FOUNDATIONS FOR FLEXIBLE PAVEMENTS

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SYNOPSIS

Flexible pavements have in recent years been subjected to wheel loads of increasing magnitude Experiments indicate that failures are primarily due to progressive plastic deformation of the foundation which, with sufficient repetitions causes excessive deflection and bending of the base and pavement The report includes (1) a discussion of the various types of pavement deflection, (2) a description of electric gauge equipment for measuring the deflection of pavements under both static and moving wheel loads, (3) a description of the California bearing ratio test and a design chart for use with this test, and (4) a summary of the deflection and traffic tests made on a special runway test section which was constructed at a western airfield by the U. S. Engineers for checking the design criteria for heavy wheel loads

It should, by this time, be unnecessary to state that the performance of flexible pavements depends, in the main, upon the adequacy of the foundation. The heavy airplane wheel loads now contemplated will stress the foundations of runways to a much greater depth and will require much thicker base courses than are customary on highways, particularly where weak soils are encountered. Regardless of the stresses transmitted to the various horizons of the base courses and foundations, flexible pavements continue to support the load as long as the pavement deflection from each application of the load is not excessive.

In the past, extensive studies have been made of the distribution of pressure of static loads through pavements and base courses in an attempt to arrive at the thickness required to distribute the wheel load to a value not in excess of the supporting strength of the soil Although these studies have thrown much light upon the subject, measurements of subgrade pressures and the deflection of pavements under static and moving wheel loads indicate that the stress transmitted and the shearing strength of the subgrade are only two of a number of important factors which influence the service behavior of pavements.

Permanent and progressive deformation of a subgrade occurs under each application of a moving wheel load even though the pressure transmitted is much below the ultimate shearing strength of the material. Also, the effects of moving wheel loads on the subgrade are radically different than those of static loads In pavement design, we are dealing with a heterogeneous system consisting of soil, base material and pavement. Under rapidly moving wheel loads the inertia of the pavement and base may be enough to greatly reduce the amount of subgrade deformation. Another factor which further complicates the problem is that some soils soften and others become "quick" when subjected to repeated applications of rapidly moving wheel loads. It appears, therefore, that a study of pavement deflections under static and dynamic wheel loads offers the best method of determining the adequacy of pavements and their foundations. One advantage of this procedure is that the measurements can be made in the field under service conditions The results may also be checked and correlated with the performance of the pavement under traffic after the moisture content of the base and subgrade has become equalized for the paved condition.

Extensive investigations have been made on California highway pavements and on both highway and runway test

sections during the last four years. Data already obtained show that the amount of pavement deflection resulting from a static load may range from 50 to 400 per cent of that caused by a moving wheel load of the same intensity This study clearly indicates that the permissible deflection under a moving wheel load depends to a considerable extent upon (1) the flexibility of the pavement, (2) the type of the deformation (elastic or permanent), (3) the radius of curvature of the pavement deformation, and, (4) the number of load repetitions which must be carried. Asphaltic concrete pavements on highways have progressively roughened, then cracked and ultimately failed where each application of a moving wheel load caused a deflection of only 0.02 or 0 03 in. A few million load repetitions were required to cause failure where the deflection was of that magnitude while serious failures developed rapidly where the deflection exceeded 0.05 in.

CAUSE AND EFFECT OF PAVEMENT DEFLECTION

Each time a wheel load passes over a pavement, deflection occurs. If the amount of deflection is excessive, repeated applications will cause roughening and cracking, ultimately resulting in complete failure as the strength of the pavement and foundation is reduced. The deflection of the pavement may be caused by elastic deformation or by a combination of elastic and plastic deformation, together with consolidation of the base and subsoils. In all three cases, a portion of the load is transmitted to the fluids (gases and liquid) contained in the pavement and foundation materials, thus setting up an internal fluid pressure or pore pressure which greatly influences the magnitude and character of the deformation.

Elastic Deformation

Elastic deflection occurs almost instantly as the load deforms the foundation materials and compresses the fluids which fill the voids of the base and subgrade at all times. As the air and other gases contained in the soil are very compressible, a large portion of the elastic deformation results from compression of these gases. In the case of a truly elastic deflection, the pavement returns to its original position immediately after the load is released and permanent roughening does not occur under repeated applications of the load Furthermore, cracking and failure will not result, if the pavement is of a type that remains sufficiently flexible to withstand the amount of this deflection for practically an unlimited number of repetitions in the case of a heavy duty highway or the required number of repetitions of the wheel load in the case of airport runways. Although some elastic deflection always occurs regardless of the thickness of the pavement and the foundation conditions, the magnitude is usually sufficiently low that flexible pavements can be designed to withstand the required number of wheel load repetitions on highways and runways.

Consolidation Deformation

Pavements also deflect when the pore pressure is enough to expel part of the fluids (air and water) and consolidate the material. Although the amount of consolidation resulting from one application of a fast moving wheel load is small in cases where the pore pressure is moderate and where the materials are somewhat impervious, the deformation is permanent, progressing with additional load repetitions until the foundation becomes completely consolidated. Even so, cracking and complete failure will not occur during the intervening period, if the traffic is well distributed so that the pavement settles as a whole and provided the magnitude of the deflection resulting from both the consolidation and the elastic deformation is not excessive for the type and quality of the pavement.

On highways, however, heavy traffic is usually concentrated in one lane and a considerable amount of consolidation deformation will ultimately produce an appreciable amount of differential settlement and cause excess bending and cracking of the pavement. Figures 1 and 2 show cracking, settlement and partial failure of flexible pavements on highways due to differential consolidation of poorly compacted subsoil. On runways, trouble may develop because traffic is concentrated near the center and with their greater width and flat grades, the differential settlement may eventually cause changes in elevation sufficiently large to prevent proper drainage of the surface. This type of failure is shown by Figure 3. The magnitude of consolidation deformation and the ultimate differential settlement resulting can, however, be reduced or entirely eliminated by proper consolidation of the base and subgrade materials to the depth affected by the wheel loads. Even if full compaction is not obtained, the amount of deflection gradually lessens as the foundation is consolidated and stabilized by traffic. Except in extreme cases, a flexible pavement can be adequately repaired by placing a thin resurfacing to restore the drainage grades. Lack of ade-

Figure 1. Cracking of a 7-In. (Center) to 9-In. (Edge) Thickness of Asphalt Concrete on a Heavy Duty Highway. Differential Settlement Occurred and Cracking Developed Due to Consolidation of an A-2 Subsoil. The Subgrade Was Thoroughly Consolidated to a Depth of 12-In. During Construction.

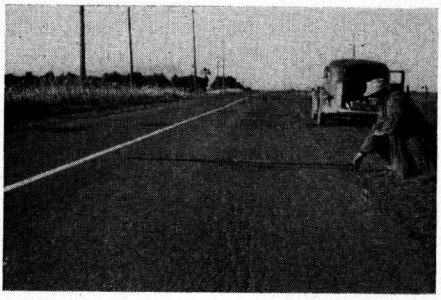


Figure 2. Settlement Failures on the Widening Strip of an Old Highway Over an A-2 Sandy Loam Soil.

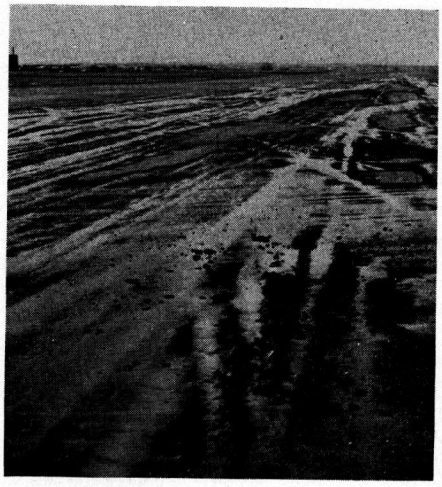


Figure 3. Showing "Bird Baths" and Failures of an 8-In. Thickness of Pavement Plus Base on a Runway in the Rocky Mountain Area. Settlement of the Center of the Runway Disrupted the Surface Drainage and Contributed Towards Early Failure of the Pavement Under 12,000-Lb. Wheel Loads.

quate compaction is much more serious, however, when plastic deformation occurs because of the low shearing resistance resulting from saturation of the material prior to consolidation.

Plastic Deformation

Deflection resulting from plastic deformation of the base and subgrade constitutes the major cause of failure of flexible pavements. Plastic deformation occurs when the pore pressure and other horizontal forces produced by the load are enough to displace the material. As the pore pressure greatly reduces the frictional resistance of the material and also, at the same time, greatly increases the shearing stresses by acting equally in all directions, it is the principal cause of lateral displacement and failure of subgrades under dynamic load repetitions. In this case, the pavement does not return to its original position after the load is removed. Although, under rapid moving wheel loads, the time of loading may not be enough to cause excessive deformations in a few repetitions, the movement is progressive, and rutting, cracking and complete failure result after a sufficiently large number of load repetitions.

Plastic deformation is even more serious as the magnitude of the deflection rapidly increases with the wheel load intensity. Disintegration of the pavement soon occurs if its thickness and strength are not sufficient to reduce to an extremely small amount the plastic deformation resulting from a single application of a moving wheel load. In addition to causing rutting, plastic deformation also causes flexible pavements and bases to weave and crack, thus progressively reducing their strength which in turn increases the magnitude of the deflection. This completes a vicious circle which rapidly destroys the pavement.

Figures 4 and 5 show typical examples of this type of failure on highways and runways. Figure 6 shows the progression of this type of failure under the traffic of a 40,000-lb. wheel load. During the traffic tests the travel of the wheel was staggered over a 5-ft. to 6-ft. width of pavement. The 3-in. asphaltic concrete pavement and

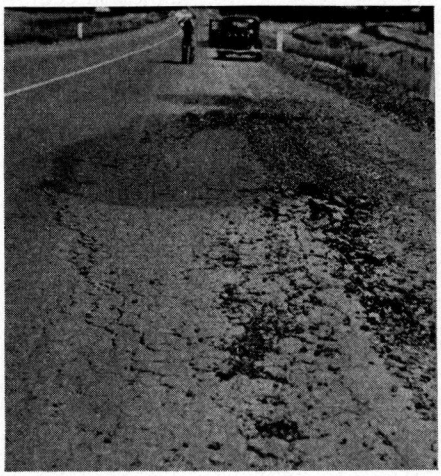
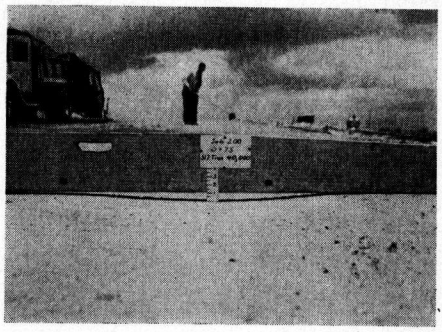


Figure 4. Typical Failure of a Flexible Pavement Caused by Inadequate Thickness of Base. Cracking First Developed Due to Excessive Deflection of Pavement Resulting from Plastic Deformation of Subgrade and With Additional Repetitions Ruts, Heaved Areas and Complete Break-Up Occurred.

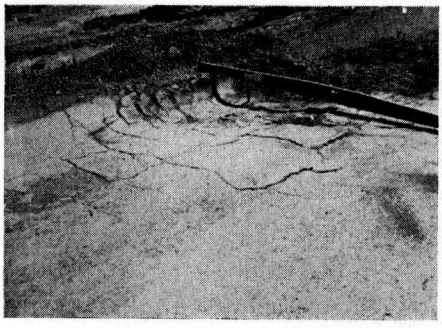
Figure 5. Rut in Runway Pavement Caused by Plastic Deformation of the Subgrade Under a 12,000-Lb. Wheel Load.

the crushed gravel base at this location (Fig. 6) totaled 24 in. thick. The subgrade consisted of A-7 adobe soil containing 30 per cent of moisture. Total deflection (elastic, consolidation and plastic) of about 0.18 and 0.16 in. occurred at the top of the pavement and at the bottom of the base, respectively, under each application of the wheel load until cracking developed after about 1,000 load repetitions. Each application of the wheel load caused some plastic deformation of the subgrade and a small increment of pavement grooving. Severe failures developed at a rapid rate after the blocks between the cracks started to punch down and weave under each wheel load repetition.



One-inch groove at 517 trips which increased to $1\frac{3}{4}$ inch depth at 635 trips

Extent of cracking at 3400 trips



8000 trips. Pavement patched during traffic, but cleared for inspection

Figure 6. Rate of Progression of Pavement Failure Under a 40,000-Lb. Wheel Load. 24 Inch Thickness of Pavement Plus Base. (From U. S. Engineer's Report.)

Oscillograph Records Showing Pavement Deflection

Figure 7 shows oscillograph records of pavement deflection on a highway test section and illustrates the elastic and permanent deformation due to moving wheel loads of various intensities. Referring to the upper chart, it will be noted that the 3,000-lb. load on the front wheel of the semi-trailer, caused the pavement to deflect approximately 0.04 in. but that the pavement returned to its original position before the rear wheels approached the point at which the measurements were being taken. In this case, only elastic deformation occurred. The curves to the right show that the pavement deflected approximately 0.1 in. under the two driven wheels of the tractor unit and the two rear wheels of the trailer on which the load averaged 8,000 lb. per wheel. This load and the deflections were excessive, causing permanent deformation of the subgrade, and the pavement did not return to its original position after the load was released. A 6,000-lb. load also caused some permanent deformation (not shown). Because of the progressive plastic deformation, this thickness of pavement and base failed under considerably less than 10,000 repetitions of a 6,000-lb. wheel load.

The oscillograph record on the bottom of Figure 7 shows the pavement deflection

due to the front and rear wheel loads of a two-axle dump truck. In this case, the deflection curve does not start at zero on the left-hand side of the chart because this oscillograph record shows results for the second trip of the truck. It may be assumed that the ordinate value at the extreme left represents the approximate amount of permanent deformation resulting from the previous trip of the rear grade deformation (not shown). This pavement also failed at about 10,000 repetitions of a 6,000-lb. wheel load.

Figure 8 shows the deflection of the top of a 3-in. asphaltic concrete pavement and the bottom of the crushed gravel base course under a moving wheel load of 40,000 lb. These measurements which also show the longitudinal distribution of the deflection were made with California's

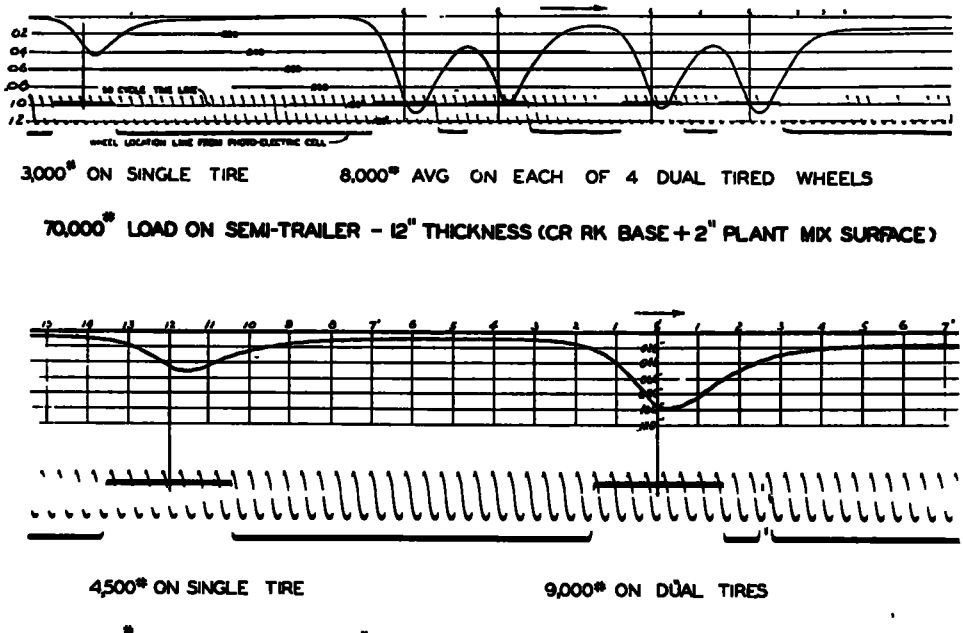


Figure 7. Oscillograph Records of Pavement Deflection Under Wheel Loads Traveling at 14 m.p.h. Horizontal Lines Superimposed on Films Show 0.02 Inch Deflection. Vertical Lines Show Each Foot of Wheel Travel.

wheel. The pavement deflected approximately 0.04 in. under the front wheel load of 4,500 lb. and 0.09 in. under the rear wheel load of 9,000 lb. Referring now to the right-hand side of the chart, it will be noted that again the pavement did not return to its original position after the second trip of the rear wheel. This permanent deformation was due, in the main, to plastic deformation of the subgrade. Each trip of a 6,000-lb wheel load also caused a slight amount of permanent sub-

electric gauge and oscillograph equipment on a runway test section (Fig. 6, 21 and 22) constructed by the U. S. Engineers at a western airfield. These curves show the elastic deflection and the approximate amount of permanent deformation. Because the subgrade consisted of an impervious A-7 adobe soil, little or no consolidation occurred. Practically all of the permanent deflection resulted from plastic deformation of the subgrade and subsoil. Although only a very small amount of

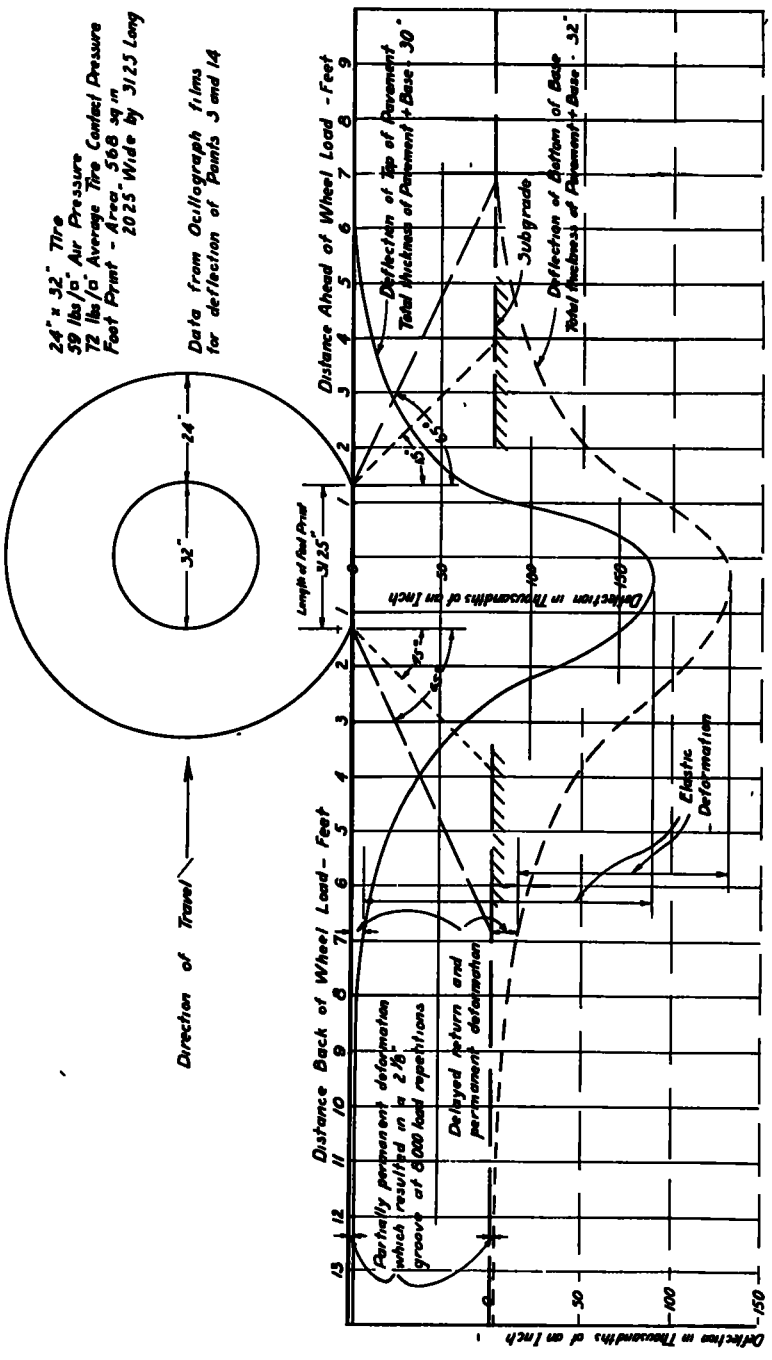


Figure 8. Typical Curves Showing Longitudinal Distribution of Pavement Deflection Under a Moving Load (10 ± m.p.h.) of 40,000 Lb. (From U. S. Engineer's Report)

permanent (plastic) deformation occurred from each load repetition, grooving developed at a progressive rate until the pavement cracked and failed The plastic deformation resulting from each wheel load repetition was greater where the pavement and base was 24 in. in total thickness (Fig 6) and as a consequence, failure developed under traffic at a more rapid rate. The deflection curves shown on Figure 8 were only included in this report to illustrate the types of deformation which (with sufficient load repetitions) contribute towards pavement failure A few thousand oscillograph records of pavement deflections have been obtained on highways and test sections during the last four years for various wheel loads, tire pressures, subgrade conditions and types and thicknesses of pavement and base A detailed discussion of the results is, of course, beyond the scope of this paper. However, a summary of the results of deflection and traffic tests on the runway test section is included.

