A COMPARISON OF THICKNESS DESIGN METHODS FOR FLEXIBLE HIGHWAY PAVEMENTS

BY

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PREFACE

The problem considered here is to determine which method or combination of methods seems to provide a solution with the least assumptions to the problem of thickness design for flexible pavements. The consideration given in each of the principal methods to each of the factors that affects the required thickness of flexible pavements will be examined and classified so that in making a choice of a design method it may be known what properties of the materials are measured, how the measured properties are related to thickness required, what assumptions are necessary bases for the method, and to what extent the solution depends on the judgment of the design engineer. Current research in pavement design is examined for means of obtaining a more reasonable solution.

The writer is indebted to many engineers who, during the past thirty-five years, have advanced the science of pavement design to its present development.

TABLE OF CONTENTS

LIST OF FIGURES

ABSTRACT

The required thickness of a flexible pavement is a function of (1) the traffic, (2) the strength of the subgrade, (3) the strength of the pavement components, and (4) the endurance limit of the surfacing.

To show how the principal methods of thickness design evaluate these factors a historical review is given of the development of the methods.

The thickness design procedure published in 1963 by the Asphalt Institute, with the subgrade evaluated by the CBR test, is the most universally adaptable and is founded on the most extensive correlations with road service.

Recent research is reviewed to formulate a suggested thickness design procedure. The procedure suggested uses a dynamic triaxial testing machine for subgrade evaluation, a stress analysis based on the elastic theory, the endurance limits suggested by Hveem, and a traffic analysis by equivalent wheel loads.

INTRODUCTION

Flexible pavement is a structure in which layers of granular materials are used to spread the wheel load sufficiently to reduce the unit pressure on the subgrade so that shear failure and plastic deformation are avoided and elastic deflections limited so that the strains produced in the surfacing are within the endurance limit.

Thickness design is the evaluation of loads and strength of materials, and the calculation of stresses and deflections to determine the most economical pavement adequate for the traffic.

Depth of frost penetration and highly expansive soils are outside the scope of this paper as is the problem of the design of bases and of surfaces, except to the extent that their properties affect the total thickness of the pavement. The problem is further narrowed by assuming, (1) that job specifications and proper supervision of construction will produce a reasonable degree of constructional excellence, (2) that the road design will provide adequate drainage of both surface and ground water, (3) that the effect of local factors are provided for, and (4) that the highway will receive adequate maintenance.

A design load should be assumed and equivalent load factors for lighter and heavier loads determined by equivalent damage to the pavement so traffic counts can be evaluated as number of repetitions of the design load.

Strengths of the subgrade and of the structural components of the pavement should be determined by static and dynamic tests with loads of the magnitude to which the materials will be subjected and with the materials in the probable worst field condition.

Calculation of stresses and deflections should be based on elastic theory using Boussinesq solution[1] (Fig. 1) if granular bases and Burmister two-layer solution[2] if stabilized bases are used, (Figs. 2 & 3).

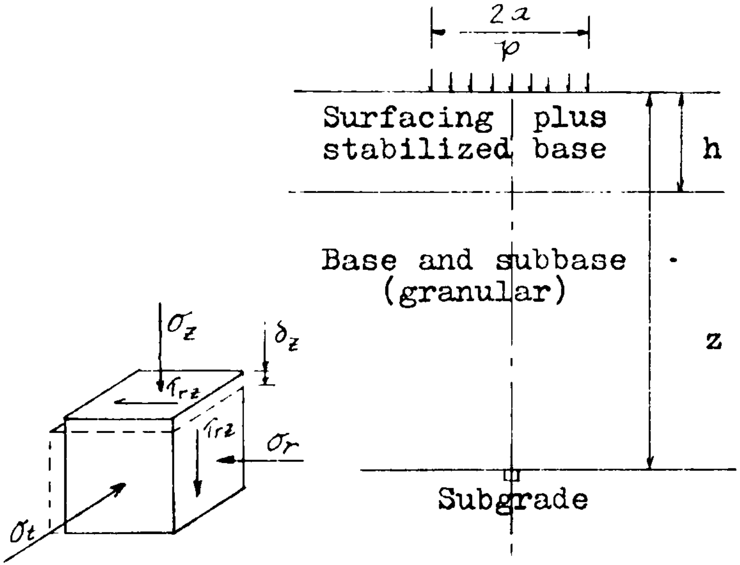

FIGURE 1. STRESSES ACTING ON AN ELEMENT

Boussinesq equations:

$$\Delta = \frac{p}{E}\left[(2-2\mu^2)(a^2+z^2)^{\frac{1}{2}} - \frac{(1+\mu)z^2}{(a^2+z^2)^{\frac{1}{2}}} + (\mu+2\mu^2-1)z\right]$$

$$\sigma_z = p\left[1 - \frac{z^3}{(a^2+z^2)^{\frac{3}{2}}}\right]$$

$$\sigma_r = \frac{p}{2}\left[1+2\mu - \frac{2(1+\mu)z}{(a^2+z^2)^{\frac{1}{2}}} + \frac{z^3}{(a^2+z^2)^{\frac{3}{2}}}\right]$$

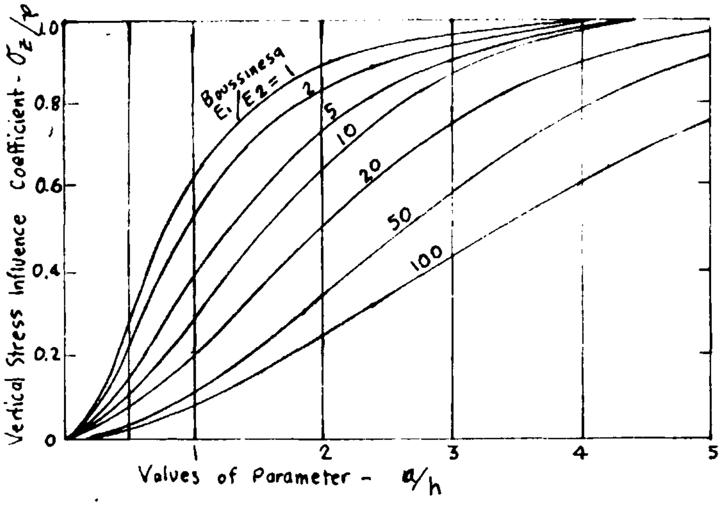

FIGURE 2. BASIC PATTERN OF TWO-LAYER VERTICAL STRESS AT INTERFACE $z = h$, $\mu_1 = \mu_2 = \frac{1}{2}$. From Burmister[3]

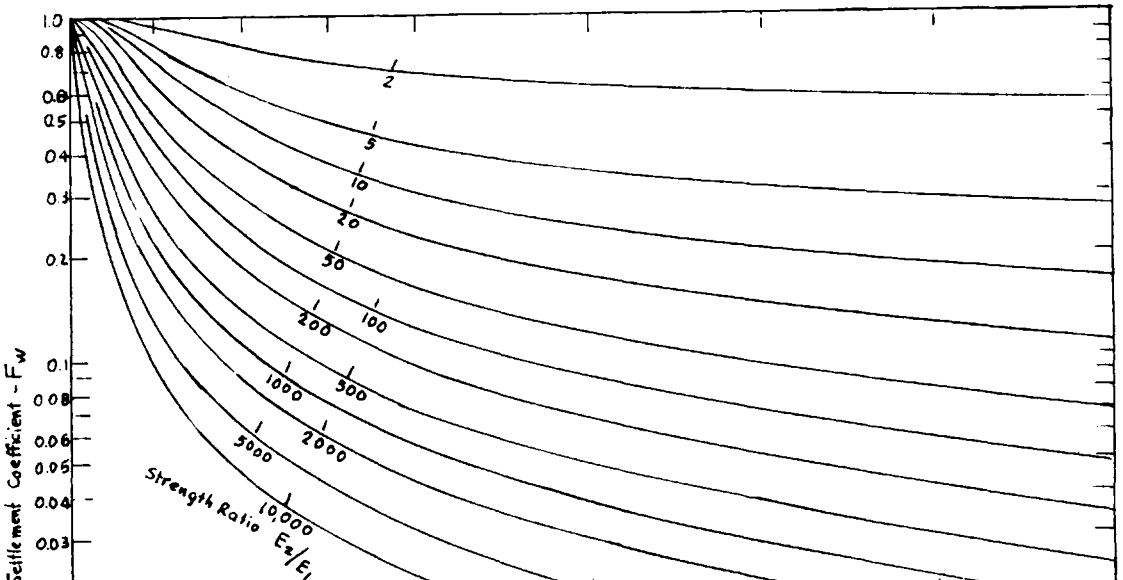

FIGURE 3. INFLUENCE CURVES OF THE SETTLEMENT COEFFICIENT From Burmister[3]

