934.

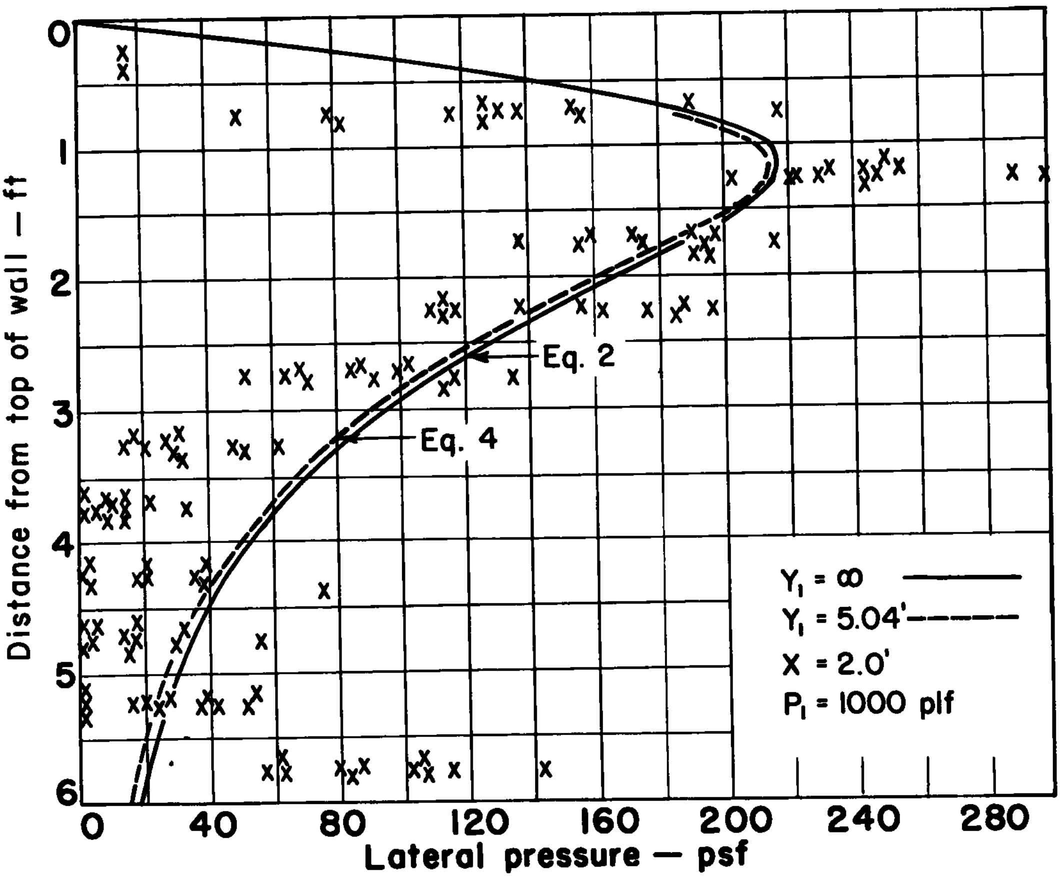

Figure 6. Lateral pressure due to uniformly distributed line load - 1934.

narrow strip load was applied by placing both rear wheels of a truck on a 6 x 8 in. timber 10 ft-1 in. long, which was laid with the 8 in. side down, parallel to the wall, and centered 2 ft back of the back face. The timber was centered directly opposite the pressure measuring devices and the rear wheels of the truck were placed symmetrically about the center of the timber. The total axle load of the truck was 19,080 lb and, assuming this load was uniformly distributed over the length of the timber, the load per unit length was 1,893 lb per ft.

The results of the line load pressure measurements are shown in Figure 6. Again the measured pressures indicate a definite correlation with those calculated by the modified Boussinesq Theory, although the usual wide scattering of data masks this relationship in any individual load application.

As previously stated, the actual length of the line load applied in the experiments was 10 ft-1 in., whereas Equation 2 represents the pressure due to a load of infinite length. If Equation 1 is integrated between finite limits of y_1 and $-y_2$, we obtain

$$h_1 = P\frac{x^2 z}{R_1^4}\left[\frac{R_1^2 y_1}{3(R_1^2 + y_1^2)^{1.5}} + \frac{2y_1}{3(R_1^2 + y_1^2)^{0.5}} - \frac{R_1^2 y_2}{3(R_1^2 + y_2^2)^{1.5}} - \frac{2y_2}{3(R_1^2 + y_2^2)^{0.5}}\right] \tag{3}$$

The maximum pressure on the wall will occur at points directly opposite the center of the finite load; that is, where y_1 and y_2 are numerically equal. For this condition we may substitute y_1 for $-y_2$ in Equation 3 which gives

$$h_1 = \frac{2}{3} P \frac{X^2 z}{R_1^4} \left[\frac{R_1^2 y_1}{(R_1^2 + y_1^2)^{1.5}} + \frac{2y_1}{(R_1^2 + y_1^2)^{0.5}} \right] \tag{4}$$

The dotted curve in Figure 6 shows pressures calculated by Equation 4, using $y_1 = 5.04$ ft, corresponding to the actual length of line load applied in the experiments. The difference between calculated pressures for the 10.08 ft load and a load of infinite length is negligible at this distance from the wall. The theoretical relationship between maximum pressures due to line loads of finite length (Equation 4) and infinite length (Equation 2) is shown in Figure 7.

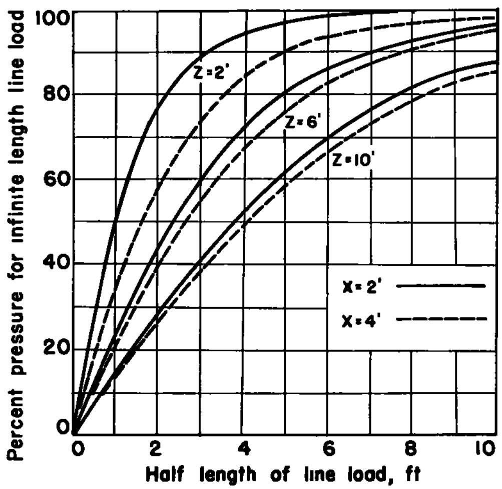

Figure 7.

1941 Experiments

The experimental work employing concentrated loads and uniformly distributed parallel line loads has demonstrated that the lateral pressures on a rigid retaining wall are closely related to those indicated by a simple modification of the Boussinesq formula. This experience led to the hypothesis that the lateral pressures due to a uniformly distributed area load may be estimated by integrating the line load formulas, Equation 2, 3 and 4, in the x-direction; that is, the direction normal to the wall.

The integration of Equation 2 has been completed and yields the following:

$$h_a = \frac{2}{3} Pa \left[\text{arc tan} \frac{x}{z} - \frac{xz}{(x^2 + z^2)} \right]_{x_0}^{x_1} \tag{5}$$

in which

- h_a = horizontal unit pressure at any point on the wall due to a uniformly distributed area load of finite width $(x_1 - x_0)$ and length greater than about 15 or 20 ft;
- Pa = load per unit area;
- x_0 = distance from back of wall to near side of load;
- x_1 = distance from back of wall to far side of load.

To obtain lateral pressures due to area loads less than about 15 or 20 ft in length, it will be necessary to integrate Equation 3 and 4 in the x-direction. These integrations have not been completed. Therefore, at present it is necessary to resort to mechanical summation procedures for these cases by dividing the applied load into a series of finite strip loads about 1 or 2 ft in width and utilizing Equation 3 or 4 to estimate the pressure caused by each strip load.

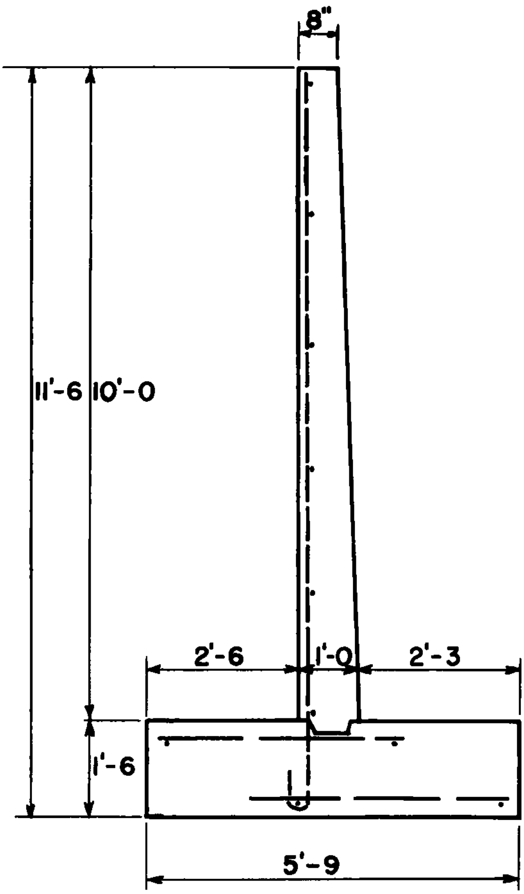

Figure 8. Cross section of experimental wall - 1940.

The 1941 experiments, which were coop-

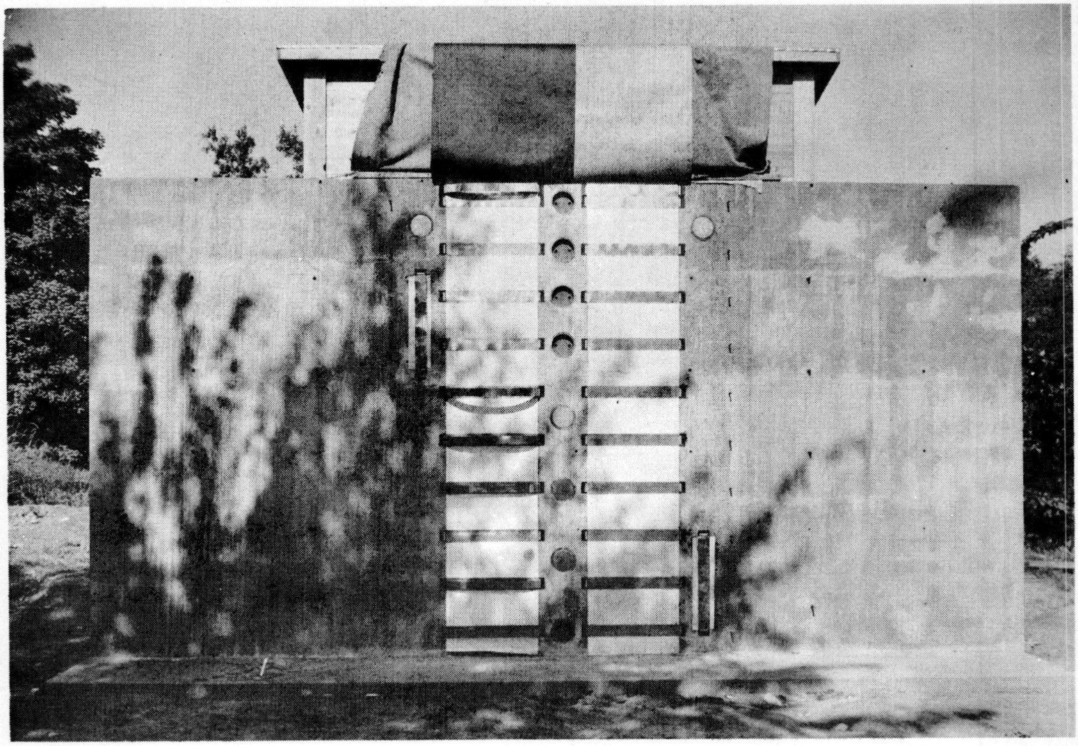

Figure 9. Experimental retaining wall - 1941.

erative with the Bureau of Public Roads, were designed to provide data bearing upon the validity of this hypothesis relative to the applicability of the modified Boussinesq equation to the case of an area load applied at the surface of a level backfill. A new experimental wall was constructed which was 10 ft high and 20 ft long, having the cross section indicated in Figure 8. This wall was fitted with pressure measuring devices in the vertical back face, consisting of a series of stainless steel friction ribbons and a series of Goldbeck pressure cells, as shown in the photograph in Figure 9.

The friction ribbons were 2 in. wide and installed with a length of 2 ft in the plane of the back face of the wall. At each end of this length the ribbons passed over a stainless steel roller and passed through the wall in such a manner that both ends of the ribbon were available for pulling from the front side of the wall. A winch was mounted in a shed at the front side for the purpose of applying pull to the ribbons.

The ribbons were mounted to slide between two sheets of stainless steel and the whole area covered with a sheet or rubber and then a sheet of tar paper to protect the ribbons from the backfill and moisture. They were calibrated individually by applying air pressure into a rubber bladder confined in an aluminum bottomless box. This box and bladder were centered directly over the ribbon to be calibrated and clamped to the wall. Pulls were applied to one end of the ribbon and the relationship between applied normal pressure and pull required to start sliding motion obtained.

The Goldbeck pressure cells were mounted in recesses in the wall with the measuring face flush with the back face. They were calibrated in essentially the same manner as the friction ribbons.

A vertical steel column was installed at a distance of 6 ft in front of the wall as a reference post for measurement of outward yield of the wall under the influence of backfill and surface loads. The column was set in a heavy concrete base entirely separate from the retaining wall structure. In order to make sure that the reference post itself did not move, a transit line was established between bench marks about 50 ft on each side of the column and well removed from the experimental wall. Frequent observations were made during the course of the loading operations, but no movement of the

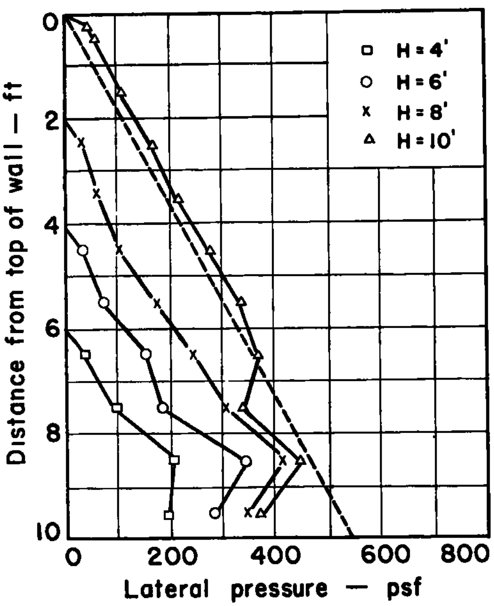

Figure 10. Lateral pressure, backfill only - 1940.

TABLE 1

Load Condition	Outward movement, in Top	Mid-height	Bottom
Backfill completed	06	02	02
First surcharge complete	60	36	20
First surcharge in place one month	91	54	34
First surcharge removed	73	45	29
Second surcharge complete	75	46	28
Second surcharge in place 2 weeks	77	46	.29
Second surcharge removed	75	.44	29
Third surcharge complete	74	45	28

reference post could be detected. The outward movement of the wall was measured by means of a micrometer caliper between the steel post and brass pins set near the top, center and bottom of the wall. These measurements are summarized in Table 1.

The first surcharge load caused relatively large outward movements, both rotation and translation, but subsequent loadings did not produce any movement of consequence. Also, when the experimental wall was again loaded during the current series of loadings, the wall movements were practically negligible. Apparently the first surcharge caused the wall to reach a state of equilibrium and no further movements occurred.

The backfill consisted of the same type of pit run gravelly sand as that used in the earlier experiments. It was placed behind the wall in the fall of 1940 by hand methods and not compacted except by its own weight. During the following winter the surface settled up to a maximum of 8 or 10 in. This settlement was made up the next spring by adding additional material up to the level of the top of the wall. During the winter the unit weight of the backfill was measured by sinking a shaft the full depth of the fill and weighing all the soil removed. The average unit weight was 111 pcf. The lateral pressures on the wall due to the backfill at various stages of its construction are shown in Figure 10.

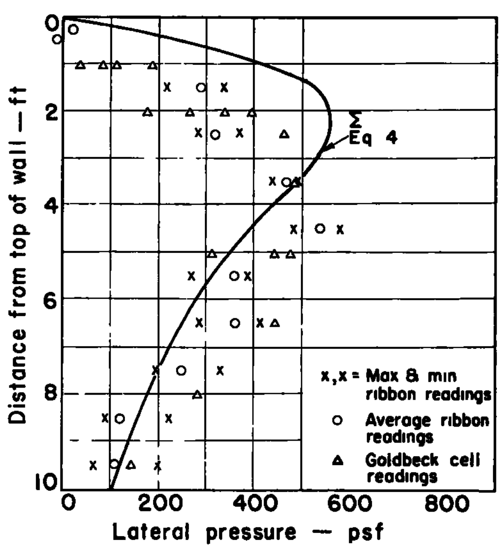

Figure 11. Lateral pressure due to area load - 1941.

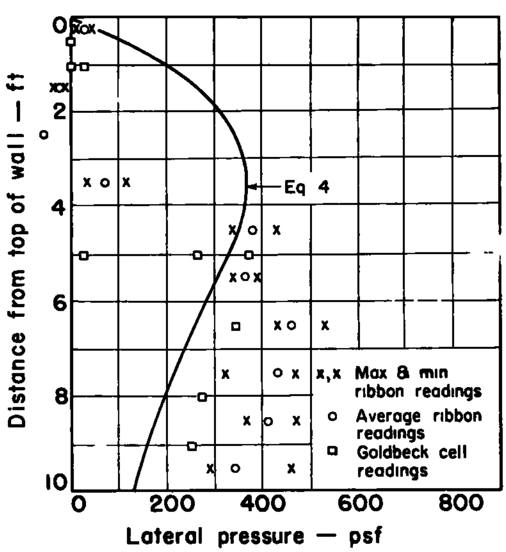

Figure 12. Lateral pressure due to area load - 1941. Load 4 feet from wall.

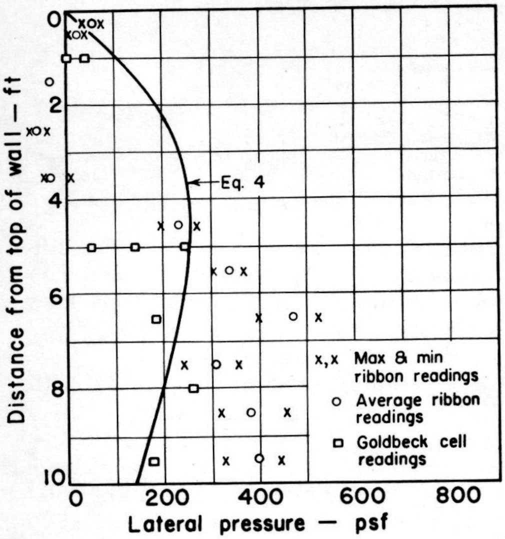

Figure 13. Lateral pressure due to area load - 1941. Load 6 feet from wall.

During the spring and summer of 1941 the backfill was loaded by piling sacks of gravel inside a wooden crib which was 6 ft wide normal to the wall and 16 ft long parallel to the wall. A series of three load applications was made, first with the load 2 ft back of the wall, then 4 ft and finally 6 ft. The magnitude of the superimposed load in each trial was 105,200 lb or 1,096 psf. The loading operation took about one to two weeks' time in each case and the maximum load was left in place from two weeks to one month. The backfill was not disturbed between the load applications. The results of these load applications are shown in Figures 11, 12 and 13.

Originally it was planned to recalibrate the measuring devices after removal of the backfill, but as stated earlier, the project was suspended at this time and the recalibration was not carried out. Hence the data obtained are not as reliable as they otherwise might have been. Also it is pointed out that only one load application is involved in each of these trials, whereas experience gained in the case of concentrated loads and line loads indicated that many repetitions of load are required to obtain a reasonably complete statistical picture of the magnitude and distribution of pressures due to loads applied at the backfill surface.

Current Experiments

The project was re-established in 1951 in cooperation with the Iowa Highway Commission. The previously constructed retaining wall was rehabilitated by pouring a 4 in. thick surfacing on the back face and by constructing 8 ft long wing walls at each end. Recesses were cast in the back face surfacing to receive the pressure measuring devices, which were soil pressure cells of the type developed by the Waterways Experiment Station of Vicksburg, Mississippi. A photograph of the experimental wall after rehabilitation is shown in Figure 14.

The pressure cells were $4\frac{1}{2}$ in. in diameter and 1 in. thick and were machined of a special grade of stainless steel to resist corrosion. They were set in the wall recesses in neat cement with the measuring face of the cell flush with the back face of the wall. The cell housing is hermetically sealed to prevent the entrance of moisture. A 4-wire electrical cable attached to the side of the cell passed through the wall and was available from the front side for connection with a strain indicator. Entrance of moisture along the wires is prevented by a special Kovar seal.

The cells consist of a metal disk, supported around its periphery. Pressure on the cell causes the disk to deflect a minute amount. The strain in the disk is measured by four SR-4 strain gages which constitute the four resistance arms of a complete Wheatstone's Bridge. The arrangement is such that strain in the disk causes an unbalance of the bridge which is a measure of the pressure causing the strain, and the relationship between pressure and bridge unbalance can be determined by calibration.

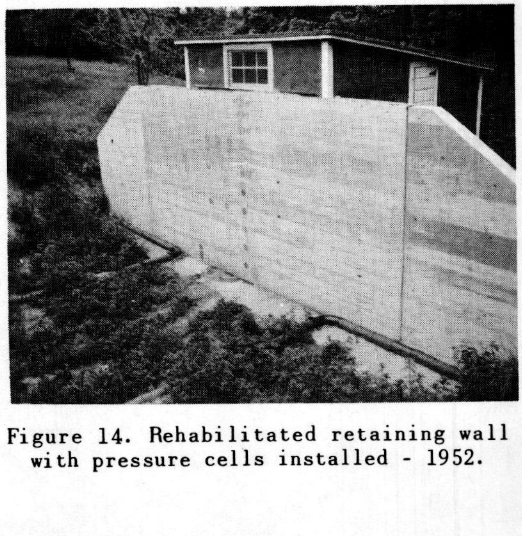

Figure 14. Rehabilitated retaining wall with pressure cells installed - 1952.

