SUPPLEMENT TO STRESS DISTRIBUTION IN A HOMOGENEOUS SOIL[1]

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The paper[1] presented on this subject in 1949 brought out the fact that the Corps of Engineers is making a long-time study of the distribution of stress and strain under airplane wheel loads. The study is directed toward a better understanding of the behavior of pavements, bases, and subgrades under loads to aid in the application of the CBR method of flexible pavement design and the possible development of a truly theoretical design. It is the purpose of this paper to report the progress that has been accomplished during the past year.

A truly theoretical method of flexible pavement design would permit: (1) computation of the true stress and strain conditions resulting from loading; (2) determination of the strength that will develop in the proposed paving materials under actual conditions by testing them in a triaxial or other device; and (3) comparison of the stress with stress-resisting ability and expression of the comparison as a factor of safety. Phase 1 is being studied by the measurement of stresses and strains in test sections, and phase 2 is being studied by conducting various types of shear tests on undisturbed samples taken from the test sections.

In the paper last year a comparison was presented between a field stress-strain curve measured by pressure cells and deflection gages in the test section and stress-strain curves obtained by unconsolidated and undrained laboratory triaxial tests on undisturbed samples. These laboratory tests are usually called "quick" tests. The field stress-strain curve and the laboratory curve for a lateral pressure of 7 psi. are shown on Figure 1. It will be noted that in the field curve the stress increases rapidly with strain and reaches high values at low percentages of strain as compared to the laboratory curve from the "quick" test. A stress-strain curve from a "cell"-type triaxial test also is shown on Figure 1. In this type of test the volume of fluid in the test chamber is maintained constant and as vertical stress is applied, the lateral pressure builds up finding its own equilibrium. The initial portion of the test consisted of preconsolidation cycles intended to restore the conditions that existed in the test section. It can be seen that the stress-strain curve for this type of test is no closer to the field curve than for the "quick" test. The fourth curve shown on Figure 1 is for a triaxial test in which the lateral pressure is not held constant but is made to vary with the vertical pressure in approximately the same relation as in the test section. This relationship was established by developing a curve from values obtained by adding 8 psi. to the test values for the major and minor principal stresses. The value of 8 psi. was selected in an attempt to duplicate the residual stress in the test section. It is evident that the resulting stress-strain curve approaches the field curve more closely than any of the other laboratory curves. This particular curve is one of the better of many trials. Procedures for making the laboratory curve duplicate the field curve in every instance have not been worked out completely, and work is being continued. The significant point is that merely conducting a triaxial test does not insure that the strength measured in the test will be the same as that developed in the field. The measured strength will be the true strength only if the test is conducted in a manner that simulates prototype conditions. The "quick" test and the "cell" test, two tests which have been proposed for use in flexible pavement design, do not do this and the use of such tests in determining shearing strength is as empirical as the use of a CBR or any other index type of test.

The paper last year mentioned the fact that residual stresses may be a very important factor in flexible pavements and pointed out that the pressure cells used in the first test section were not adapted to measuring

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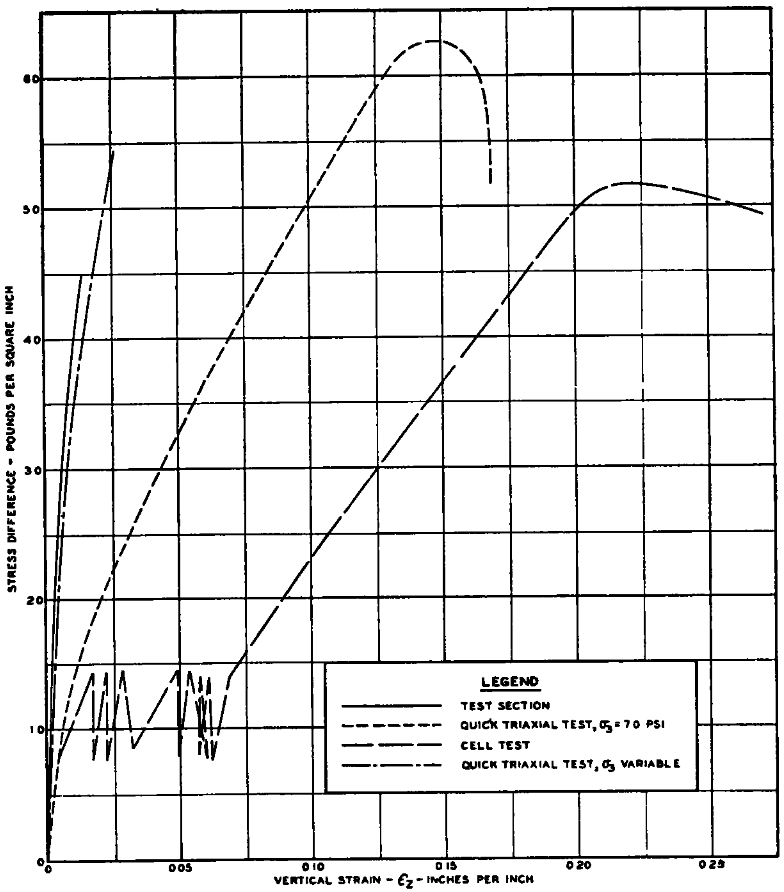