Deflection:

$$\Delta = 1.5\ (pa/E_2)\ F_w$$

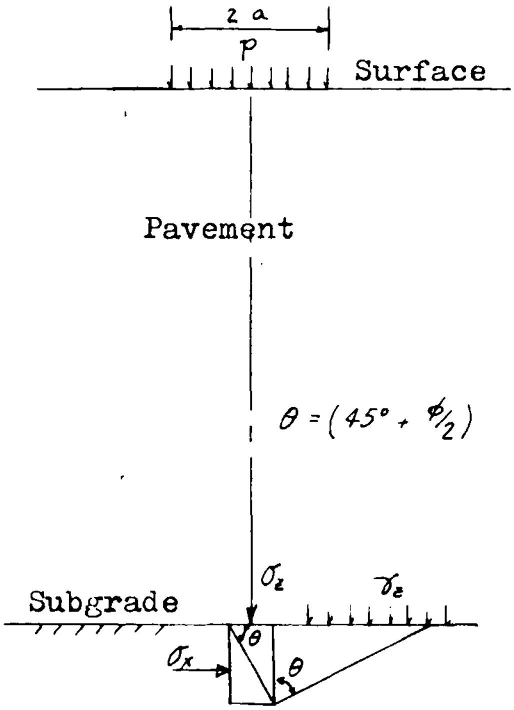

FIGURE 4. STRESSES ON AN ELEMENT OF SUBGRADE

From Tschebotarioff's expression[4] for the resistance to failure of an element of soil triaxially loaded:

$$\sigma_z = \sigma_x \tan^2\left(45° + \frac{\phi}{2}\right) + 2c\tan\left(45° + \frac{\phi}{2}\right)$$

And by assuming that σ_x is equal to the passive lateral pressure provided by a wedge of soil subjected to a vertical load equal to the overlying weight of the pavement, Hewitt[5] developed an equation for thickness design based on shear strength of the subgrade.(Fig. 4):

$$\gamma z - p\left[1-\left(\frac{1}{1+\left(\frac{a}{z}\right)^2}\right)^{3/2}\right] = \gamma z\left[\frac{1+\sin\phi}{1-\sin\phi}\right]^2 + \frac{4c}{1-\sin\phi}\left[\frac{1+\sin\phi}{1-\sin\phi}\right]^{\frac{1}{2}}$$

where c is cohesion and ϕ is angle of internal friction determined from Mohr diagram of test data obtained from the triaxial test of the subgrade material.

Asphalt concrete is subject to fatigue so service life is a function of the magnitude of surface deflections which must be controlled to keep the strains within the endurance limit for the design traffic. Monismith[6] and Saal and Pell[7] have shown fatigue to be a function of strain and repetitions of load, (Fig. 5).

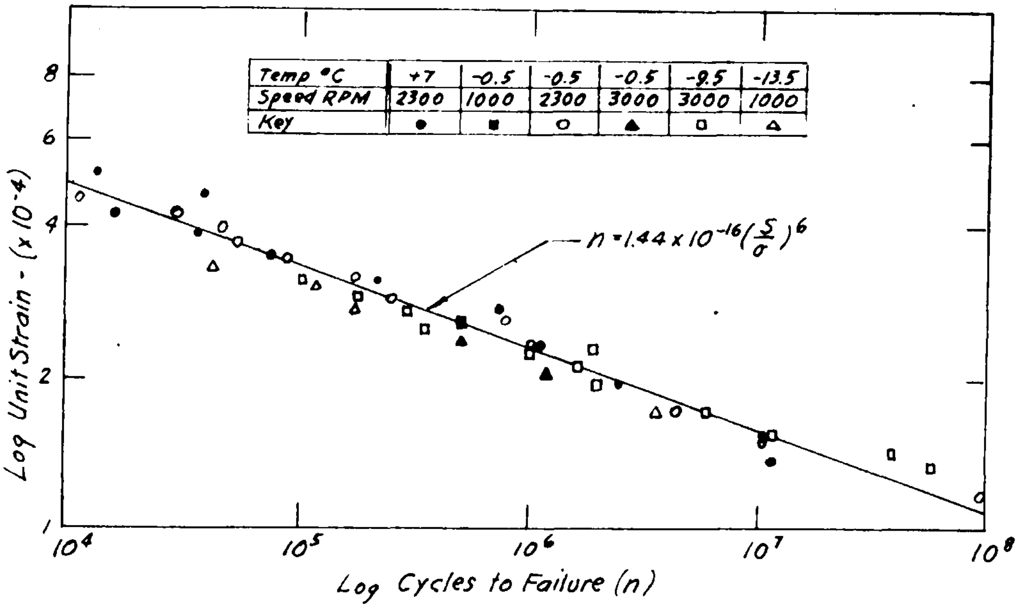

FIGURE 5. FATIGUE TESTS OF ASPHALT CONCRETE[7]

A mathematical relation between strain and deflection of the surface is not known but limiting deflections proposed by Hveem[8] for several million repetitions have been confirmed by many sources. These are given on page 33.

CBR METHOD

The history of the development of the various thickness design methods is outlined to show the bases on which the methods are founded.

An extensive investigation of pavement failures in California in 1928-9 and the original CBR method that was developed as the result is described by Porter.[9, 10] The test devised to evaluate subgrades and bases uses a 6-in diameter cylindrical mold and soil samples of varying moisture content that compact to about 5-in thickness under a 2000 psi static load to determine optimum moisture content. The specimens to be tested are prepared at optimum moisture and are compacted at 2000 psi static load. A surcharge weight equal to the anticipated weight of the pavement is placed on the specimen and the specimen is then soaked for 4 days to approximate probable worst field conditions. Per cent expansion is noted during soaking. A penetration test is then made with a 1.95-in diameter piston with an estimated surcharge weight in place. The percentage ratio of the load required to penetrate the specimen 0.1 in to the load required for the same penetration of a standardized crushed rock specimen is the CBR value. Many CBR tests of sections of flexible pavements, both sections that were adequate and ones that had failed, furnished data to correlate the CBR values with road service and from this correlation design curves were compiled.

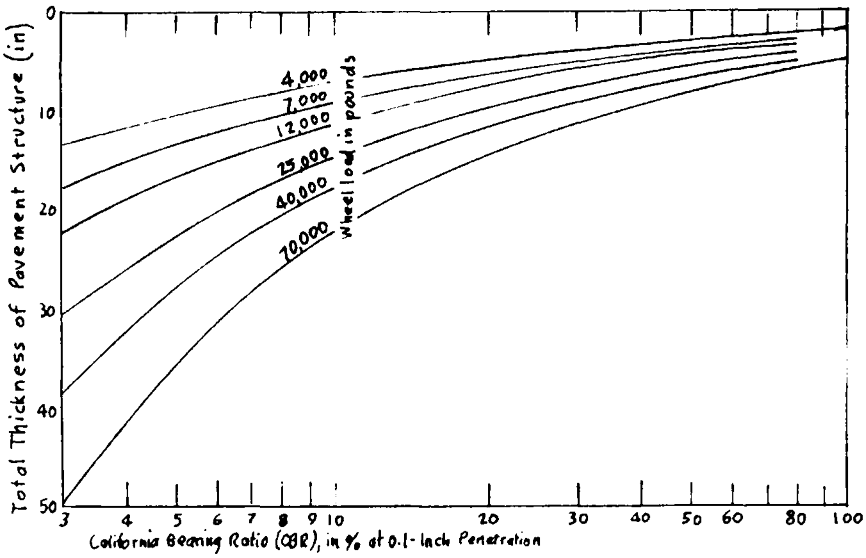

FIGURE 6. THICKNESS DESIGN CURVES FOR CBR[11]

Jervis and Eustis[11] describe the modified CBR method developed by the Corps of Engineers at the beginning of World War II. This design procedure has been used throughout the world, and the soil test was adopted by the American Society for Testing Materials in 1961 as ASTM - D 1883. The principal modifications to the original method are that 1-in layers of soil are compacted in the mold with a 10-lb hammer with an 18-in drop; specimens are prepared with 12, 26, and 55 blows per layer to obtain a family of curves relating density, water content, and CBR; design CBR is selected for the expected field results - usually 95 per cent density as determined by AASHO T-180 Method D; for expansive soils samples for a wide range of moisture content are prepared to establish the relationship between moisture content, density, swell, and CBR; a design CBR is selected to correspond to a moisture condition that can be specified for construction.