An SR-4 Type L portable strain indicator manufactured by the Baldwin-Lima-Hamilton Corporation was used to measure the unbalance of the bridge. It was housed in a constant temperature box at 95 deg F to minimize the effect of temperature changes on the indicator. It was checked from time to time with a Baldwin constant resistance box. Power imput to the indicator was furnished through a Baldwin transformer early in the experiments, but this was abandoned after a short time because of poor line voltage. Batteries were substituted as a source of power during the balance of the study. Individual cells were connected to the indicator by means of a Baldwin twenty pole selector.

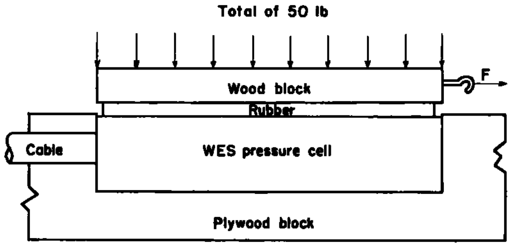

Figure 15. Apparatus for applying a shearing force on the face of a pressure cell.

Thermometers were inserted into the holes through which the cables passed through the wall, to a point 4 in. in front of the cells. It was noted that temperature variations changed the unbalance of the bridge, but no correlation between temperature change and bridge unbalance could be established.

Calibration curves for the pressure cells were furnished by the supplier. However, results obtained with the first backfill placed behind the wall led to the conclusion that conditions prevailing in the factory calibration and the actual installation were not the same, and an extensive in-place calibration program was carried out. This was done by clamping a 9-in. diameter hemispherical vessel over a cell and introducing air pressure into a rubber bladder which impinged directly on the cell. Calibration curves obtained in this manner were reproducible and appeared to be satisfactory. The cells were recalibrated after removal of each of the backfills placed behind the wall, and little change in calibration was noted.

At this stage of the investigation a question arose relative to the effect of shearing forces on the measuring face of the pressure cell on the indicated normal pressure. In order to study this question, a cell was removed from the wall and mounted in a wood block in the laboratory with the face of the cell flush with the surface of the block, in much the same way as the cells were mounted in the retaining wall. A piece of thin rubber was placed over the cell to develop frictional resistance to tangential force. Then a circular piece of plywood the same diameter as the cell was placed on the rubber directly over the cell. A diagram of this arrangement is shown in Figure 15. A 50 lb weight was placed on the plywood disk which actuated the pressure cell at about 3 psi. Next, a shearing force was applied by pulling the plywood disk at right angles to the radial axis of the cell; that is, parallel to the measuring face. Tangential forces up to more than one-half the normal force were applied, but they did not change the unbalance of the bridge. However, when the tangential force was applied at a slight angle with the face of the cell, the influence of a normal component was readily detected on the SR-4 indicator. From these trials, it was concluded that the cells measured normal components of pressure only, uninfluenced by shearing forces acting on the back face of the wall.

Up to the time of this report, four backfills have been placed behind the experimental wall. The first three consisted of a sandy loam glacial till, which contained 6 percent gravel, 57 percent sand, 25 percent silt and 12 percent 5-micron clay. The liquid limit was 18 and the plasticity index 4.

TABLE 2

Backfill No	Date placed	Date removed	Unit weight pcf
1	June, 1953	Aug , 1953	115 3
2	Oct , 1953	May, 1954	-
3	July, 1954	Oct , 1954	117 6
4	Dec , 1954	Apr , 1955	115 5

Backfill number 4 was a pit run gravel which contained 56 percent gravel, 36 percent sand, 5 percent silt and 2 percent 5-micron clay. The liquid limit was 21 and the plasticity index was 1. The dates of placement and removal of the backfills are shown in Table 2.

TABLE 3

Backfill No	Surcharge No	Unit Load psf	Distance wall to load, x_o, ft
1	1-A	938	3
2	2-A	938	3
3	3-A	938	3
	3-B	1,448	3
	3-C	1,448	1.5
4	4-A	1,448	2
	4-B	1,448	2
	4-C	1,448	3

All of the backfills were placed by essentially hand methods. A small dragline was used to move the material from a nearby stockpile to the general area behind the wall. It was then hand shoveled up to the wall and brought up in horizontal layers. Care was exercised to see that no stones or lumps of soil were placed in the vicinity of the pressure cells. In the case of backfill No. 3, a vertical layer of clean river sand about 6 in. thick was placed next to the retaining wall as the backfill was built. The soil was not compacted behind the wall in any of the experiments.

Surcharge loads consisted of a wooden crib 6 ft wide and 10 ft long, filled with 50 lb bags of pea gravel. The bottom of the crib was made of loose 2 in. by 12 in. planks 2 ft long laid end to end with joints staggered in adjacent rows. The purpose of this arrangement was to enable the gravel bags to conform to the surface of the backfill at all times. The bags were piled in orderly arrangement to attain a uniform distribution of pressure over the 6 ft by 10 ft area. The crib and gravel bags were kept covered with a heavy tarpaulin at all times to prevent rainfall from changing the load.

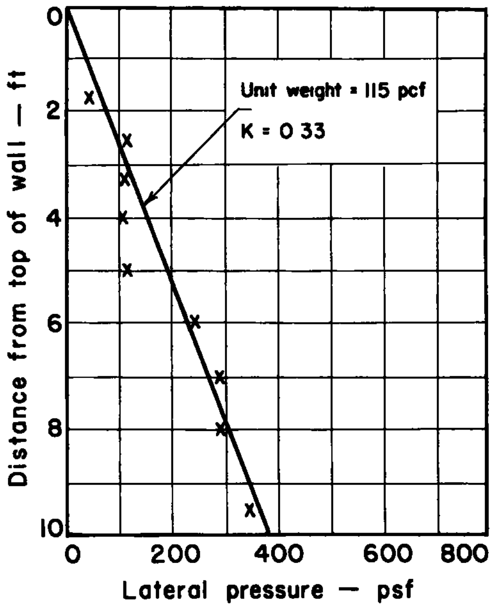

Figure 16. Lateral soil pressures caused by backfill only. Backfill number one before surcharging.

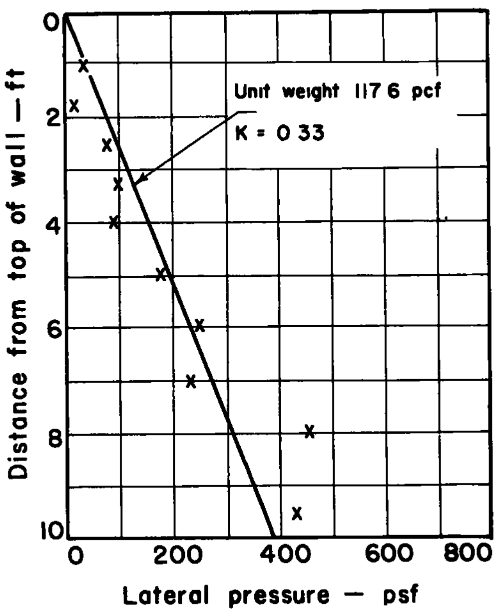

Figure 17. Lateral soil pressures caused by backfill only. Backfill number three before surcharging.

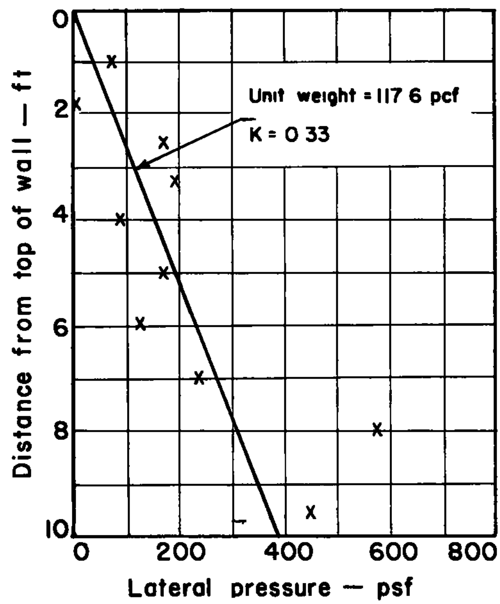

Figure 18. Lateral soil pressures caused by backfill only. Backfill number three after removal of first surcharge.

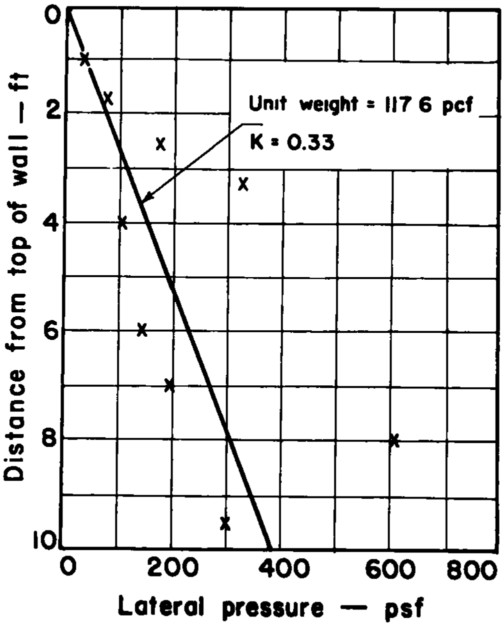

Figure 19. Lateral soil pressures caused by backfill only. Backfill number three after removal of second surcharge.

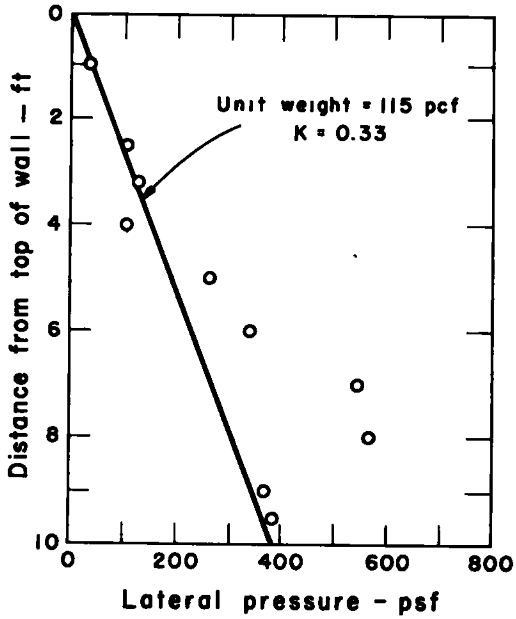

Figure 20. Lateral soil pressures caused by backfill only. Backfill number four before surcharging.

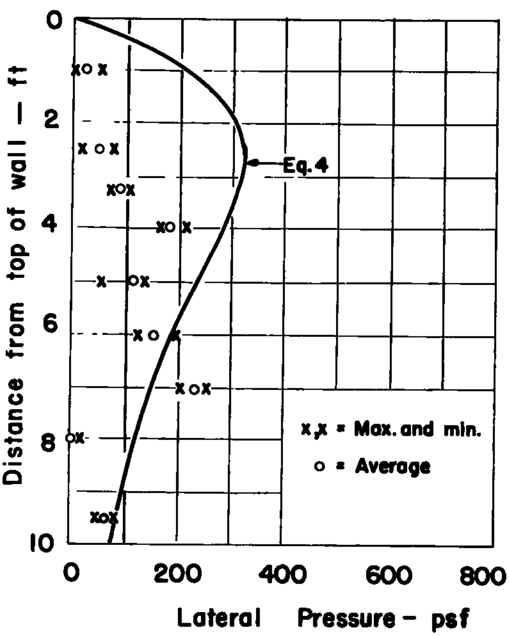

Figure 21. Lateral pressures caused by surcharge only, backfill number one. Surcharge number one, 938 psf at 3 feet clear distance from wall.

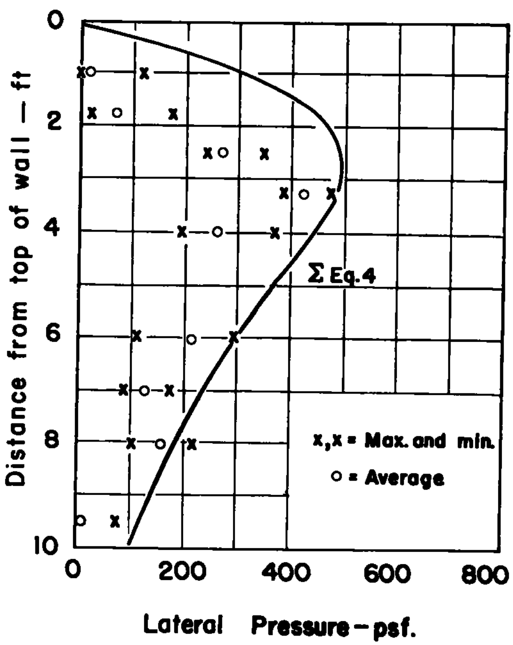

Figure 22. Lateral pressures caused by surcharge only, backfill number four. Surcharge number two, 1,448 psf at 2 feet clear distance from wall.

A total of eight surcharge loads have been placed; one each on backfills 1 and 2, and three each on backfills 3 and 4. The center of the surcharge area was placed opposite the center of the wall in each case. The magnitude and position of each surcharge are shown in Table 3.

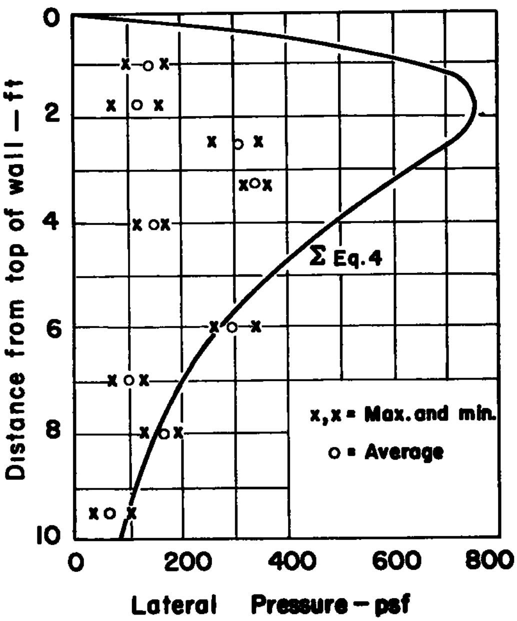

Figure 23. Lateral pressures caused by surcharge only, backfill number three. Surcharge number two, 1,448 psf at 3 feet clear distance from wall.

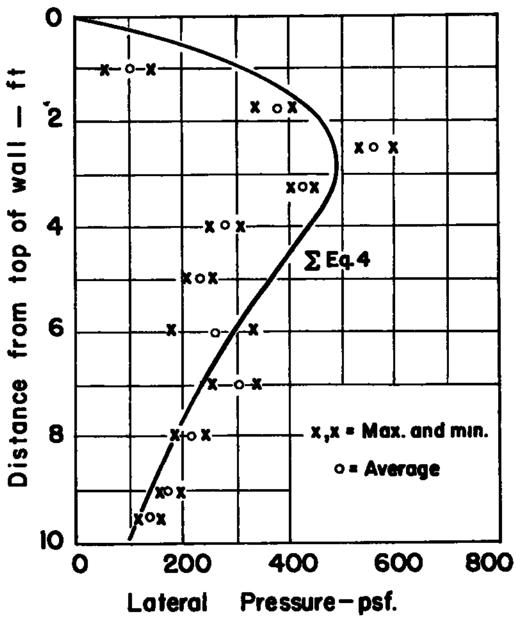

Figure 24. Lateral pressures caused by surcharge only, backfill number three. Surcharge number three, 1,448 psf at 1 foot 6 inches clear distance from wall.

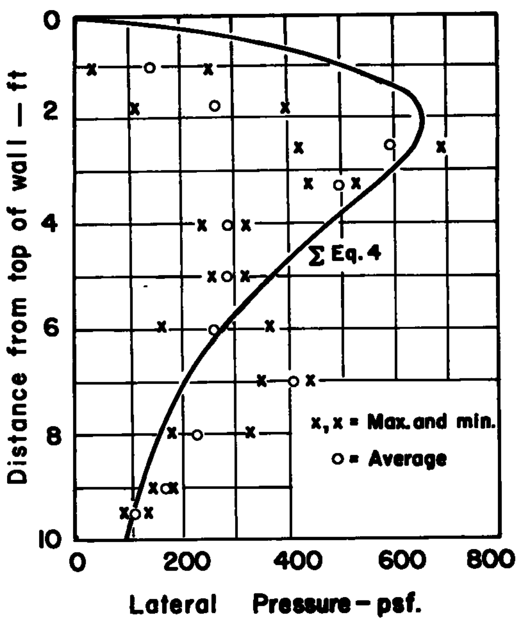

Figure 25. Lateral pressures caused by surcharge only, backfill number four. Surcharge number three, 1,448 psf at 3 feet clear distance from wall.

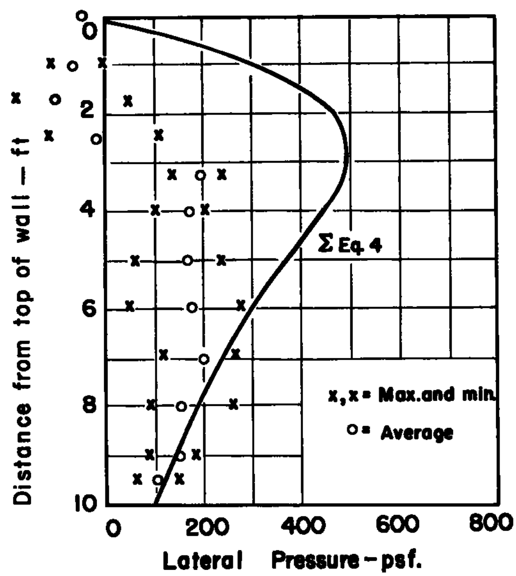

Figure 26. Lateral pressures caused by surcharge only, backfill number four. Surcharge number one, 1,448 psf at 3 feet clear distance from wall.

The procedure employed in interpreting the data has been to measure the pressures due to backfill only. Then a surcharge was applied and the total pressure observed. The difference between the total pressure and that due to backfill only was deemed to be the increment of pressure caused by the surcharge load. This procedure is logical, but has been fraught with difficulties and uncertainties because of wide fluctuations in the pressure cell readings with no apparent change in loading conditions. Temperature changes, rainfall, periods of dry weather all seemed to effect the cell readings, but no logical or consistent relationship between these phenomena and the cell readings could be identified.

In those cases where more than one surcharge was placed on the same backfill, the pressure on the wall due to backfill alone was frequently greater after removal of a surcharge than it was prior to loading. In other words, there were residual pressures against the wall after removal of the first surcharge. The initial backfill pressure readings prior to application of any surcharge has been subtracted from the total pressure readings in order to obtain the net increment of pressure due to surcharge alone.

The results of the current series of pressure measurements are summarized in Figures 16 to 26. Pressures due to backfill alone are shown in Figures 16 to 20 and pressure increments due to surcharge are shown in Figures 21 to 26.

The data points representing the measured pressures are widely scattered and fall far short of accurate coincidence with the curves representing the modified Boussinesq formula for lateral pressures due to surcharge loads. Nevertheless, the general statistical trend of the measured pressures, both as to magnitude and distribution, appears to be compatible with the theory and the authors believe that the modified formulas, Equations 1 to 5, are appropriate for estimating lateral pressures on retaining walls caused by concentrated loads, line loads, and area loads respectively.

In the early phases of this research program, it was assumed that the scattering of lateral pressure data was primarily due to shortcomings of the pressure measuring devices or the technique of their use. After long and extensive experience with a rather wide variety of pressure cells, the authors are convinced that a substantial part of the dispersion of data is not necessarily due to lack of precision of the measuring devices, but rather, is inherent in the problem itself. A soil backfill, even though reasonably homogeneous as a soil, is far removed from a homogeneous material as that term is used in the science of mechanics. Therefore, it is probably futile to expect that stresses transmitted through the soil to a retaining wall should consistently conform to a well defined theoretical pattern. It seems very probable that local variations in density and other properties of the soil will cause deflection and discontinuity of stress lines which may account for a substantial part of the dispersion which has been observed. This statement is not to imply that there is no need for further improvement in pressure cells and the technique of their use. Rather it is to say that the nature of the problem of measuring pressures on retaining walls is such that wide dispersion of measured data is a characteristic with which the researcher must contend.

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Discussion

EDWARD S. BARBER, Civil Engineer, Arlington, Virginia — The excellent data of this paper substantiate the use of theory of linear elasticity for calculating lateral pressures on walls, due to live loads. The charts presented herewith facilitate such calculations.