Figure 1. Stress-Strain Curves—Test Section Data Compared with Laboratory Triaxial Data

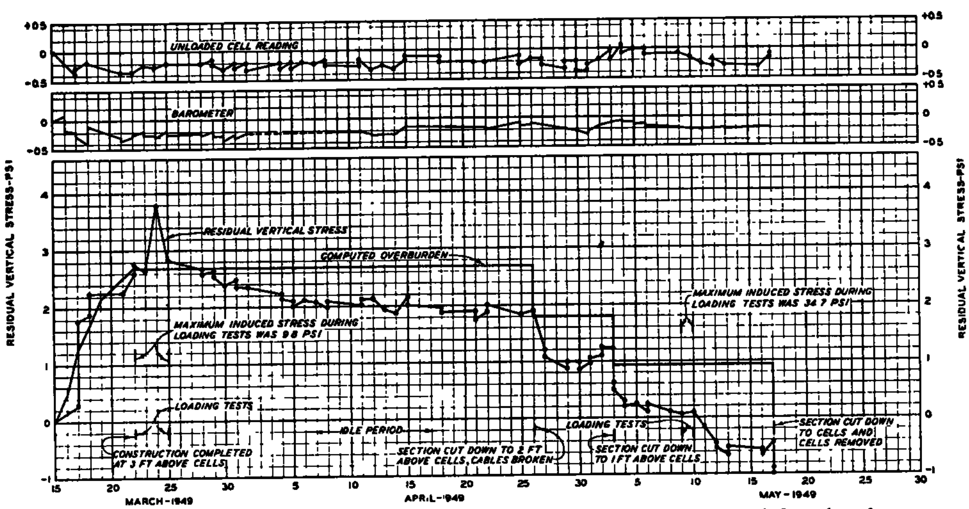

Figure 2. Comparison of Residual Vertical Stress with Weight of Overburden

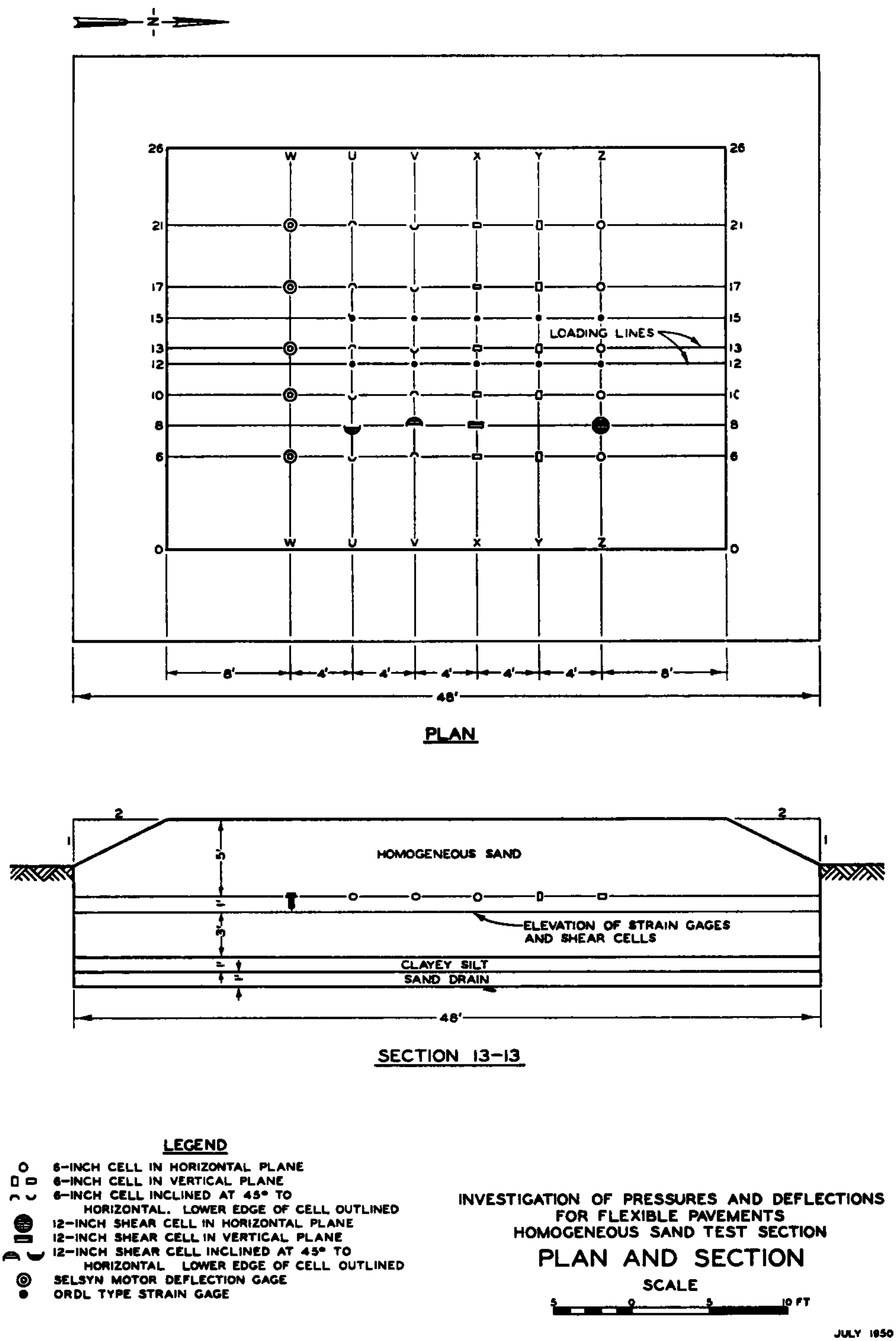
N
W U V X Y Z
26 21 17 15 13 12 10 8 6 0
LOADING LINES
8' 4' 4' 4' 4' 4' 8'
48'
PLAN
HOMOGENEOUS SAND
ELEVATION OF STRAIN GAGES AND SHEAR CELLS
CLAYEY SILT
SAND DRAIN
48'
SECTION 13-13
LEGEND
6-INCH CELL IN HORIZONTAL PLANE
6-INCH CELL IN VERTICAL PLANE
6-INCH CELL INCLINED AT 45° TO HORIZONTAL. LOWER EDGE OF CELL OUTLINED
12-INCH SHEAR CELL IN HORIZONTAL PLANE
12-INCH SHEAR CELL IN VERTICAL PLANE
12-INCH SHEAR CELL INCLINED AT 45° TO HORIZONTAL. LOWER EDGE OF CELL OUTLINED
SELSYN MOTOR DEFLECTION GAGE
ORDL TYPE STRAIN GAGE
INVESTIGATION OF PRESSURES AND DEFLECTIONS FOR FLEXIBLE PAVEMENTS
HOMOGENEOUS SAND TEST SECTION
PLAN AND SECTION
SCALE
5 0 5 10 FT
JULY 1950

Figure 3.

residual stresses because of zero drift. A new four-gage cell is now available which is better adapted to the measurement of residual stresses. This cell was developed by the Waterways Experiment Station as part of the Corps of Engineers Civil Works studies for earth