SOIL TESTS

The stress-strain characteristics of subgrade soils and their resistance to plastic deformation probably are influenced by, although not directly dependent, on the internal friction and cohesion of the material. These properties should, therefore, be carefully considered in theoretical studies of pavement and foundation design procedure. However, internal friction as commonly measured by shear, triaxial and stabilometer tests is greatly influenced by the pore pressure developed in the specimen during the test In such cases, the results do not indicate the action of the materials under service conditions unless the pore pressure developed in the field is of the same intensity. Unfortunately, simple equipment for quickly determining accurate quantitative values of cohesion and internal friction has not yet been developed. Furthermore, in connection with routine design and construction control, thousands of tests must be made to determine the quality and variations of subgrade materials encountered and to evaluate the subbase and base materials available in each locality A simple and quick test procedure must, therefore, be used even though the results cannot be quantitatively applied to all construction without allowing for the many pavement, surcharge and traffic conditions involved

Field Bearing Tests

As indicated, static load tests are influenced by the elastic, consolidation and plastic deformation to a variable degree, depending upon the soil conditions. Under static loads, the pore pressure does not reach the maximum developed under dynamic conditions, if some of the air and water escapes during the test As a consequence, consolidation of the underlying material may, in some cases, constitute the major portion of the deformation. If sufficient fluids are expelled during loading to greatly reduce the pore pressure, little or no plastic deformation occurs during the test whereas an appreciable amount may take place under each repetition of a dynamic load. Because of these factors, the results measured often cannot be evaluated so as to indicate the deflection and performance of pavements under moving wheel loads One other difficulty is that the test usually cannot be made in the field when the material is at a state of density and moisture comparable to that which will exist subsequent to construction It is difficult and sometimes impossible to properly moisten the soil in the field to the depth affected by the load test, and as a consequence, the result may only represent a test condition which bears little or no relation to the ultimate condition and action of the subgrade during service.

Bearing Ratio and Expansion Tests

The bearing ratio test was devised in 1929 in an attempt to eliminate some of the objections to field loading tests and to provide a quick method for comparing local base and subbase materials. Samples are first thoroughly consolidated to the density obtainable under good construction methods and to approximate the density ultimately produced by traffic in good subgrade and base materials. This procedure eliminates to a large degree the consolidation deformation which often influences static load tests made in the field. The compacted specimens are next soaked under a surcharge representing the weight of the pavement, to permit the specimen to swell and reach the adverse state of moisture which is usually present in the subgrade under normal drainage and climatic conditions. A penetration test is then made on the specimen to determine the resistance to lateral displacement, thus measuring the combined influence of cohesion and internal friction. As the increments of penetration are large in respect to the size of the loaded area, the results are only influenced to a minor degree by elastic deformation.

The test provides a quick method of comparing local base and subgrade materials and determines the two most important subgrade properties affecting pavement service, namely, the resistance to displacement under moist to wet conditions and the volume increase and up-lift resulting from absorption of moisture subsequent to construction. By continually investigating pavement failures and testing the materials, an empirical relationship was subsequently established between the test values and the suitability of base and subgrade materials. The favorable correlation found between the test data and the field service of the materials is the principal reason for the continued use of these tests in connection with our routine design and construction control procedure.[1]

A detailed outline of the "Method of Test for Determining the Bearing Ratio and Expansion of Subgrade Soils and Base Materials" is appended to this report (see Appendix A).

Typical Tests

Tests indicate that clay and other adverse soils usually have a good supporting value when in a thoroughly compacted and relative dry state. This ideal condition, however, does not continue over a period of years. Beneath impervious pavements, such materials usually take up sufficient moisture by capillarity or by vapor pressure and condensation to cause expansion and as a consequence of the increase in water content, the soil often reaches the state of wetness and compaction comparable to the soaked specimen in the bearing test. Typical load-penetration curves for a number of untreated soil and base materials are shown on Figure 9. Good base and subbase materials usually show expansion values of less than 1 per cent during the soaking period. California specifications for imported subgrade material (imported borrow) include an expansion test requirement of not more than 1 per cent or 2 per cent the lower value being used on all projects where the better grade of local material can be obtained at low cost.

Figures 10 and 11 show a comparison between dry weight, bearing ratio and expansion of 257 samples of soil, base and subbase materials commonly encountered in California. These data show that no relation exists between the density and the bearing ratio or expansion value of these materials. Many of the samples showing densities ranging between 50 and 90 lb. per cu. ft. consist of soil, siliceous

[1] O. J Porter, "The Preparation of Subgrades," *Proceedings*, Highway Research Board Vol. 18, Part II, p 324 (1938)

shales and other materials containing some diatomaceous earth The samples in this density range also include soils which contain varying amounts of light volcanic particles, such as pumice, cinders or other grades of altered and weathered tufaceous materials In contrast to the experience reported from eastern and mid-western States, many good subbase and subgrade materials are found in western States which show compacted densities of less than 100 lb. per cu ft. Because of their

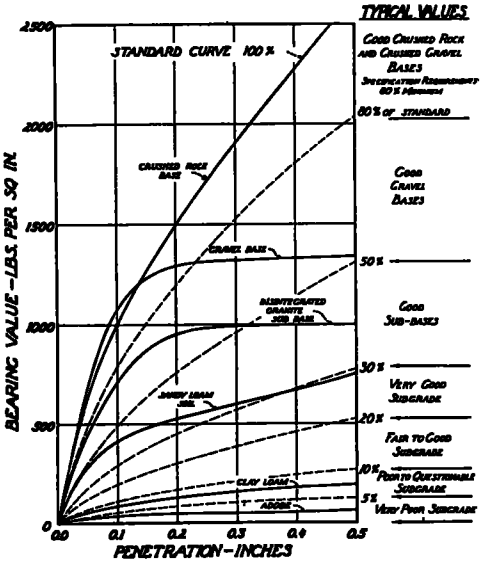


Figure 9. Bearing Value Tests Compacted and Soaked Specimens

light weight and the porous nature of the particles, many of these materials show high liquid limits, but low to moderate plasticity indices.

Figures 12, 13, 14, and 15 show the relations between bearing ratios and the plasticity index as of typical soils, subbase and base materials available for highway and airport construction in California. The relationships between the bearing ratios and the expansion of these same materials are shown on Figures 16, 17, 18, and 19. It will be noted that the soils which consist largely of passing the No 8 sieve (Fig. 12) show a low bearing ratio when the fine material passing the No. 40 sieve shows a high plasticity index. These materials also show high expansion (Fig. 16). The A-6 and A-7 soil types usually show a very low bearing ratio and high expansion values. The bearing ratio and expansion characteristics of A-2 soils, on the other hand, may range from very good to poor, depending on the amount and angularity of the coarse particles, the plasticity index of the fines and the grading of the material. Samples which expand appreciably during the soaking period consistently show relatively low bearing ratios.

A study of the data on Figures 12 to 19, inclusive, indicates that material having a plasticity index of less than 10 may show a bearing ratio ranging from a low to a high value, depending on its expansion characteristics and to a lesser extent on the amount of coarse particles and grading of the material.

Samples showing a bearing ratio of 100 per cent or more usually contain a large amount of crushed or naturally angular particles Many of these samples showing high bearing ratios consist of crushed rock, crushed gravel or disintegrated granite produced, selected or blended for base course construction. The data shown on Figures 18 and 19 indicate, however that granular materials containing a relatively large amount of coarse particles will show a low bearing ratio, if the compacted specimens expand appreciably during the soaking period as do some of the shales and other types of soft rock available for subgrade and subbase construction. Such materials have proved unsatisfactory for subbase and base construction and they are eliminated by including low expansion and appropriate bearing ratio test requirements in the specifications

DESIGN OF FOUNDATION FOR FLEXIBLE PAVEMENTS

Numerous methods have been proposed from time to time for the design of

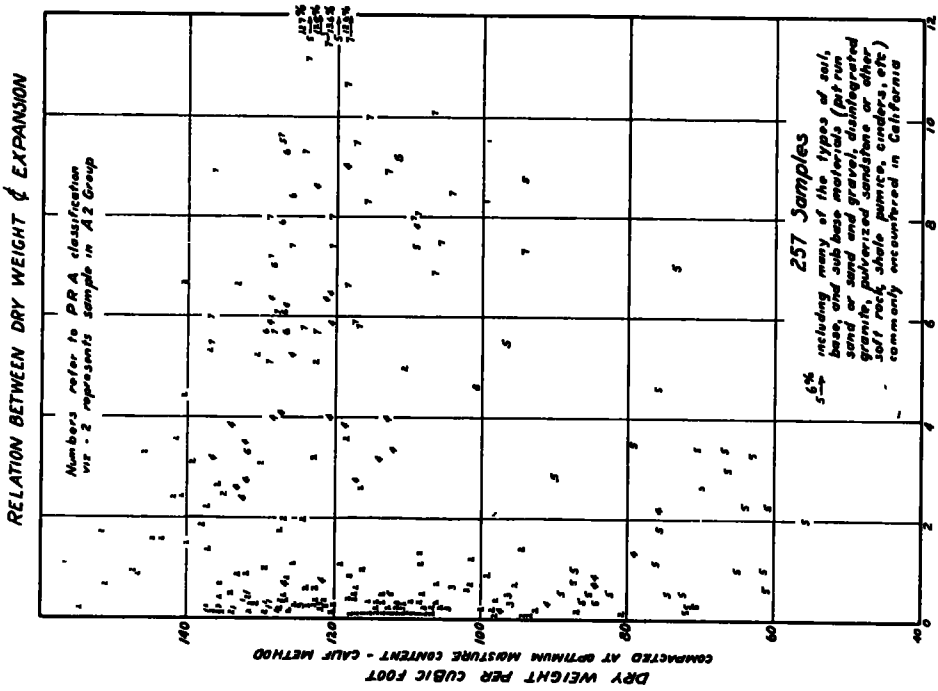


Figure 11. Relation Between Dry Weight and Expansion

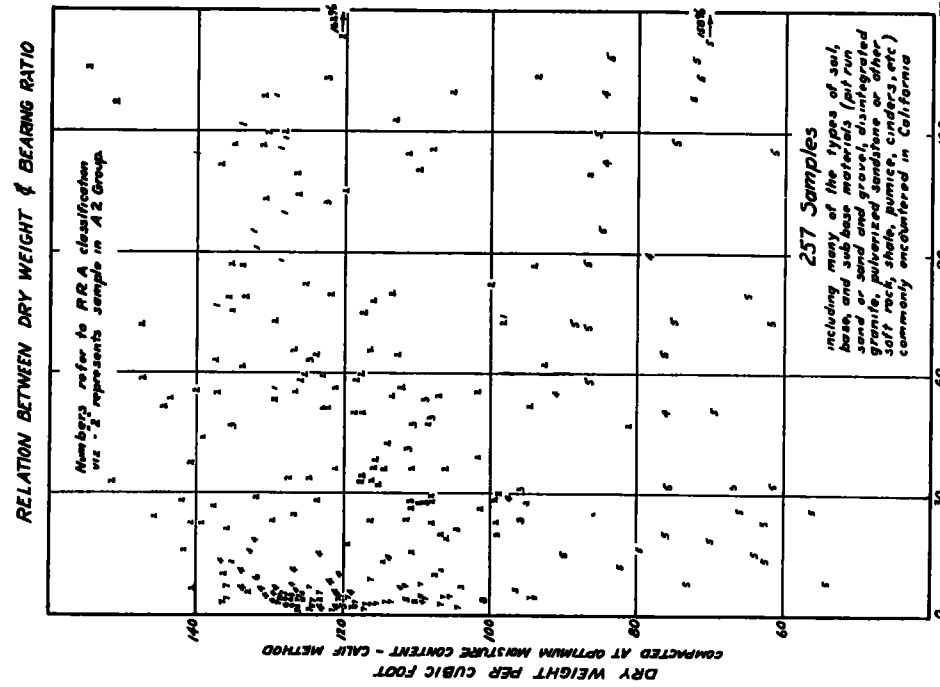


Figure 10. Relation Between Dry Weight and Bearing Ratio

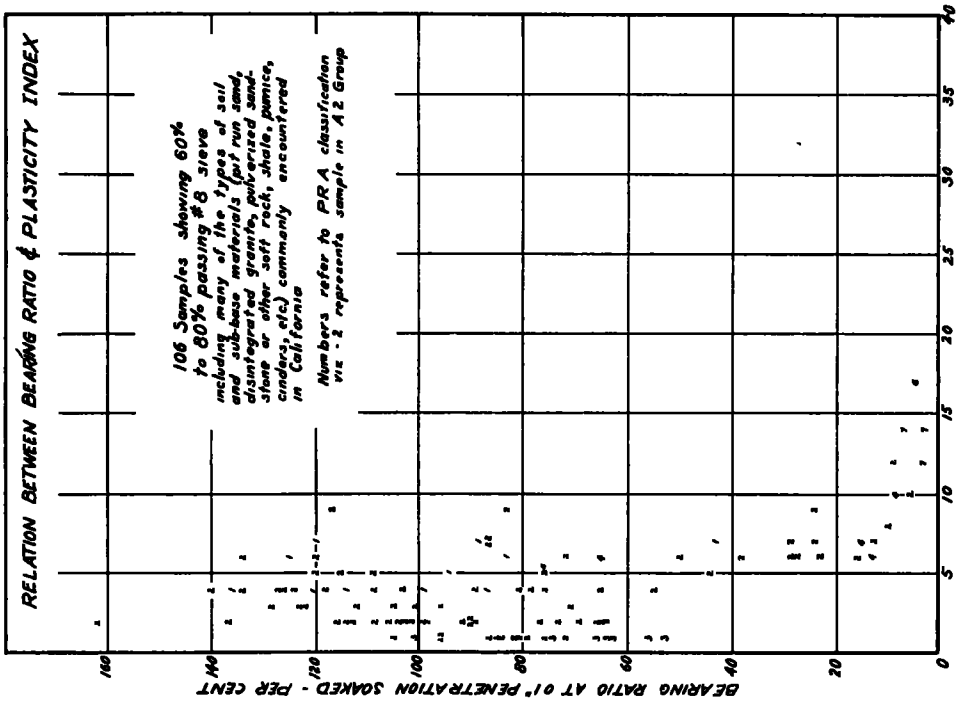


Figure 13. Relation Between Bearing Ratio and Plasticity Index

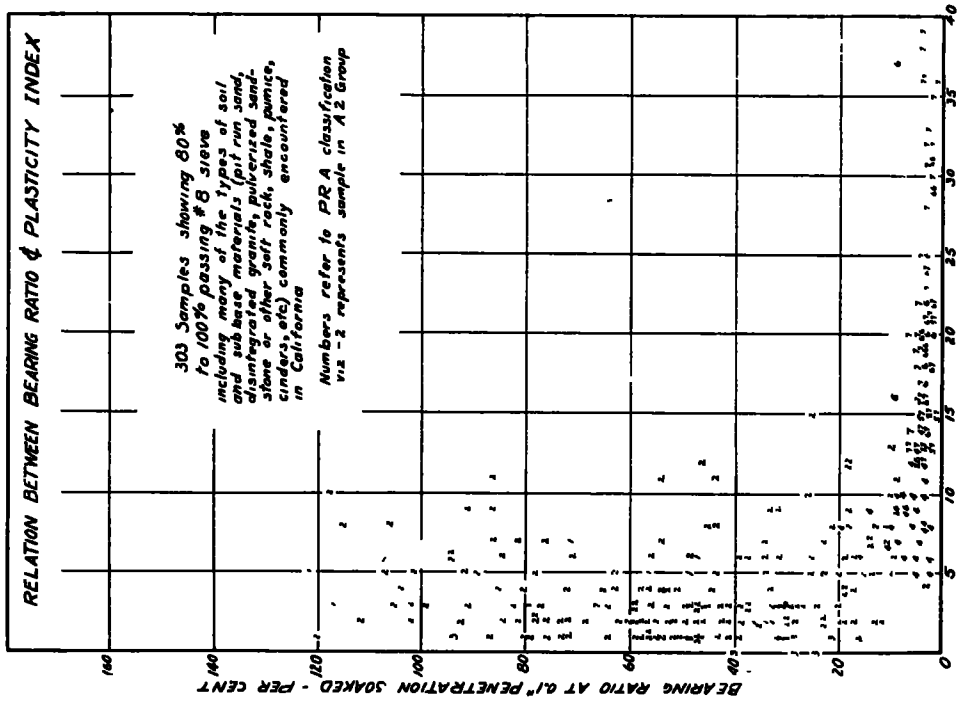
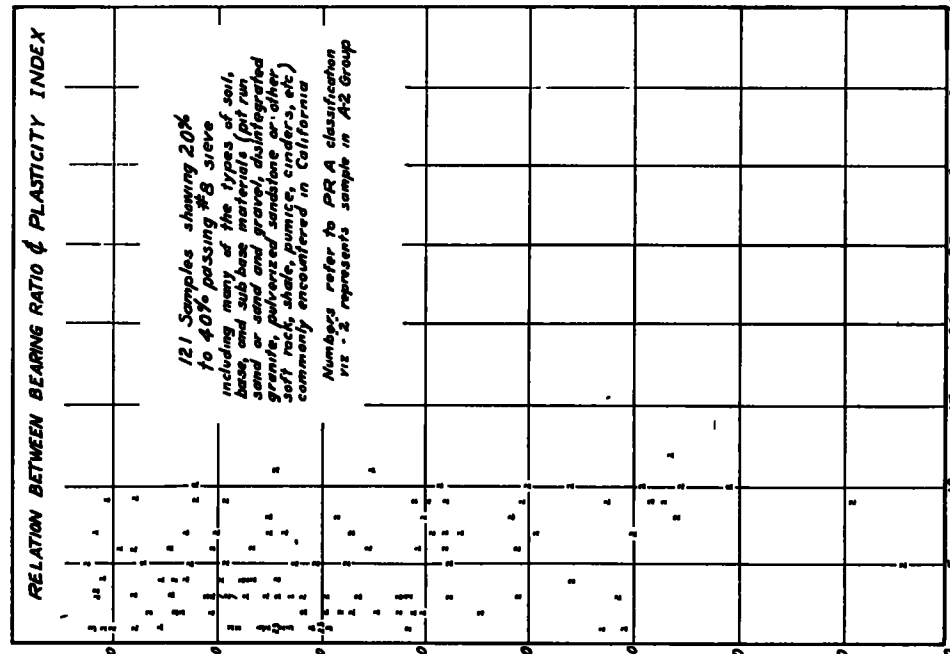