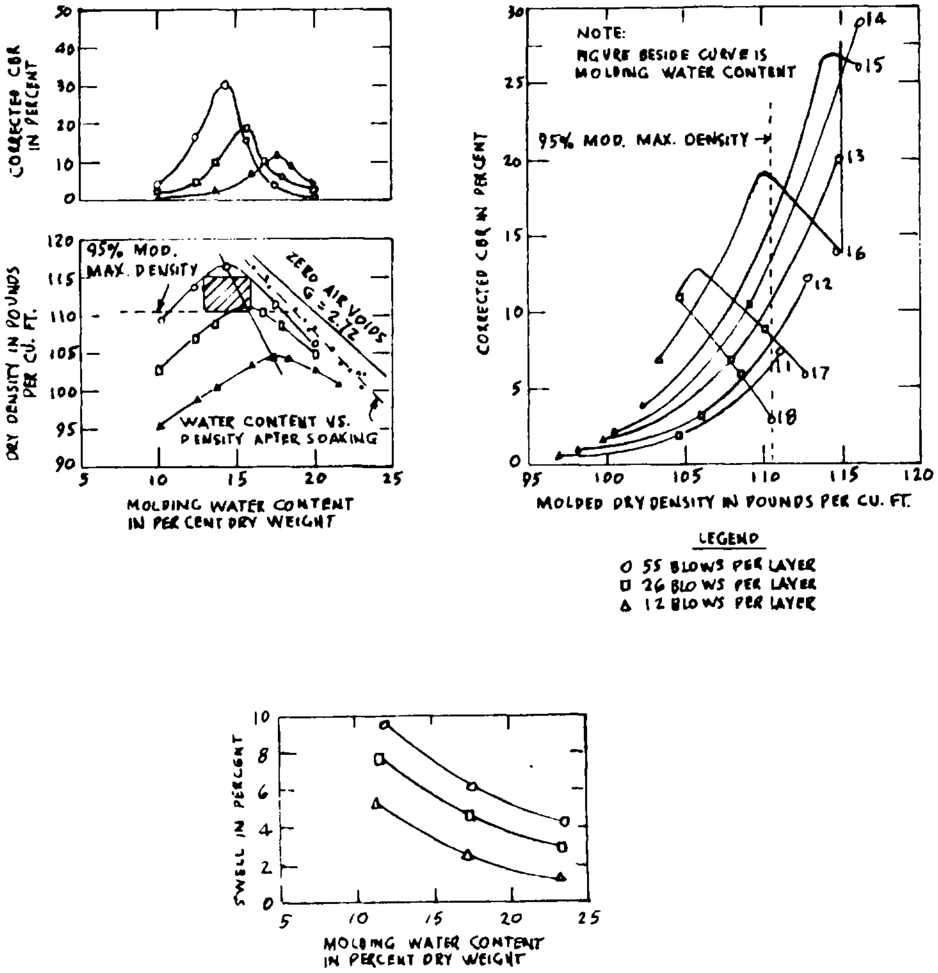

FIGURE 7. PLOT OF CBR TEST DATA FOR DESIGN[12]

Design CBR and density are selected toward the bottom of the ranges in CBR and density. The lower plot is used to select a moisture and density for use with expansive soils.

Russell and Olinger describe[13] the modified design curves used with the CBR test in Wyoming. With CBR curves for wheel loads from 4000 to 15000 lb as a base, an empirical relation was established to evaluate the affects of the amount of precipitation, frost action, site conditions, and the estimated number of equivalent wheel loads for the design period. Olinger[14] reports that satisfactory results are obtained with this modified procedure for overall thickness, but that a stabilometer is used to evaluate granular materials, bases and pavements.

Woodson reports[15] the use in Virginia of the Corps of Engineers CBR procedure except for soaking specimens until no further expansion occurs. Nickols describes[16] test roads built in Virginia with overall thickness determined by CBR method and a 13-in base-surfacing in varying combinations of thickness. Deflections from 0.046 to 0.060 in and several failures within 10 months indicate the pavement may be inadequate.

Lee describes[17] the successful use of the original CBR method in Maryland where a 10 per cent impact factor added to the legal load requires a 15000 lb wheel load for design. After a 3-in asphaltic macadam surface on a 8-in macadam base proved inadequate, two standard surface-base sections were adopted; 3½-in asphalt concrete on 10-in macadam base is used for traffic over 2000 vehicles per day and 2-in asphalt concrete on 8-in macadam base for traffic less than 2000 vehicles per day. On three roads where deflections have been measured with a Benkleman beam and a 11,200-lb wheel load and where the condition of the road is good, deflections ranged from 0.02 to 0.03 in.

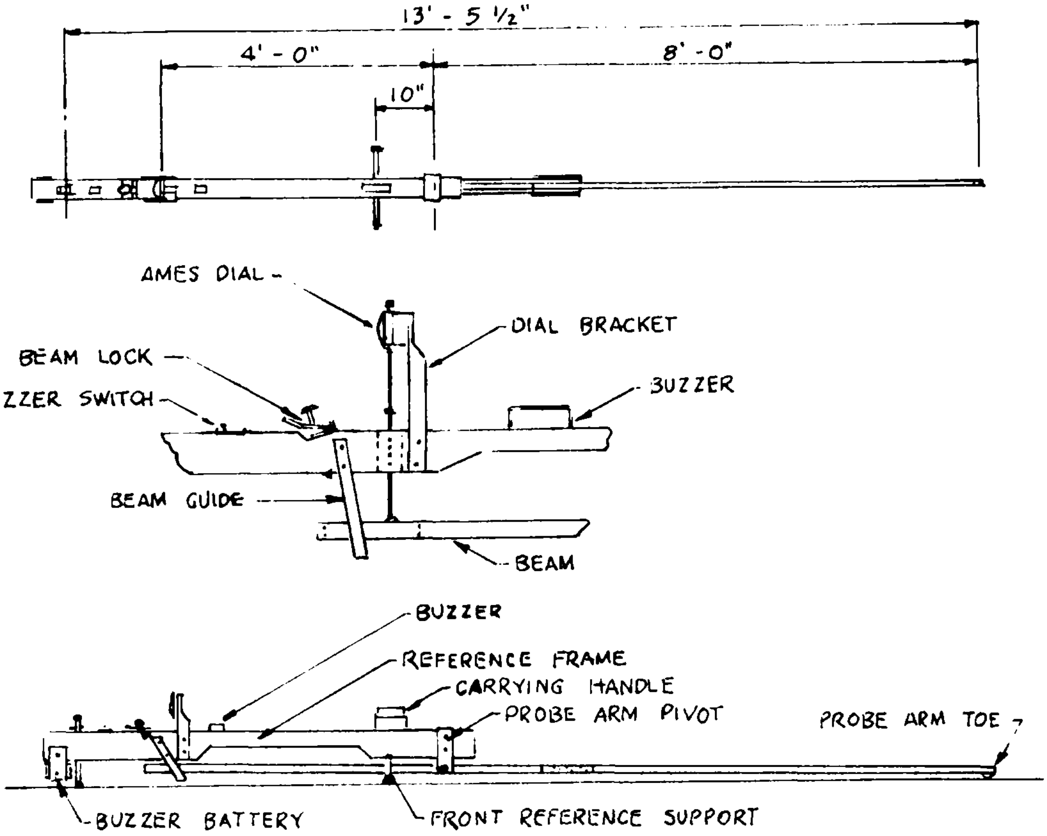

FIGURE 8. THE BENKLEMAN BEAM[18]

The development in Kentucky of a design method using the CBR test correlated to road deflections and traffic is described[19] by Drake and Havens. Based on limiting deflections determined from a statistical study of actual pavements, the accumulated equivalent wheel loads that had been sustained, and the subgrade CBR value, design curves were compiled.

Shook and Finn developed[20] the thickness design relationship of the 1963 Asphalt Institute method from a statistical analysis of the AASHO Road Test, the WASHO Road Test, British road tests, and other unspecified data. The variables considered were (1) the pavement thickness represented by a linear combination of the thicknesses of surfacing, base, and subbase, (2) the number of applications of a given axle load to a serviceability index, p, of 2.5, and (3) axle load and configuration. The elements of pavement condition that were found to contribute to an index that correlated highly with subjective opinion of serviceability were: roughness, SV; rut depth, RD; cracking, C; and patches, P. These elements were combined in an equation whose coefficients were determined by a regression analysis to be:

$$p = 5.03 - 1.91 \log(1 + SV) - 0.01(C + P)^{\frac{1}{2}} - 1.38\ RD^2$$

An 18-kip axle load was used as a base and equivalency factors determined for other loads from a damage ratio. The subgrade of the AASHO Test Road had a CBR of 2.5 and modified CBR curves published by the Asphalt Institute in 1960 were used to extend the design relationship to other CBR values. This was done by determining thickness requirements for a range of CBR values along the 18-kip curve to correspond to very heavy, heavy, medium, and light traffic; minimum thickness requirements and layer equivalencies were used to convert design sections to design factor, T; a plot of log T vs log CBR was then made for each traffic classification; a straight line relationship was found to exist for each traffic classification so thickness adjustments for CBR were made using the slope of the straight line. The final design equation is:

$$T = (-20.5 + 5.53 \log W + 0.669L_1 + 0.0932L_1L_2) \times (2.5/\mathrm{CBR})^{0.4}$$

where W = applications of a given load
L_1 = single-axle load in kips
L_2 = 0 for single-axle loads and 1 for tandem-axle
T = thickness factor = $2.0D_1 + D_2 + 0.75D_3$

Discussion: Of the methods discussed that of the Asphalt Institute, Shook and Finn,[20] best treats the factors affecting thickness, (1) loads and repetitions are evaluated from traffic counts, (2) a reasonable evaluation of strength of materials facilitates an economic solution, and (3) stresses and deflections are controlled by a correlation to serviceability which is an index of how well deflections are limited. The test equipment and strength test of the subgrade are simple and the results easily interpreted. The soaking of the test specimens to approximate worst field conditions is probably as close an approach to possible conditions as can be devised. How nearly the CBR value represents the actual strength of the subgrade is probably best illustrated by its relation to the dynamic modulus of elasticity which was measured by Heukelom and Klomp[21] and Jones[22] for about 70 different soils varying from CBR = 2 to CBR = 150 and the relation was found:

$$E = 100\ \text{CBR (within a factor of 2)}$$

E in kg/cm^2.