dams. During the past year, six of these cells were installed in a pilot test section to determine their suitability. Figure 2 is a plot of the residual vertical stress against time for two of the cells. One of the cells was installed in lean clay and 3 ft. of fill placed above it. The other cell was placed in a box at the same elevation in an unloaded position as a control cell. It can be seen that

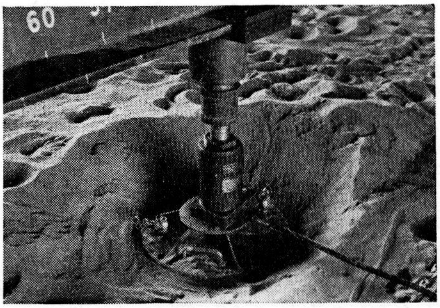

Figure 4. Jack in Loading Position on 250–Sq. In. Plate

the vertical pressure built up in direct proportion to the weight of the fill, but that release of pressure did not occur in exactly the same manner although the cell practically returned to zero when finally unloaded. The control cell showed very small variations throughout the period. The variations followed the barometer reading very closely. The readings of the cells were corrected for changes in barometric pressure, and it was found that the zero drift for the six cells ranged from −0.11 to +0.41 psi. It was concluded that the cells were satisfactory and several were constructed. These cells have been used in the sand test section, described later, and are giving very satisfactory service.