Figures A, B, and C give lateral pressures in a semi-infinite mass and must be doubled to give pressures on a rigid wall. The formulas presented in the paper are higher by the ratio of $\pi \div 3$. Figure A gives pressures from an infinite strip load parallel to the wall and of variable pressure perpendicular to the wall. Figure B gives

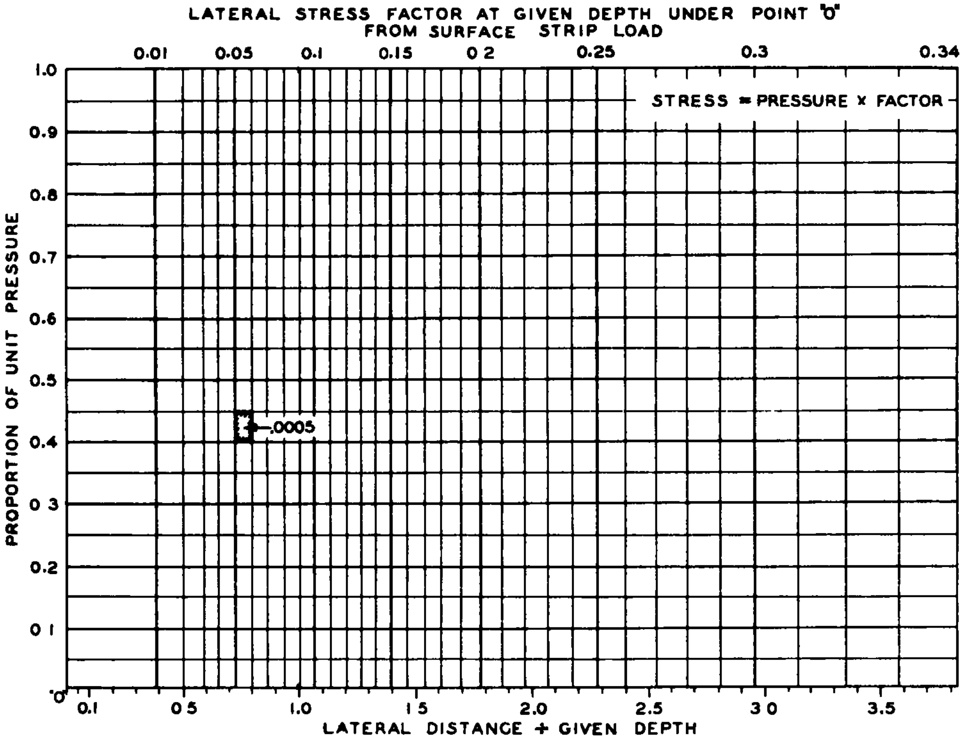

Figure A. Chart for computing lateral stress at any point from surface strip load.

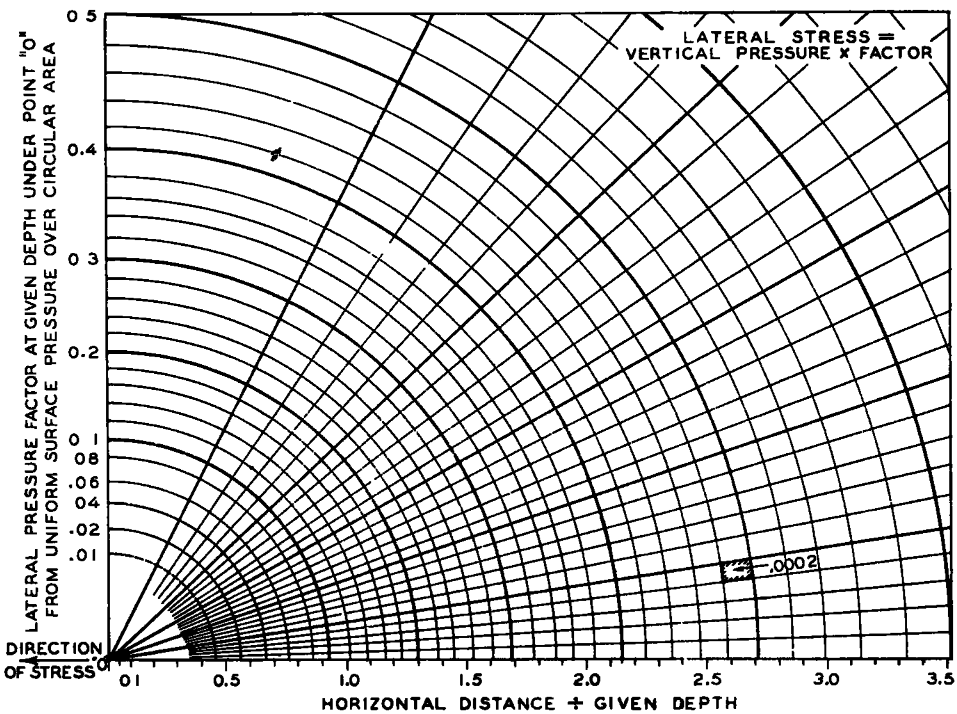

Figure B. Chart for computing lateral pressure at any point from finite surface load - Poisson's ratio = 0.5.

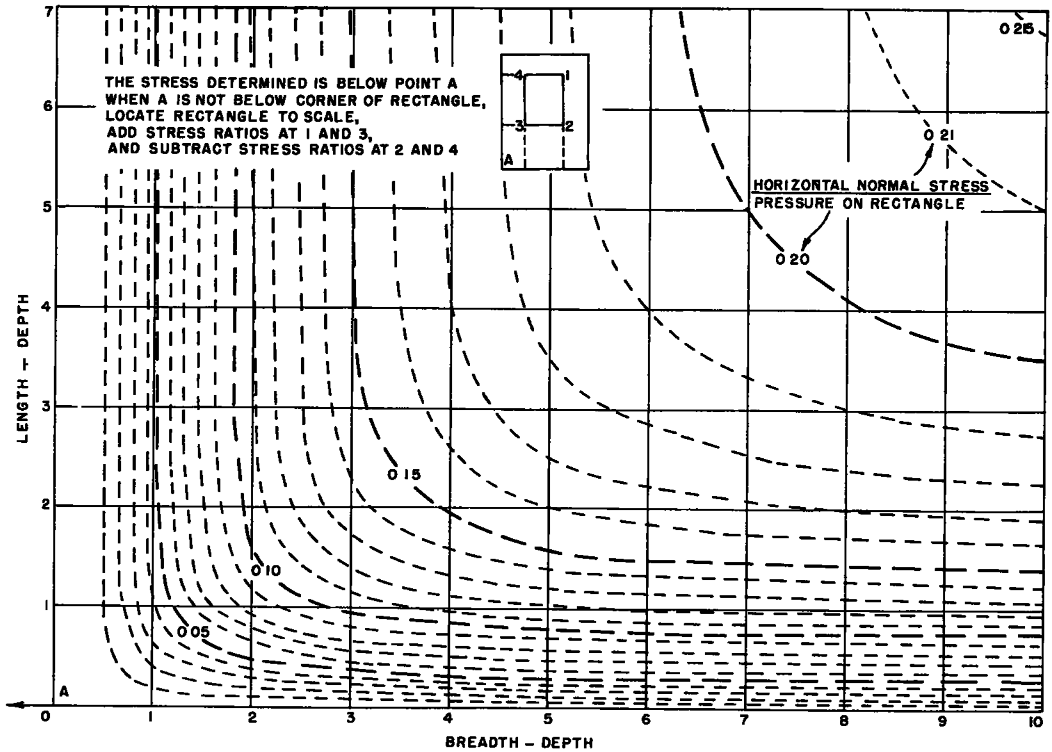

Figure C. Graph of horizontal normal stress under corner of rectangle loaded with unit pressure.

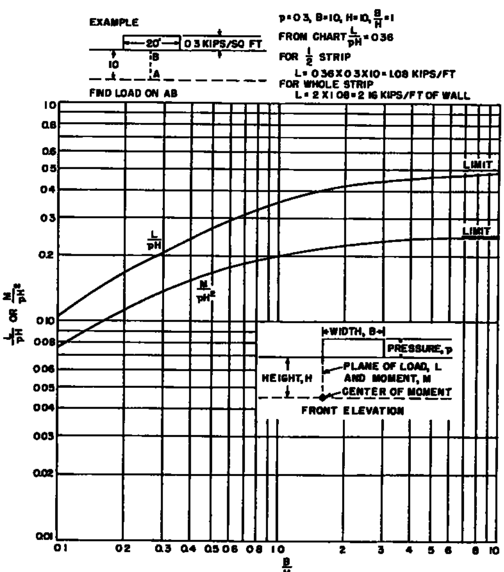

Figure D. Lateral load and moment on smooth boundary due to parallel uniform strip of infinite length.

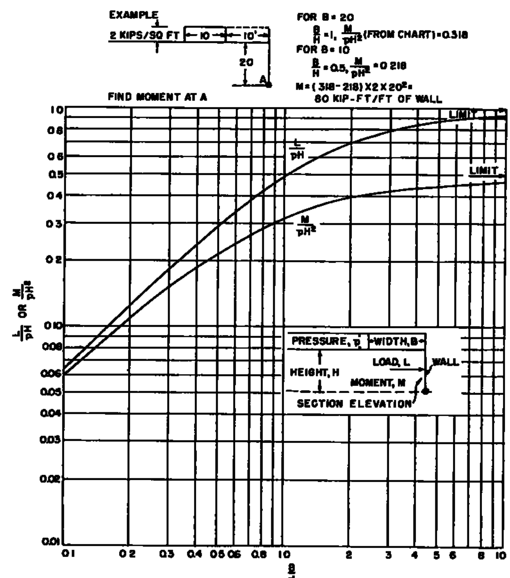

Figure E. Lateral load and moment on smooth rigid boundary due to perpendicular uniform strip load of infinite length.

lateral pressures from a uniform pressure over an area of any shape. The influence factor is 0. 0002 x the number of influence areas covered by a plan of the loaded area plotted according to the scale given on the abscissa. Figure C gives the lateral pressure directly for uniform stress over a rectangular area with one side parallel to the wall.

Figures D and E give total stresses from strip loads on a vertical strip of unit width on a rigid wall and the moment of this total stress. The stress in a semi-infinite mass has already been doubled. In Figure D the strip load is parallel to the wall, while in Figure E the strip load is perpendicular to the wall, as for a highway going over an abutment.

Waterways Experiment Station Large Triaxial Apparatus

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Flexible Pavement Branch, Soils Division, Waterways Experiment Station
Corps of Engineers, U. S. Army, Vicksburg, Mississippi

● A SERIES of investigations in connection with studies of the distribution of stresses and strains in soil masses has led to the development of the large triaxial apparatus which is the subject of this paper. These investigations are being conducted by the Waterways Experiment Station of the Corps of Engineers.

The apparatus was not built primarily for testing soil specimens, but to provide a substantially known stress condition in a mass of sand. This was needed in connection with a study of pressure cell action. Pressure cells are used to measure stresses in soil masses, but it is known that difference in stiffness between the soil and the cell can lead to improper registrations. The large triaxial apparatus was developed to help study this problem.

The triaxial loading was dictated by a need for controlled and known stress conditions; the size of specimen, by a need to install pressure cells within the triaxially loaded mass (the Waterways Experiment Station pressure cell presently used is 6 in. in diameter and 1 in. thick); and the use of sand as a sample medium, by a need to interpret more precisely a large number of measurements made in sand under an earlier phase of the program.

Details of the large triaxial apparatus and its operation and use are given in this paper. Comments on the preliminary results are included as a matter of interest. Since vacuum was used for confinement and only one soil, and that dry, was tested in the apparatus, any results bearing on the triaxial testing of soils, generally, are limited in scope.

DESCRIPTION OF THE APPARATUS

The Waterways Experiment Station large triaxial apparatus uses a specimen 70 in. tall and 35.68 in. in diameter (see Figures 1, 2, 3 and 4). This diameter gives a cross-sectional area of 1,000 sq in. The specimen is confined within a rubber membrane between a head and base plate and evacuated to gain lateral load from normal air pressure. The headplate is rigidly made of sections of ½-in. steel plate welded together. In addition to its faceplate and circumferential plate, it has both radial and tangential stiffening ribs and a partial top plate. The base plate is of porous concrete set in a welded steel pan. This base is pierced by an access port to accommodate pressure cell cables. The specimen is evacuated through the porous concrete base and its access port. The membrane is of $^1/_{32}$-in. black rubber. Membranes have been made of sheet rubber by vulcanizing longitudinal seams. Soft white rubber membranes were tried but proved to be too fragile, becoming pierced repeatedly at the top of the forming jacket during specimen construction operations. This occurred despite the fact that a sheet metal rim was used to protect the membrane at this critical point.

During construction, the triaxial specimen is confined within a forming jacket consisting of three 120-deg segments. These segments were built up by welding together parts formed from sheet steel. They consist of the main curved sheet of ¼-in. steel, reinforced vertically at the edges with ⅜-in. ribs and horizontally by six curved ⅜-in. steel ribs. The three segments are bolted together to provide a cylinder. Rubber gaskets are placed between segments so that vacuum may be applied between the forming jacket and membrane during sample construction. Experience has indicated that this feature probably is unnecessary.

The entire device is seated on a heavily reinforced concrete base 6 ft square and 10 in. thick. Load is applied by use of hydraulic jacks working against a heavy load-reaction truss. This is a truss built in the stress distribution shelter at the Waterways Experiment Station for use in loading full-scale test sections constructed beneath it. It

Figure 1.

has been described in an earlier publication (1). A hand winch and trolley crane mounted on the load-reaction truss provide the means of handling the headplate, forming jacket segments, and other heavy items.

Loads on the specimen are measured with a 50,000-lb capacity Baldwin, Type C SR-4 load cell. Vacuum is measured at both the top and bottom of the specimen. The vacuum is applied through buried pipes by a pump, and a cable outlet chamber is provided to permit electrical connections to pressure cells installed within the specimen.

A three-leg pipe frame is placed around the specimen to provide support for deflec-

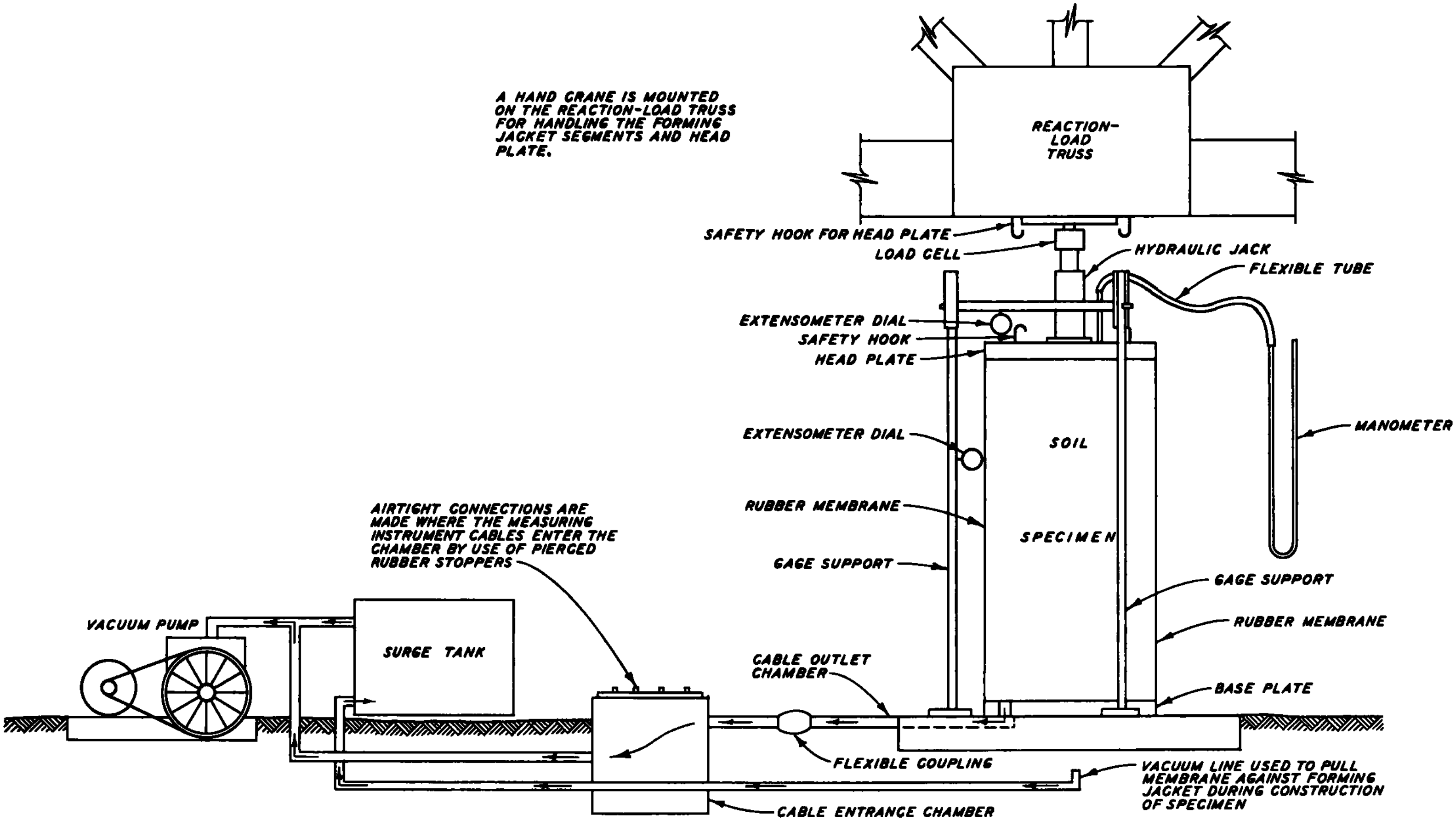


Figure 2. Large triaxial apparatus.

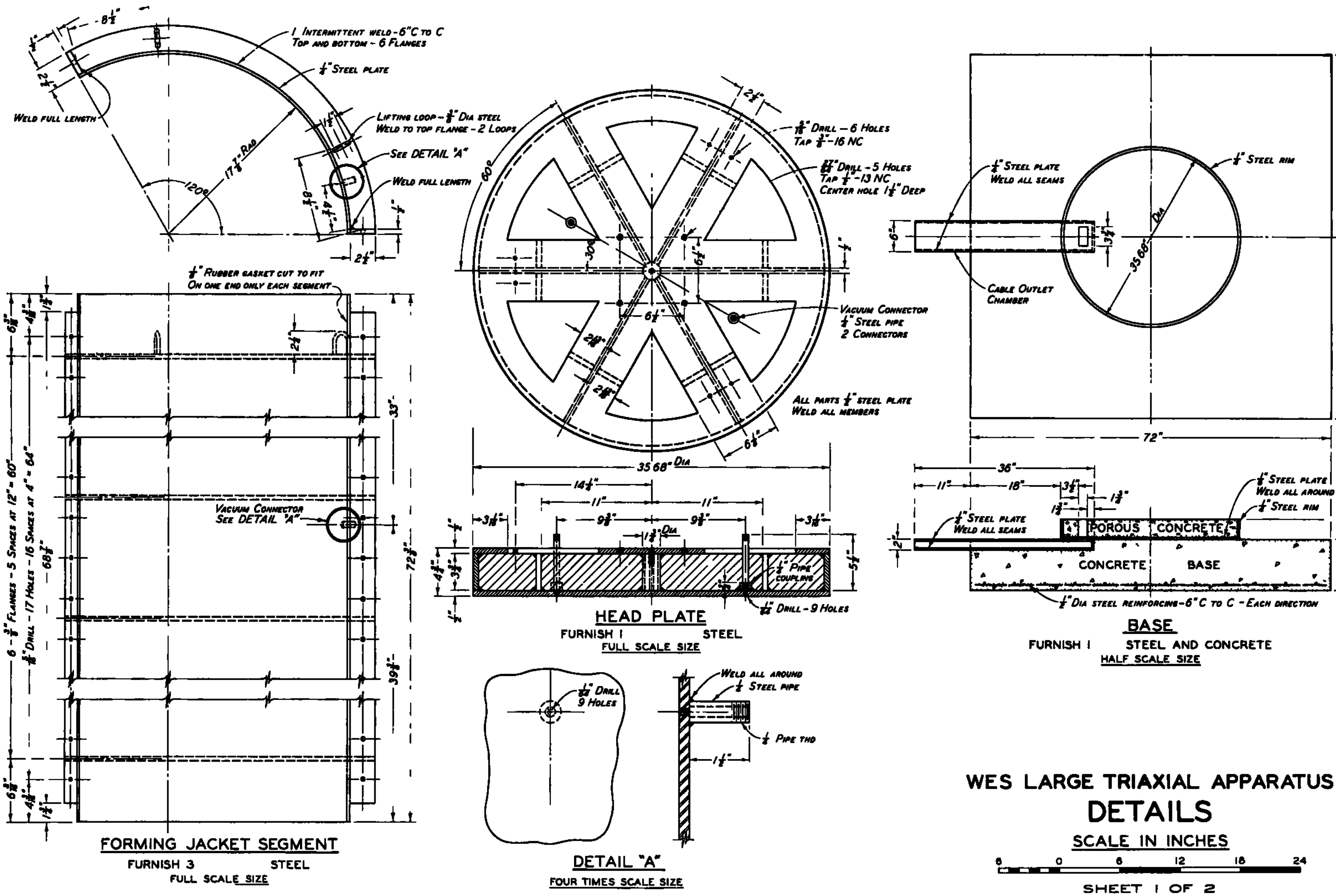
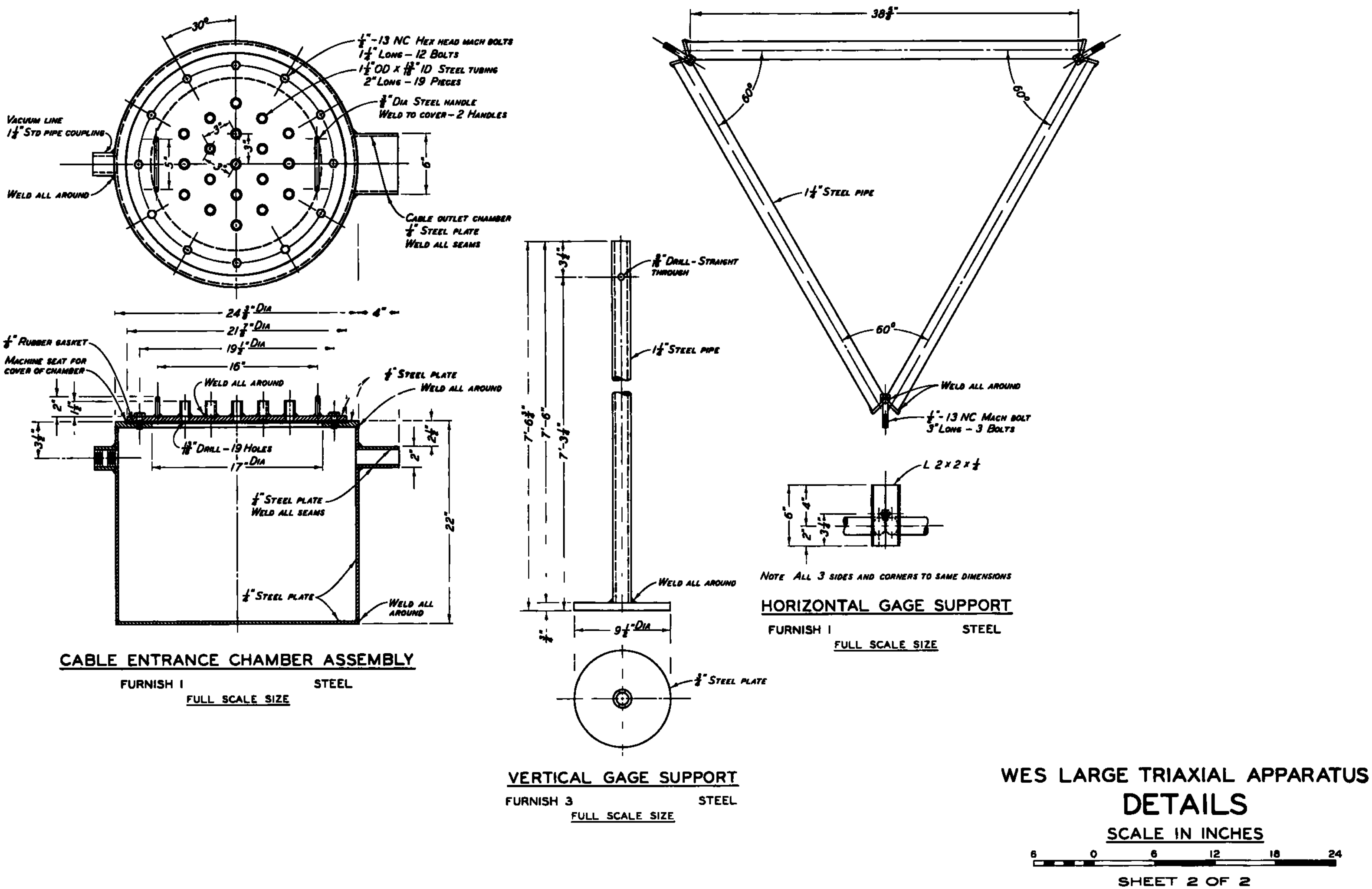