The soil property measured in the CBR test is a fair index of the strength of the subgrade, and for this reason it has long been a widely used test; however, it would be better to use a dynamic triaxial test that measures directly the strength of the subgrade. For example, in a design for a primary highway with a 9000-lb wheel load, 80 psi tire pressure, equivalent radius of contact of 6 in and a subgrade CBR of 5 (E of 500 kg/cm^2),

$$\text{Thickness} = \sqrt{\left(\frac{3pa^2}{2E\Delta}\right)^2 - a^2} = \sqrt{\left(\frac{3(80)36}{0/04E}\right)^2 - 36}$$

$$= 30\ \text{in}$$

If the strength of the subgrade is only measured to within minus 50 per cent to plus 100 per cent as reported above, this would result in a thickness range of 14 in to 60 in. This is an extreme case as most of the points on the curve[21,22] are close to the relation, E = 100 CBR kg/cm^2, and the assumption that all the error is in the measurement of CBR values is questionable.

STABILOMETER METHOD

The stabilometer method was developed[23] by the California Division of Highways to replace the CBR method because it was considered that the mold effect of the CBR method is critical and that a better analysis of material would result from measurement of soil properties in several tests. Hveem and Carmany determined 62 factors[23] that affect pavement thickness and developed soil tests and traffic data to evaluate them in a design formula. The problem was approached with a shear-strength theory of pavement failure similar to that for metals proposed[24] by Prandtl and adapted for soils by Terzaghi[25]; however, the design formula developed is a correlation of soil test values and traffic with road behavior. The testing machine to evaluate the subgrade, the stabilometer, is a modified triaxial test of a 4-in diameter x 2½ in specimen that is saturated and subjected to a consolidated-undrained

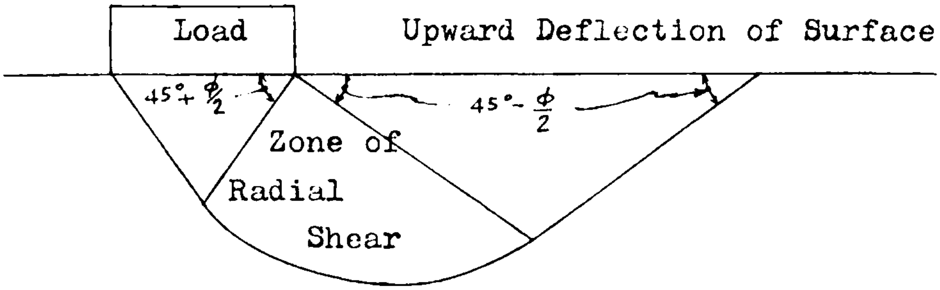

FIGURE 9. PRANDTL'S PLASTIC EQUILIBRIUM THEORY[24]

test. An initial lateral pressure of 5 psi is allowed to increase as the vertical pressure is raised to 160 psi. The strength of the material is considered to be a function of the ratio of the lateral pressure to vertical pressure when the latter is 160 psi. A later revision of the test modifies the strength by a function of the horizontal displacement that occurs while the lateral pressure is reduced to 5 psi after first reducing the vertical pressure to 80 psi. The triaxial test as modified by Hveem was considered to be an index of the strength of the material in the zone of radial shear. These strength values have been correlated to road service and traffic data and the correlation used to prepare a thickness design chart. A cohesiometer, Figure 11, was developed to measure an index of the resistance of the surfacing and bases to

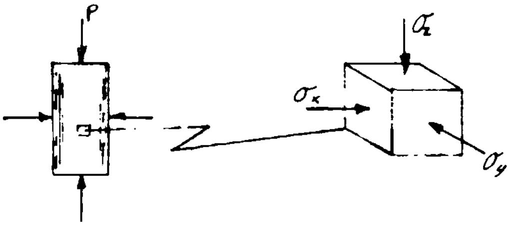

FIGURE 10. SKETCH OF TRIAXIAL TEST

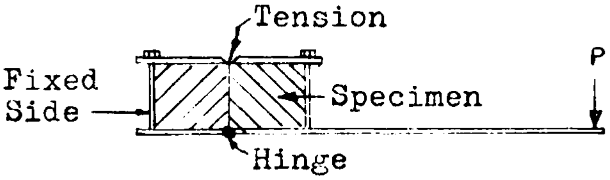

FIGURE 11. SKETCH OF COHESIOMETER

upward deflection of the surfacing shown in Figure 9; for this the modulus of rupture of the specimen when bent about a neutral axis outside the specimen is measured. From observations of test tracks and road performance it was determined that the effectiveness of bases and pavements is a function of the fifth root of the tensile strength as measured in the cohesiometer. Field data indicate that the thickness required is a function of the log of the heavier wheel load repetitions. Loads are assigned factors determined from estimated damage done the road in comparison to a 5000-lb wheel load and all traffic is converted to an equivalent number of 5000-lb wheel loads. From traffic counts an estimate of equivalent load repetitions is made for a 10 year design period. Thickness required was also found to be a function of tire pressure, and for loaded areas in the range of tire prints it is a function of the square root of the area. Similar evaluations of these individual traffic effects have been made by Barkan,[26] Drake and Havens,[19] and McLeod.[27] All the effects of traffic are combined with a correlation factor to adjust thickness by $1.35(EWL)^{.11}$. This traffic index was developed from field observations that the required thickness is a log function of the number of heavier wheel load repetitions, and coefficients were determined to make the index fit observed road service.

To prepare test specimens a kneading compactor was developed which was believed to produce a particle structure more nearly like that obtained from construction equipment since the foot of the kneading compactor more nearly duplicates the action of the sheepfoot roller. To estimate the worst condition that would probably occur in the subgrade samples are prepared at varying moisture content and tested saturated. The samples are compacted with enough moisture so that after 100 applications of the compactor foot at 350 psi water will be exuded by a uniform pressure of 100 to 800 psi over the area of the sample. The pressure at which this occurs is called the exudation pressure. The samples are then confined and soaked 16 to 20 hours and the expansion pressure that is developed is measured. Thickness of cover material required to confine the soil or the thickness required to support the load at the moisture content equivalent to 300 psi exudation pressure which ever is greater is the design thickness.

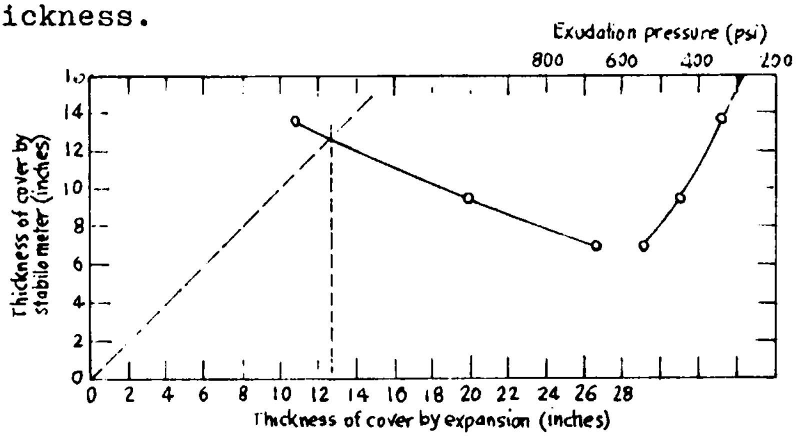

FIGURE 12. PLOT OF STABILOMETER TEST DATA FOR DESIGN[23]

A further development of the stabilometer method has been reported by Hveem.[8] Another test machine, a resiliometer, to measure elastic deflections of soil samples under repeated loads has been developed by the California Division of Highways, and it is suggested that an additional design procedure should be added to that described above. To provide sufficient depth of base or slab strength of pavement to reduce pavement deflections to within the endurance limit, deflections produced by estimated loads on samples of proposed pavement structural components are measured in the laboratory to estimate total deflection. Where the estimated deflection is greater than permissible, as determined from road performance, the proposed pavement design is strengthened to reduce the deflections to within allowable limits.