Figure 12. Relation Between Bearing Ratio and Plasticity Index



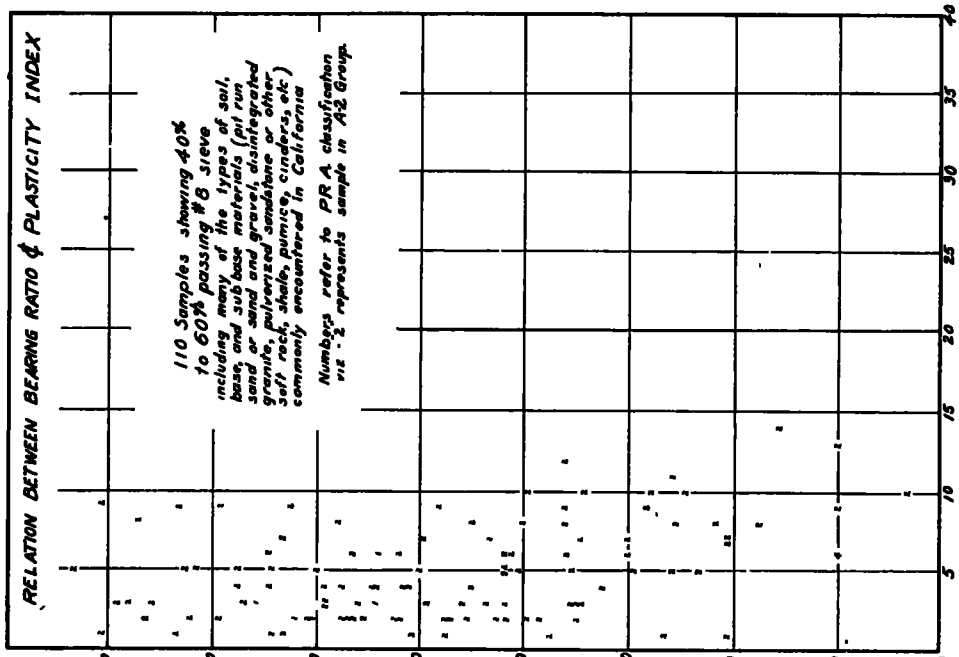


Figure 14. Relation Between Ratio and Plasticity Index Figure 15. Relation Between Bearing Ratio and Plasticity Index

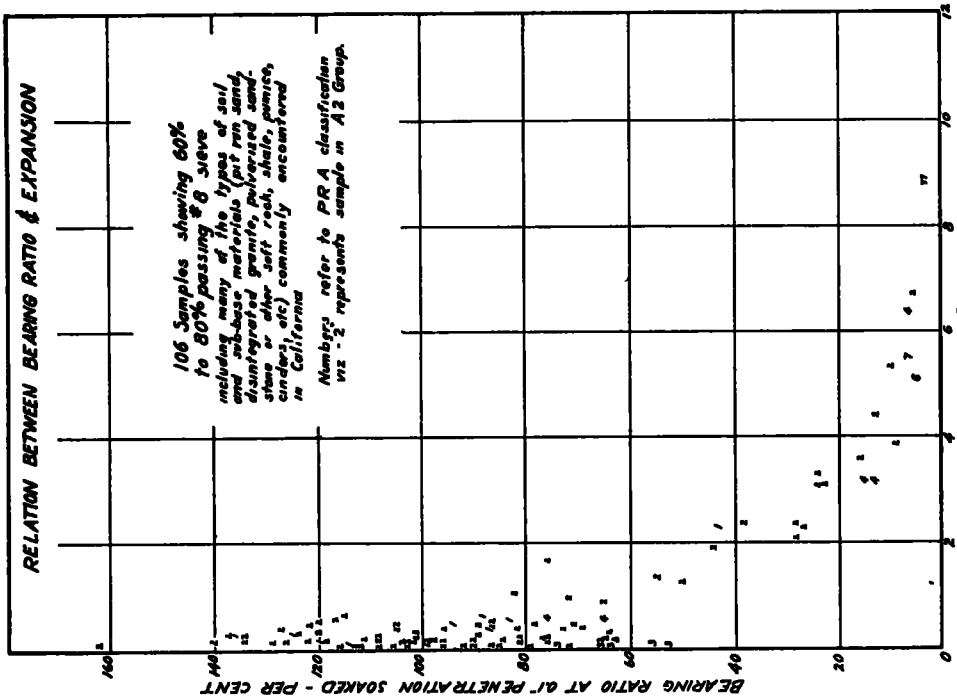


Figure 17. Relation Between Bearing Ratio and Expansion

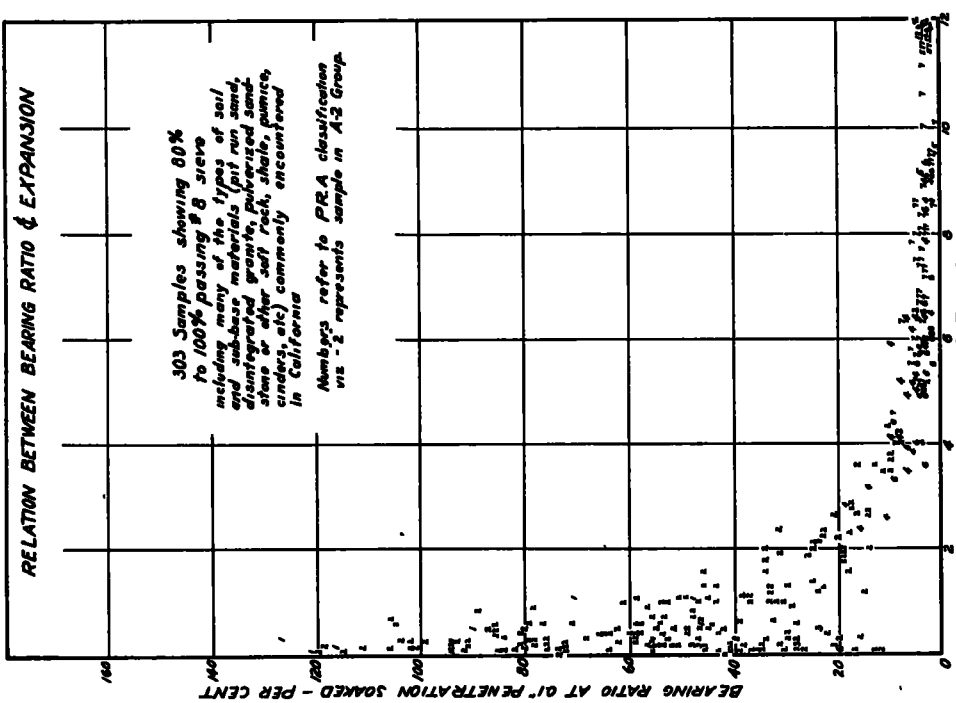


Figure 16. Relation Between Bearing Ratio and Expansion

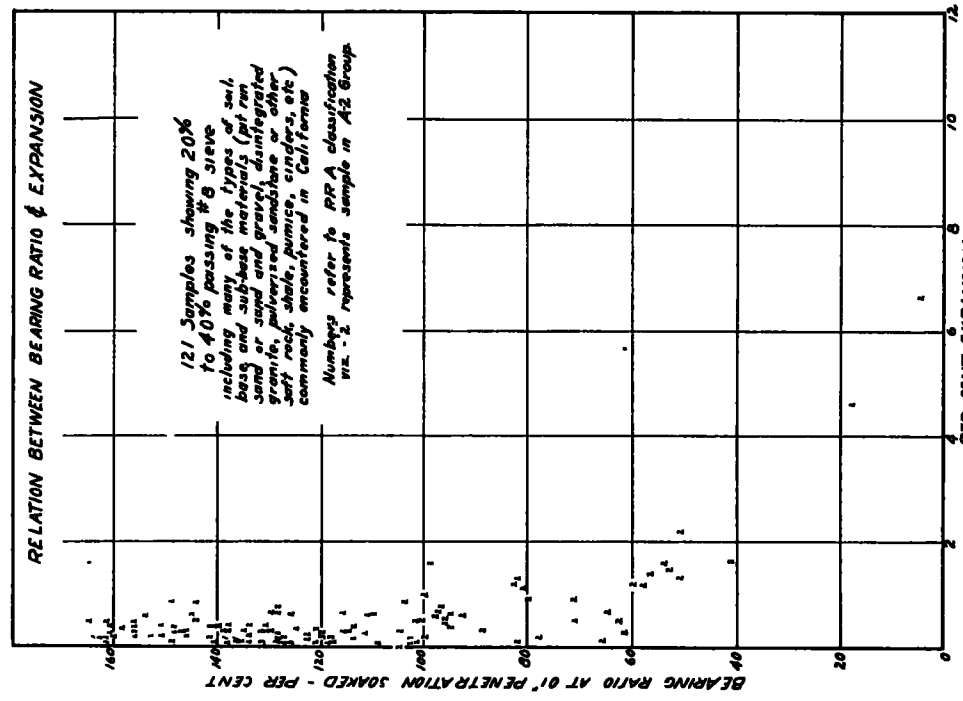


Figure 19. Relation Between Bearing Ratio and Expansion

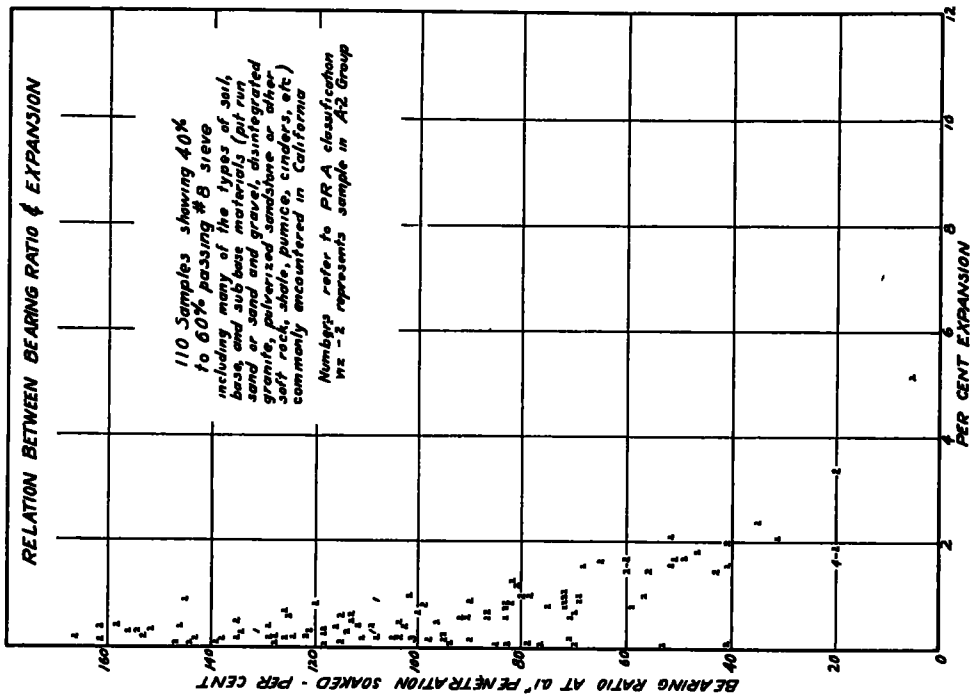


Figure 18. Relation Between Bearing Ratio and Expansion

flexible pavements. All of them recognize that the character of the foundation is of primary importance. Many of these methods use static load tests to determine the bearing value of the soil and in practically all cases some deflection value, ranging from 0.1 to 0 5 in has been set as a limiting value of deformation for use in design. It is usually assumed that the thickness of base and pavement should be equal to the amount required to reduce the average subgrade pressure to a value less than its supporting value after assuming that the total applied load would be spread through the base and pavement at an angle of approximately ±45 deg Although these assumptions are based on extensive studies of the distribution of pressure under static conditions of loading, investigators, until recent years, have not attempted to use such methods for designing pavements for the heavy wheel loads now contemplated and to verify the design assumptions by actual measurements of the pavement behavior under moving loads.

In California attempts were also made to utilize static load field bearing tests. Little or no encouragement was furnished by data obtained on a limited number of projects and hence study was directed towards attempting to correlate the service behavior of foundations with a simpler laboratory test.

Although the bearing ratio test was first used to compare local granular materials, such as soft sandstone and shale, disintegrated granite, pit run sand and gravel with the crushed rock bases conventionally used prior to 1929, it was also utilized to test the soils either as found in place or compacted in the subgrade. In line with a sound research procedure, studies were made to determine the qualities of base, subbase and subgrade materials under both failed and good sections of flexible pavements throughout the California highway system.

Curves "A" and "B" of Figure 20 show the minimum thicknesses of reinforcement now used for light and medium-heavy traffic on the California Highway System and represent the extensive empirical data collected from our studies during the past 12 years. Figure 20 also shows additional curves for much heavier, as well as lighter wheel loads which are proposed by the office, Chief of Engineers, Washington, D. C., as a basis for the design of flexible pavements for airfields. Because curves for very heavy wheel loads have not been adequately substantiated by experience, they should be adjusted as additional data from static and dynamic load tests and the performance of runway pavements under traffic become available The thicknesses derived from Figure 20 apply for average conditions and for a tire pressure of 60 lb. per sq. in. and it is recommended that they be increased, or decreased by 20 per cent depending on the quality of the base, characteristics of imported fill material, tire pressure, ground water and drainage conditions, possible effects of frost action and frequency of load.

This design procedure contemplates the use of compacted soils. Whenever the subgrade is in a loose condition, it is desirable that it be thoroughly compacted to a depth of at least 1 ft Where the subgrade and subsoil cannot be economically compacted to an adequate depth, the bearing ratio of the materials must be determined for the soil in its naturally loose condition.

Special attention is called to the fact that many good grades of sandy soil become "quick" and displace under the action of the airplane wheel loads and the vibration of the motors, unless they are thoroughly compacted to a depth of at least one and one half times the width of the loaded area. This action will not occur in moderately compacted soils, if the ground water is low and the material does not become completely saturated. The need for extra high compaction in per-

vious soils is, therefore, considerably reduced by good drainage. On the other hand, most adverse clay soils become

tion of the ends of the runways where the maximum number of wheel load repetitions, together with the motor vibration,

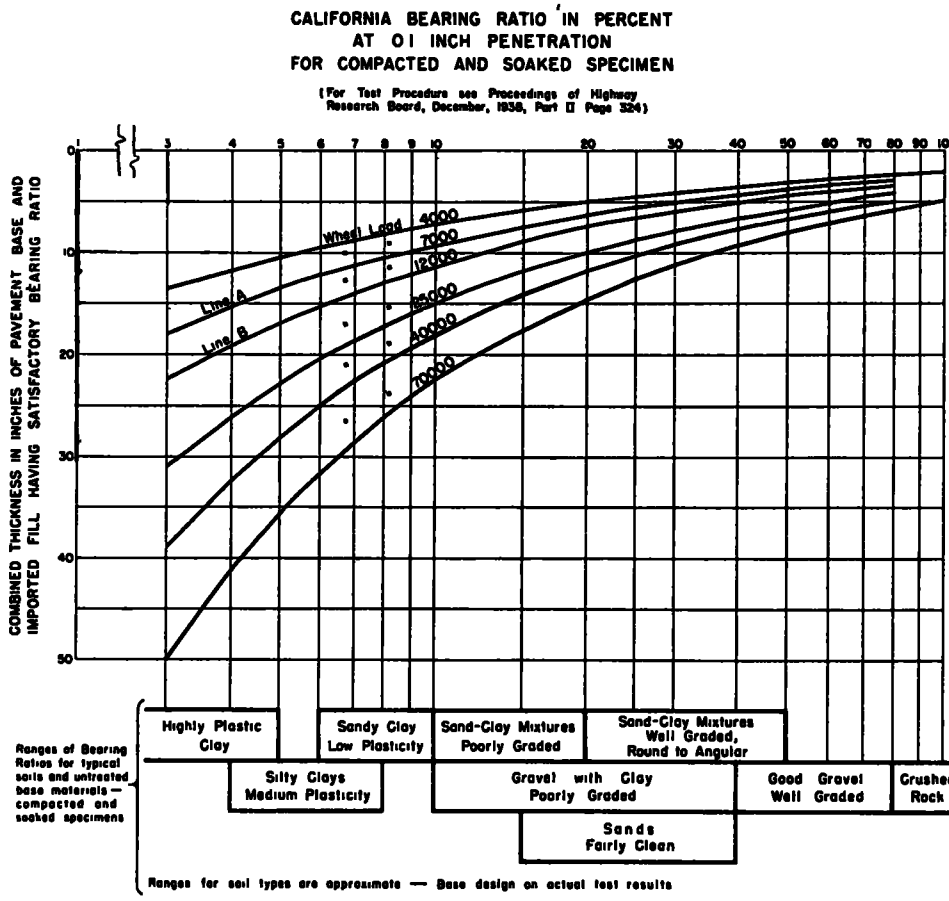


Figure 20. Tentative Design of Foundations for Flexible Pavements

1 Line "A"—California experience for light highway traffic
Line "B"—California experience for medium heavy highway traffic. (See "California Highways and Public Works," November, 1941, page 7)

2 Thicknesses derived from this chart are for average conditions based on a tire pressure of 60 lb per sq in and they should be increased or decreased up to ± 20 per cent, depending on tire pressure, thickness and type of pavement base, characteristics of imported fill, ground water and drainage conditions, possible effects of frost action, and frequency of loading

3 The minimum bearing ratio shall be used for determining the required thickness of pavement and base for total thickness of less than 6 in. instead of the bearing ratio at 0.1 in penetration (From Chapter XX, Engineering Manual, Office, Chief of Engineers).

plastic under impervious pavement regardless of their initial compaction and the drainage conditions. As a consequence, the thickness of pavement should still be increased on taxiways and the central portion of the runways where the maximum number of wheel load repetitions, together with the motor vibration, will tend to increase the subgrade deformation and pavement deflection.