1 Intermittent weld-6" C to C Top and bottom - 6 Flanges
¼" Steel plate
Weld full length
Lifting loop - ⅜" Dia steel Weld to top flange - 2 Loops
See DETAIL "A"
Weld full length
120°
17⅞" Rad
¼" Rubber gasket cut to fit On one end only each segment
Vacuum Connector See DETAIL "A"
6 - ⅜" Flanges - 5 Spaces at 12" = 60"
⅝" Drill - 17 Holes - 16 Spaces at 4" = 64"
33"
72⅞"
39⅞"
68⅞"
FORMING JACKET SEGMENT
FURNISH 3 STEEL
FULL SCALE SIZE
⅝" Drill - 6 Holes Tap ⅜" - 16 NC
Drill - 5 Holes Tap ½" - 13 NC Center hole 1½" Deep
Vacuum Connector ½" Steel pipe 2 Connectors
All parts ½" steel plate Weld all members
35.68" Dia
½" Pipe coupling
Drill - 9 Holes
HEAD PLATE
FURNISH I STEEL
FULL SCALE SIZE
Drill 9 Holes
Weld all around
½" Steel pipe
½" Pipe thd
DETAIL "A"
FOUR TIMES SCALE SIZE
¼" Steel plate Weld all seams
¼" Steel rim
Cable Outlet Chamber
35.68" Dia
72"
36"
¼" Steel plate Weld all around
¼" Steel rim
¼" Steel plate Weld all seams
POROUS CONCRETE
CONCRETE BASE
½" Dia steel reinforcing - 6" C to C - Each direction
BASE
FURNISH I STEEL AND CONCRETE
HALF SCALE SIZE
WES LARGE TRIAXIAL APPARATUS
DETAILS
SCALE IN INCHES
6 0 6 12 18 24
SHEET I OF 2

Figure 3.



CABLE ENTRANCE CHAMBER ASSEMBLY
FURNISH 1 STEEL
FULL SCALE SIZE
VERTICAL GAGE SUPPORT
FURNISH 3 STEEL
FULL SCALE SIZE
HORIZONTAL GAGE SUPPORT
FURNISH 1 STEEL
FULL SCALE SIZE
NOTE ALL 3 SIDES AND CORNERS TO SAME DIMENSIONS
WES LARGE TRIAXIAL APPARATUS
DETAILS
SCALE IN INCHES
SHEET 2 OF 2

Figure 4.

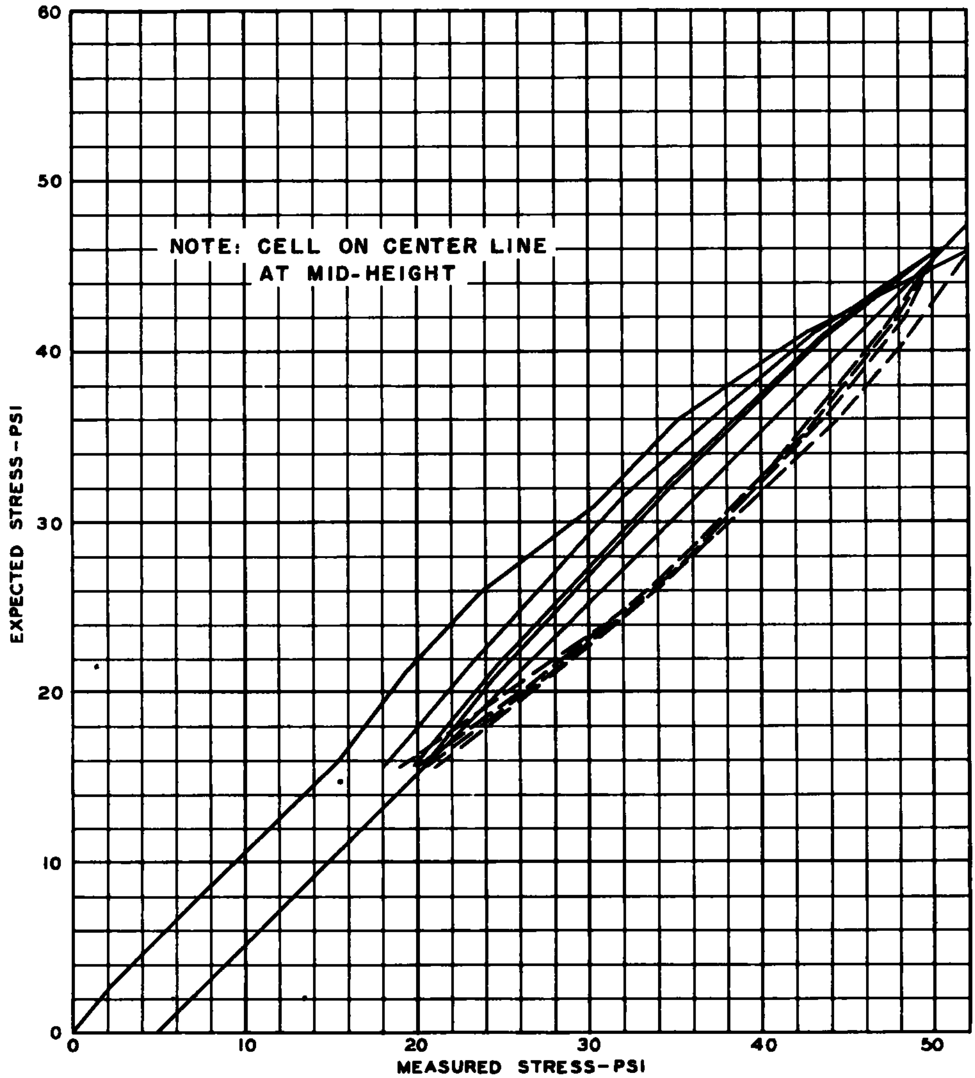


Figure 5. Typical vertical stress plot.

tion gages. Vertical gages are placed 120 deg apart at the edge of the headplate, and horizontal gages can be placed at any desired height along the three legs of the pipe frame.

A uniform sand is the only soil thus far used in the Waterways Experiment Station large triaxial apparatus. It is a processed mortar sand, substantially all of which falls between the 10- and 200-mesh sieves, and which has about 50 percent passing the 40-mesh sieve. In all cases the sand was air dry.

As has been mentioned, Waterways Experiment Station pressure cells were installed in the large triaxial specimens to measure stresses. These cells are 6 in. in diameter and 1 in. in thickness. They have a 100 percent sensitive face and use a mercury-filled pocket beneath the faceplate to transmit strain to a sensitive diaphragm. Details of this cell are not sufficiently pertinent to this paper to warrant further description here, but the cells are described in detail in another reference (2). In addition to the pressure cells, a number of soil strain gages were installed in several specimens.

These gages utilize differential transformer elements to measure relative displacement of reference diaphragms. The particular gages used were developed at the Ohio River Division Laboratories of the Corps of Engineers and are described in some detail in another reference (3). Figure 4 shows a typical plot of some of the data being collected.

FINDINGS OF INTEREST

The project has not progressed to the point that comprehensive results can be presented, but certain preliminary or tentative findings can be stated, as follows:

1. Considerable variation exists between magnitudes of stresses at various points within a triaxially loaded sand mass.
   a. At mid-height, vertical stresses near the center are appreciably greater than those at the edge of the specimen.
   b. Near the head and base plates, vertical stresses near the center are appreciably lower than those at the edge of the specimen.
2. Considerable variation exists between magnitudes of strains at various points within a triaxially loaded sand mass. Vertical strains in the center of the specimen may be several times as large at mid-height as near the top and bottom.
3. Though only a single soil at a single moisture condition (dry) was used, specimens at various densities were tested. The pressure cell results indicate no effect of density on the distribution of stresses throughout the specimen.
4. Since the pressure cells were stiffer than the surrounding soil, they might be expected to yield readings somewhat too large. This was the case, though the magnitudes of over-registration have not yet been fitted to a neat pattern.
5. In comparing the results from the large apparatus and equivalent small tests, several trends can be noted:
   a. Large specimens showed average vertical strains less than those for the small laboratory specimens when subjected to the same external stresses.
   b. Similarly, the slopes of stress-strain curves (stress plotted vertically) were greater for the large specimens.
   c. The angle of internal friction, ϕ, for the dry sand when determined from large triaxial specimens was generally about a degree larger than when determined from the small laboratory specimens.

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Granular Earth Pressures on Steel Tunnel Lining

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E. VEY, Professor of Civil Engineering, Illinois Institute of Technology

A number of field measurements was made to determine the pressures and corresponding deflections for an arch type steel lined tunnel under storage piles of crushed stone. The stone was $1\frac{1}{2}$ inch material and uniform throughout.

The tunnel was constructed of $\frac{1}{4}$ inch steel plate sections, flanged on all edges inwardly. These sections were bolted together through the flanges on the inside of the tunnel, thus leaving the outside surface fairly smooth and uniform and making the tunnel quite flexible. The width of the tunnel, at the concrete base slab, was approximately 7 ft and the height at the crown 7 ft.

Goldbeck pressure cells were carefully calibrated and mounted underneath the base slab and around the outside of the tunnel during construction. Deflection gages were also mounted inside the tunnel at the section where the pressure gages were placed.

Measurements were made over a period of several months during which time the crushed stone overburden was placed on the tunnel to a maximum height of 70 ft and then removed. It was possible, therefore, to obtain measurements at various heights and configurations of overburden load.

The results indicated that the tunnel lining experienced the full overburden pressure at the crown and the pressures around the rest of the tunnel tapered off to a value approximately 0.3 times the vertical overburden at the base.

A recommended loading is given, based on the experimental data. It consists of a trapezoidal pressure distribution over the horizontal projection of the tunnel and a distribution over the vertical projection which is rectangular on the upper one-third and trapezoidal on the lower two-thirds.

● ONE phase of the general field of bulk materials handling has to do with the storage and the reclamation of material from storage.

The material in question might be sand, gravel, various sizes of screened or unscreened crushed limestone, coal, crushed, screened and unscreened ore, slag, etc. The piles might be stacked up to 120 ft in height by means of stackers, self-unloading boats, trippers of belt conveyors, grab buckets, etc., as shown in Figure 1.

One of the mechanical methods used to reclaim stored material in great volumes is by means of a belt conveyor installed in a tunnel under a storage pile (reclaiming systems up to 8,000 tons per hour are known). A number of gates and chutes suspended from the tunnel top can be opened or closed to allow the flow of material from the pile to the moving belt conveyor in the tunnel. The capacity of material flowing to the belt may be controlled by the size of the gate opening, slope of the chute, speed of the belt and the number of open gates.

As soon as the gate has been opened, material directly over the gate loses its support and slides down the chute to the belt, thus producing a cavity in the storage pile directly over it. This cavity increases in size as time goes on and finally becomes an inverted cone with its apex at the gate and its base as large as the angle of repose permits. If the material arches and stops the flow, the next gate can be opened, thus building another cavity and eventually destroying the leg of the arch at the first gate. This procedure may be repeated, providing complete control of the reclaiming process.

As a consequence of this type of reclamation the pile becomes separated into two smaller piles along both sides of the tunnel with the sides sloping at the angle of repose of the material.

Meanwhile, the original pile may be reloaded fully or partially, or with the help of a bulldozer one or both piles on the sides of the tunnel may be pushed toward the tunnel gates and reclaimed.

In the design of the tunnel the following varying load conditions have to be taken into account:

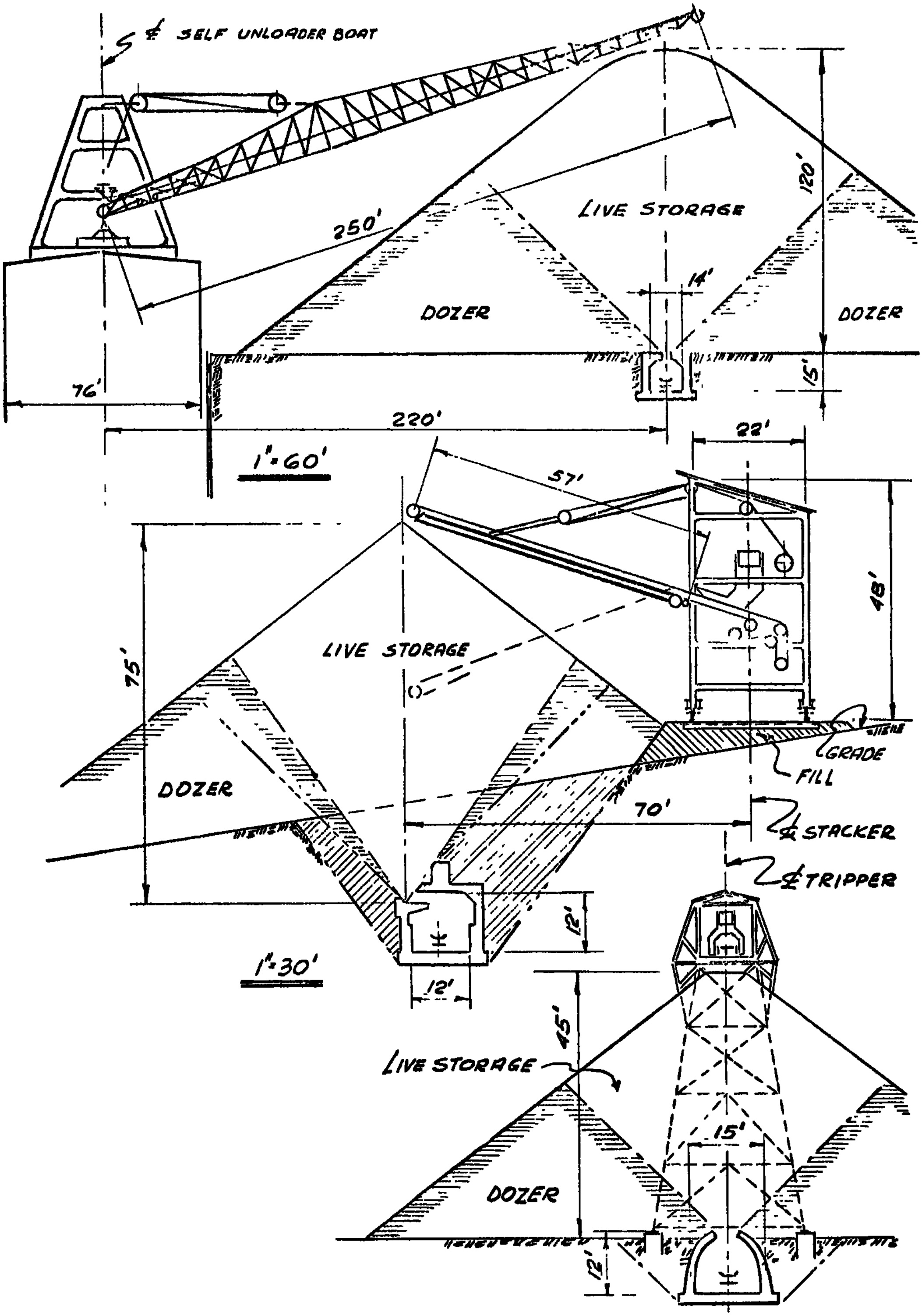


Figure 1. Sections through typical storage piles.

1. Tunnel fully loaded with either a triangular or prismatically shaped pile in cross section (full load).
2. Tunnel loaded from both sides by two triangularly shaped piles and no overburden load on the center of the tunnel.
3. Tunnel loaded from one side by triangularly shaped pile and no overburden load on the center or far side.
4. Intermediate loadings between a, b and c.

In order to design the tunnel shell it is necessary to know the pressure distribution around the tunnel under all conditions of loading. But because of the flexibility of the tunnel and the unusual configuration of some of the loadings, the use of available earth pressure formulas are questionable.

It has been the custom with some designers to take two-thirds of the pile height and consider it as a full weight on the horizontal projection of the tunnel and forty to fifty percent of that load on the vertical projection of the tunnel.

For the (2) and (3) loading conditions stated above, designers have used Culman's graphical method taking an angle of internal friction ϕ slightly larger than that of repose and fully or partially developed wall friction angle ϕ_1. An angle of sliding of 55 to 60 degrees, half developed ϕ and fully or half developed ϕ_1 have also been used.

Of the many tunnels so designed and constructed, only two failures are known to the writers, and they seem to have been the result of poor workmanship in the field rather than inaccuracy in design. However, the selection of the type of loading for which to design, the type of tunnel to be used and the questionable validity of the method of load distribution used, prompted the present experimental investigation.

An opportunity for this investigation arose through the cooperation of the consulting engineering firm of John F. Meissner Engineers, Inc. of Chicago, the Commercial Shearing and Stamping Company of Youngstown, Ohio, manufacturers of the tunnel lining

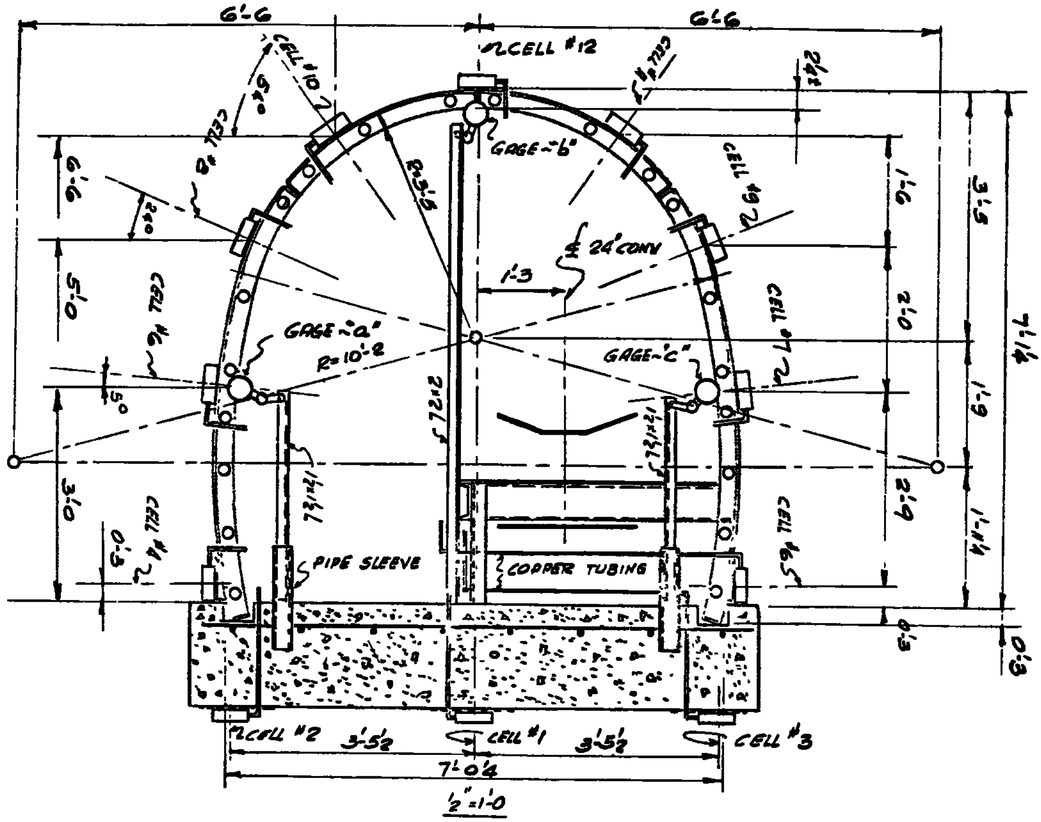


Figure 2. Arrangement of dial gages and Goldbeck pressure cells.