Livingston reports[28] that the Colorado highway department adopted the stabilometer method because of erratic results with the CBR method on A-1 and A-3 soils and because road performance had not substantiated the Group Index method which had been used.

LeClerc describes[29] the use of the stabilometer in Washington highway department where it replaced the CBR method in 1951 because it is faster, and more consistent test results were obtained on clayed gravels and clean sands. A design chart of stabilometer values vs total surfacing is used with equivalent wheel load curves obtained from correlations with road performance and traffic. Sample preparation is modified from California practice to 40 blows at 100 psi and design exudation pressure is 400 psi from a range of 100 to 600 psi.

Erickson describes[30] the Idaho highway department method which replaced the CBR method in 1950. The California procedure is used except that an air-driven kneading compactor was developed with a foot pressure of 250 psi.

By 1957 traffic data and road service studies indicated that a small change in the original design formula was desirable. After the data from the WASHO and the AASHO Test Roads was evaluated Hveem and Sherman recommended[31] a revised scale of cohesion values for gravel bases and bituminous bases and a slightly modified design equation to correlate the stabilometer procedure to the test results.

Discussion: A reasonable evaluation of loads and repetitions is made from traffic surveys. A more direct evaluation of strength of surfacing and bases would be obtained from a triaxial test. The tension measured by cohesiometer is an index of the modulus of elasticity at rupture, and the stabilometer R-value is related to the modulus of elasticity of the materials tested, but simpler tests should correlate better. The California study reported by Hveem[8] relating surface deflection to the endurance limit of surfacing is the most practical solution of fatigue the writer has discovered, and since Hveem[8] suggests that the permissible deflections be used as a final criteria to check the designs obtained from the stabilometer method, it would seem to be more reasonable to base the design procedure directly on fatigue and the elastic theory as suggested in the Introduction.

The kneading compactor eliminates much hand labor so would probably increase production in a laboratory where many remolded samples are tested, and it produces a more consistently uniform compactive effort than can be obtained by hand; however, where possible, it is desirable to use undisturbed samples taken directly from the subgrade.

TRIAXIAL METHOD

A solution of the thickness design problem with soil parameters determined by the use of the triaxial test seems to have first been suggested[32] by Palmer and Barber. From the theory of elasticity Boussinesq had derived[1] formulas for the stress components in a homogeneous isotropic mass caused by a point load, and Love had integrated[33] these over a circular area. Triaxial test machines were being used by Delft Laboratories in Holland, Shell Development Company, and the U.S. Corps of Engineers when Worley first used[34] the triaxial test in pavement design in Kansas. The machine used was patterned after the one used by the Corps of Engineers, and details of the test are given[35] by Finney. Samples are either undisturbed or compacted to estimated field density. The sample sizes are 2.8-in diameter by 8 in long for minus No. 4 material and 5-in diameter by 14 in long for minus $1\frac{1}{2}$-in material. A porous stone at the bottom of the sample is placed in water, a surcharge equal to the estimated weight of the pavement is placed on top, and a vacuum applied at the top until the sample is saturated. Consolidated-drained tests are run on the saturated samples with lateral pressures of 10 psi and 30 psi. Volume is assumed constant so $\mu = 0.5$, and an average secant modulus is determined over the range of estimated vertical stresses. A wheel load of 5,000 lb and a permissible deflection of 0.10 in were the criteria for design in 1943. Thickness of pavement is calculated, as suggested[36] by Hogentogler, by the relation derived[32] from elastic theory by Palmer and Barber and modified[37] by Marguerre's theoretical solution for deflection in a double layered system. The latter modification was confirmed[2] by Burmister.

Kansas reported[38] the addition of variable factors for traffic and for rainfall based on road service, and Lacy gave[39] the values being used; for annual precipitation less than 35 to 45 in thickness is reduced and for traffic greater or less than 1500 vehicles per day the thickness is increased or reduced accordingly. The theoretical vertical and horizontal stresses in the subgrade due to the design load are calculated and the subgrade modulus determined from the triaxial test at these values.

McDowell describes[40,41] the triaxial test procedure developed in the Texas highway department. Optimum moisture is determined; then 6 samples are compacted at that moisture in 6-in diameter by 8 in long molds with 10 lb modified AASHO hammers. The samples are then saturated by capillarity with a surcharge of 1/3 to 1 psi and a lateral pressure of 1 psi. Swell is determined by weight and volume change. Triaxial tests of lateral pressures of 0, 3, 5, 10, 15, and 20 psi are run and a Mohr's envelop drawn. A chart, Figure 13, consisting of Mohr's envelops that have been correlated with road service is used to decide the class of the sample. In Figure 14 curves of soil classes have been correlated by road service with thickness of pavement and design wheel load which is the average of the 10 heaviest wheel loads per average day. Two scales for wheel loads are given, one for roads to last 10 years and another for roads of 20 to 30 years life expectancy. Ahlvin notes[42] the similarity of the Texas design curves to the CBR design curves used by the Corps of Engineers.

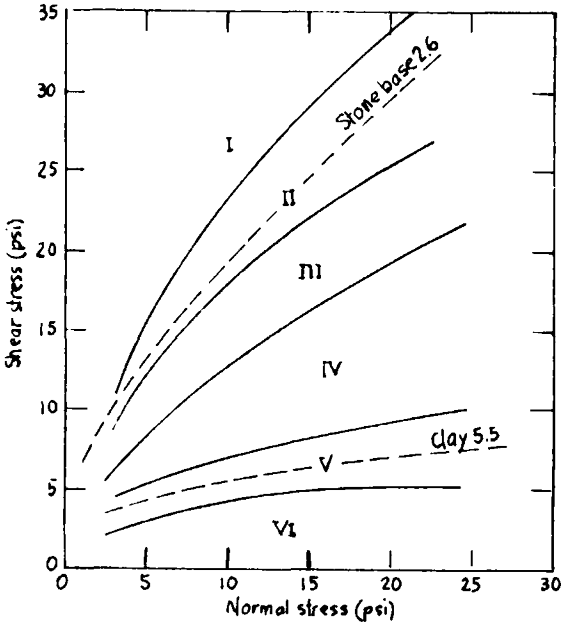

FIGURE 13. TEXAS CLASSIFICATION CHART[41]

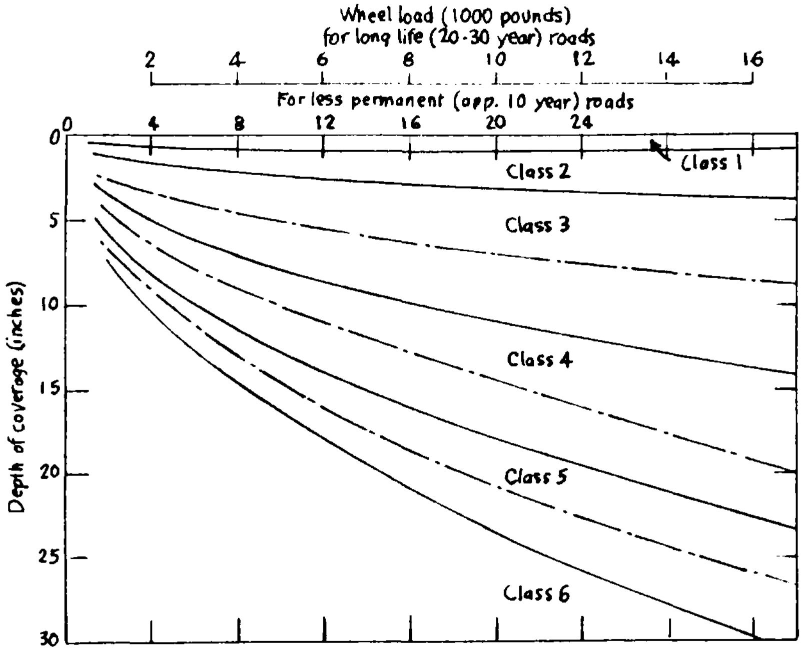

FIGURE 14. TEXAS DESIGN CHART[41]

Discussion: The triaxial test is a direct measurement of soil resistance to deflection caused by the load; a dynamic **triaxial** test would give more exact results with some soils. The Texas procedure correlates the measured strength to road service so is an index of dynamic strength; while the Kansas procedure is a mathematical solution.based on elastic theory with assumptions of ideal conditions. The Kansas design deflection of 0.10 in is beyond the endurance limit of surfacing, but is corrected for by the correlation with traffic and road service. The evaluation of traffic by equivalent wheel loads would provide a more exact load and number of repetitions factor.