Measurements of the action of flexible pavements under the dynamic influence of light and heavy planes indicate that the

vibration of the motors during taxiing and takeoff increase the deflection 20 to 100 per cent. The minimum values were obtained where the pavement and base were amply strong to carry the rolling wheel loads while the greater increases in deflection occurred where the thicknesses of the pavement and base were 20 to 40 per cent less than the former amount. Even when the soils do not become "quick" the vibration may increase the rate and amount of plastic deformation. These studies, together with field observations indicate that even greater increases in the deflection, often resulting in complete failure of the pavement, may occur where the pavement is not sufficiently thick to carry the rolling wheel loads for more than a relatively small number of repetitions. This action readily accounts for the numerous pavement failures which have occurred on runways while planes were taxiing or "revving up" the motors preparatory to taking off.

In order to provide a balanced design, it appears that the total thickness of base and pavement should be increased approximately 10 to 20 per cent on taxiways and the central portion of the principal runways where the pavement will be subjected to the maximum number of repetitions and the dynamic influence of the motors along with relatively slow moving wheel loads. This precaution will not be necessary, however, on the remainder of the runways where the fast moving wheel loads will not remain for a sufficient period to cause an appreciable amount of subgrade displacement and pavement deflections.

An example of a typical design of a flexible pavement over adverse soil will further explain the use of the design curves (Fig. 20).

Let us assume that:

1. The airfield will be permanently used.
2. A large percentage of the wheel loads will be about 25,000 lb.
3. The natural subgrade on the field consists of an A-7 soil showing a bearing ratio of 3 per cent.
4. The construction will be carried out during favorable weather.
5. The subgrade will be thoroughly compacted at a moisture content of about 20 per cent, which has previously been found (from special expansion test) to be the most desirable water content for use when the material is confined by the weight of about 30 in. of subgrade reinforcement and pavement.

Because of the poor character of the soil, it is further assumed that investigations have been made of all imported fill and subbase materials readily available and that a deposit has been located which contains an ample amount of two grades of material which are suitable for fill and subbase construction. The upper part of the deposit, to a depth of 10 ft., consists of sandy top soil and fine sandy loam soil showing a bearing ratio of 15 to 25 per cent at 0.1-in penetration. We will, therefore, conclude that selected fill material showing a bearing ratio of not less than 15 per cent can be obtained adjacent to the runway at low cost. Next, a well graded mixture of sand and gravel showing a bearing ratio of not less than 50 per cent at 0.1-in penetration underlies the sandy material in the pit. A subbase consisting of pit run gravel is, therefore, proposed for use over the selected fill material.

The design curve for the 25,000-lb. wheel load (Fig. 20) calls for a combined thickness of pavement, base and imported borrow of about 32 in. over this class (bearing ratio 3 per cent) of subgrade soil. The curve also indicates that the sandy material may be used as a fill over the adobe up to 12 in below the top of the pavement. Two layers of pit run gravel subbase totaling 6 in in compacted

thickness would next be provided on top of the sand. The curve permits this grade of material to be used up to within 6 in. of the top of the pavement. To complete the runway, a base course consisting of 3 in. of well-graded crushed gravel (bearing ratio of 80 per cent) and 3 in. of asphaltic concrete pavement, or any other suitable combination of base and pavement would be provided.

In the event funds are very limited and stage construction is proposed, the pit run gravel might also be used for the base course beneath a 3 or 4-in. thickness of pavement, provided that the material was carefully selected and properly compacted. In this case, however, an additional 2 or 3-in. thickness of bituminous pavement would then be proposed as resurfacing to improve and complete the construction after sufficient traffic developed to justify the additional expense.

It will be noted that this design follows the minimum values permitted by the curve and does not provide for the maximum safety factor possible at reasonable cost for the local conditions assumed in this example. Although this minimum design would be justified in many cases where the pit run gravel subbase involved a longer haul and an appreciably greater cost than the selected fill material it would be desirable in this case to decrease the thickness of selected fill to the minimum which will remove the overburden from the pit, and increase the thickness of pit run gravel subbase.

This design example also contemplates average service conditions and does not provide for a large number of load repetitions nor for extra severe service conditions on taxiways and at the ends of the runways. To accomplish a balanced design, it would, therefore, be desirable to increase the thickness of the imported fill and subbase materials to provide a total thickness, including the pavement, of 32 in. plus 10 to 20 per cent, or say 36 in., for any heavily traveled taxiways and for the central portion of the first 500 ft. of principal runway at the end where the maximum number of take-offs will be commenced.

Because the central portions of wide runways usually carry most of the load repetitions of heavy planes, it may be possible in many cases to effect economies by progressively reducing the total thickness of reinforcement from the center to the outer edge of the runway. If the runways were approximately 200 ft. wide, it is probable that the anticipated load repetitions would be sufficiently low so that the thickness at the outer edge could be safely reduced by 20 per cent, or say from 32 to 26 in., for this example.

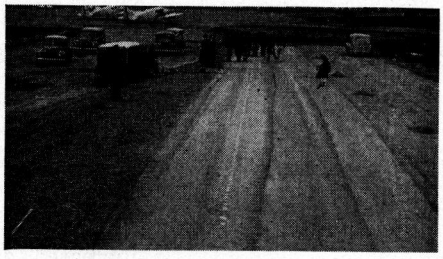


Figure 21. Army Engineers Inspecting Runway Test Pavement at a Western Field. White Line Indicates the Location of the Outside Wheel Track. Pavement and Base 4 Ft. Thick in Foreground, Tapers to 12 In. in Background Where Engineers Are Inspecting Failures Resulting from About 500 Trips of a 40,000-Lb. Wheel Load.

Where the ground is relatively flat, this gradual reduction in thickness could be applied so as to take care of a portion of the crown of the runway.

A WESTERN RUNWAY TEST SECTION

A runway test section (Fig. 21) was constructed at a western field by the Sacramento District, U. S. Engineers, as a part of an extensive research program conducted by the Engineering Department of the Office of the Chief of Engineers for the purpose of verifying or modifying the tentative design curves (Fig. 20). An abandoned taxiway at this western airfield

which had not failed due to lack of use was selected for this series of tests because:

1. The old runways constructed in the same manner as the taxiway and consisting of approximately 6 in. of sandy loam soil stabilized with emulsion, had failed under the action of light training planes (2,000 to 2,500-lb. loads) during the first year of the defense program.

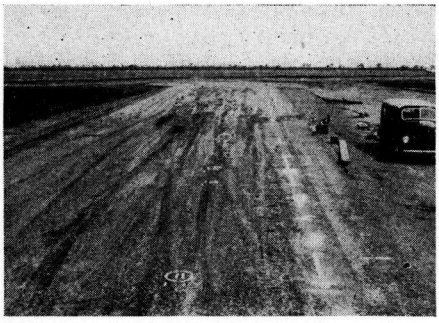


Figure 22. The Western Runway Test Pavement Before Deflection and Traffic Tests. 12 In. Thickness Under Electric Gauge Point 11 in Foreground and 4 Ft. Thickness in Background. Electrical Recording Equipment for Measuring Subgrade Pressure and Pavement Deflection Is Mounted in the Station Wagon.

2. The adobe subgrade (A-7 Classification) contained approximately 30 per cent of moisture to a depth of 6 ft. and had become equalized for the paved condition.
3. It was desired to determine the thickness of foundation required to support loads due to heavy bombers on a very poor, low bearing ratio soil.

The test section was constructed on approximately a 2 per cent grade so that the total thickness of pavement and base varied from 6 in. to approximately 50 in. The entire base course, except for the 6-in. thickness of old pavement which was scarified and recompacted, consisted of a mixture of sand and gravel and crushed gravel showing a minimum bearing ratio of 85 per cent at a penetration of 0.1 in. and a minimum of 62 per cent at a penetration of 0.5 in.

An asphaltic concrete pavement 3 in. thick was constructed throughout the length of the test section so that the only variable in the test would be the thickness of the base course.

Electric deflection gauges (Fig. 22) were installed to measure the deflection of the base and pavement under the action

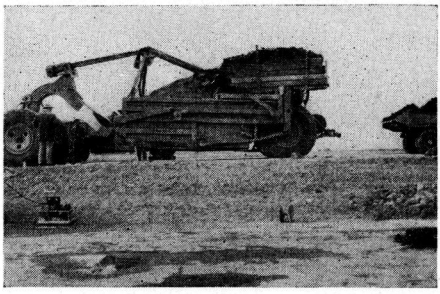


Figure 23. R. U. Le Tourneau Scraper Loaded With Sand and an Additional Load of Scrap Iron Over Left Rear Wheel to Obtain a Load of 5,000 Lb. on a Single 24.00 by 32 Tire.

of both moving and static loads. One set of deflection gauges was placed in the old taxiway to determine the movement at the bottom of the base and another set in the new pavement to measure the total deflection. The test program also included static load bearing tests on circular rigid plates on both the subgrade and the pavement for comparison with the deflection of the pavement under wheel loads having footprints and contact pressures comparable to those of typical airplanes.

Deflection Tests

The deflection of the base and pavement was progressively determined under wheel loads of approximately 5,000, 10,000, 25,000, 40,000 and 54,000 lb. (Fig. 23). Trucks and Le Tourneau

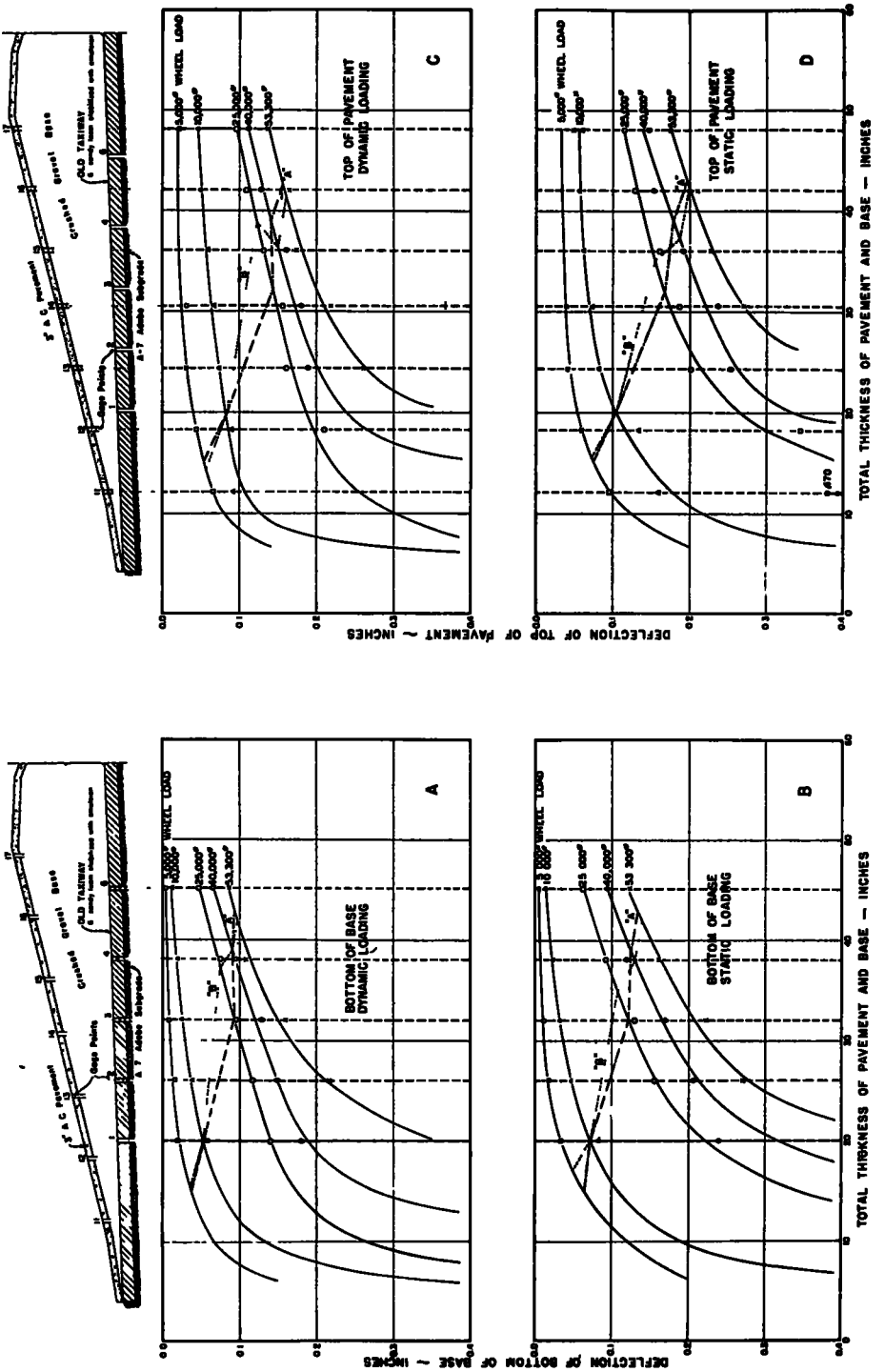


Figure 24. Deflection of Pavement. Western Runway Test Section

scrapers were used to obtain the desired wheel loads and, in general, the tire pressures were adjusted to obtain a footprint contact pressure ranging between 60 and 70 lb. per sq. in Appendix B includes a description of the electric gauge equipment.

Figure 24 shows a profile of the test section, location of the electric deflection gauges and the deflection obtained under both static and dynamic wheel loads. It will be noted that for a given thickness of pavement plus base the deflection increased rapidly with the wheel load intensity. The deflection of the bottom of the pavement under dynamic and static wheel loads (Figs 24A and B) furnishes the best indication of what action takes place in the subgrade. These values include the movement due to the elastic, consolidation and plastic deformation of the subgrade. Because the subgrade consisted of an impervious adobe soil, little or no consolidation occurred and, therefore, the permanent deformation and grooving which developed under traffic was almost entirely due to plastic flow of the soil The deflections measured on top of the pavement (Fig. 24-C and D) include both the consolidation and elastic deformation of the base in addition to the subgrade movement. These results indicate the total amount of deflection the pavement must continue to withstand without cracking to give adequate service.

Lines "A" shown on Figures 24-A, -B, -C and -D indicate the approximate thickness called for by the design chart (Fig. 20) for a subgrade showing a bearing ratio of 3 per cent. This is the average value obtained on this adobe soil in the regular test. Undisturbed samples from beneath the test section showed bearing ratios of 3 to 6 per cent.

Lines "B" on Figures 24-A, -B, -C, and -D indicate the approximate thickness required for the various wheel loads if the permissible deflection is increased in direct proportion to the square root of the footprint areas. It is believed that except for actual traffic tests this is the best method of determining the thickness of base required from the deflection measurements as it allows for approximately the same radius of curvature of the deformation of the pavement under the various loads. In view of the fact that Line "B" (Fig 24-A) indicates thicknesses for heavy wheel loads which are within about 10 per cent of the values shown by Line "A," it does not appear that any major adjustment should be made in the design

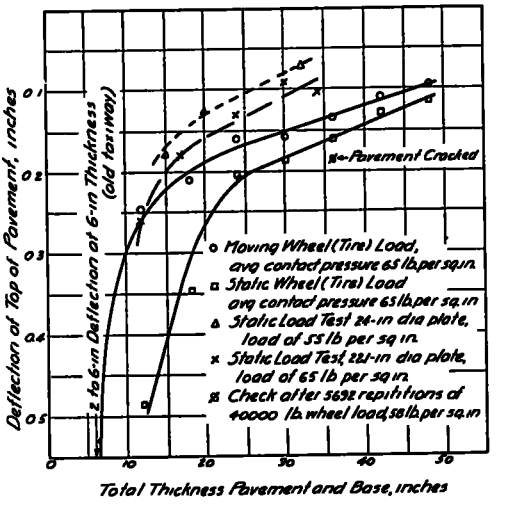


Figure 25. Deflection of Top of Pavement, 25,000-Lb. Loads, a Western Runway Test Section.

curves (Fig. 20) for the class of subgrade and base material encountered on this project.

The test furnished much additional information of value in connection with the design of flexible pavements. As an example, Figure 25 shows the deflection of the top of the pavement under a 25,000-lb wheel load and also the results obtained from static load bearing tests using a rigid plate. It will be noted that, in the bearing test, the amount of pavement deflection was much lower than that measured when the same load was applied to

the pavement through a tire. Similar discrepancy has been found in other tests. It is believed that the shape and rigidity of the plate greatly affects the results of bearing tests. Under a tire load, the pavement and base can bend throughout the loaded area while in the case of rigid plates practically all of the bending must occur outside of the loaded area

Traffic Tests

After the deflection and static load bearing tests were completed, the test section was subjected to the traffic of 25,000-lb. wheel loads on one side and 40,000-lb wheel loads on the other to determine the rate of failure in relation to the total thickness of pavement plus base and load repetitions. During these traffic tests each wheel load was staggered and spread over a width of 5 to 6 ft to approximate more nearly actual service conditions.