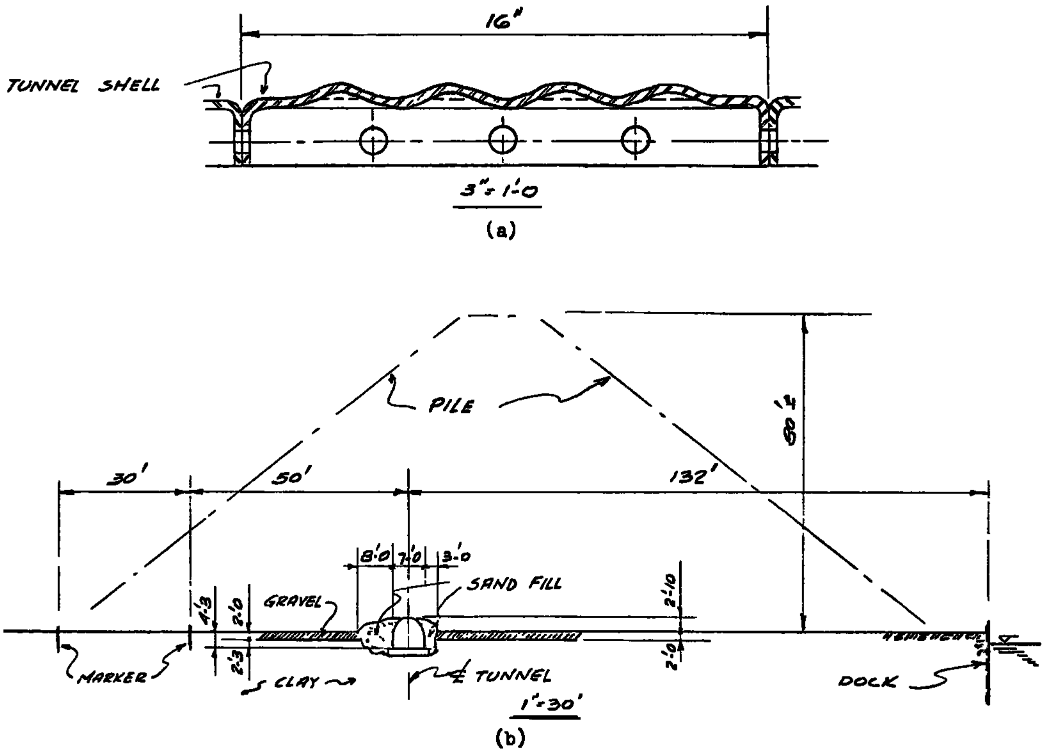


Figure 3. (a) Longitudinal section through liner plate, (b) section through tunnel and maximum size stock pile.

used, and the Marblehead Lime Company of Chicago, Illinois, who granted permission and the necessary labor to perform tests on a newly installed tunnel at their South Chicago Plant.

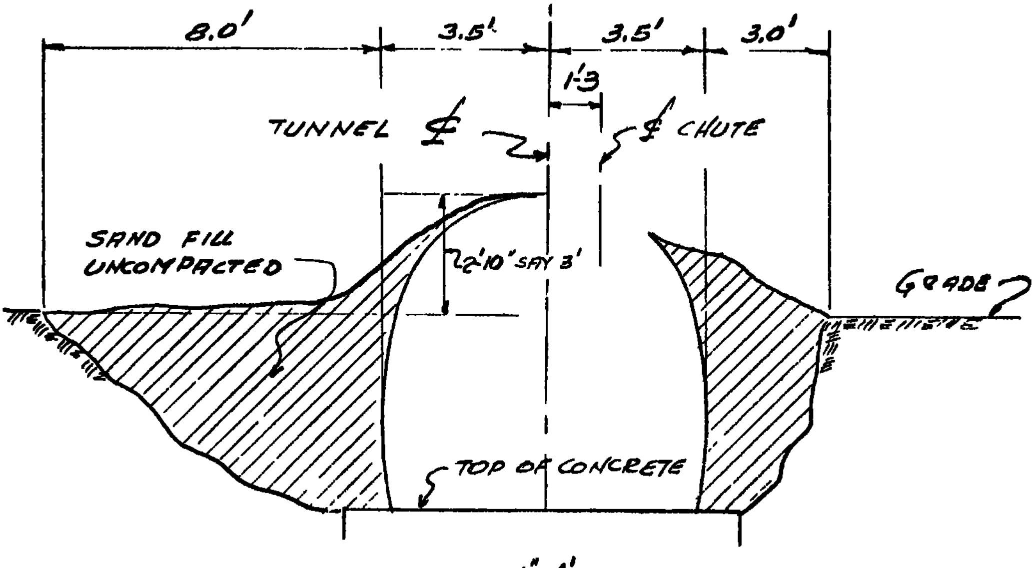
It was decided to try direct field measurements of pressures on the tunnel shell, together with corresponding deformations of the tunnel walls.

SITE OF TEST TUNNEL AND INSTALLATION OF GAGES

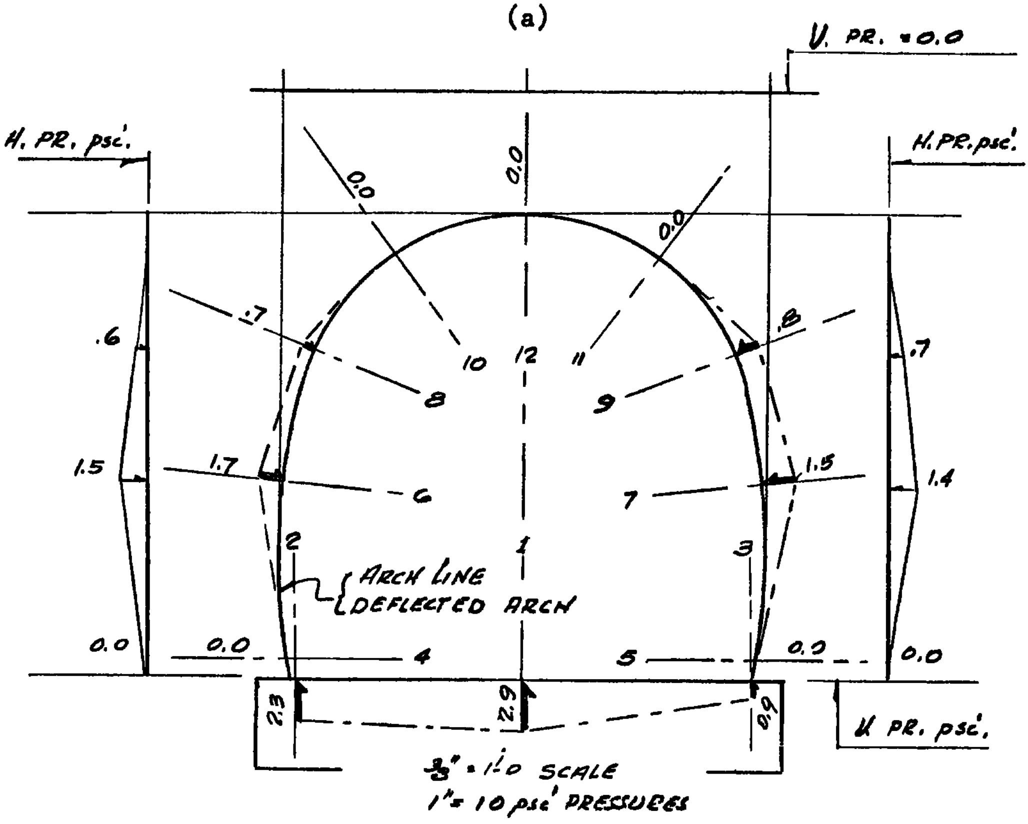
The tests were made on a new tunnel being constructed at the South Chicago Plant of the Marblehead Lime Company. The tunnel under study was to serve as a reclaiming unit for 1½ in. crushed limestone where the storage piles over the tunnel would at times reach a maximum height of as much as 80 ft.

Figure 2 shows a section through the completed tunnel. It was constructed of ¼ in. steel liner plate manufactured by the Commercial Shearing and Stamping Company. The sections of plate used were 16 in. wide and of different lengths to fit tunnel sections of varying radii. They were corrugated in one direction and flanged on all sides with bolt holes in the flanges for connection to adjacent sections. Figure 3 (a) shows a section through a typical liner plate and Figure 3 (b) the relative proportions of the tunnel and stock pile. The springing line of the tunnel was located approximately 4 ft - 4 in. below the grade and bolted to an 18 in. reinforced concrete slab. The crown was exposed approximately 2 ft - 9 in. above the grade. The excavated area surrounding the tunnel was backfilled with sand after the tunnel was constructed, Figure 4 (a).

Goldbeck pressure cells were used for measuring pressures and these were mounted underneath the slab and around the outside of the tunnel at the locations shown in Figure 2. Cells 1, 2 and 3, underneath the slab failed to function shortly after being installed so no readings were obtained from these cells. All other cells seemed to function satisfactorily during the entire test period. All cells had been carefully calibrated and checked in the laboratory before installation.

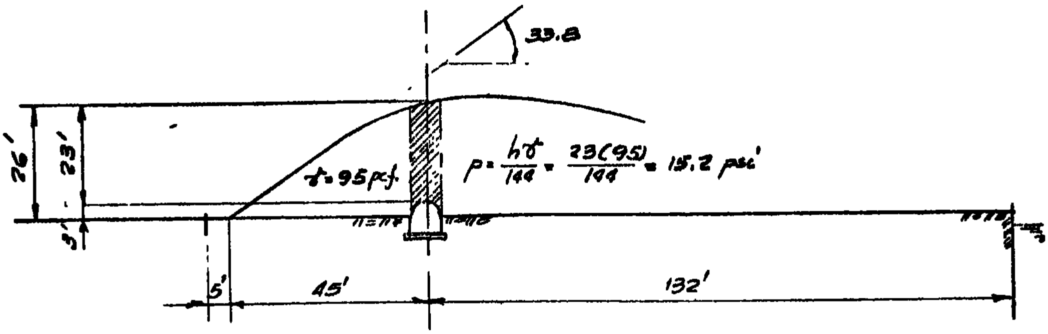


(a)

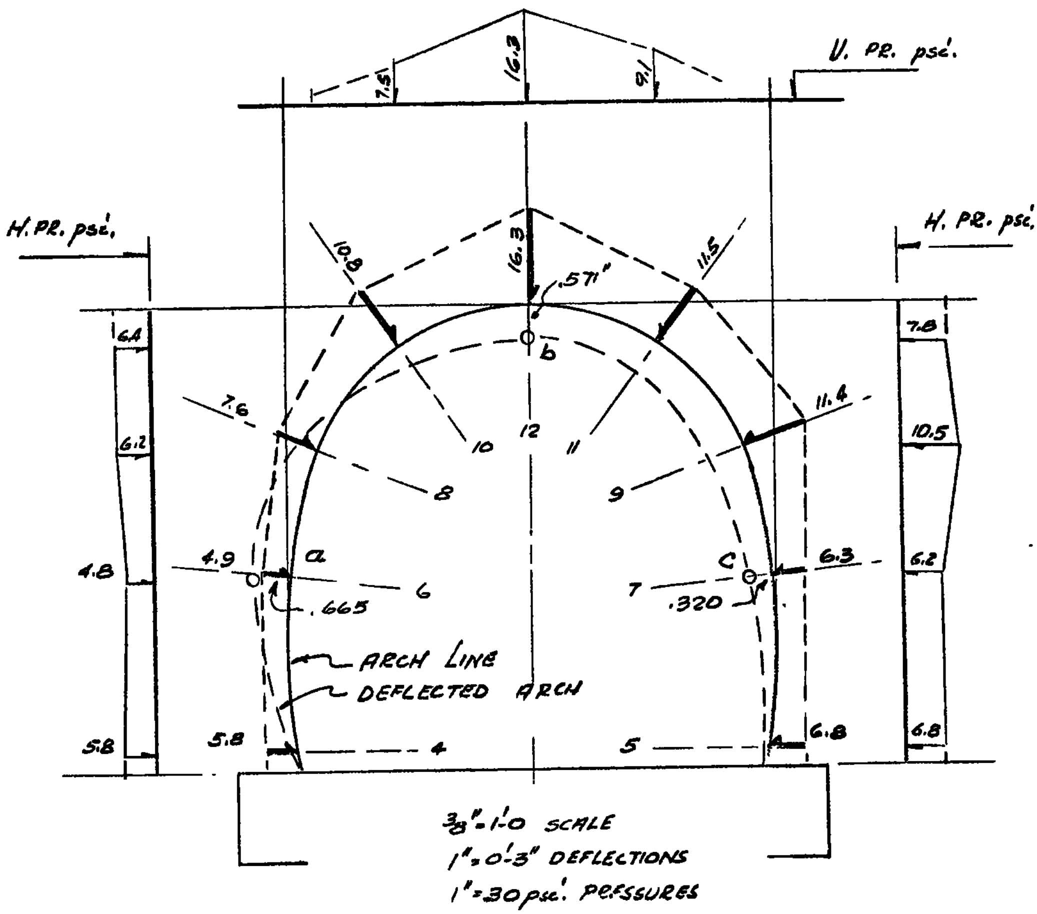


(b)

Figure 4. Field measurements 1, (a) backfilled tunnel before loading (b) gage measurements.

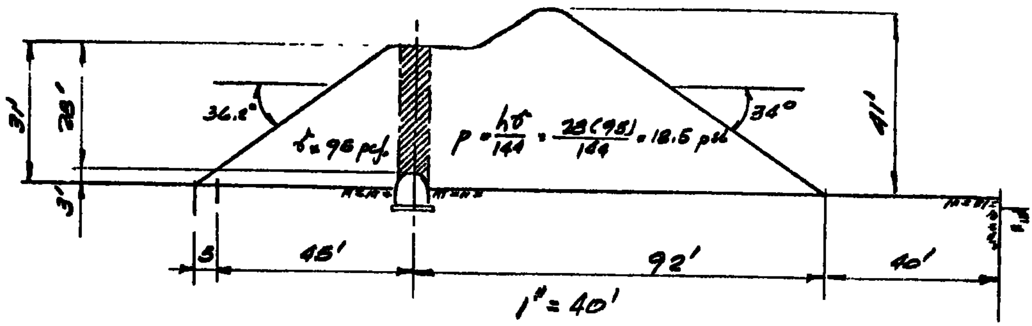


(a)

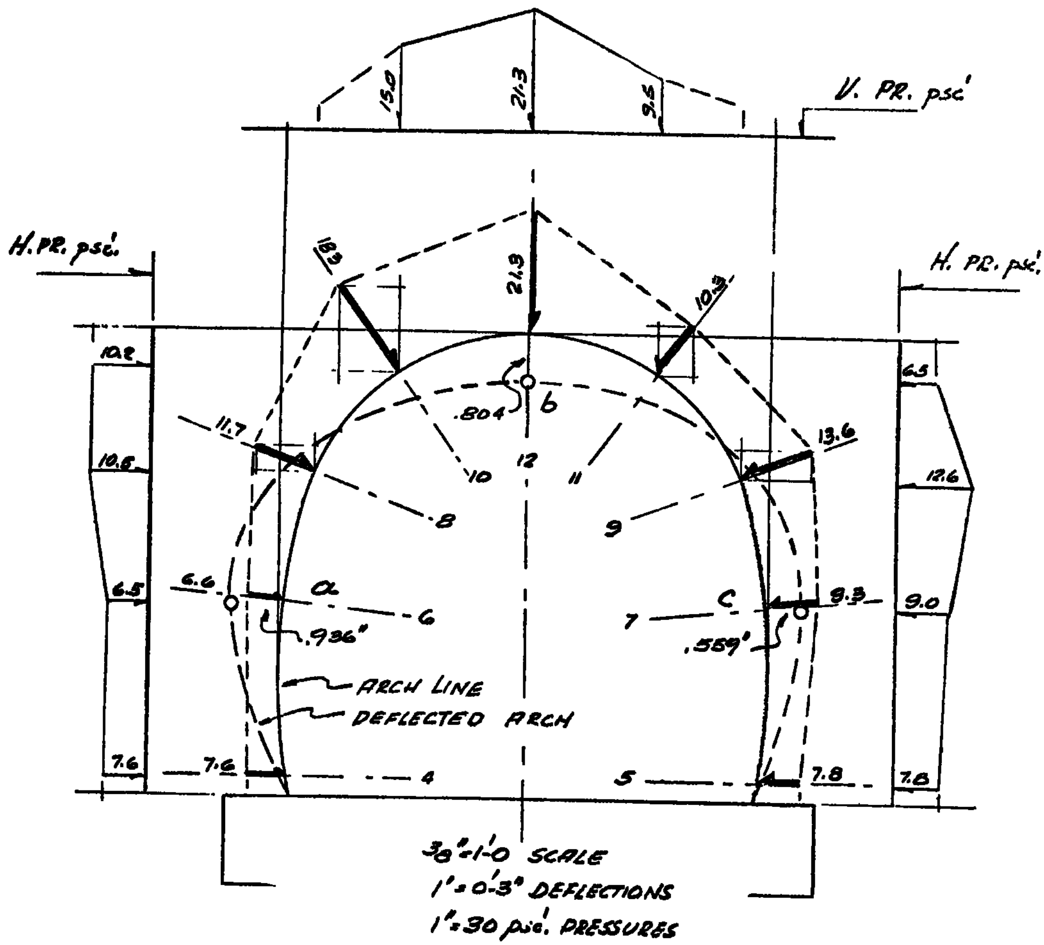


(b)

Figure 5. Field measurements 2, (a) loading diagram, (b) gage data.



(a)



(b)

Figure 6. Field measurements 3, (a) loading diagram, (b) gage data.

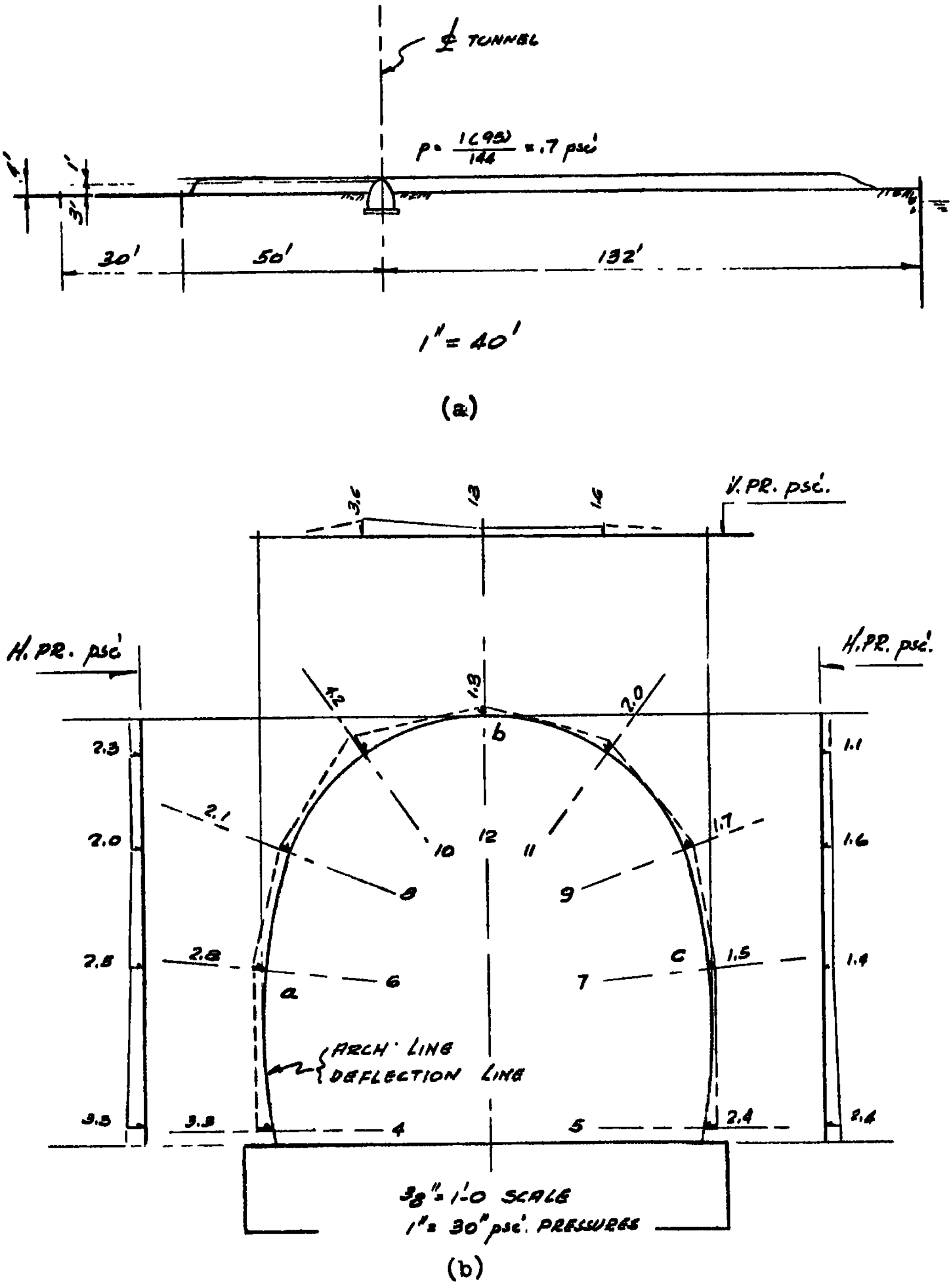


Figure 7. Field measurements 4, (a) loading diagram, (b) gage data.

In addition to the pressure cells a number of dial gages reading to one thousandth of an inch were mounted inside the tunnel at pressure gage locations 6, 7 and 12. These were used to determine how the deflections varied with pressures. The location of the dial gages are shown in Figure 2. They were mounted on steel angles cantilevered from 2½ in. pipe sleeves set in the concrete base slab.