PLATE BEARING METHOD

Burmister developed[2] the theory of stresses and displacements in a two-layered system by the mathematical theory of elasticity and presented the solution in terms of the ratio of the radius of the bearing area to the thickness of the surfacing, and the ratio of the modulus of elasticity of the subgrade to that of the surfacing, and evaluated the results by plotting influence curves in terms of these ratios as shown on page 3. The usual assumptions of elastic theory that the materials and conditions are ideal were made in the solution. Burmister's solution[2] became the basis for the design method used by the U.S. Navy Department for airfields where an allowable deflection is assumed and moduli of elasticity of the subgrade and the pavement are determined from field test sections.

McLeod made[27] extensive field investigations of airfields in Canada in which the subgrade and pavement components were evaluated by the plate bearing test. CBR, cone bearing, and triaxial tests were also run to determine relative values. The load-deflection value obtained with a 12-in diameter plate at 10 repetitions and 0.20 in deflection was adopted as the subgrade support value. An idealized plot of the data obtained[27] by McLeod is shown in Figure 15 and from this data the design formula was derived as follows:

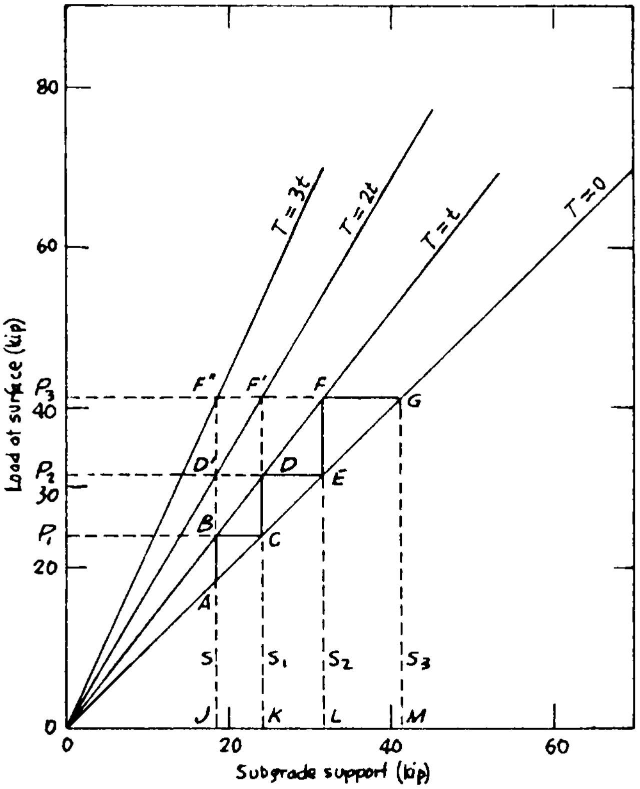

FIGURE 15. DIAGRAM ILLUSTRATING MCLEOD ANALYSIS[27]

From similar triangles

$$BJ/AJ = DK/CK = FL/EL$$

$$P_1/S = P_2/S_1 = P_3/S_2 = P_n/(S_n - 1)$$

$$P_1 = S_1,\ P_2 = S_2,\ .\ .\ .\ .\ ,\ P_n = S_n$$

Substituting

$$P_1/S = P_2/P_1 = P_3/P_2$$

Solving for P

$$P_2 = P_1^2/S, \quad P_3 = (P_1/S)(P_1^2/S), \quad P_3/S = (P_1/S)^3$$

$$P_n/S = (P_1/S)^n$$

If the layers of n number are 1 inch thick

$$P/S = (P_1/S)^T \text{ and } T = \left[\log(P_1/S)\right]^{-1}\log(P/S)$$

Assuming that P_1/S is independent of the type of base so equal to a constant

$$T = K \log(P/S)$$

where T = the required thickness of granular base in inches
P = gross wheel load in pounds
S = total subgrade support in pounds for the same contact area, deflection, and number of repetitions of load pertaining to the applied load P
K = base-course constant, Figure 16
S_1 = subgrade support contributed by first layer
S_n = subgrade support contributed by n layers
P_1, P_2, P_n = load capacities

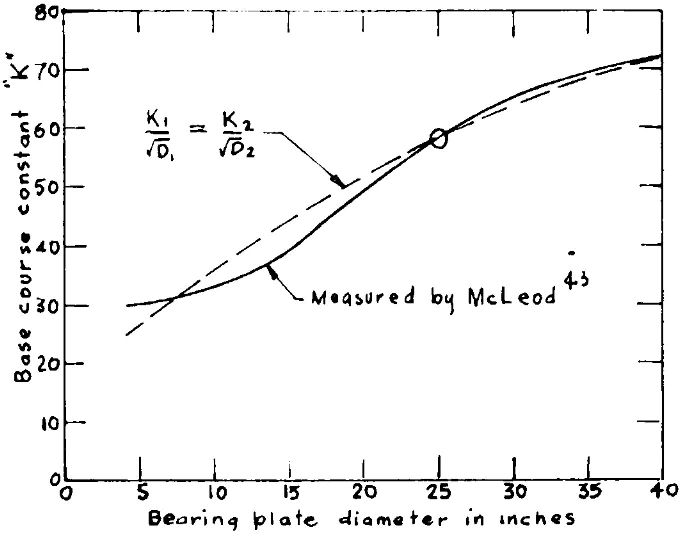

FIGURE 16. INFLUENCE OF BEARING-PLATE DIAMETER ON VALUE OF K[43]

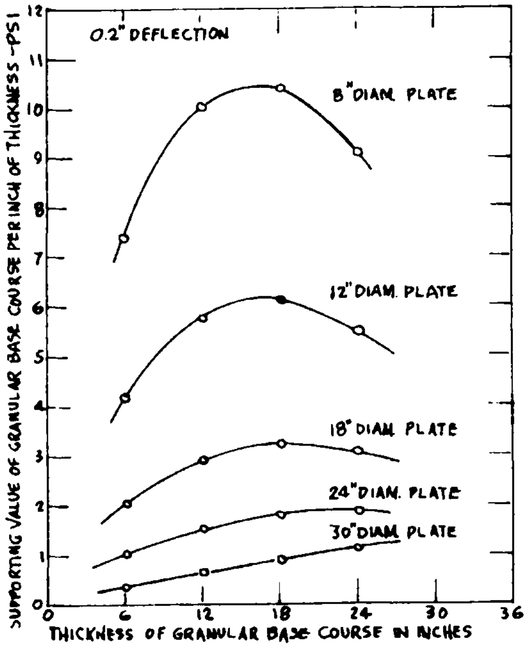

FIGURE 17. BASE COURSE LOAD SUPPORT PER INCH OF THICKNESS[44]

A deflection of 0.20 in was adopted as the design deflection for highways for unlimited traffic. McLeod reiterates[43] the criterion of deflection of 0.20 in for unlimited traffic, or for 1 million equivalent wheel loads, plots data to confirm the relation suggested by other investigators that thickness required varies as the log of the number of repetitions, and recommends a reduction of thickness based on a relation reducing thickness necessary for 1 load to 25 per cent of the thickness for 1 million loads with thickness varying as the log of the number of repetitions for intermediate amounts of traffic. An equivalency is suggested that the total thickness be reduced ½ in for each inch of asphalt concrete surfacing.

When the data from the Hybla Valley, Virginia tests, given[45] by Benkleman and Williams, and data collected earlier in Canada were plotted, Figure 17, McLeod noted[44] that there exists an optimum thickness for pavements of

$1\frac{1}{2}$ to 2 times the radius of the loaded area - that is that strength gain per unit of increased thickness decreases at greater thicknesses - so the possible economic advantage of increasing strength of base by stabilizing with cement or asphalt should be investigated in each design. McLeod also demonstrates[46] that for a given bearing plate size and equal deflections of the pavement and subgrade that a theoretical relationship exists between thickness as determined by the Canadian formula, $T = K \log (P/S)$ and K, the reciprocal measure of increase in strength provided by the first unit of thickness of pavement placed on the subgrade. McLeod further notes[46] that the Canadian Good Roads Association studies have found that deflections measured by a Benkleman beam and a 9000-lb dual wheel are very close to those obtained by equivalent plate bearing tests, and that under a 9000-lb load the stress-strain relation of flexible pavement structures measured by a 12-in steel bearing plate is approximately equal to dual tires with 80 psi tire pressure. In light of more recent studies the 0.20-in design deflection recommended earlier is reduced to 0.05 in for heavy traffic and 0.10 in for light traffic. McLeod recommends[46] that subgrade and pavement moduli be evaluated at design deflection with moisture at the most critical service condition and using a bearing plate approximately equal to the contact area of the design wheel.

Discussion: Ritter and Paquette report[47] that no civilian agency in the United States is using the plate bearing method for thickness design; however, it is included in this paper because of the excellent field studies that have been conducted with it. A direct measurement of the static modulus of elasticity is made, but no method is knownto obtain the dynamic modulus. It is difficult to obtain conditions of the soil in the worst probable conditions for the test. The evaluation of loads and repetitions suggested for the Canadian method has been verified by several other agencies, but the design deflection of 0.05 in is greater than has been found satisfactory by most investigators.