The pavement cracked and failed up to the 24-in. thickness of pavement plus base and a groove or rut ¾ in. deep progressed up to about the 32-in thickness under 5,772 trips of the 25,000-lb wheel load. The rate of failure in relation to thickness of pavement plus base and load repetition indicates that a total thickness of 30 to 32 in would be required to withstand 10,000 to 20,000 repetitions of the wheel load without serious cracking. This is in close agreement with the thickness called for on the design chart (Fig. 20).

Figure 26 shows the rate of roughening and failure of the pavement under the 40,000-lb wheel load in relation to pavement thickness, pavement deflection and load repetitions. It will be noted that the thickness of pavement plus base required to avoid serious failures depends to a large extent on the number of traffic repetitions. Data previously obtained on highway test sections for lighter wheel loads and various types of bases also indicates that the rate of failure progresses as a straight line up to 30,000 repetitions on this type of semi-log chart. Figure 26 shows that 8,000 trips caused the pavement to fail to the 39-in thickness and that, at this point, the bottom of the base showed a deflection of less than 0 1 in under the moving wheel load prior to

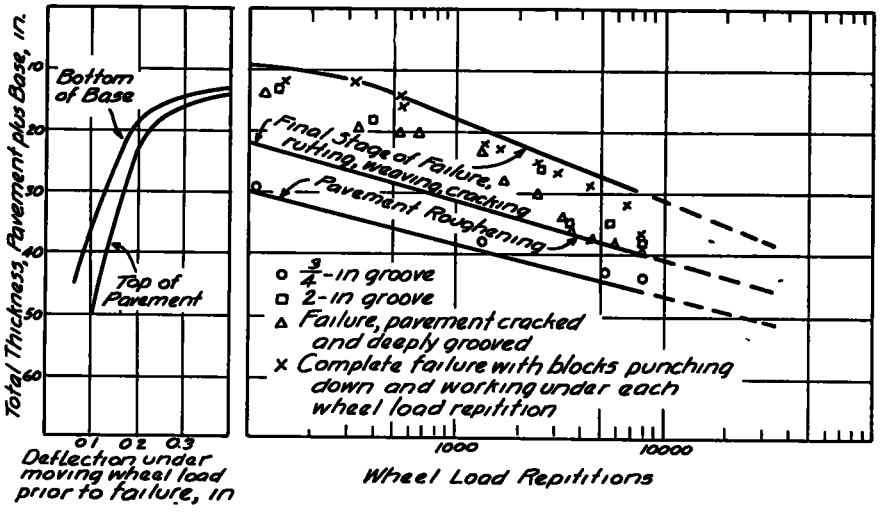


Figure 26. Deflection of Pavement and Rate of Failure Under Traffic of a 40,000-Lb. Wheel Load. The Thickness Includes 3 In. of Asphaltic Concrete Pavement Plus Variable Thicknesses of Crushed Gravel Base.

failure. It appears that even a somewhat greater thickness of base, resulting in lower deflections, would be required to adequately support the pavement for 20,000 to 30,000 repetitions of the wheel load The traffic test clearly indicates that the 38-in. thickness called for on the design chart (Fig 20) for this wheel load is the minimum which should be used for the type of subgrade and base material encountered on this project.

CONCLUSIONS

1. The so-called bearing value or modulus of deformation as determined by static load field bearing tests on a rigid plate may, in many cases, be at least double the value obtained when the test is made with the static load acting through an actual tire having a footprint and average contact pressure equal to that of the rigid plate.

2. Because of inertia, permeability of the soil and the influence of time on the stress-strain characteristics of all of the materials, moving wheel loads, in many cases, affect the pavement, base and subgrade radically differently than static wheel loads. Data already obtained from a few thousand oscillograph records show that the pavement deflection under a static wheel load may range from 75 to 400 per cent of the amount caused by a moving wheel load of the same intensity.

3. The extensive traffic tests conducted on the western runway test section at the direction of the Engineering Department of the Office of the Chief of Engineers, U. S Army, closely checked the design curves for the 25,000 and 40,000-lb wheel loads. The data clearly indicate that the design curves are not over-conservative for low bearing ratio soils and that the thicknesses probably should be increased somewhat, say 10 to 20 per cent, for any taxiway or runway which will be subjected to 10,000 to 30,000 load repetitions or more.

4 In California, during the past twelve years, good correlation has been found between the bearing ratio and expansion values and the service behavior of subgrade and base material under heavy truck traffic. The data now available indicate that the tests are equally applicable for use in connection with the design and construction of flexible pavement runways. Although some details of the testing and design procedure may require modification as additional information becomes available, it appears that, at least during the present emergency, the use of this simple and direct approach is fully justified even though instances develop where the design curves should be modified because of special climatic or unusual foundation conditions. Judgment, backed by intelligently interpolated factual data from practical experience, will, of course, be required to evaluate the test data and to arrive at a proper design in many cases

ACKNOWLEDGMENTS

Soils and foundation investigations for California highway construction are conducted by the Materials and Research Department, Division of Highways, California Department of Public Works Mr. C. H Purcell is Director of the Department of Public Works and Mr G T. McCoy is State Highway Engineer. Mr T. E Stanton is the Materials and Research Engineer

The runway test section at the western airfield was constructed by the Sacramento District, U. S Engineers, Col R. C Hunter, District Engineer, in accordance with plans prepared under the direction of Col James H Stratton, Chief, Engineering Branch, Office of the Chief of Engineers, Washington, D C Most of the tests were made by Messrs J. E Barton, Assistant Physical Testing Engineer, H L. Ferron, Assistant Highway Engineer, of the Division of Highways, and W. L. Davis, Engineer, Sacramento District, U S Engineers The writer, acting as consultant for the Army Engineers, supervised the test program

APPENDIX A

Method of Test for Determining the Bearing Ratio and Expansion of Subgrade Soils and Base Materials

Scope

1. This method of test is intended for determining:

(a) The relative bearing value (bearing ratio) of untreated base, subbase and subgrade material at

1 Optimum moisture and maximum density and
2. After soaking the compacted specimens of the material

(b) The extent to which subgrade and base materials will expand or swell during a soaking period of four days.

Note: To develop the stability indicated by this standard bearing ratio test, it is essential that subgrade and base materials be completely compacted in the field at optimum moisture content. When the roadbed or runway will be constructed over unconsolidated subsoils or whenever it is impractical to properly consolidate the subgrade and base materials in the field, the tests shall be made on undisturbed samples of soil at their in-place density or on disturbed samples compacted in the laboratory to the anticipated field density.

Apparatus

2 (a) A cylindrical mold 6 in in diameter by 8 in. high. The mold is fitted with a detachable base plate, a piston or plunger 5 in. high for compacting specimens and a penetration piston with an end area of 3 sq in (See Fig. 27).

(b) A testing machine, calibrated hydraulic press or other static loading apparatus of 60,000 lb capacity.

Note: Figures 28, 29 and 30 show hydraulic jack and compression frame assemblies suitable for use in the field. Although this portable equipment may also be used in the laboratory, a standard testing machine is recommended for permanent use.

(c) A scale or balance of 20-kilo capacity sensitive to 1 g

(d) A perforated plate with an adjustable stem and a 10-lb weight (See Fig 27)

(e) A dial or other suitable gauge for measuring the penetration of the 3-in area piston. (See Figures 27 and 31)

(f) A dial or gauge mounted in a tripod for measuring the expansion of the specimen during the soaking period (See Figures 27 and 32)

(g) A suitable water tank or vat for immersing the specimens during the soaking period.

(h) A suitable drying oven with forced ventilation

(i) Miscellaneous equipment including a depth gauge graduated in hundredths of an inch, mixing bowl, graduates, spatula, spoons, etc

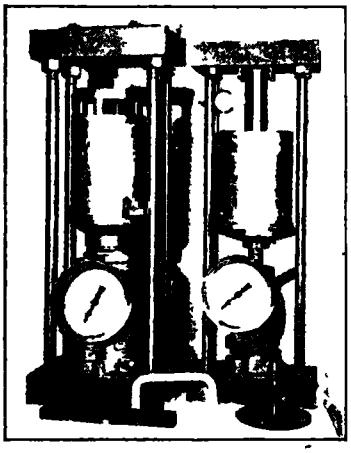


Figure 28. Field Bearing Value and Swell Testing Equipment. Hydraulic Jacks Compression Testing Assemblies. A—for Consolidating Soil Sample. B—for Bearing Test.

Preparation of Sample

3 A representative sample of sufficient size for the tests desired (usually 20 to 40 lbs) shall be oven-dried to constant weight at a temperature of 100 to 104 deg C. The dried material shall be separated into three sizes by screening on a ¾-in sieve and a No. 4 sieve In the event a grading analysis has not been made on the sample, the percentage of each of these three sizes shall be determined

Procedure

4 (a) A test sample of approximately 4,000 g (see Note 1) containing the same percentage of material passing the No 4 sieve as is present in the original sample (see Note 2) shall be weighed into a mixing bowl to an accuracy of 5 g. Three to six such samples shall be prepared in the event the maximum dry weight

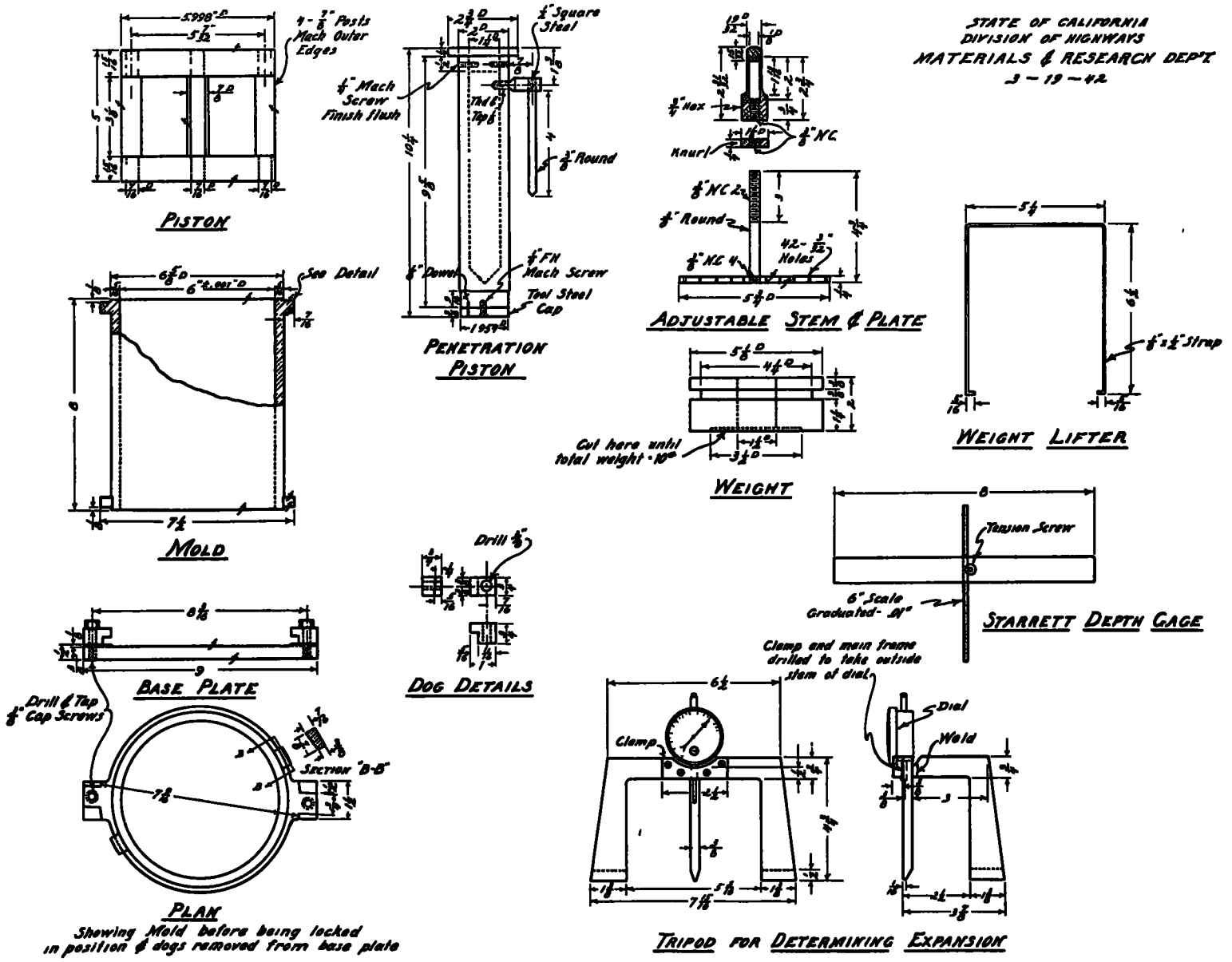


Figure 27. Details of Equipment for Determining the Bearing Ratio and Expansion of Soils and Untreated Base Materials. See Proceedings—Highway Research Board, 1938, Vol. II, Pages 324 to 331 Inc.

per cubic foot and the optimum moisture content has not been previously determined

NOTE 1 This weight of sample is satisfactory for average material When either light weight or heavy weight materials are proposed for use in the work Rock particles retained on the ¾-in sieve are too large for inclusion in the test Small rock passing the ¾-in and retained on the No 4 sieve shall, therefore, be substituted for the same percentage of larger rock particles retained

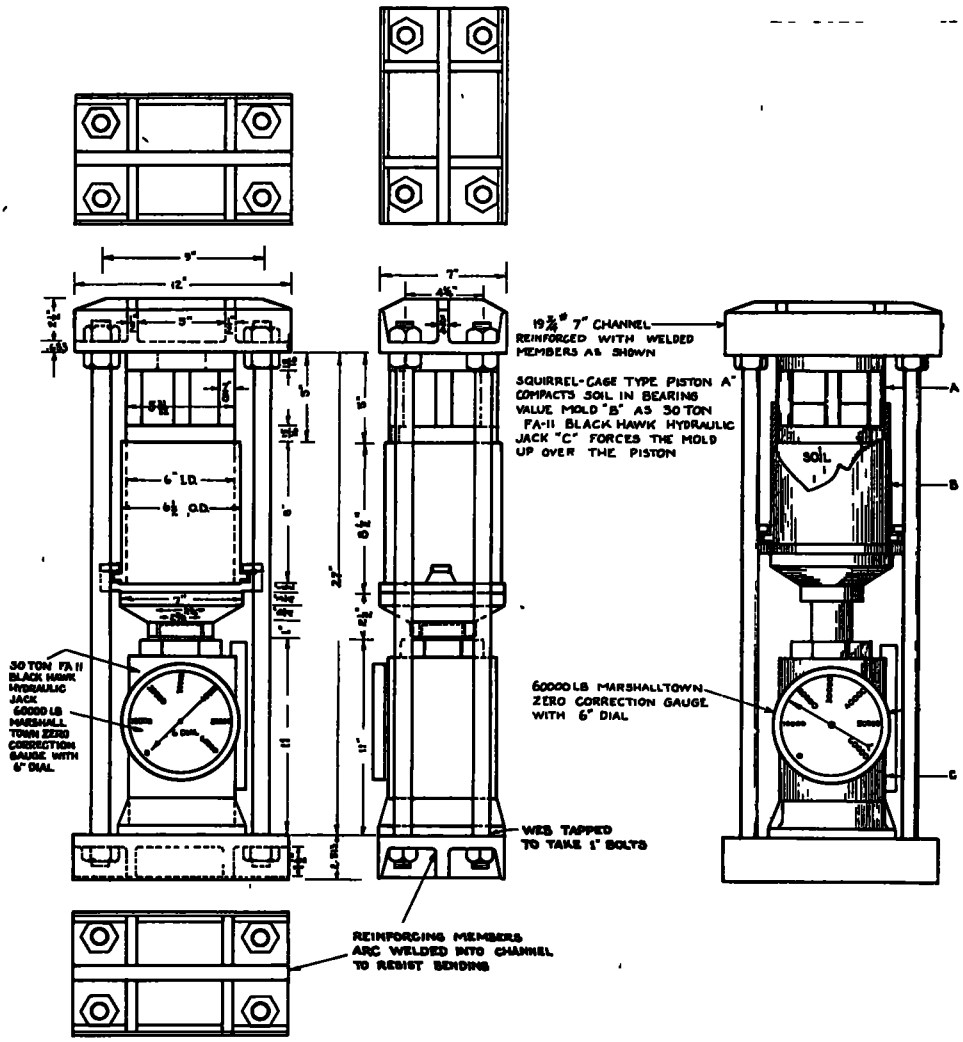


Figure 29. Jack and Compression Frame for Static Load Compaction of Soils in the Field

tested the weight of the samples shall be reduced or increased to an amount which will result in a compacted specimen 4 to 6 in high.

NOTE 2. The tests should be made, so far as possible, on a representative sample of material, including any rock particles, as on the ¾-in sieve in the materials as proposed for use Where the amount of rock particles in pit run or other materials proposed for use varies considerably, it is suggested that at least one test be made on a sample containing the lowest percentage of coarse particles which may be incorporated in the work.