Cells 4 through 12 were secured against slipping and tilting by means of welded back plates, clips and hooks. The diaphram of every cell was kept unobstructed, and parallel to the slope of the tunnel wall. The cells were checked before and after sand backfilling around the tunnel and no appreciable readings on the cells could be recorded after backfilling.

FIELD DATA

Readings of pressures and deflections were recorded at various stages of loading as the crushed stone pile was built up over the tunnel. The cross section of the stock pile was determined by making a survey of the pile at the gage section. Figures 4 through 12 show the plotted data. Each figure shows a cross section of the tunnel and stock pile at the gage location and the measured radial pressures are plotted around the tunnel as well as their horizontal and vertical projections. The displacements of the tunnel are also plotted from the original arch line, at an exaggerated scale for clarity.

The measurements were performed in two cycles. Figures 4, 5 and 6 represent a full cycle of loading only. Figures 7 through 12 show a full cycle of loading, reclamation and complete unloading. The displacement of the tunnel in every case was measured with respect to the unloaded condition Figure 4 in the first cycle and Figure 7 in the second cycle. Figure 12 shows the final pressures and deflections of the unloaded arch before the gages were removed.

The radial pressure data plotted on Figure 4 through Figure 12 were made up of averages of second, third and fourth readings. The first readings usually were fifteen to twenty percent higher than the following ones. This could have been caused by the piston sticking to the body of the cell requiring an additional internal pressure to break this seal. The first release of air and the second and third injections or releases showed about the same pressures. These pressures were recorded for use. With an increasing number of injections within a short period of time the pressures on the cell pistons gradually decreased. The phenomenon could be due to the movement of the cell cylinder. Even a very short stroke could temporarily displace the sand behind the piston thus decreasing the pressure on the cell. The average of the few readings as explained, were corrected for zero error as previously determined in the laboratory and the results plotted in Figure 4 through Figure 12.

SUMMARY OF FIELD DATA

In developing general pressure distributions for use in design, two conditions were considered. The first condition was when the greatest height of stock pile was directly over the tunnel crown as represented in Figures 5, 6, 8 and 9. The second was when two stock piles of maximum height stood on both sides of the tunnel with no load on the crown as shown in Figure 11. It is apparent that for design purposes the most severe condition of loading is the first. From the nine sets of data taken, only four showed a full load and these are represented in Figures 5, 6, 8 and 9. However, in the summary the data shown in Figure 8 was omitted because some of the gage readings were so far out of line with the other measurements that they looked unreasonable.

For the first condition the measured pressures were averaged for the remaining three cases and the vertical component of the radial pressure over the full width of the tunnel computed. This pressure was then expressed as the product of a pressure factor and the overburden weight and the derived pressure diagram was approximated by a trapesoid. The lateral pressure was then computed as the horizontal component of the radial pressure on a vertical plane from the crown of the tunnel to the base and the pressure distribution approximated by another trapezoid. The vertical pressure was constant and approximately equal to 1. 1 times the overburden over the middle third of the horizontal projection. Over the outside third on each side the pressure tapered to

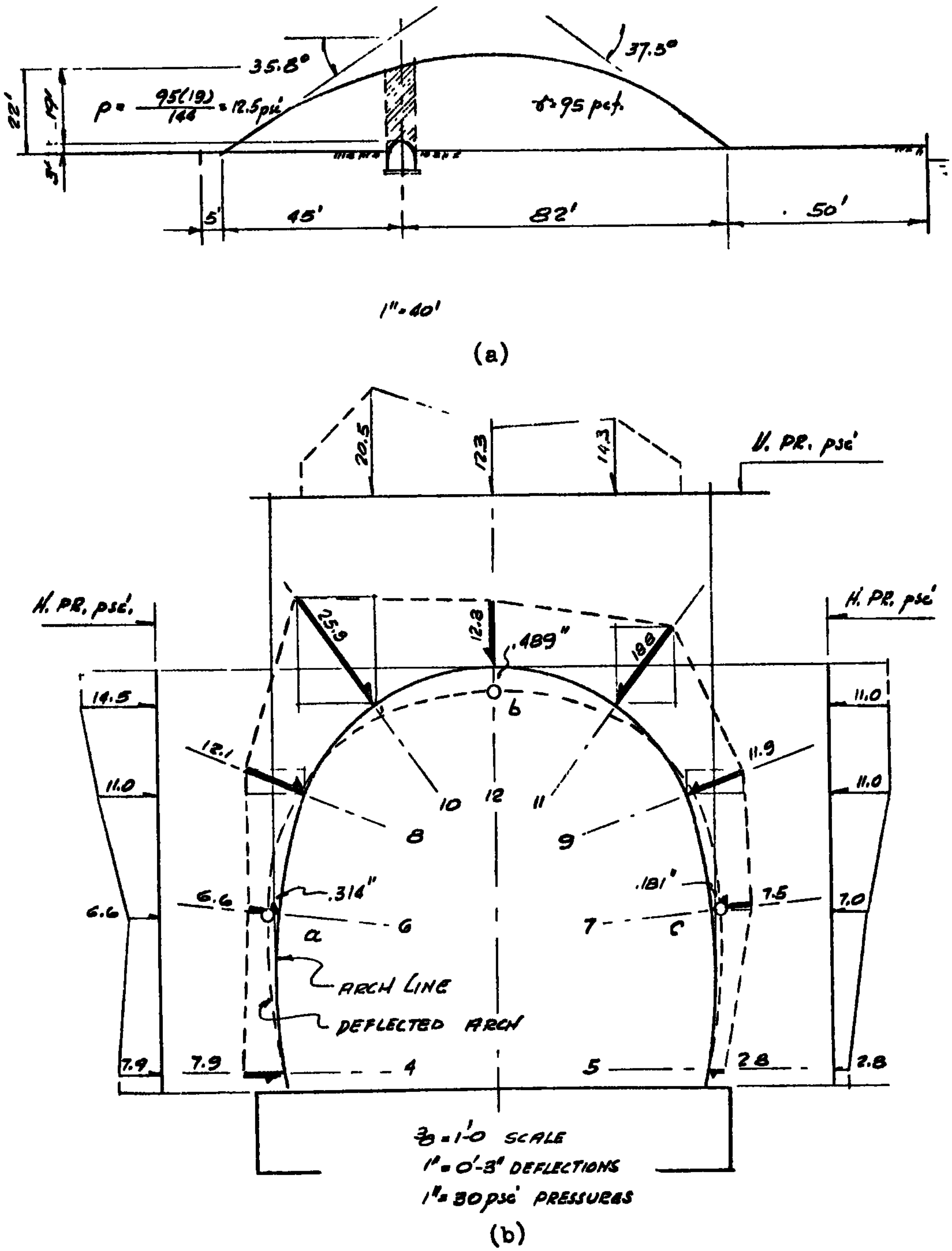


Figure 8. Field measurements 5, (a) loading diagram, (b) gage data.

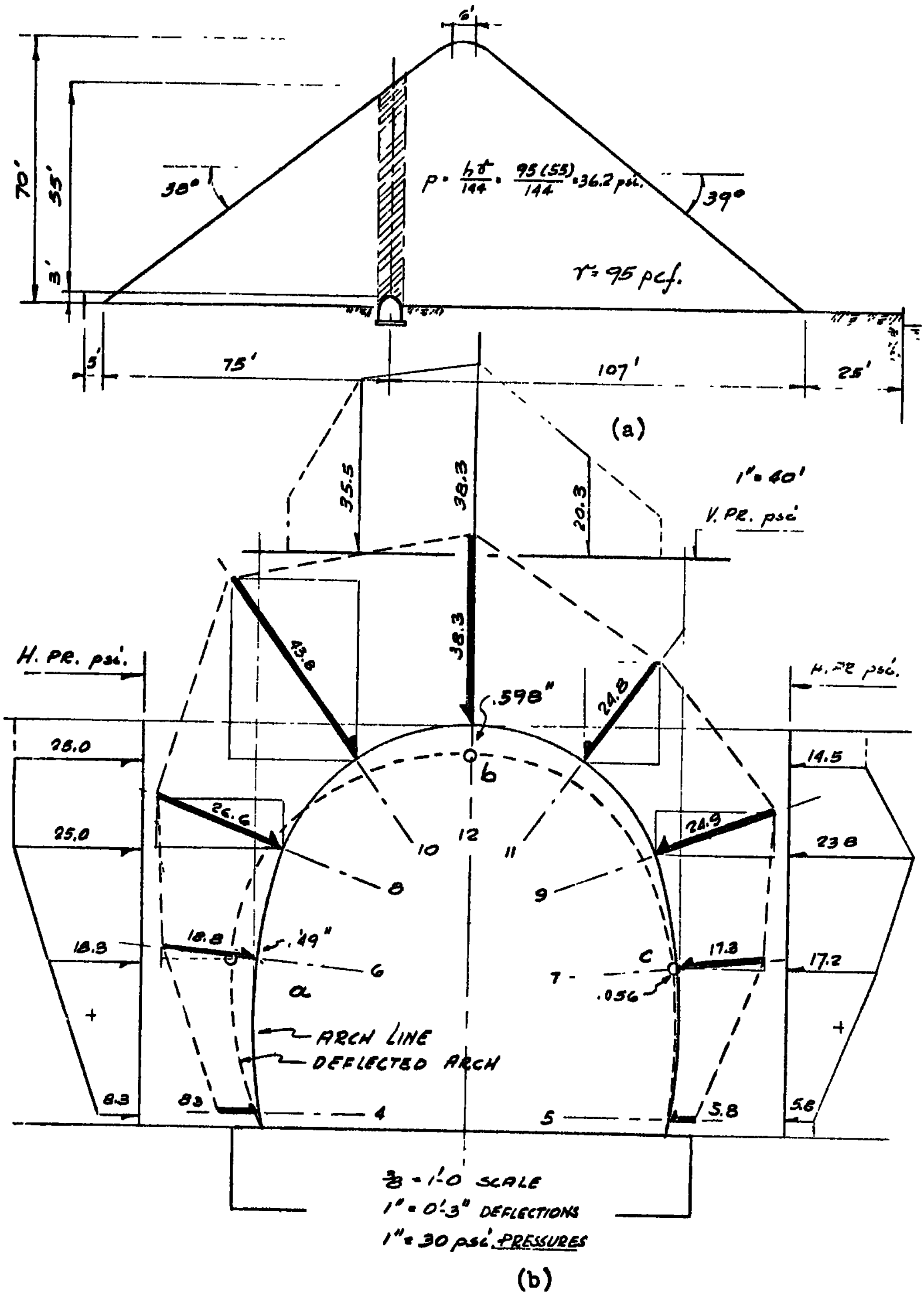


Figure 9. Field measurements 6, (a) loading diagram, (b) gage data.

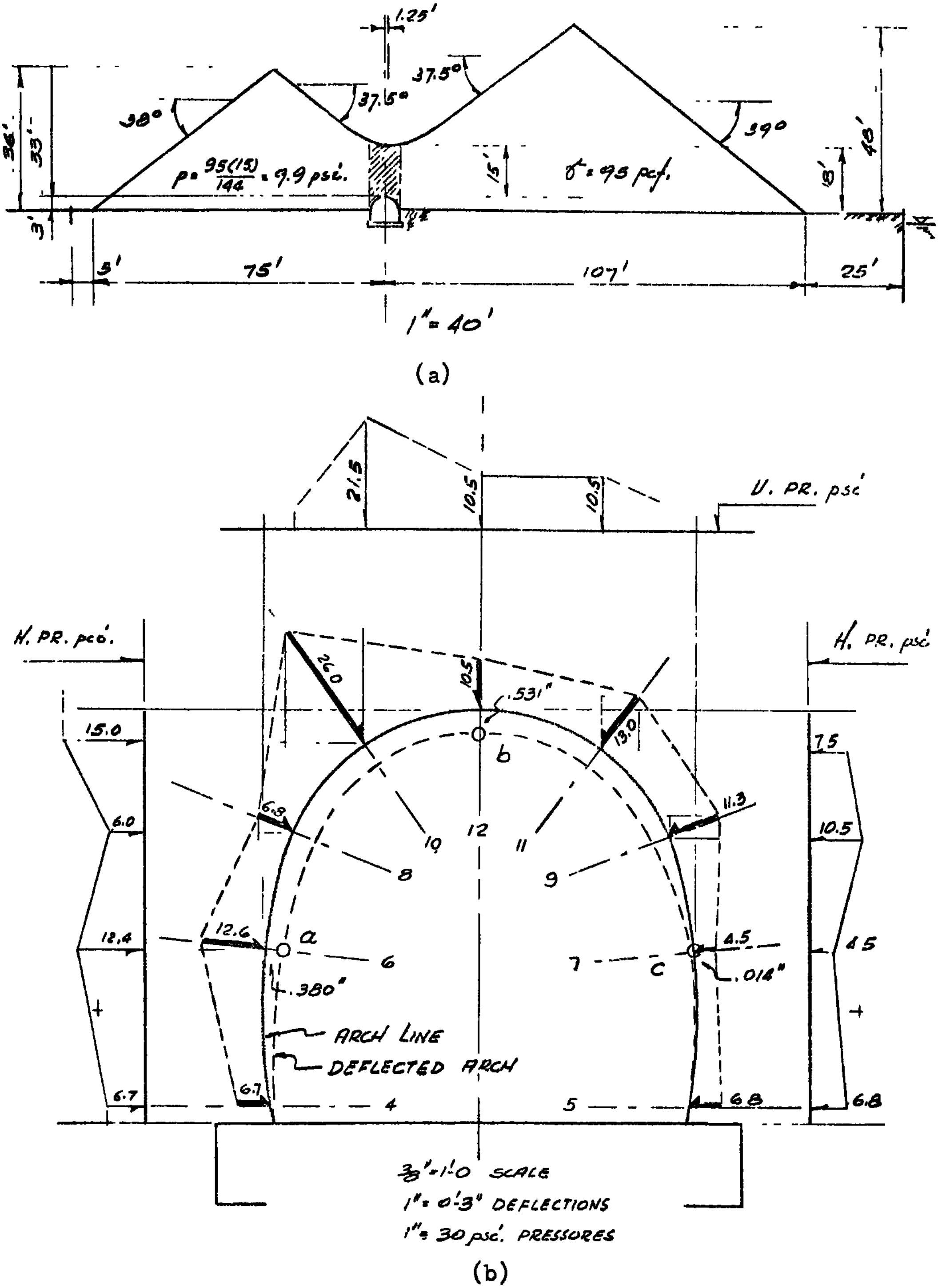


Figure 10. Field measurements 7, (a) loading diagram, (b) gage data.

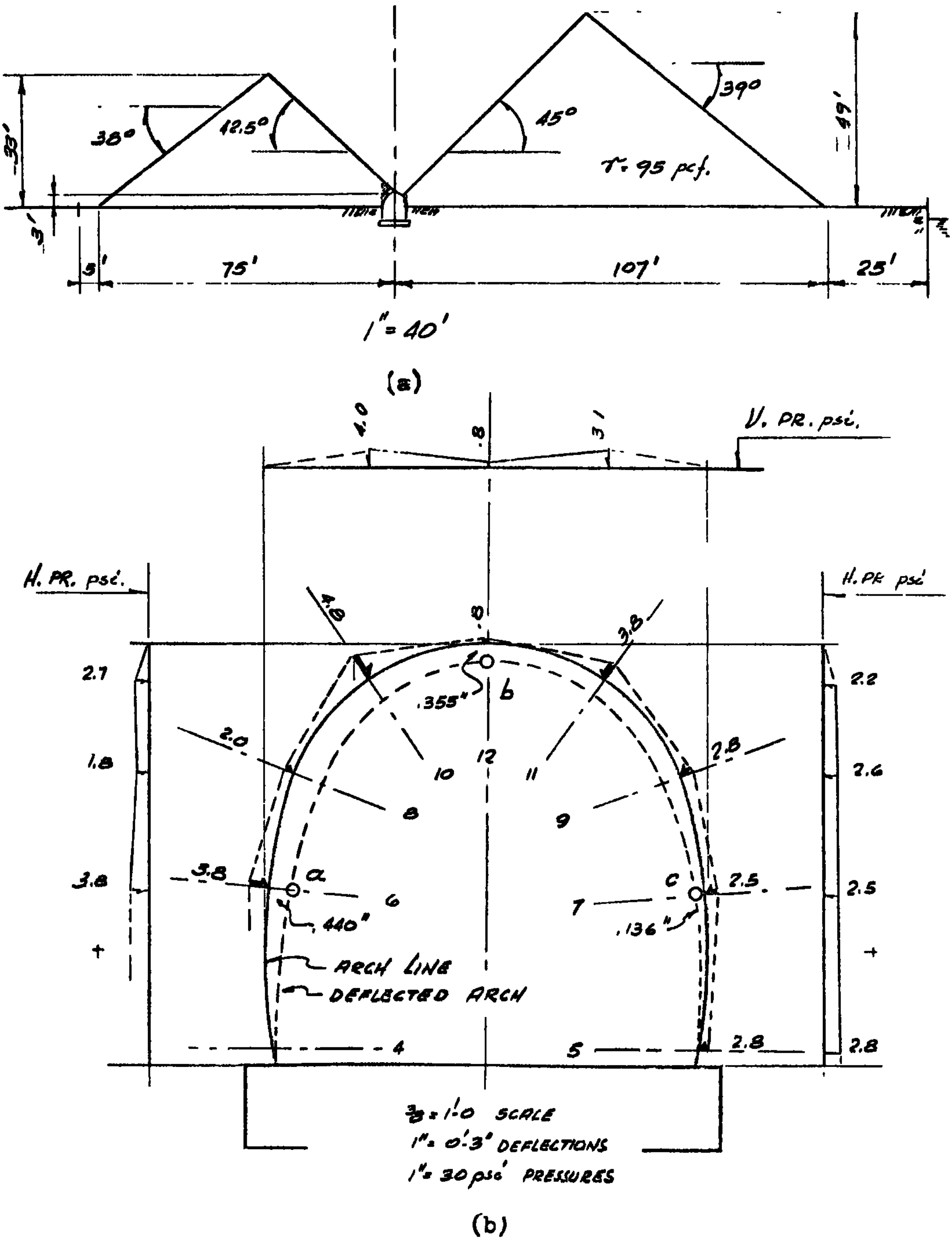


Figure 11. Field measurements 8, (a) loading diagram, (b) gage data.

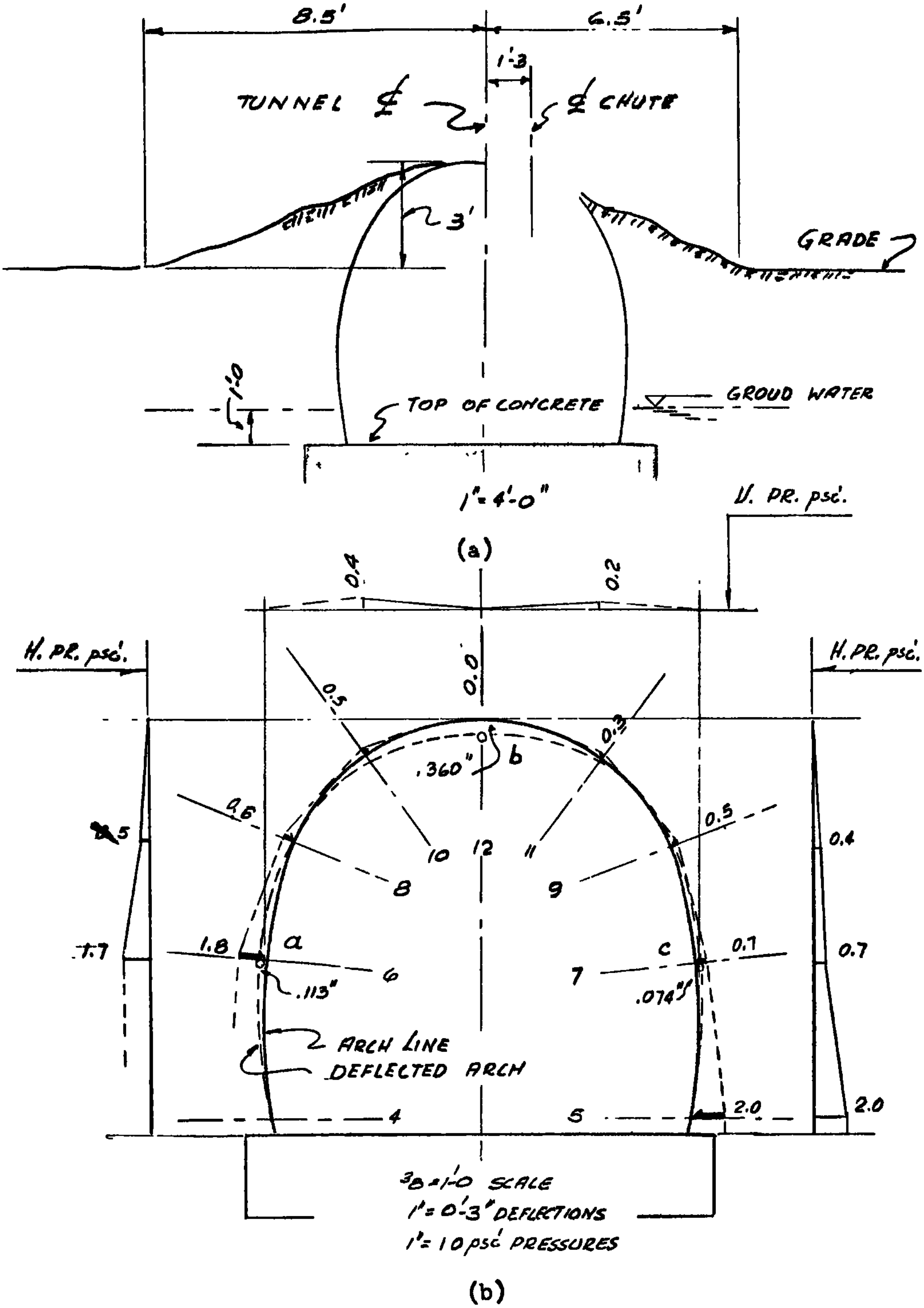


Figure 12. Field measurements 8, (a) unloaded tunnel, (b) gage data.

zero. The same procedure was followed for arriving at a lateral pressure diagram. In this case the pressure was approximately constant over the upper third to the height of the tunnel and equal to about 0.55 times the overburden (γH) over the crown. Over the lower two-thirds the pressure varied linearly from the upper value of 0.55γH to approximately 0.12 times the full overburden pressure γ(H + h) at the bottom.