PAVEMENT DEFLECTIONS AND FATIGUE FAILURES

The type of pavement failure characterized by alligator cracking of the surfacing is generally agreed to be due to fatigue. That this type of failure is related to repeated deflections of the surface due to loads is accepted by most investigators. Although magnitude of strain and number of repetitions of load have been demonstrated to be the exact functions of fatigue for a given surfacing material as shown in Figure 5, and although radius of curvature may be a better measure of strain than

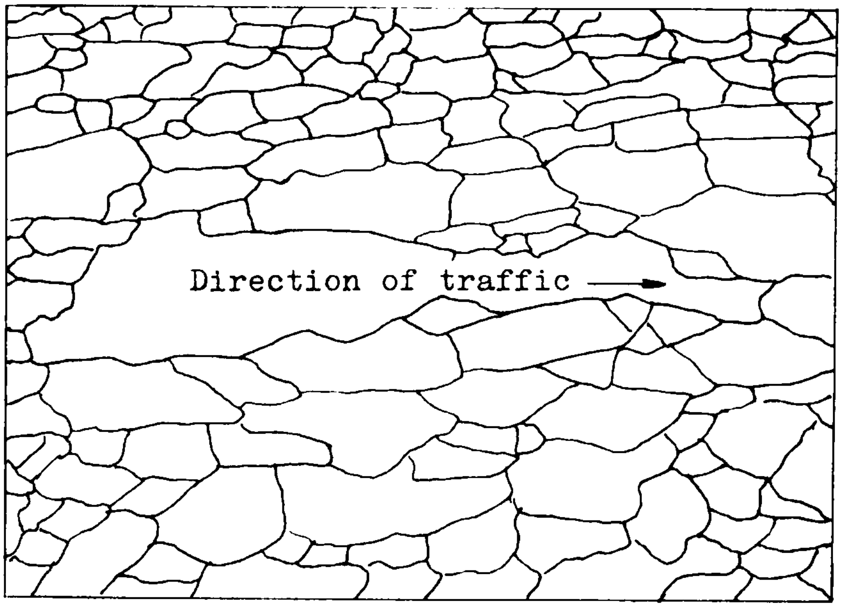

FIGURE 18. ALLIGATOR CRACKING OF FLEXIBLE SURFACE

pavement deflection, there appears to be sufficient data to indicate that pavement deflections are an approximation of strain that may be used for a design. Radius of curvature is defined as the absolute value of the reciprocal of the curvature of the surface of the pavement due to the load deflection and is a function of size of loaded area and intensity of load.

Porter in summarizing data gathered[10] in road performance studies in 1928-9 in California noted that where pavement deflections exceeded 0.02 to 0.03 in failure occurred after several million load repetitions.

Studies by Nijboer and Poel have shown[48] that where flexural stresses caused by traffic exceed the endurance limit failure occurs. In fatigue tests using alternating bending an endurance limit decreased as stresses were increased. Hveem[8,49] reported extensive field studies of deflections and failures in California and suggested the following tentative allowable deflections under a 15,000 lb axle load as a design criteria for roads to sustain several million repetitions of the design load. The general range of deflections has been confirmed by many investigators, and many of the reports of pavements with greater deflections having performed satisfactorily have based their data on deflections of greater loads and/or lesser number of load applications so when these data are adjusted to a 15,000 lb axle load at several million repetitions the agreement is remarkable.

Pavement Thickness (in)	Type	Maximum Deflections for Design (in)
8	Portland Cement Concrete	0.012
6	Cement Treated Base (with Bituminous Surface)	0.012
4	Asphalt Concrete	0.017
3	Asphalt Concrete on Gravel Base	0.020
2	Asphalt Concrete on Gravel Base	0.025
1	Road-mix on Gravel Base	0.036
½	Surface Treatment	0.050

Williams and Maner measured[50] deflections in Virginia that concurred with Hveem's allowable deflections for roadmix.

Williams and Lee determined[51] allowable deflections on 3½- and 4-in asphalt concrete surfaces in Maryland to be 0.012 to 0.028 in when corrected for residual recovery on roads 1 to 11 years old.

From 814 loading tests on 123 pavement sections at the CAA Technical Development Center Herner concluded[52] that limiting deflections might be used as a direct method of thickness design.

In Benkleman's analysis[53] of the WASHO Test Road the conclusion was reached that pavements (2- and 4-in asphalt concrete) could withstand deflections of 0.030 in in winter to 0.045 in in summer for the test period considered.

From a statistical study of actual pavement performance in Kentucky, Drake and Havens obtained[19], Fig. 19, a deflection - number of equivalent-wheel-loads curve which indicated a fatigue deflection of about 0.020 in at 30 x 10^6 repetitions. Traffic groups in 10^6 EWL's are 1A = less than $\frac{1}{2}$, I = less than 1, II = 1-2, III = 2-3, IV = 3-6, V = 6-10, VI = 10-20, VII = 20-40, VIII = 40-80.

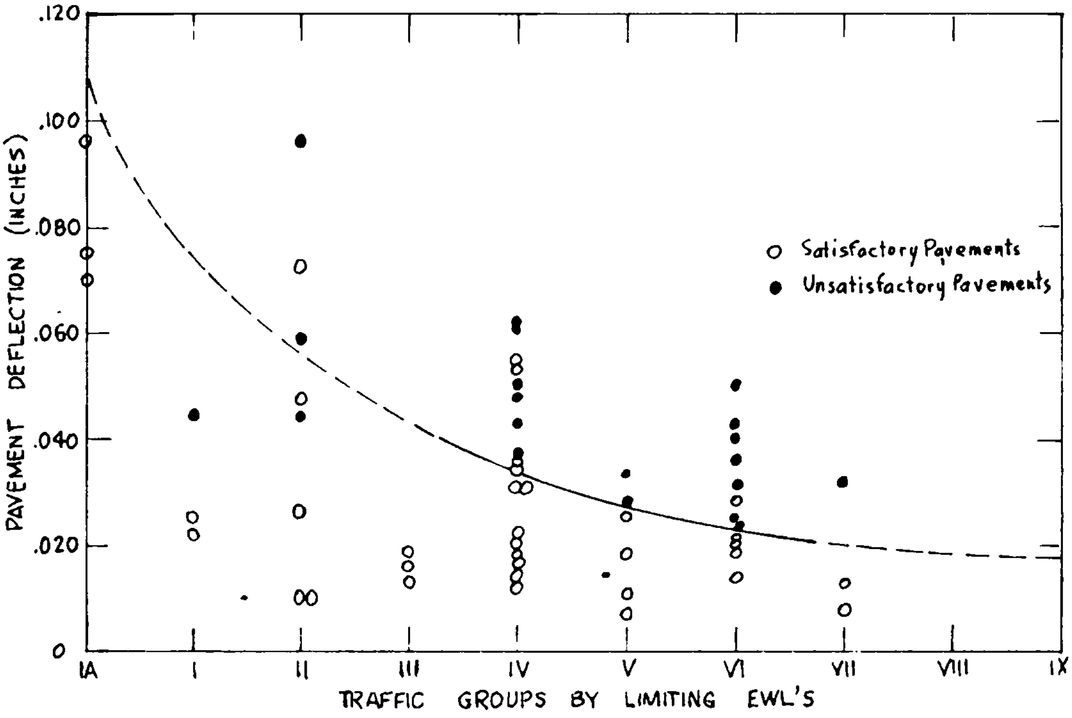

FIGURE 19. DEFLECTION - NUMBER OF EQUIVALENT WHEEL LOADS[19]

Deflections were measured by Benkleman beam and 18,000-lb axle load. The fatigue curve formed by separating the deflections of roads that had failed from those that had not at repetitions up to 30 x 10^6 had a shape similar to that usually found for fatigue in concrete.

Saal and Pell reported[7] fatigue tests run on asphalt concrete. At temperatures from -13.5°C to + 7°C, dynamic loads at 1000 to 3000 RPM, and repetitions from 10^4 to 10^8 a very narrow band of points indicated the relation, $n = 1.44 \times 10^{-16}(S/\sigma)$ where σ is bending stress, and S = stress/strain; S approaches E at rapid loadings.

Laboratory fatigue studies of asphalt mixes reported[54] by Monismith, Secor, and Blackmer confirm field observations of fatigue in asphalt concrete. The limiting of strain in the surfacing is reported to be an excellent criterion for preventing fatigue failures, but the quantitative determination of strain is complex as it is a function of deflection and radius of curvature of the surface due to load-deflection.