(b) The sample shall be mixed with the amount of water (optimum moisture content) giving the maximum dry weight per cubic foot (see Note 3) and lightly tamped into the tared mold and then compacted to its maximum dry weight per cubic foot under a load of 2,000 lb per sq in In applying the load between 1,000 lb per sq in and 2,000 lb per sq in, the head of the testing machine or the hydraulic press shall be operated at the rate of approximately 0 05 in per min The static load of 2,000 lb per sq in shall be maintained on the sample for 1 min and then gradually released during a period of about 20 sec

NOTE 3 If the maximum density and optimum moisture content under this test procedure has not been previously determined, the additional samples prepared as specified under 4(a) shall be moistened and compacted at varying moisture contents so that a range well on each side of optimum is covered

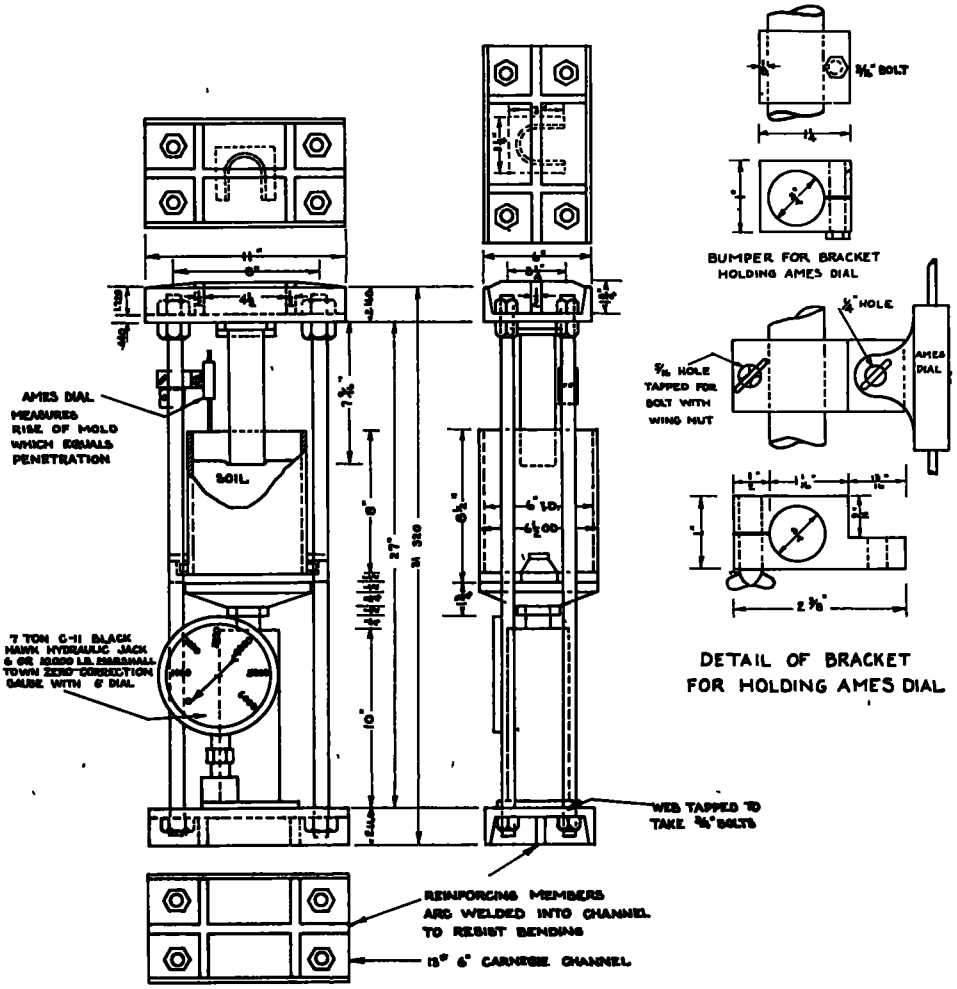


Figure 30. Bearing Value Apparatus for Field Use

(c) The molds shall be removed from the testing machine or hydraulic press and the height of the compacted specimen measured with the depth gauge to the nearest hundredth of an inch The dry weight per cubic foot of the specimen shall be calculated from the height and diameter of the specimen and the dry weight of the sample

Penetration of Compacted Specimen before Soaking

5. The compacted specimen, retained in the mold, shall be placed in the testing machine and tested by penetrating with the small piston (end area of 3 sq in.). Before starting the penetration test, the piston shall first be placed on the center of the compacted specimen and firmly seated by applying an initial total load of 10 lb after which the dial shall be set at zero to measure the penetration of the piston (see Figure 31) During the penetration test the head of the testing machine shall be operated at the

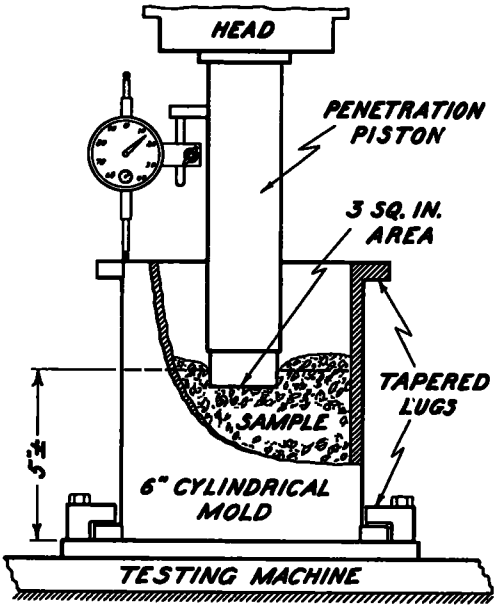


Figure 31. Bearing Test

rate of 0 05 in per min and the total load and the load in lb per sq in shall be recorded at penetrations of 0 1 in, 0 2 in, 0 3 in 0 4 in and 0 5 in

Expansion Test

6 (a) After completing the penetration test for the compacted condition, the top of the specimen shall be loosened to a depth of approximately 1 in with a screwdriver or suitable rod A 7-in filter paper shall next be placed on top of the mold after which the base plate shall be removed from the bottom and fastened to the top of the mold over the filter paper The mold shall then be turned up-side-down and the specimen recompacted under a load of 2,000 lb per sq in applied as previously specified under 4-b

(b) The height of the recompacted sample shall be recorded to check the height and the dry weight per cubic foot of the specimen as originally compacted

(c) A 6-in. diameter filter paper shall first be placed on top of the compacted specimen, followed by the perforated plate and then a 10-lb weight (See Note 4) The gauge and tripod assembly shall next be set on the mold and the stem on the perforated disc adjusted to a zero gauge reading (Fig. 32). After removing the gauge, the mold assembly shall be placed in the tank and water poured into the top of the mold The water level in the tank and in the mold

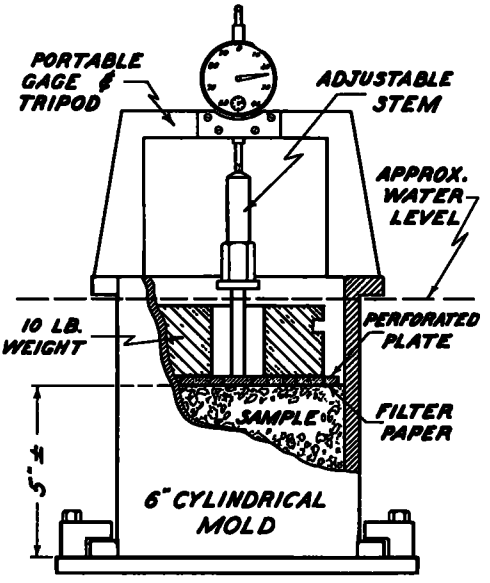


Figure 32. Expansion Test for Determining Swell of Bearing Value Specimens

shall at all times be kept between 1 in above and 2 in below the top of the mold

NOTE 4 The 10-lb weight and the perforated plate are approximately equivalent to the weight of a 5-in. thickness of base and pavement Some subgrade soils which expand considerably and show a low bearing ratio in the regular test will show lower expansion and a higher bearing ratio, if confined by a heavier weight during the soaking period If such materials are thoroughly compacted in the field and then confined by a heavier base and pavement, an additional test may be made using a surcharge weight of 20, 30 or 40 lb The weight used shall be equal to or slightly

less than the weight of the pavement and base course proposed for use

(d) The specimen shall be soaked for four days (see Note 5) The expansion shall be measured each day with the gauge and tripod assembly and recorded to the nearest one thousandth of an inch. The percentage of expansion shall be calculated in terms of the original height of the recompacted specimen before soaking.

NOTE 5: The four-day soaking period can be reduced to two or three days on very porous materials, such as relatively clean sand and gravel, if all expansion of the specimen has ceased and the material is fully saturated.

(e) After the four-day soaking period, the specimen shall be removed from the tank and with the 10-lb weight and the perforated plate held in place the mold shall be tipped up allowing the specimen to drain for one minute (see Note 6). The 10-lb weight, perforated plate and filter paper shall next be removed after which the specimen, contained in the mold, shall be weighed to determine the moisture content of the material after the soaking period

NOTE 6 Care must be taken to prevent disturbing the surface of the sample and to avoid vibration which will cause saturated material, particularly sand and silts, to become "quick."

Penetration of Soaked Specimen

7. The soaked specimen shall be tested for penetration as specified above under 5

Moisture Content

8 A sample shall be taken from the top of the specimen to a depth of 1 in weighed and dried to constant weight at 100 to 104 deg C The moisture content of this portion of the material, as tested for bearing after soaking, shall be recorded

NOTE 7· Difficulty may be experienced in obtaining a representative sample and the correct moisture content of granular materials which contain a considerable percentage of rock particles on the No 4 sieve This moisture content normally indicates the maximum amount of water the soil will take up when confined by the weight of a 6-in pavement and is particularly useful in checking the ultimate expansion of clayey materials This portion of the test may be omitted on non-swelling, granular materials which contain a considerable amount of coarse rock particles.

Calculation of Bearing Ratio

9. (a) The bearing ratio (relative bearing value) for both the compacted and the soaked condition of the material shall be computed in percentage of the following standard loads for each increment of penetration

Penetration, in	Standard load, lb per sq in
0 1	1000
0 2	1500
0 3	1900
0 4	2300
0 5	2600

(b) Unless otherwise specified, the bearing ratio of a material shall be considered as the minimum value shown for any increment of penetration (see Notes 8 and 9).

NOTE 8: In general, the bearing ratio of the soaked specimen at 0 1-in penetration is used for evaluating subgrade materials while both the value of 0 1-in penetration and the minimum value (usually at 0 5 in) may be specified for the high-quality, crushed and graded base materials, normally used in the upper 6 to 9 in of the base and pavement

NOTE 9 In making routine comparisons of subgrade and base materials, time may be saved in performing the test by eliminating the penetration test on the compacted (unsoaked) specimen as specified under 5 As most materials show minimum bearing ratios for the soaked condition, this value is normally used in design, but the test on unsoaked specimens may be considered in evaluating embankment materials at maximum density and optimum moisture content or at any other density and moisture content proposed for use

Report

10 The following data shall be recorded and reported for each of the samples tested

(a) Grading analysis for the sample as received

(b) Grading analysis for the specimen used in each bearing ratio test.

(c) Moisture content, or optimum moisture for compaction

(d) Average moisture content of sample after soaking

(e) Moisture content of top 1 in of the soaked specimen

(f) Bearing ratio in percentage of the standard load at each increment of penetration for both the compacted and the soaked specimen

(g) The maximum dry weight per cubic foot of the material as compacted at optimum moisture content

(h) The dry weight per cubic foot and the corresponding moisture contents for each specimen compacted at moisture contents above or below the optimum

(i) Expansion in percentage of the height of the original specimen

(j) Notes as to condition of sample during tests or other information of value in comparing local material available for use

Alternate Procedures

11 When subgrades and subsoils are not consolidated and it is desired to test the material for bearing ratio at the density found in place, the penetration test shall be made on undisturbed

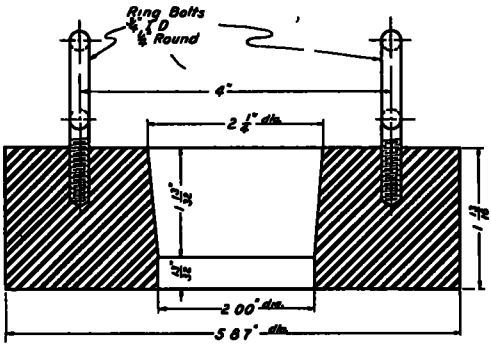


Figure 33. Surcharge Weight Used While Penetrating Cohesionless Materials in Bearing Ratio Test (Total Weight 12¼ Lbs.).

specimens in their natural state, both before and after soaking. When it is desired to determine the bearing ratio and expansion characteristics of materials at densities lower than the maximum dry weight per cubic foot, the test shall be made as outlined previously except that the compaction load shall be reduced to that which results in a specimen of the desired height and density.

Bearing Test with Surcharge

12. Round-grained, poorly graded, clean sand which is lacking in cohesion and particle interlocking usually shows low expansion, but also a relatively low bearing ratio (15 to 30 per cent at 0 1 in penetration) when tested without a surcharge as specified under 5, and unless confined by a heavy pavement, this class of material is subject to shifting under the action of moderate wheel loads and vibration To evaluate such materials for use in the foundation under thick layers of base and surface, penetration tests shall be made as specified under 5 and 7 except that a surcharge weight (Fig 33) (see Note 10) shall rest upon the specimen during the penetration test.

NOTE 10. This surcharge (Fig. 33) is equivalent to the weight of a 5 to 6-in thickness of pavement. When cohesionless material will be confined by thicker bases and pavement, the test may be made with heavier surcharges of 20, 30 or 40 lb The surcharge used shall be equal to or somewhat less than the weight of the pavement and base course proposed for use over the material being tested

APPENDIX B

DESCRIPTION OF ELECTRIC GAUGE EQUIPMENT FOR DETERMINING THE DEFLECTION OF PAVEMENTS

BY O J. PORTER AND J E BARTON *

The electric gauge equipment used by the California Division of Highways for the past four years was designed for measuring the deflection of rigid and flexible types of pavement under both dynamic and static wheel loads. The equipment offers a means of determining the action of pavements under service conditions including various wheel loads, speeds, tire footprints, and contact pressure.

The recording equipment is mounted in a station wagon and is intended primarily for use under field conditions Power is obtained from storage batteries. A portable gasoline engine-driven generator for charging the batteries is included, which provides for operation independent of any outside source of power The measuring unit is a part of a wheatstone bridge circuit The deflection is recorded at some distance from the point under test, and thus measurements may be made directly under the center of a wheel load

The equipment will measure deflections up to 0 5 in. and when set for the most sensitive range will measure movements up to 0 1 in to an accuracy of 1 per cent or 0 001 in An electronic amplifier is used when greater sensitivity is required, and with this additional equipment measurements have been made to an accuracy of 0 0001 in for a maximum deflection of less

*Senior Physical Testing Engineer and Asst. Physical Testing Engineer, respectively, Materials and Research Department, California Division of Highways

than 0 01 in The equipment is capable of measuring and recording any transient deflection which exists for 0 001 sec

The electric gauge equipment consists essentially of a gauge head (see Fig 34) which is connected with a pair of wires to the controls and recording equipment The gauge head is designed for mounting in a standard 3-in pipe coupling, which is grouted into the pavement at any point where deflection measurements are desired. The gauge heads may be readily moved from one coupling to another, allowing one unit to be used at any number of locations

In a recent airport runway test the pipe couplings with a short length of nipple were grouted into a hole in the surface of the old taxiway

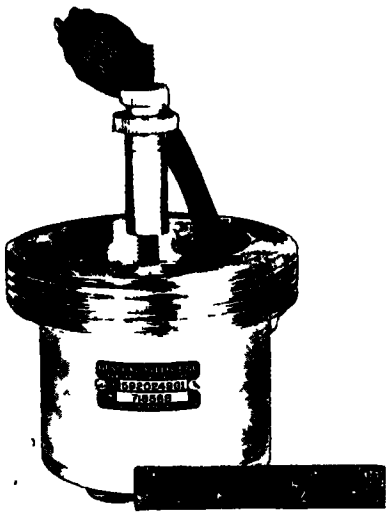


Figure 34. Electric Gauge Head

under the runway test section After the test ramp was completed, pipe couplings were also placed in the new pavement

A reference rod was driven into the material below each coupling. A hole was first made through the center of each coupling with a sampler down to 5 ft below the old taxiway surface. A casing consisting of 1-in thinwall conduit was next pushed to the bottom of the hole The reference rod consisting of a length of ½-in pipe, topped with a 1-in by ¾-in bushing and a ¾-in brass cap, was then placed through the casing and driven into the clay subsoil to a depth of 6 ft to 7 ft below the surface of the old taxiway.

Five gauge heads were permanently installed in the couplings in the old taxiway surface to measure the deflection of the bottom of the base. The lead wires were extended outside of the area of the runway test section, which permitted normal construction operations to be carried on over the units Another gauge head was used to measure the deflection of the top of the pavement and it was moved from one coupling to another as the tests were made

The special electric gauge equipment was constructed by the General Electric Company to comply with design requirements furnished by the Materials and Research Department of the California Division of Highways and includes the following units

1 General Electric gauge heads.
2 General Electric balancing units
3 General Electric power units
4 General Electric power supply unit
5 General Electric P M oscillograph
6 General Electric photo-electric relay and light source
7 General Radio 50-cycle tuning fork.
8 Connecting wires, batteries and gasoline-engine-driven generator.

Electric Gauge Heads

Figure 34, is a photograph of one of the electric gauge heads Figure 41 shows a cross section of the gauge head It is designed to mount in a standard 3-in pipe coupling The pipe coupling is fastened to the pavement at the location at which it is desired to measure the vertical movement

The gauge head consists essentially of a brass mounting unit containing a special iron-clad solenoid coil with a hole through its center and a short iron plunger mounted on a threaded brass rod The solenoid is rigidly attached to the pavement through its mounting unit and a 3-in pipe coupling, and the plunger is attached to the reference rod with a threaded brass rod The relative motion of the solenoid coil in respect to the iron plunger (attached to the reference rod) produces a deflection of the oscillograph.

Balancing Unit

An iron-clad coil and iron plunger, having the same electrical characteristics as the gauge head, are mounted as a unit Provision is made for adjusting the position of the iron plunger and locking it in position Six individual balancing units are mounted in one case as shown on Figure 35

Power Unit

The power unit contains the electric circuit for the gauges This equipment (see schematic wiring diagram, Fig 36) contains the differen-

tial transformer (ratio coils) of the wheatstone bridge circuit, sensitivity control, filter system, rectifier, switches and terminals for connecting the various parts of the circuit The cases containing the six individual balancing units (Fig 35) and the six individual power units are of the same length so they may be mounted side by side and interconnected by means of links

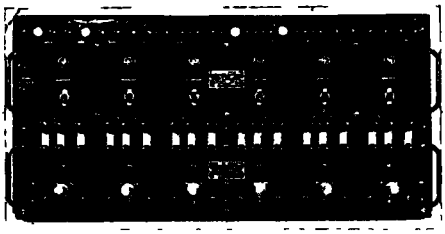


Figure 35. Electric Gauge Power Unit and Balancing Unit

Power Supply Unit

The power supply unit (Fig 37) consists of a 12-volt direct current, 2000-cycle, 110-volt alternating current motor generator set, with an output transformer to reduce the output to 30 volts The panel board includes meters and rheostats for measuring and controlling the output frequency and voltage A meter is also provided to measure the direct current input voltage.