These pressure diagrams were constructed to give the total measured force over the vertical and horizontal projections and at the same time to approximate as closely as possible the actual distribution of the three loadings. A summary of the results is shown in Table 1 and Figure 13 shows the derived pressure distribution diagrams.

The second loading condition represented in Figure 11 shows the tunnel loaded on each side with no load directly over the crown. The recorded data show comparatively small pressures. Apparently the shear resistance on planes parallel and adjacent to the slopes is large enough to resist nearly all the shear forces. This is reasonable since fracture and sliding of the outside of the pile occurred as the material was withdrawn until the final inclinations of the separated pile slopes were reached. These slopes represent final fracture planes.

Deflections in the first loading cycle were measured from the initial condition shown in Figure 4. Figure 5 shows some lateral movement of the arch toward the left. Apparently the load was built up on the tunnel from the right side due to the boom location of the self-unloader boat. A higher overburden load (Figure 6) produced a much higher deflection of the crown resulting in almost a symmetrical deflection of the sides. In this

TABLE 1

SUMMARY OF FIELD DATA

Fig. No.	Lateral Pressures Gage No. psi 8	10	11	9	Avg	Height of Pile H ft	Vert Load Hγ psi	Lat Pr / Hγ %	Height H + h ft	Vert Load (H + h)γ psi	Lat Pr / (H + h)γ %	
5	6.2	6.4	7.8	10.5	7.7	23.0	15.1	51.0	23 + 7			
6	10.5	10.2	6.5	12.6	10.0	28.0	18.6	53.7	28 + 7			
8[a]	11.0	14.5	11.0	11.0	11.9	19.0	12.5	~~95.4~~	19 + 7			
9	25.0	25.0	14.5	23.8	22.1	55.0	36.2	61.3	55 + 7			
	Gage No.					Av Fig 5, 6, 9		55.3				
	4	6	7	5	Avg	Av Fig 5, 6, 8, 9		~~65.3~~	a			
5	5.8	4.8	6.2	6.8	5.9				30.0	19.7	30.0	
6	7.6	6.5	9.0	7.8	7.7				35.0	23.0	33.4	
8[a]	7.9	6.6	7.0	2.8	6.1				26.0	17.1	~~35.6~~	
9	8.3	18.3	17.2	5.8	12.4				62.0	40.9	30.4	
									Avg Fig, 5, 6, 9		31.3	b

Fig. No.	Vertical Pressures Gage No. psi 10	11	Avg			%	Vert Pr (data average) / Hγ
5	7.5	9.1	8.3	23.0	15.1	55.0	
6	15.0	9.5	12.3	28.0	18.6	51.3	
8[a]	20.5	14.9	17.7	19.0	12.5	~~119.0~~	
9	35.5	20.3	27.9	55.0	36.2	56.2	
				Avg Fig. 5, 6, 9		54.2	c
	Gage No. 12 psi						
5	16.3			23.0	15.1	108.0	
6	21.3			28.0	18.6	114.5	
8[a]	19.0			19.0	12.5	~~98.7~~	
9	38.3			55.0	36.2	105.5	
				Avg Fig. 5, 6, 9		109.3	d

[a] Figure 8 data in this summary not used.

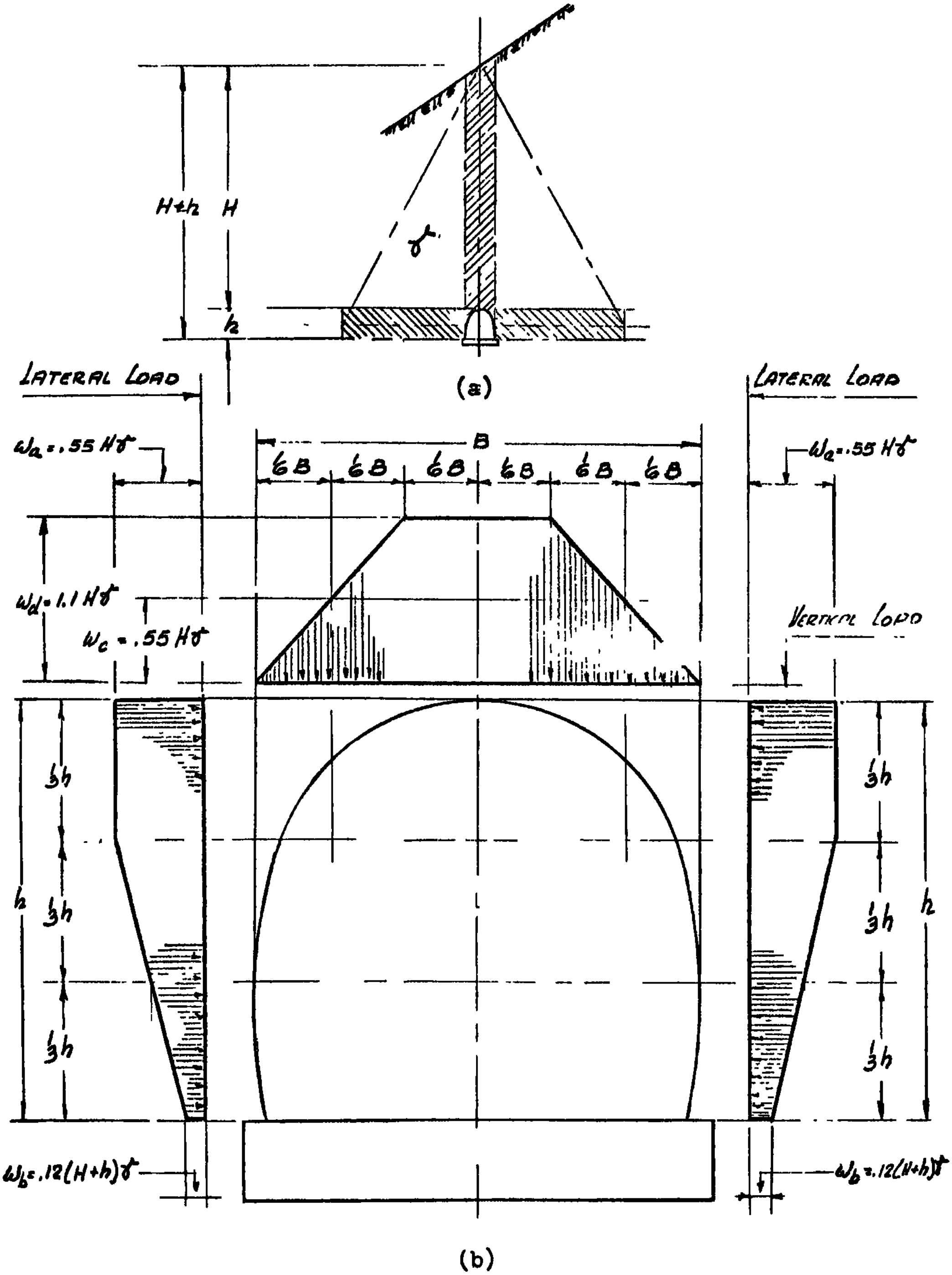


Figure 13. Recommended design loading, (a) loading diagram, (b) pressure diagram.

cycle no measurements of the deflections during the unloading of the tunnel were taken.

The second cycle started with the measurements shown in Figure 7 and this position of the arch was taken as the base for all following loading conditions. A nearly symmetrical deformation of the tunnel shell is shown in Figure 8 followed by some sidesway due to the unsymmetrical loading shown in Figure 9.

As the overburden load decreased, leaving in place two piles on the sides of the tunnel (Figures 10 and 11), the lateral deflections changed their directions and the tunnel moved inwards. At the same time the crown moved up a little, but did not reach the original position of Figure 7.

The deflection line did not reach the original tunnel line even at the complete unloading, Figure 12. This was probably caused by some permanent bending, shortening of the shell or readjustment of the bolted and painted joints.

The deflections are larger on the left side of the tunnel than on the right side. This could be due to a wider sand backfilled area on the left side, thus allowing more compression and larger deflections of the left rib of the arch.

DISCUSSION OF RESULTS

It is believed that the Goldbeck pressure cells were quite reliable and the data which they gave were quite accurate measurements of the pressures acting on the face of the cell. There are several factors, however, which may have influenced those pressures and this raises the question as to whether the values measured at any given location were the true pressures acting on the tunnel lining.

The cells were mounted on the outside surface of the tunnel which meant that the cell protruded beyond the surface of the wall approximately 2 in. This together with the rigidity of the cell and the stiffening effect of the small angles used to hold the cell in place undoubtedly had some effect on the yielding of the tunnel lining and hence on the pressure. A factor which apparently has a very pronounced effect on the measured pressure is the movement of the sensitive part of the cell face necessary to actuate the gage. It has already been pointed out that for a particular cell reading the pressure necessary to get the initial reading was abnormally high. The next three or four pressure cycles gave essentially the same value and subsequent cycles gave decreasing pressures. The first high value was attributed to sticking of the movable piston. The values which could be duplicated three or four times were taken as the true pressures, but it is obvious that the nature of the gage operation, i. e., movement of the cell face, although extremely small, against the granular fill is an undesirable feature of the gage. Any cell which undergoes a deflection whether produced by the fill or by being forced against the fill is going to be affected by such movement.

Probably the most meaningful pressure measurements are those under the heaviest overburderns, because the effect of the factors described above would be proportionately small compared to such effects as rigidity of the base slab, relative flexibility of the joints and the initial distortion of the tunnel. The latter would vary depending on where the first overburden loads were placed. The derived pressures shown in Figure 13 are therefore believed to be accurate enough for ordinary design purposes.

The average value of the pressures directly over the crown was found to be 10 percent greater than the overburden. This was based on the three loadings shown in Figures 5, 6 and 9. For the loading shown in Figure 8 the value was almost 2 percent less than overburden and if this value were included the average would be about 7 percent greater than overburden. Ordinarily the crown pressure might be expected to be less than overburden due to the shear resistance of the fill on vertical planes and to arching effects. If the tunnel had been made through an existing fill there probably would have been enough vertical movement of the overburden for this to occur. But because the fill was built up after the tunnel was in place and because the material flowed out over the crown from the unloader boom it was difficult for the static shear resistance and arching to develop. The static deflection which actually occurred took place very gradually and during the early stages of loading there were probably some small sudden deflections due to the dynamic effects of the moving stone. These deflections and corresponding dynamic pressures would tend to be sealed in by the resistance of the existing overburden. The same effect has been observed to produce abnormally high anchor tensions in anchored sheet piling as a result of compacting the fill above the anchor level.

The lateral pressures do not vary much over the depth of the tunnel for moderate overburden heights but they seem to taper off near the bottom for the greater overburdens. This is much more pronounced on the side of the tunnel which deflects inward from the original arch line. It is quite apparent that the lateral pressures are very much influenced by the way the tunnel lining deflects during loading operations. The pressures are largest where the tunnel shows the greatest outward movement and smallest where the greatest inward movement occurs. The manner in which the tunnel lining deflects depends principally on the looseness and symmetry of the initial sand fill and the side from which the tunnel is first loaded. The relative flexibility of the bolted joints also affects the deflection picture.

Results of the tests described in this report show that there are a number of variables which influence the pressures of granular earth on flexible tunnel lining of this type. As a next step in the investigation it is recommended that an attempt be made to separate the variables and evaluate each one independently, so that it may be possible to estimate their relative effects under actual field conditions.

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Soil Deformations in Normal Compression And Repeated Loading Tests

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This paper presents data comparing the deformation characteristics in repeated load tests of specimens of two soils having similar stress-strain relationships in normal compression tests. The tests were performed on a silty clay from Vicksburg, Mississippi, and a clayey silt from the subgrade of the Idaho test road. Specimens were prepared to a degree of saturation of about 90 percent by the method used by the California Division of Highways. Comparable samples were then tested in normal triaxial compression tests and in triaxial compression tests in which the axial load was repeatedly removed and re-applied 1,000 times. By interpolation the behavior in repeated load tests of samples of the two soils having similar stress-strain characteristics in the normal compression tests was determined and shown to be quite different, both insofar as irrecoverable deformation and resilient deformation are concerned.

● IN a previous paper presented before the Highway Research Board it has been shown that the method of load application to a soil has an important effect on the magnitude of soil deformation (1). For example, a specimen subject to repeated loading has been found to deform many times more than an identical specimen subjected to a sustained load of equal magnitude. This difference in soil behavior under different types of loading raises the question whether tests performed under conditions of slowly increasing stress can satisfactorily indicate the performance of a soil under the repetitive type of loading to which it is subjected under a pavement.

A pavement may be considered to have failed when the deformation of the soil below the wearing surface is of such a magnitude as to cause an uneven riding surface or to cause cracking of the surfacing material. The object of pavement design procedures is to determine the thickness of pavement and base which must be placed over a subgrade in order that the deformation of the subgrade will not be excessive. Thus, for a satisfactory method of pavement design, it is necessary to devise some means of evaluating the resistance to deformation of the subgrade when it is subjected to a series of repeated loads of different magnitudes, durations and frequencies.

Recent research (2) has shown that it is not sufficient to evaluate only the resistance to permanent or plastic deformation of the subgrade, but also the elastic or resilient properties of the subgrade soil. A series of investigations conducted by the California Division of Highways have shown that there is a close correlation between observations of cracking and fatigue-type failures in bituminous pavements and the measured deflections of these pavements due to passing wheel loads. It appears, therefore, that large elastic deformations in a soil are a primary cause of pavement failure.

While in many cases soils having low resistance to plastic deformation may also exhibit high resilient deformations, it seems likely that some soils may exhibit extremely small plastic deformations and yet have high elastic deformations. Such soils would probably cause fatigue failure in the surfacing much more readily than would a soil exhibiting a larger plastic deformation but a much smaller elastic movement. It is apparently necessary, therefore, to evaluate separately the resistance of a soil to plastic flow and the elastic or resilient properties of soil in order to design a satisfactory pavement.

Most methods of pavement design now in use are based on an index of soil strength or resistance to deformation determined by some type of test in which the total load is slowly applied over a period of several minutes. These indices of strength have been correlated empirically with the performance of soil underlying actual pavements and thus provide a fairly reliable index for design. It does not, however, necessarily follow that a strength index determined under conditions of slow stress increase will satisfactorily indicate either the plastic deformation of the soil or the resilient deformation of the soil under conditions of repeated loading. If soils having the same strength index behave in similar fashions under repeated loading, then any difference between the effects of repeated loads and gradually increased loads will be taken into account in the empirical correlation with pavement performance. If, however, soils having the same strength index are affected to different extents by repeated loading, then the correlation of strength index with pavement performance can be only approximate.

The investigation described in this paper was conducted to throw some light on the extent to which soil strength tests carried out in the normal manner using a gradually increasing stress can be used as an index of the plastic deformation and resilient deformation of the soil under conditions of repeated loading. Series of tests were performed on saturated specimens of two different types of soil, prepared at various water contents, to determine their stress versus strain characteristics in triaxial compression tests under normal loading conditions and also when subjected to a series of 1,000 applications of a constant load. By interpolation from these results, the behavior under repeated loading conditions of specimens of the two soils having similar stress versus strain characteristics in the normal type of tests were compared.

SOILS USED IN THE INVESTIGATION

The soils used in the investigation were:

1. A silty clay from Vicksburg, Mississippi. This soil had a liquid limit of 37 and a plastic limit of 23.
2. A silty soil from the subgrade of the WASHO Test Road in Malad, Idaho. This soil had a liquid limit of 36 and a plastic limit of 26.

TESTING PROCEDURES

Preparation of Specimens

All of the tests were performed on compacted samples of soil prepared in a manner similar to that used in the California Division of Highways pavement design procedure. The soil was mixed to the desired water content, allowed to condition for a period of 24 hours and then compacted in a 1.4-in. diameter mold using the Harvard Miniature Kneading Compactor (3). Samples having a height of 4.5 in. were compacted in 10 layers using 20 tamps per layer and a tamping pressure of such a magnitude as to result in a sample having a degree of saturation between 85 and 90 percent. The samples were then subjected to a static pressure until moisture was exuded, at which stage the pressure was released. This procedure resulted in samples having a degree of saturation of about 90 percent.

For each soil two samples were prepared at each water content; one of these samples was subjected to a normal triaxial compression test and the other to a repeated loading test. After compaction each specimen was placed between a lucite cap and base and surrounded by two thin rubber membranes which were sealed against the cap and base by neoprene O-rings. The specimens were then assembled in the triaxial compression cells for testing.

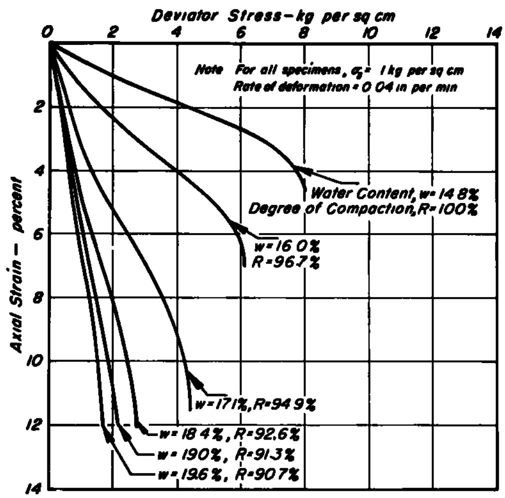

Figure 1. Stress versus strain characteristics of specimens of Vicksburg silty clay in normal triaxial compression tests.

Normal Triaxial Compression Tests

For the normal triaxial compression tests, the specimens were surrounded by water in the triaxial compression cell, a confining pressure of 1 kg per sq cm was applied by means of air pressure in the upper part of the cell, and the specimen was loaded axially by placing the cell in a standard type of compression testing machine. The rate of load increase was controlled to maintain a constant rate of deformation of the specimen of 0.04 in. per minute. The deformation of the specimen during loading was measured by a dial indicator attached to the piston applying load to the specimen.

Repeated Loading Tests

In the repeated loading tests a specimen was assembled in the triaxial compression cell as for the normal type of test, a confining pressure of 1 kg per sq cm was applied, and the specimen was then subjected to 1,000 applications of a 12.5 kg load corresponding to a deviator stress of about 1.25 kg per sq cm. Each load application was for 0.2 second duration with an interval of 3 seconds between applications.

RESULTS OF NORMAL COMPRESSION AND REPEATED LOAD TESTS ON VICKSBURG SILTY CLAY

The results of a series of normal triaxial compression tests performed on specimens of Vicksburg silty clay at various water contents are shown in Figure 1, and the deformations of essentially identical specimens in the repeated loading tests are shown in

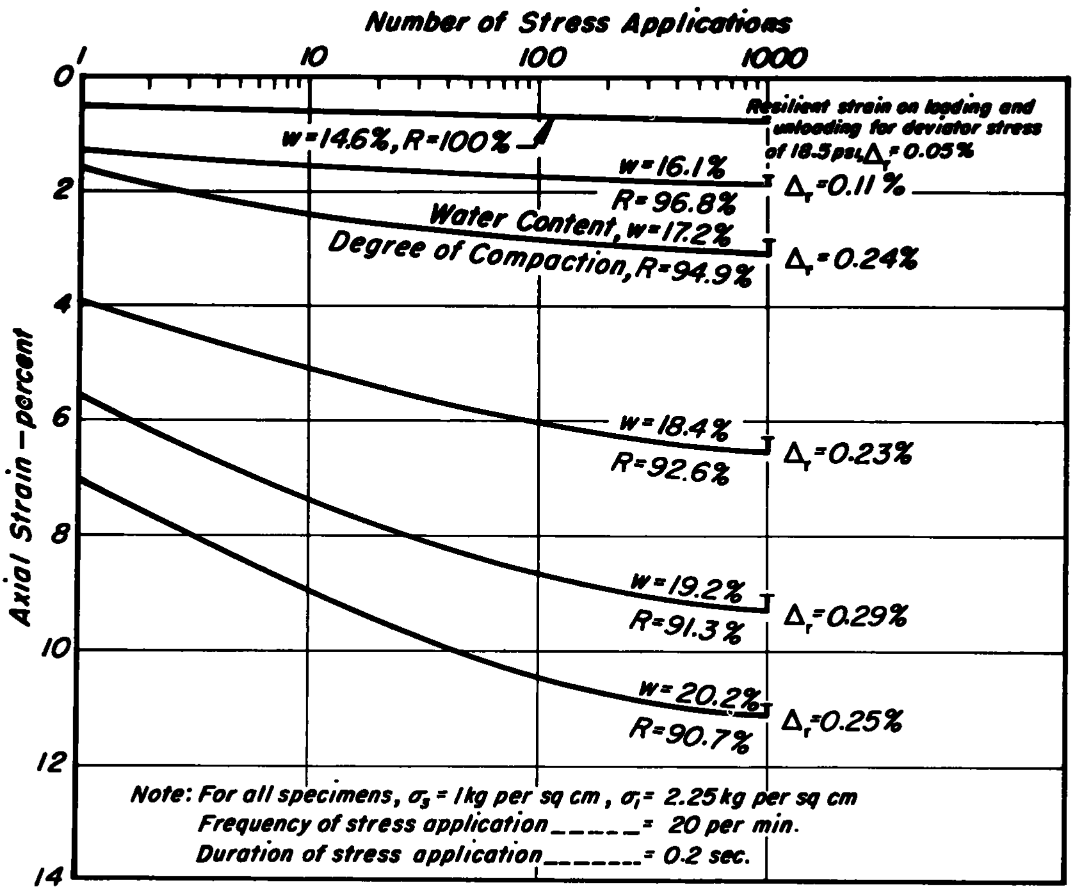

Figure 2. Deformation characteristics of specimens of Vicksburg silty clay in repeated loading tests.