At the time of the Highway Research Board meeting in 1949 it was anticipated that the next test section would be a two-layered system consisting of a base course and a subgrade. The board of consultants, however, decided that the available information on residual stresses was not adequate and, accordingly, a homogeneous-sand test section has been constructed. It was not mentioned in the first paper, but the following consultants have been employed on this study since 1946: Professors D. M. Burmister, Arthur Casagrande, N. M. Newmark, P. C. Rutledge, and D. M. Taylor, and Messrs. T. A. Middlebrooks and R. R. Philippe.

The sand test section is quite similar to the clayey-silt test section. A plan and section are shown on Figure 3. An ordinary mason's sand, with 100 percent passing the No. 10 sieve and less than 1.0 percent passing the No. 200 sieve, is being used. It was placed in an air-dry condition and compacted to an average density of 108 lb. per cu. ft. with heavy tractors. The maximum density that can be achieved in the laboratory by vibration is 111 lb. per cu. ft., therefore it is apparent that the sand in the test section is in a relatively dense state. Figure 4 is a view of the loaded jack in place on a 250-sq. in. plate. This picture shows the texture of the sand. Pressure cells and deflection gages were installed during construction at an elevation 5 ft. below the finished grade. Loading will be applied through circular flexible-face plates. After the loading is completed at the 5-ft. elevation, the test section will be cut down in increments and loads applied, successively, at heights of 4, 3, 2, 1, and 0.5 ft. above the cells. In addition to the pressure cells and deflection gages, this test section includes four shear cells developed by Professor D. W. Taylor and Mr. R. R. Philippe, strain gages developed by Mr. R. R. Philippe, and a strain gage developed by the Waterways Experiment Station. These instruments are performing satisfactorily. Both strain gages use a differential transformer as the measuring device. Very consistent induced and residual stresses are being obtained. It is anticipated that testing and analysis will be completed in time for a more detailed paper for the next meeting of the Highway Research Board.

DISCUSSION

The following closing remarks refer to the discussions by Professor Krynine and Mr. Scrivner which were made of the original paper and published in Research Report 12-F. These closing remarks were inadvertently omitted from the original publication.

CHARLES R. FOSTER AND S. M. FERGUS, *Closure*—Professor Krynine suggests that some densification of the test section occurred during the load testing which changed the characteristics of the homogeneous fill. The authors believe that the increase in density was very small and did not appreciably affect the test results. In support of this view, the following points are cited. First, no point on the surface of the test section at any level received more than 18 applications of the maximum testing load of 60 psi. Second, the tests were performed in triplicate, and no significant difference in the three values was noted. Third, although some slight drying out of the test surface was observed by the technicians who performed the tests, the moisture and density values obtained after the loads had been applied were in the same range as those obtained before the testing began. Fourth, such densification as did occur was almost certainly in the top one-foot layer which was cut off as the testing progressed. Fifth, the higher CBR values (16 per cent) were obtained in March 1948 after all load testing was completed. Tests performed in the loading area in February 1948 had an average value of 11 percent.

Professor Krynine's reasons for the difference between the test values and those computed from elastic theory are fully appreciated. The authors are only too aware of the possibilities for serious error involved in the unconsidered application of the theory of elasticity to soil problems. The fact that in this study a satisfactory similitude was found between the actual and the theoretical patterns of stress distribution should not lead anyone to a false conclusion. The absence of the linear stress-strain relationship renders invalid both the principle of superposition and the use of constant values for Poisson's ratio and the modulus of elasticity. The theory of elasticity is only one avenue of approach to the basic stress-strain problem in soils and must be so regarded. The nonlinear relationships found in the tests show that a broader theory is required and the present study is one contribution to that end.

With respect to the occurrence of the maximum deflection under one of the dual loads rather than at the centroid, this same pattern has been encountered previously. For a very shallow depth, the maximum deflection occurs under one of the duals; for a very deep depth it occurs under the centroid. Between these two depths the point of maximum deflection will vary. It is believed that no eccentric loading occurred because both jacks were connected to one manifold which in turn was connected to the pump.

The authors are willing to accept the wording "satisfactory similitude" for conclusion *a*.

Mr. Scrivner's discussion and study based on the sums of stresses measured by pairs of mutually perpendicular cells which show that there was a fair degree of cell accuracy are quite welcome.