The output of the power supply unit is held

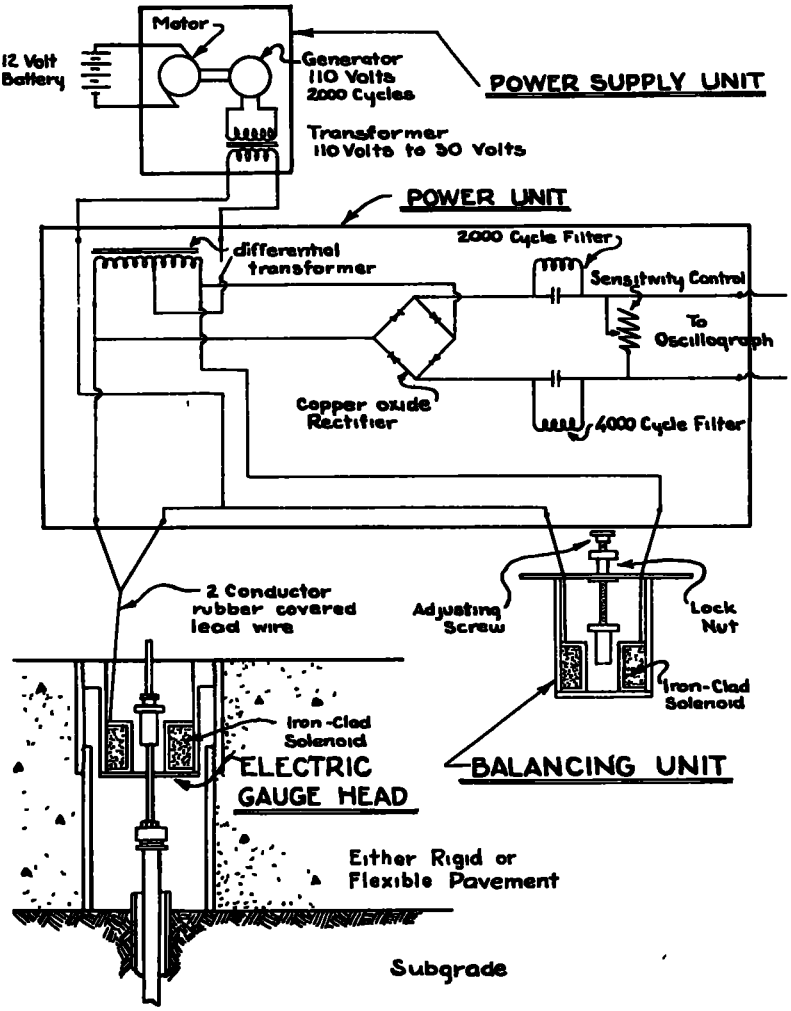


Figure 36. Schematic Wiring Diagram of Electric Gauge Circuit

at exactly 2000 cycles and 30 volts for all calibrations and measurements with the electric gauges.

P.M. 14 Oscillograph

The oscillograph (Fig 38) is a standard magnetic type fitted with four supersensitive galvanometers and a continuous-drive film holder. The supersensitive galvanometers are of the light beam type, having a frequency response up to 500 cycles per second, without a correction factor, and a current sensitivity of approximately 0.17 milliampere per millimeter deflection of the light beam as recorded on the oscillograph film. The galvanometers are also fitted with stationary adjustable zero mirrors. These mirrors record as straight lines on the oscillograph film for use as reference or zero lines.

The optical system is constructed so that a beam of light from a 50-C.P. automobile lamp is reflected by a mirror, through an adjustable orifice, to the mirror on the galvanometer and then back to a cylindrical lens which focuses the beam as a spot at the front of the oscillograph. The front of the oscillograph is designed for mounting either a ground glass calibrating screen or a continuous-drive film holder. The continuous-drive film holder is designed for films up to 6 in. wide and 15 ft. long. The film is driven at a uniform speed, by means of an adjustable speed electric motor, and records a continuous picture of the position of the light beams as shown on Figures 43 and 44.

The characteristics of the electric gauge and recording equipment are such that transient movements lasting up to 0.001 sec. may be accurately measured.

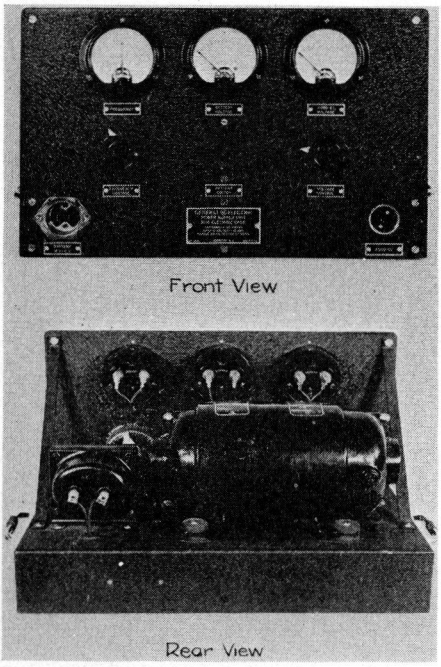


Figure 37. Power Supply Unit for Electric Gauge

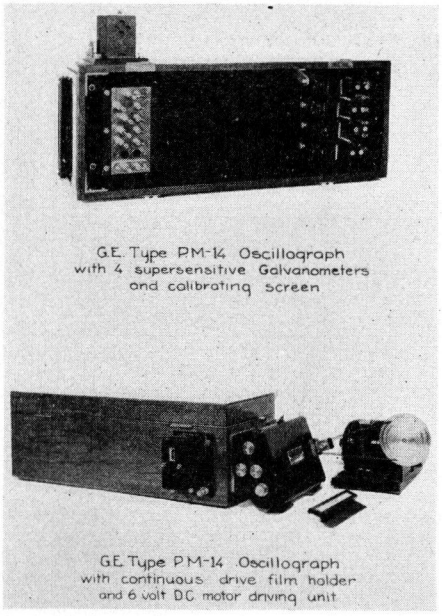


Figure 38.

Photo-electric Relay

The photoelectric relay (Fig. 39) consists essentially of a photo-electric cell, condensing lens, and a vacuum tube amplifier mounted in a metal container. Leads are brought out to facilitate connecting with the oscillograph and batteries. The output of the vacuum tube amplifier is connected directly to one of the oscillograph galvanometers. The batteries furnish power to operate the photo-electric cell and vacuum tube amplifier.

50-cycle Tuning Fork

A 50-cycle, electrically maintained, tuning fork is used to record a 50-cycle time line on the oscillograph film. The tuning fork is equipped with a micro-phone button and an

impedance matching transformer for connecting directly to a galvanometer in the oscillograph

Batteries and a Gasoline-Driven Generator

One 12-volt and two 6-volt storage batteries are required to operate the equipment The

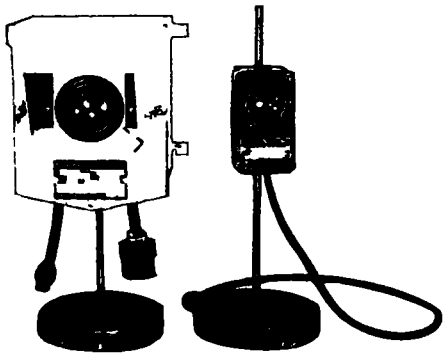


Figure 39. Photo Electric Relay and Light Source

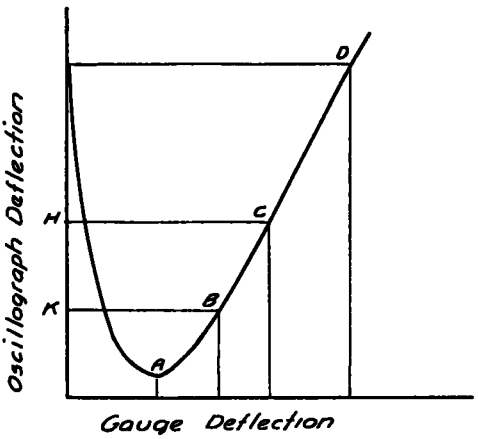


Figure 40. Typical Oscillograph Gauge Deflection Curve

12-volt battery supplies the input current for operating the power supply unit and the 12-volt, 30-C P. lamp in the oscillograph One 6-volt battery supplies power for operating the motor for the continuous-drive film holder. The second is used for lighting the 6-volt 50-C P globe used in connecting with the photo-electric relay

The combined power drawn from the 12-volt battery is about 15 amperes A gasoline-driven generator is floated across the battery to supply most of this current and to keep the batteries charged. It is also used to furnish lights for night operations. The 6-volt batteries are only used for short intervals of time during actual recording and, therefore require infrequent charging.

Dry batteries are supplied for the 50-cycle tuning work circuit and for the photo-electric relay.

CALIBRATION

The component parts of the electric gauge circuit are connected as shown in Figure 36 The gauge head is connected to the power unit by the length of wire that is to be used in actual field measurements The gauge head is mounted in the calibrating device as shown on Figure 42

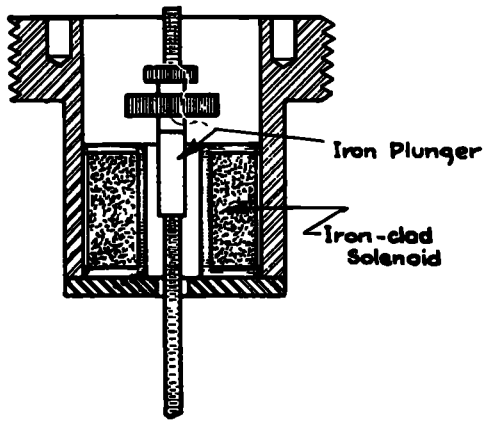


Figure 41. Cross-Section Electric Gauge Head

A typical curve of oscillograph deflection plotted against gauge deflection is shown in Figure 40. The righthand slope of this curve is a straight line over a considerable range of gauge deflection The deflection of the gauge is controlled to operate over this portion of the curve. The slope of the curve is determined by the setting of the sensitivity control on the power unit panel.

The iron plunger in the gauge head is set to a predetermined point which will cause the gauge to operate over the desired portion of the calibration curve. The position of the balancing unit is selected by switching on the power and adjusting the iron plunger for the desired deflection of the oscillograph at zero gauge reading The adjustment on the balancing unit is locked in position. The calibrating device Fig. 42 is then moved by increments and a record made of the corresponding oscillograph deflections

The six electric gauge heads were each cali-

brated, using the power unit and corresponding balancing unit and a specific oscillograph galvanometer. The oscillograph contains four galvanometers Three galvanometers may be used for recording three electric gauge units at one time and the fourth galvanometer used for recording the time line and the output of the photo-electric cell

The six electric gauge heads were then calibrated, using unit number 1 of the power units

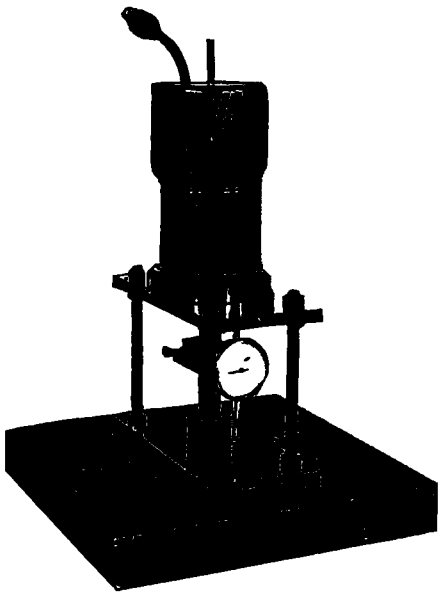


Figure 42. Calibrating Device for Electric Gauge Heads

and its corresponding balancing unit with the sensitivity control set so that a deflection of the electric gauge head of 0 1 in caused a 5-cm deflection of the oscillograph galvanometer The same gauge units were next calibrated, using unit number 2 with the sensitivity control set so that a deflection of the electric gauge head of 0 2 inch caused a 5-cm deflection of the oscillograph galvanometer and then calibrated, using power unit number 3 with the sensitivity control set so that a deflection of the electric gauge heads of 0 4 in. caused a 5-cm deflection of the oscillograph galvanometer

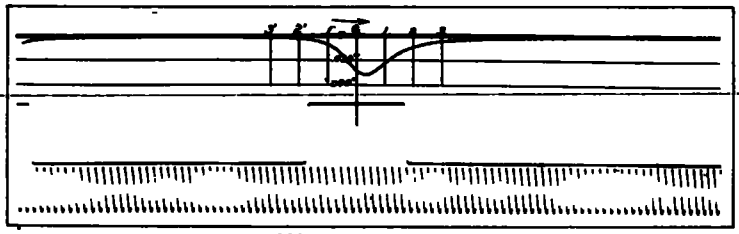
With three ranges, it is possible to secure large deflections of the oscillograph galvanometer for small pavement deflections and still measure greater pavement deflections when necessary. The three ranges also provide for the measurement of large to small deflections without recalibrating the gauge heads in the field

When only one electric gauge unit is measured at a time, the other three galvanometers in the oscillograph are used for recording the time line and the output from two photo-electric cells which are set to determine the position of the wheel with respect to the point under test.

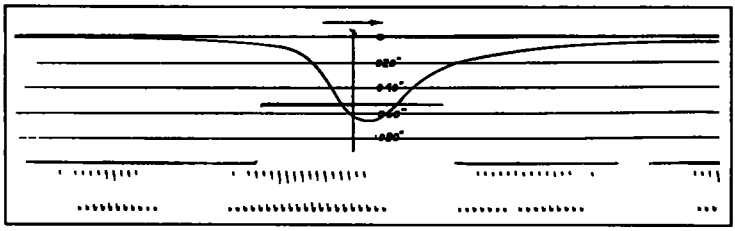
Typical Oscillograph Records

Figures 43 and 44 show typical oscillograph films obtained for various wheel loads on a western runway test section. The upper line on each film of Figure 43 shows the deflection of the top of the pavement as recorded by one galvanometer for a wheel load moving at approximately ten miles per hour. The 50-cycle time line shown on the bottom of each film was recorded by a second galvanometer. The other one or two lines (see 53,300-lb wheel load, Fig. 43) record the output of the photoelectric cells and mark the position of the wheel in respect to the point under test Where two lines are shown the photo-electric cells were spaced 3 ft apart with one cell placed so that the light beam was shining directly across the point under test The light beam, shining on the photo-electric cell, was cut by the tire for an appreciable time while approaching and leaving the point under test The center of jump in the line occurs at the time the tire is directly over the deflection point.

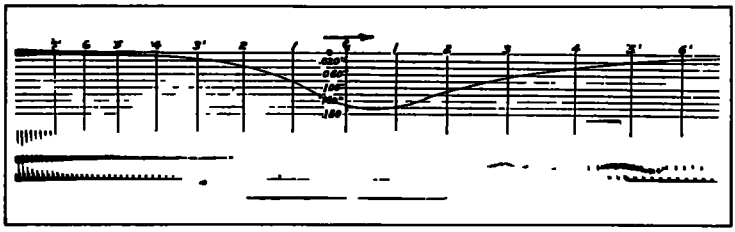
Figure 44 is similar to Figure 43 except that the measurements were made of the movement at the bottom of the base at the point where the total thickness of pavement plus base was 32 in Comparisons may be made of the deflection of the top of the pavement and the bottom of the base from the data shown on these two figures (43 and 44) after making due allowances for the small difference (2 in) in the total thickness of the pavement and base



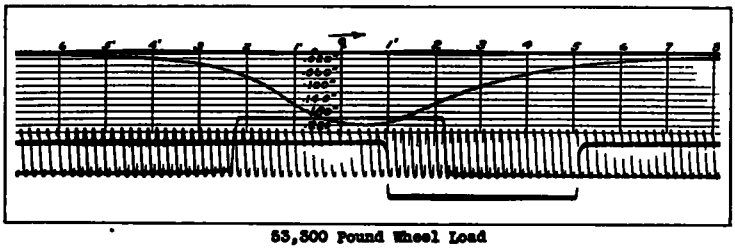
8000 Pound Wheel Load



10,000 Pound Wheel Load



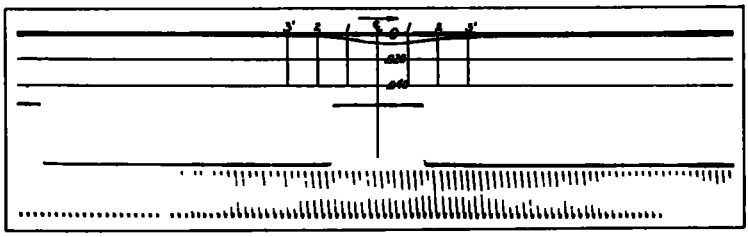
40,000 Pound Wheel Load



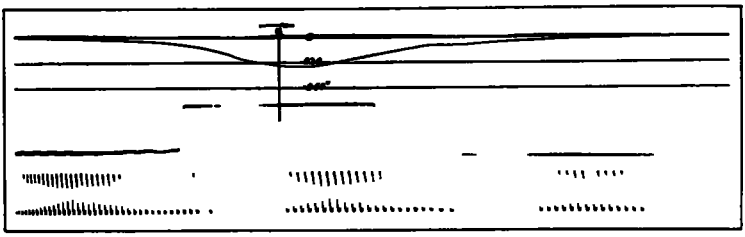
53,300 Pound Wheel Load

Vertical lines superimposed on film indicate one foot increments of wheel travel.
Horizontal lines superimposed on film indicates each 0.02" deflection.

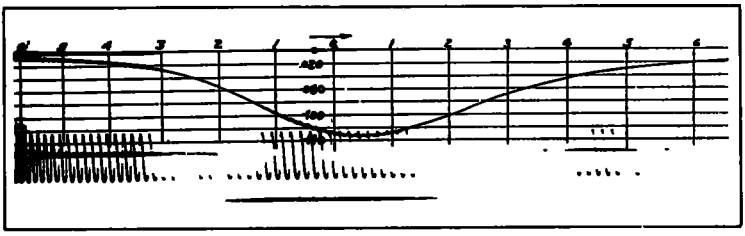
Figure 43. Typical Oscillograms Showing Deflection of Top of Pavement 30" Thickness Surface and Base



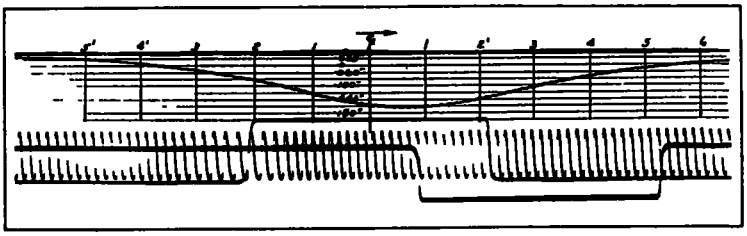
5000 Pound Wheel Load



10,000 Pound Wheel Load



40,000 Pound Wheel Load



53,300 Pound Wheel Load

Vertical lines superimposed on film indicate one foot increments of wheel travel.
Horizontal lines superimposed on film indicates each 0.02" deflection.

Figure 44. Typical Oscillograms Showing Deflection of Bottom of Base at 32" Thickness Surface and Base

DISCUSSION ON FOUNDATIONS FOR FLEXIBLE PAVEMENTS

MR. H. G. NEVITT, *Socony-Vacuum Oil Company*· The California flexible pavement design method, based upon the findings of the Porter soil test, makes no distinction in the thickness of specified base for materials of different characteristics, except for some minor qualifications as to minimum thickness of the top layer, which must be selected material. No credit is given for any additional strength imparted by the bituminous surfacing, nor is any distinction made between thick foundation courses of superior material and those of less load bearing and distributing power.