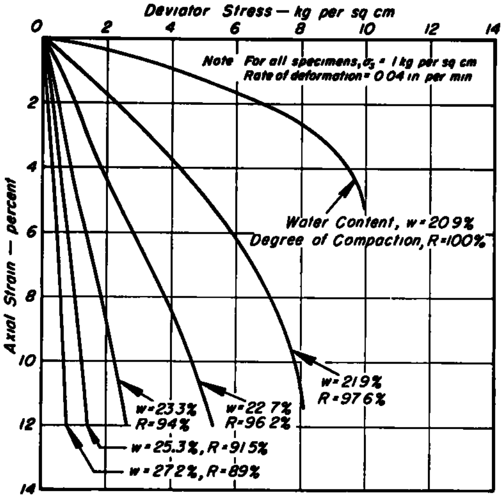

Figure 3. Stress versus strain characteristics of specimens of Idaho clayey silt in normal triaxial compression tests.

Figure 2. The specimens had water contents ranging from 14.6 percent to 20.2 percent and a constant degree of saturation of about 90 percent. Since the degree of saturation is the same for all specimens, the water contents are directly related to the dry densities which varied from 116.1 pcf at a water content of 14.8 percent to 105.1 pcf at a water content of 20.2 percent. The degrees of compaction of the specimens, based on the modified AASHO compaction test, are shown in the figures. As would be expected, the specimens having the higher water contents and lower densities exhibited the larger deformations under equal stresses in the normal triaxial compression tests and at equal numbers of stress applications in the repeated load tests.

It is interesting to note the large change in deformation occurring in the repeated load tests for a small change in degree of compaction. For example, a specimen

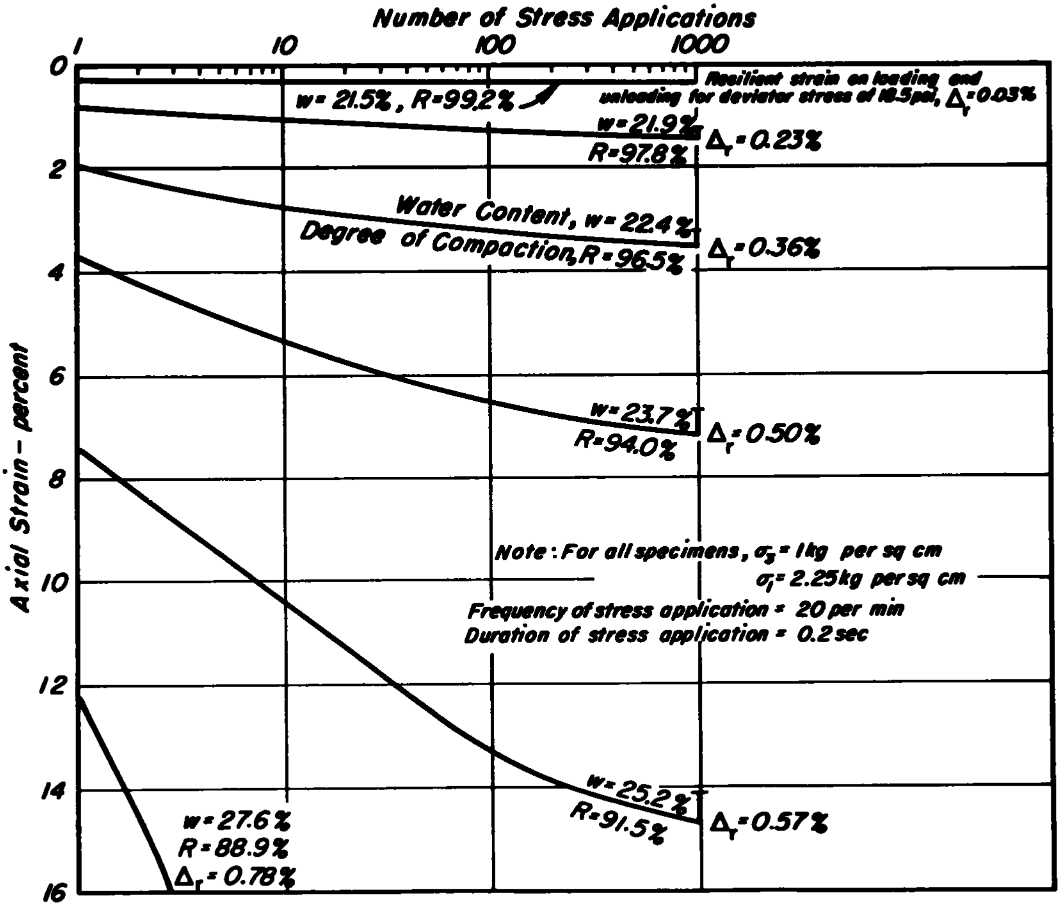

Figure 4. Deformation characteristics of specimens of Idaho clayey silt in repeated loading tests.

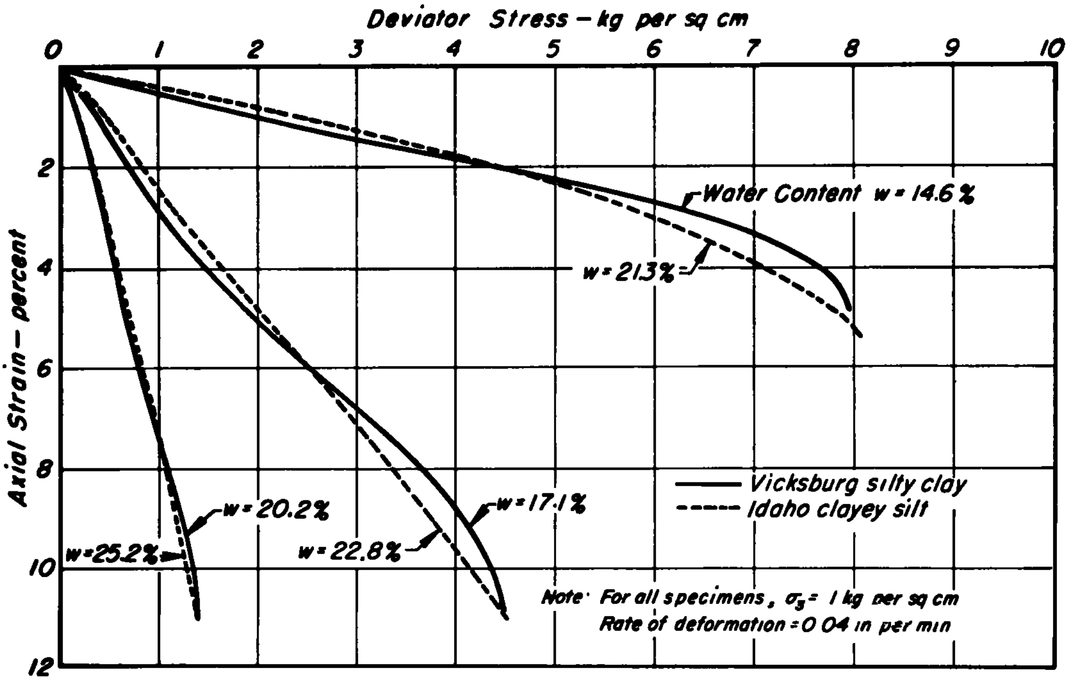

Figure 5. Comparable stress versus strain relationships for specimens of silty clay and clayey silt in normal triaxial compression tests.

having a degree of compaction of 95 percent deformed only 3 percent after 1,000 stress applications, but a specimen having a degree of compaction of about 91 percent deformed more than three times this amount.

In addition to showing the total deformation of the specimens due to a series of applications of the same load, Figure 2 also shows the resilient deformation occurring on application and removal of the load. It is interesting to note that this resilient strain changes only slightly from 0. 24 percent to 0. 29 percent for a change in degree of compaction from 95 percent to 91 percent. Thus although the density of the specimen has a large effect on the total deformation, within this range of densities it has only a small effect on the resilient deformation.

RESULTS OF NORMAL COMPRESSION AND REPEATED LOAD TESTS ON IDAHO SILTY CLAY

The results of the tests performed on specimens of Idaho silty clay at various water contents are shown in Figures 3 and 4. For this soil the specimens had water contents ranging from 20. 9 to 27. 6 percent and a constant degree of saturation of 92 percent; this range of water contents corresponds to degree of compaction values ranging from 99 to 89 percent, based on the modified AASHO compaction test.

These data also show clearly the influence of a small change in the degree of compaction on the deformation characteristics. For this soil a specimen having a degree of compaction of 96. 5 percent deformed only 3. 5 percent after 1,000 stress applications, but a specimen at a degree of compaction of 91. 5 percent deformed about 15 percent. However, the change in resilient deformation was not so marked, the same change in degree of compaction causing an increase in resilient strain from 0. 36 to 0. 57 percent. Thus, for this soil also, density changes would appear to have a much greater effect on total deformation than on the resilient properties of the soil.

COMPARISON OF DEFORMATION CHARACTERISTICS OF VICKSBURG SILTY CLAY AND IDAHO CLAYEY SILT

The data presented in Figures 1 to 4 permit a comparison of the deformation characteristics of the silty clay and the clayey silt. For example, it is possible by interpolation in the data in Figures 3 and 4 to compare the behavior in the repeated load tests of specimens of the two soils having similar stress versus strain characteristics in the normal compression tests.

Consider the stress versus strain curve for a specimen of the Vicksburg silty clay at a water content of 14.8 percent shown in Figure 5. By interpolation in the stress versus strain curves for the Idaho clayey silt in Figure 3, it may be shown that a specimen of clayey silt at a water content of 21.3 percent has a similar stress versus strain relationship in the normal type of triaxial compression test. For comparison purposes these two curves are shown in Figure 5. Having determined that specimens of the two soils at these water contents behave in a similar manner in the normal type of test, it is now possible to compare their behavior in the repeated load tests. This comparison is shown in Figure 6. The curve for the silty clay was obtained by test and is reproduced from Figure 2 while that for the clayey silt was interpolated from the data in Figure 3. It will be seen that the specimens behave differently in the repeated load tests, although in both cases the deformation is extremely small.

A similar comparison can be made for specimens exhibiting greater deformations in the normal triaxial compression tests. For example, in these tests a specimen of silty clay at a water content of 17.1 percent has a stress versus strain relationship similar to that of a specimen of clayey silt at a water content of 22.8 percent (Figure 5). The deformations of corresponding specimens in the repeated load tests are shown in Figure 6. In this case, the specimens deform almost equal amounts under the first load appli-

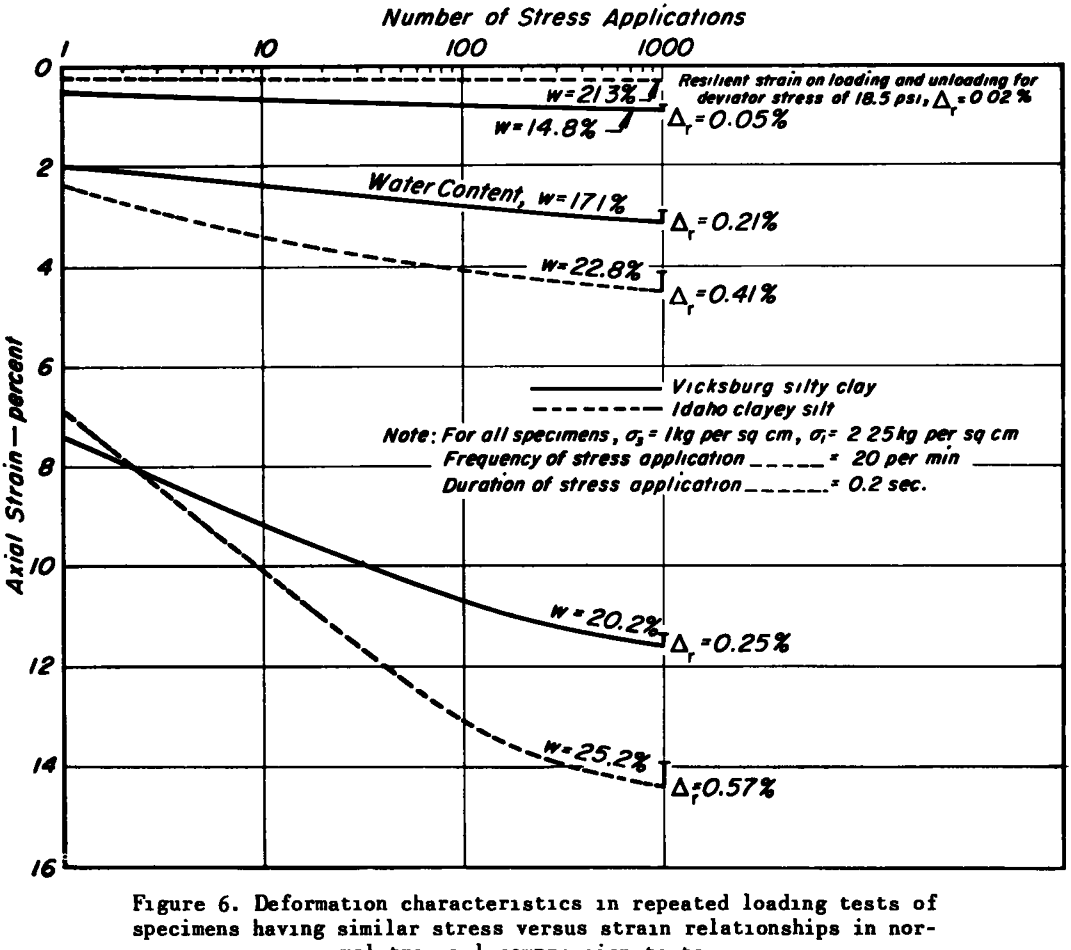

Figure 6. Deformation characteristics in repeated loading tests of specimens having similar stress versus strain relationships in normal triaxial compression tests.

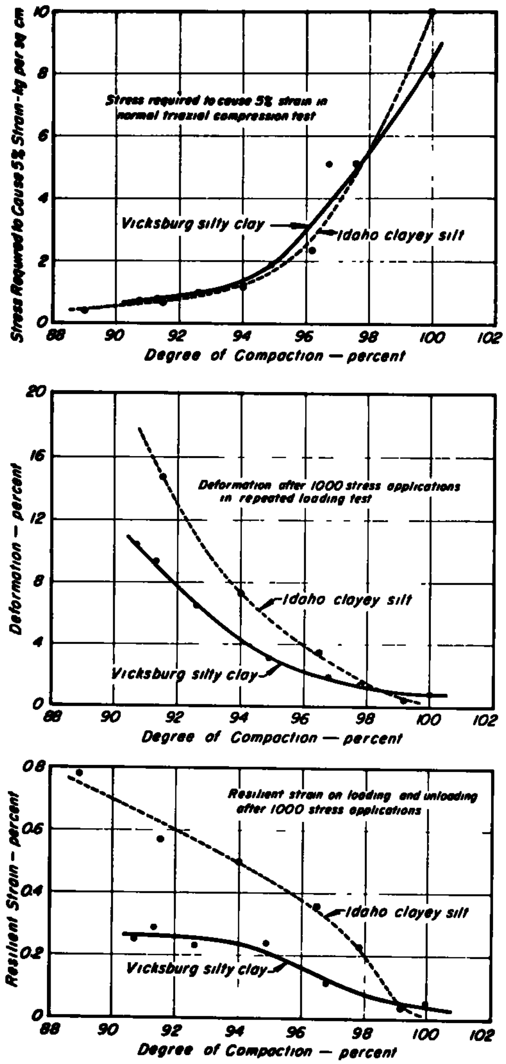

Figure 7. Deformation characteristics of silty clay and clayey silt at equal degrees of compaction.

cation but after 1,000 applications the clayey silt has deformed about 50 percent more than the silty clay. Furthermore, the resilient deformation of the two specimens is considerably different, the Idaho soil exhibiting a resilient strain of 0.41 percent as compared with only 0.21 percent for the silty clay.

At still higher water contents the difference in behavior of the two soils in repeated load tests is even more marked. For example, a specimen of silty clay at a water content of 20.2 percent and a specimen of clayey silt at a water content of 25.2 percent also have similar stress versus strain characteristics in normal triaxial compression tests. However, it will be seen from Figure 6 that there is a great difference in the behavior of these two specimens in the repeated load tests. Under the first few applications the clayey silt deforms less than the silty clay, but subsequently the deformation of the clayey silt is appreciably greater than that of the silty clay. The resilient deformations also differ greatly, a specimen of clayey silt having a resilient strain of about 0.57 percent as compared with only 0.25 percent for the silty clay.

One of the difficulties in making a comparison of the deformation characteristics of the two soils used in this investigation is that of determining the degree of similarity which should be achieved between the stress versus strain curves of the corresponding specimens. The general shape of the stress versus strain curves in normal triaxial compression tests for specimens of the two soils is somewhat different, with the result that curves which are closely alike at low strains may be considerably different at high strains and vice versa. Thus, for example, the stress versus strain curve for a specimen of silty clay having a water content of 14.8 percent is similar at low strains to that of a specimen of clayey silt having a water content of 21.5 percent and at higher strains to the curve for a specimen of clayey silt at a water content of 21.2 percent. In comparing the effects of repeated loading on specimens, it is therefore necessary to define the range of strain in the normal compression tests at which comparable specimens of the two soils should have similar stress versus strain curves.

In most pavement design procedures the index of soil stability is determined as the stress required to cause a certain amount of strain. However, the strain at which the strength index is determined varies considerably. In some procedures, specimens requiring the same stress to cause 5 percent strain are considered to have equal strength indices, while in other procedures specimens might be considered to have equal strength indices if they require the same stress to cause 3 or 8 percent strain. The stress versus strain curves used for comparison in Figure 5 have been selected to coincide at about half the maximum strength. Corresponding specimens in repeated load tests have been shown to have different deformation characteristics. This difference in deforma-

Figure 8. Pavement surface fatigue failure caused by resilient deformations of the subgrade.

tion characteristics in repeated load tests may be shown to exist regardless of the axial strain selected as a basis for establishing the similarity of stress versus strain relationships obtained in normal compression tests.

COMPARISON OF DEFORMATION CHARACTERISTICS OF SILTY CLAY AND CLAYEY SILT AT EQUAL DEGREES OF COMPACTION

It is revealing to compare the influence of degree of compaction on the deformation characteristics of these soils. Figure 7 shows the total deformation after 1,000 stress applications, the resilient strain in the repeated load tests and the stress required to cause 5 percent strain in the normal type of triaxial compression tests for specimens at various degrees of compaction.

It will be seen that for both soils the higher the degree of compaction the smaller is the resilient deformation during repeated loading. However, for the Vicksburg silty clay the resilient strain changes only slightly for degrees of compaction ranging from 90 to 95 percent, while for a similar range of degrees of compaction the Idaho clayey silt shows an appreciable change in resilient deformation. Furthermore, for the range of degrees of compaction of practical interest the Idaho soil exhibits much higher resilient deformations than the silty clay.

At equal degrees of compaction the two soils require approximately equal stresses to cause 5 percent strain in the normal compression tests. In the repeated load tests, however, a specimen of the Idaho soil deforms about 50 percent more than a specimen of silty clay having an equal degree of compaction. This fact again indicates that deformation characteristics determined under normal loading conditions will not necessarily indicate the behavior of soil under repeated loading conditions.

CONCLUSION

The data presented in this paper are not intended to show that established methods of testing soils for the design of pavements are necessarily unreliable. Long experience has indicated that these established methods provide reasonably satisfactory data for design purposes, and this fact cannot be disregarded on the basis of a single investigation of limited scope. The differences in behavior of the soils used in this investigation may, however, serve to explain some of the pavement failures which occur from time to time even though established design methods have been used. The investigation shows clearly that deformation characteristics determined under normal loading conditions are not necessarily indicative of soil deformation under repeated loading conditions, but further investigations are required before the applicability of this result to other types of soil can be ascertained.

It has recently been shown that the resilient or elastic deformation of subgrade soils may be an important factor causing failure or cracking of a flexible pavement. Figure 8 shows a pavement with extensive cracking even though there is little evidence of any plastic deformation of the subgrade (4). The cracking may well be attributed, therefore, to high resilient deformations leading to fatigue failure in the surfacing material. A secondary purpose of this investigation, then, was to demonstrate that the resilient characteristics of a soil are not necessarily related to test data obtained from normal compression tests. For the two soils used in this investigation a strength index determined at high strains does not provide an index of resilient deformation; in fact, the initial tangent moduli of the stress versus strain curves do not appear to serve this purpose. Thus if resilient deformations are to be included as a factor in pavement design, a new testing procedure would appear to be required to determine their relative magnitudes. Repeated loading of specimens in triaxial compression tests provides a convenient means of measuring this type of movement, and at the same time determining the relative resistance to permanent deformation of different soils.

Finally the investigation throws some light on the characteristics of the subgrade soil from the Idaho test road. This soil exhibits an appreciably higher resilient deformation than the silty clay with which it is compared, even though the Atterberg limits of the two soils do not differ appreciably; hence, in view of the correlation between pavement failures and resilient deformation of the pavement under load, some degree of caution would perhaps be warranted in extending the conclusions drawn from this test road to pavements constructed on less resilient types of soil.

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