There are two phases of this situation. The first is the effect of the strength imparted by higher type materials in the base. Obviously granular aggregates of good interlock and high stability will serve to dissipate the load intensity more rapidly, and therefore should be required in less thickness, than would be the case with a relatively thin layer of high stability material below which are placed much less favorable aggregates or soils to comprise the total mat thickness specified by the formula. Since this has been amply commented on elsewhere, no further remarks on this point seem necessary.

The other phase of this failure to evaluate the characteristics of the base constituents is the effect of rigidity. An observation of the interesting pictures presented by Mr. Porter leads at once to the thought that the failure may be just as much due to the inability of the thin pavement to withstand the distortion incidental to the necessary deflection of the base under these heavy loads as it is to failure in this base. In other words, a base might be sufficiently strong to withstand the load, and yet distort too much for the pavement to stand up (with perhaps later actual failure of the base after the aiding power of the pavement to help support the load had been lost) or it might be a direct base failure which would be little affected by increasing the pavement thickness. I am inclined to think that it may have been the former, and that certainly there are many cases in which the failure may be attributed more correctly to an insufficient bituminous surfacing thickness than to a failure of the base alone. This is borne out by past experiences in certain areas where unusual thicknesses of base were resorted to in test sections in order to permit the construction of a minimum thickness bituminous surface. The result was almost inevitably failure despite the base thickness supplied, due to this lack of balance between the structural characteristics of the two supporting elements which are grouped together in the California method

It seems obvious that this matter of rigidity is an important factor in the design. It is quite possible that highway designs are essentially for strength, while airport designs with these extremely heavy loads are for rigidity. A necessary corollary would be that highway design experience could not be properly extrapolated to cover these airport conditions, a conclusion for which there seems to be considerable justification in practice.

A qualitative analysis of this situation quickly brings out the necessity for ample bituminous surfacing thickness, with the resultant addition of beam strength which this provides. If there is no beam effect whatever, the load concentration, as it is carried down into the base and subgrade, will only spread at the angle of dispersion provided by the soil characteristics. It is evident that this may involve many feet of depth with these extremely heavy loads, and consequently elastic deflections of considerable magnitude. If however there is an ample thickness of surfacing of such type that it provides some beam action, although still able to adjust itself to slow changes in the base, the situation is greatly

improved. The heavy stress concentrations are much more widely distributed by the bituminous mat, so that the intensity of stress on the base below it, and hence the necessary deflection to bring about its reaction, is much less This means that the applied load will be dissipated over a wider area and descend to a much less depth. Obviously the total deflection produced will be less, the pavement strains will be correspondingly decreased, and there is much less likelihood of failure

It seems evident that instead of ignoring the thickness and characteristics of the bituminous surface, as in the California design, it should be recognized as playing an important part in the support of these heavy loads. Experience with highway loadings where this factor is less critical, and under bituminous surfaces (such as those constructed with SC materials) which have little beam strength, must not be given too much consideration. In brief, an effective beam effect, although not that requiring a rigid surface, seems essential. The fact that surprisingly low foundation courses are called for in the Army designs for these heavy loads with thin rigid slabs seems to be an immediate confirmation of this point, although it is one which seems obvious upon careful consideration.

MR. A C. BENKELMAN, *Public Roads Administration*· In replotting the data presented in Figure 26 of Mr Porter's report, in the manner shown in Figure 1, a clear picture is had of the importance of the magnitude of the deflection upon the performance of the test pavement. As pointed out by Mr. Porter, the deflection values represented by curve A for different thicknesses of pavement were those recorded under each passage of the 40,000-lb. wheel load before the pavement exhibited marked signs of distress. Although, as stated, some permanent displacement resulted under each passage of the load, the measured deflection values were reasonably constant up to the time when the pavement surface showed definite indications of impending failure

According to these data, an increase in the pavement thickness from about 24 to 50 in. resulted in a reduction of the pavement deflection from 0 20 to 0 10 in The corresponding increase in resistance of the pavement is reflected by the number of repetitions of the 40,000-lb wheel load required to produce roughening of the surface, *i e.,* 20 as compared to about 50,000 repetitions (curve B).

While these data are applicable only to the one set of conditions as described by Mr. Porter they do suggest the importance of placing a reasonable limit upon the magnitude of the deflection that is to be used for design purposes. Those responsible for the conduct of the investigation are to be commended for this significant contribution to the problem of design of flexible type runways.

MR W. H. CAMPEN, *Omaha Testing Laboratories*. Mr. Porter has done an excellent piece of work in developing a formula for the designing of flexible pavements in California with the materials at hand. I think we will have to agree that he has solved his flexible pavement problem by using certain materials in a certain manner However, in arriving at his conclusions and in describing his method he uses certain tests and makes certain assertions which are questionable in my opinion As an advocate of the load bearing method of designing flexible pavements, I wish to discuss several points in Mr. Porter's paper and thereby hope to add the results of my observations to this complicated subject. Parts of my discussion of Mr. Middlebrook's paper [1] apply to Mr. Porter's paper as well I will repeat as little as possible and for this reason those interested are urged to read both discussions

[1] See page 173.

Measuring Natural Subgrade Strength

Mr. Porter emphasizes the fact that subgrade support is one of the major factors to be considered in determining the required thicknesses of compacted soils to be used as subgrade reinforcement and comparing them to the results obtained on a sample of crushed limestone. I am of the opinion that the complicated subgrade reactions cannot be measured accurately by testing the upper 8 in. of it in molds in the laboratory. If the sub-

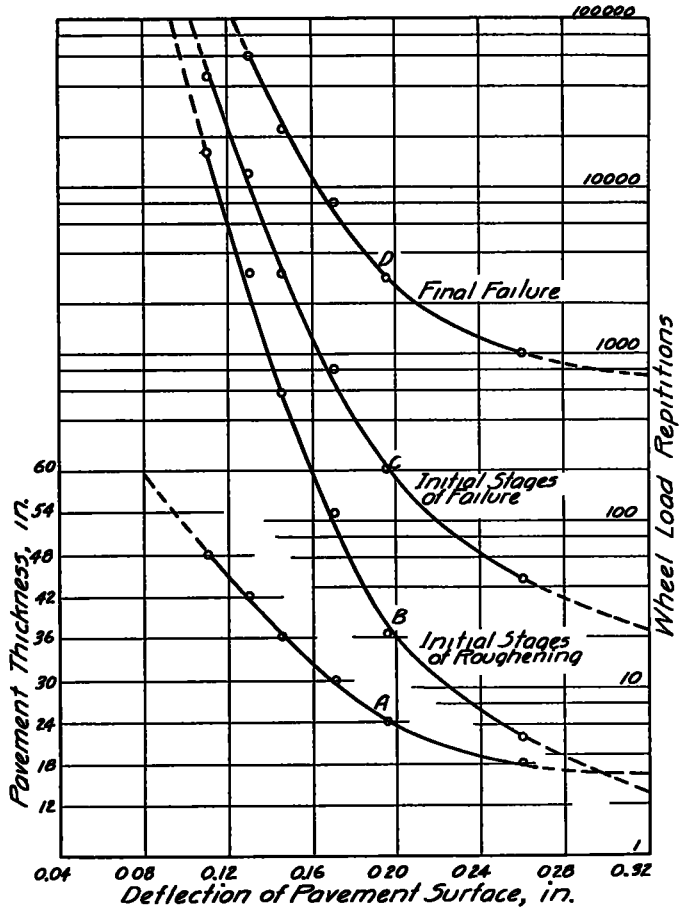


Figure 1. Runway Investigation, 40,000-Lb. Moving Wheel Load. Data from "Foundations of Flexible Pavements" by O. J. Porter

of compacted soil aggregate mixtures or crushed rock to be used as subbases or bases. I agree with this statement but do not agree on the method of determining the subgrade strength. Mr. Porter determines the strength of the subgrade by means of a laboratory penetration test and expresses the results as a bearing ratio by

grade is of uniform composition and consistency for a depth of at least 4 ft. his test may have some value but on subgrades which vary continually from the top down, the test has little value I have come to the conclusion, after having tried this very method, that Mr. Porter now proposes, that the only real way to evaluate a sub-

grade is to test it in place under the worst anticipated conditions.

Method of Saturating Subgrades, Subbases, and Bases

In attempting to determine the effect of water on natural subgrades, we cover the top of the subgrade with sufficient sand to represent the weight of the courses to be superimposed, then add sufficient water to fill the voids in the sand and to maintain this condition for four days. This represents the worst possible condition. The top of the subgrade is tested before and after this water treatment.

In testing compacted mixtures for subbases and bases, we submit samples to absorption by capillarity as described in our method. We believe that this represents field conditions more closely than does Mr. Porter's method. The thing that we wish to bring out as being important in this connection, and with this phase of the discussion, is the fact that properly selected and compacted soils or soil-aggregate mixtures will not take up any appreciable amount of water when submitted to capillary water. We therefore cannot agree with Mr. Porter that all soil mixtures will either soften or quicken as a result of absorbed water plus traffic. It is true that compacted plastic mixtures will deform permanently, if enough load is placed upon them but this can be controlled by decreasing the optimum water content at maximum density or by limiting the load intensity. The first can be accomplished by increasing the compactive effort and the latter by increasing the thicknesses of stronger superimposed layers.

To show that all compacted soil mixtures will not take up more water than their optimum requirement, I submit some tests made at the Omaha, Nebraska, airport. The base mixture on this project consists of 20 per cent soil binder and 80 per cent graded $\frac{3}{8}$-in. sand-gravel. Its optimum water content is about 7 per cent. The mixture was compacted to maximum density at optimum water and was laid in 1936. A number of tests made in 1937, 1940 and 1941 show a maximum water content of 7.1 per cent and the average of all tests is about 5 per cent. The runways are located on a former Missouri River bed and the water level comes to within 3 ft. of the surface when the river is high.

Densification of Subbases and Bases

Mr. Porter states that he finds no significant relationship between dry weight per cubic foot and strength of compacted mixtures. As we have said elsewhere, it is not the dry weight that is the determining factor but rather the air void content at maximum density and optimum water. The percentage of air voids determines the amount of water which the mixture can take up and the strength is a function of the water content The density and composition of a mixture may also affect the strength but this effect is insignificant compared to the effect of water. I wish to emphasize that unless the water holding capacity of a plastic soil mixture is limited, this mixture is not stable and its strength behavior cannot be anticipated.

Mr. Porter does not include any air void content figures in his paper, nor does he show the percentages of maximum density to which the soils used by him were densified. Apparently in the selecting, designing, and compacting of subbase and base mixtures, he did not use the air void content at optimum water and maximum density as a guide.

Deflections and Load Repetitions:

Mr. Porter has shown, (1) that dynamic loads cause less deflections than static loads of the same magnitude, (2) that loads applied through tires produce more deflections than equivalent rigid circular plates, (3) that dynamic loads which produce deflections in excess of 0.05 in. will cause rapid failure. This is much

needed information and we are indebted to Mr. Porter for obtaining it. Judging from these results, load bearing tests should be made with tires or tire-shaped plates and the strength of the finished pavement should be such as to prevent the development of permanent deflections greater than 0.05 in. I wonder though if we should not continue to use circular plates and design for low permanent deflections, until more data are available on the Porter method. We have had no difficulty in designing and constructing runways, which show no or very little permanent deflection, when tested with circular plates to a total deflection of 0.25 in. or more.

In conclusion I would like to present some data on the construction and performance of the runways on the Omaha Airport with the thought in mind of checking Mr Porter's curves The natural subgrade on this project varies greatly but much of it is an A-7 soil and has a bearing ratio of from 3.3 to 3.8 at 0.125-in deflection. At 0.10 in. it would be even less. On the average 12 in of well compacted A-5 soil and 6 in. of the base mix discussed under "Method of Saturating Subgrades, Subbases and Bases" was constructed above the subgrade. Either 3 in. or 2 in of asphaltic concrete was placed above the base. Some of these runways have been in use since 1936 and until about a year and a half ago handled the mainliner plane, having 13,500-lb. wheel loads For about a year and a half now these runways have been handling the Flying Fortress and Liberator planes, having about 25,000-lb wheel loads. The runways are in fine condition, showing no longitudinal or transverse cracks and very little cracking of any kind. According to Porter's curves the Omaha runways should be about 32 in. thick, whereas they are only 20 or 21 in

MR PORTER: Referring to Mr. Nevitt's discussion, the failures shown in Figure 6 occurred due to displacement of the subgrade soil and not instability of either the pavement or base. As previously stated, measurements showed that approximately the same amount of deflection occurred at both the bottom of the base and the top of the pavement. If failure had occurred in either the pavement or base, the deflection at the top of the pavement would have been very much larger than at the bottom of the base. Furthermore, the failures would not have developed at a uniform and progressive rate in relation to the thickness of base and pavement as shown in Figure 26. No failures occurred where the pavement and base exceeded 40 in. total thickness. This fact conclusively demonstrates that both the pavement and base were sufficiently stable to withstand the stress transmitted by the wheel load.

After the traffic tests were completed, the pavement and base were removed Measurements were then made to determine the amount of subgrade deformation beneath the failed areas. These measurements also clearly indicated that the failures were due to displacement of the subgrade and not to internal displacement of either the pavement or the base course

California experience has not been based on the performance of thin bituminous pavements as inferred by Mr Nevitt. Up to a few years ago, most of the asphalt concrete pavements constructed in this State ranged from 6 to 9 in. in thickness. The failures shown in Figure 1 developed on a highway where the asphalt concrete pavement ranged from 7 in. in thickness at the center to 9 in. at the edge. Pavement test sections constructed in California and tested to destruction under traffic have included bituminous pavements and bituminous treated bases ranging to 20 in. total thickness.

Although much additional information should be obtained on the influence of pavement thickness, the data now at hand

indicate that the total thickness of the pavement and base usually cannot be economically reduced by constructing asphalt concrete pavements more than 5 to 8 in thick. Heavier (thicker) pavements, however, reduce the stresses transmitted to the base, thus permitting the use of somewhat lower quality base materials.

I agree with Mr Nevitt that highway experience cannot be applied to airport runway design without evaluating and correctly compensating for the different traffic conditions Traffic on a heavy truck highway is more severe than on an airfield runway in regard to contact pressure and number of repetitions. Both of these factors require a somewhat higher quality pavement and a greater total thickness of base and pavement than that necessary to support a wheel load of like intensity under airplane runway usage The added effect on the subgrade of motor vibration of airplanes is more severe, however, than that of trucks moving rapidly over a highway Airplane wheel loads are very much larger and are spread over a greater area and, therefore, affect the soil to a much greater depth than the heaviest of truck wheel loads now in general use on highways.

The extensive investigations conducted included a study of the effect of the following factors on the pavement, base and subgrade·

1. The wheel load intensity
2. Impact
3 Motor vibration
4 Tire contact pressure
5 Wheel load repetitions

The complete results of these studies are too extensive for inclusion in this report An attempt has, of course, been made to evaluate all of these factors from the data now available in adopting tentative design standards for foundations for flexible pavements on airfields.

The points raised by Mr. Campen are discussed in the order of their sequence.

Measuring Natural Subgrade Strength

Mr. Campen is under the erroneous impression that the bearing ratio is only measured on the upper 8 in of the subgrade, whereas the method actually requires that undisturbed samples from the subgrade and subsoil be tested to the depth affected by the wheel loads In the case of non-uniform soil conditions, the bearing ratio of each layer must be determined and then the design prepared, starting from the bottom, to determine the total thickness of reinforcement (compacted subgrade, subbase, base and pavement) required over each material

Method of Saturating Subgrades, Subbases, and Bases

The writer did not state "that all soil mixtures will either soften or 'quicken' as a result of absorbed water plus traffic," but that "some soils soften and others become 'quick' when subjected to repeated applications of moving wheel loads." Thoroughly compacted grades of granular material, showing high bearing ratios and low expansion values do not soften enough to become unstable Observations indicate that such materials will not become "quick" and displace under the action of heavy airplane wheel loads and the influence of motor vibrations.

Densification of Subbases and Bases

The writer agrees that where practicable subgrades should be compacted to maximum density and minimum air void state Other things being equal (surcharge, density, etc.), minimum expansion will occur subsequent to construction, thus insuring minimum ultimate water content and maximum stability, if the subgrade is constructed at minimum air void state. The use of optimum or slightly more than optimum moisture content and compaction of subgrades to maximum density has been standard practice in California since 1929. (See test procedure submitted

November, 1930, to the American Road Builders' Association, page 6, "Grading Methods and Grading Equipment," Bulletin No. 17, American Road Builders' Association, 1931, also "Compaction of Embankments" by T E. Stanton, *Proceedings*, Highway Research Board, Vol 18, Part II, 1938, page 151)

Deflections and Load Repetition

Static load tests on rigid plates with areas and contact pressures equal to that of each of the wheel loads were made on a runway test section. If Mr. Campen's criteria for design, namely, 0.25-in deflection were used, the total thickness of base and pavement required would only be about 12 in. for the 25,000-lb. wheel load (see Fig. 25) and 18 in. for the 40,000-lb. wheel load. Under the traffic test, these thicknesses failed in less than 200 trips which clearly indicates that a deflection of 0 25 in. is excessive. As previously stated "the pavement cracked and failed to the 24-in thickness of pavement plus base . . . . under 5,772 trips of the 25,000-lb. wheel load." By referring to Figure 25, it will be noted that the 25,000-lb. load on the rigid plate corresponding to the footprint of the tire caused a deflection of about 0 13 in. Even this deflection is too great and the thickness of the base and pavement too thin to support traffic for 10,000 to 20,000 repetitions The complete results of the traffic tests indicate that the deflection should not be more than about 0 1 in for this wheel load. The data for the 40,000-lb wheel load (Fig. 26) also show that the pavement failed rapidly under traffic where the deflection greatly exceeded 0.1 in.

Mr Campen shows a bearing ratio of approximately 3 per cent for the A-7 subgrade soil which he states is satisfactorily supporting 25,000-lb wheel loads on the Omaha Airport under 20 to 21 in total thickness of select soil, base and pavement No information was furnished, however, regarding the number of repetitions of the 25,000-lb wheel load The bearing ratio results given are identical with those obtained on A-7 adobe soil where traffic tests conclusively showed that a 20-in. thickness of base and pavement would only support a 25,000-lb wheel load for a few thousand repetitions Although a 15-in. thickness supported this wheel load for a few hundred repetitions, the traffic tests indicated that approximately a 30-in. thickness would be required to adequately support the load for approximately 10,000 repetitions. The data furnished by Mr Campen do not indicate whether or not the test was made on an undisturbed sample of the subgrade from beneath the runway or on a disturbed specimen after being soaked under a surcharge equivalent to the weight of base and pavement.