Havens noted[55] the paradox in stresses decreasing as the thickness of surfacing decreases as might be inferred from Hveem's suggested allowable deflections and pointed out that the stresses due to a wheel load are a combination of bending stress and tension, being chiefly bending stresses in thicker surfaces and changing to nearly pure tension in the case of very thin surfaces.

Yoder notes[56] that deflection patterns, deflections of components, radius of bending, thickness of pavement, speed of loading, and temperature are all functions of the load deflection and questions the use of allowable deflections as a design criterion, but concludes that in general deflections greater than 0.02 in can cause distress on conventional pavements with some exceptions.

Ahlvin reports[57] that theoretical deflections were nearest to the measured ones in controlled sections when Poisson's ratio of $\frac{1}{2}$ for clay-silts and of 0.3 for sand was used in the calculations. Deflections in both clay and sand differed somewhat from theoretical, but deflections by superposition was shown to be reasonably valid. Barkan reports[26] finding these same values for Poisson's ratio in dynamic tests of soils.

In a summary of investigations relating deflection measurements to pavements Monismith concluded[6] that deflections are symtomatic of fatigue, and although there are other factors involved, deflections are a good means of correlating field performance and research.

A definite trend for deflections to vary with pavement thickness on high type pavements was found[58] by Ford and Bissett, but the data, which included 7000 Benkleman beam tests with a 9000-lb wheel load on 8 test sections in Arkansas was too scattered for deflections alone to be sufficient information to indicate pavement performance. The ratio of radius of influence to deflection was found to be a good criteria for over-all pavement performance.

Walker reported[59] that deflections were found to be inconclusive on a 4-year old test road in Indiana that failed by developing longitudnal cracks and rutting. The section was 5 in of asphalt concrete, 8 in of water-bound macadam on 5 to 8 in of open-graded subbase. Deflections varied from 0.015 to 0.035 in. The type of failure described does not appear to indicate failure by fatigue.

Concluding a study of dynamic testing of silt, clay, and sand-clay in a triaxial testing machine (Fig. 20) Larew and Leonard report[60] that a critical level of repeat deviator stress exists at which the slope of the deformation vs number of repetitions curve is constant after the first few applications. For higher deviator stress the slope of the curve is positive and increases until failure occurs. Ahmed and Larew found[70] the ratio of the static modulus

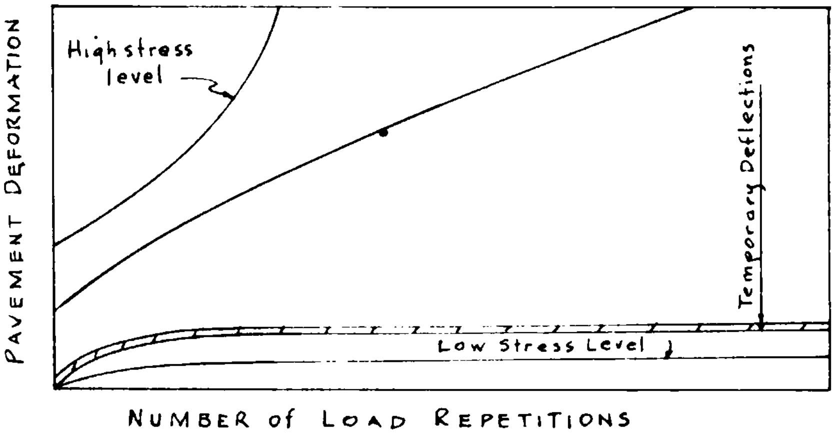

FIGURE 20. SCHEMATIC REPRESENTATION OF DEFLECTIONS VS LOAD REPRESENTATIONS[60]

to the repeated load modulus of three fine-grained soils, micaceous silt, limestone residual, and sand-clay, varied from 1.05 to 2, demonstrating the improvement in accuracy that can be obtained in the case of some soils.

Dehlen has reported[61] the results of a 4-year study of flexure of road surfaces and fatigue of surfaces in South Africa. A distinct relation was found to exist between both the pavement deflection, and the radius of curvature and the condition of the surfacing. The load deflection relation was found to be practically linear in the range of stresses caused by traffic. A fairly good agreement was found to exist between measured deflections and theoretical as computed by Burmister's solution.[2]

In a summary of pavement research in Virginia Nichols reported[62] that the use of lime or cement stabilization of the subgrade had been found to be the most effective manner of reducing fatigue from excessive pavement deflections, with better control of subgrade and base compaction next in effectiveness. Relating pavement deflections, as measured with a Benkleman beam and an 18000-lb axle load, to road performance those roads with deflections greater than 0.036 in could be expected to develop early distress in the form of alligator cracking and rutting.

<u>Discussion</u>: The review of research presented in this chapter demonstrates the importance of designing

for fatigue in the surfacing and suggests relationships that can be incorporated into design procedures. The advantage of using dynamic modulus over static modulus in dealing with some soils is demonstrated.

SUGGESTED THICKNESS DESIGN PROCEDURES

The development of the principal thickness design methods have been examined to determine the bases of the solution that is obtained from the use of each. Although the Kansas triaxial method and the plate bearing method have a rational approach, the final thickness in both methods is much influenced by empiricism and personal judgment so no rational method is known to exist at this time.

With the modification[20] by Shook and Finn of the CBR design curves correlated to fatigue (if it may be considered that serviceability index is an expression of fatigue) this method, presented in detail in *Thickness Design - Asphalt Pavement Structures for Highways and Streets*, The Asphalt Institute, 7th Ed., 1963, used in conjunction with the CBR test as described in *Soils Manual*, Asphalt Institute, 1963, appears to be the most practical method for universal application. The test and design procedure are given in the appendix. Although it is empirical in its entirety, it is based on the most extensive correlations of any method.

The research work that has been studied suggests that a rational design could be made at this time with the most uncertain assumption necessary being the relation between strain in the surfacing and deflection of the pavement. With the various steps taken from the works of authors already described a possible procedure is suggested as follows:

Obtain an undisturbed subgrade sample with a thin-walled sampling tube or a trimming auger as suggested[63] by Housel; if the material is coarse, disturbed samples could be taken and prepared in a split mold as described[64] by Bishop. Saturation by capillarity aided by a vacuum with a surcharge equal to the estimated weight of the pavement would provide the probable worst field condition. The measurement of swell during saturation would call attention to highly expansive soils that would require special treatment.

With an estimated thickness of pavement, given design wheel load, and permissible deflection the vertical and lateral stresses for the design load at the surface of the subgrade can be obtained by Boussinesq's solution[1] provided a granular base is to be used; if the base is to be stabilized with cement or asphalt, Sowers,[65] and Zimpfer[66] have shown that Burmister's solution gives results close to those measured in the field but the moduli ratio would have to be estimated and recomputed if found later to be in error. With the computed vertical and lateral stresses used to determine the repeated deviator stress to be used in the triaxial test the resulting equilibrium strain could be measured and the dynamic modulus of elasticity obtained; thus accurate moduli for the conditions of loading should be obtained. Osterberg describes[67] a triaxial test machine for repeated loads developed at Northwestern University, and Larew describes[68] the modifications made on the original[69] by Leonards and used[60] by Larew and Leonards which could be used, Figures 21 and 22.

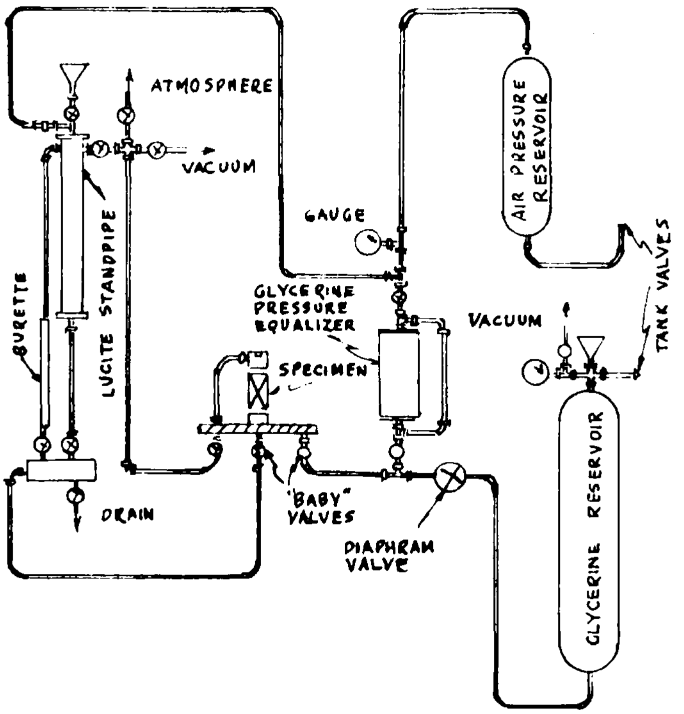

FIGURE 21. SCHEMATIC PIPING DIAGRAM FOR THE TRIAXIAL COMPRESSION TESTING APPARATUS OF FIGURE 22

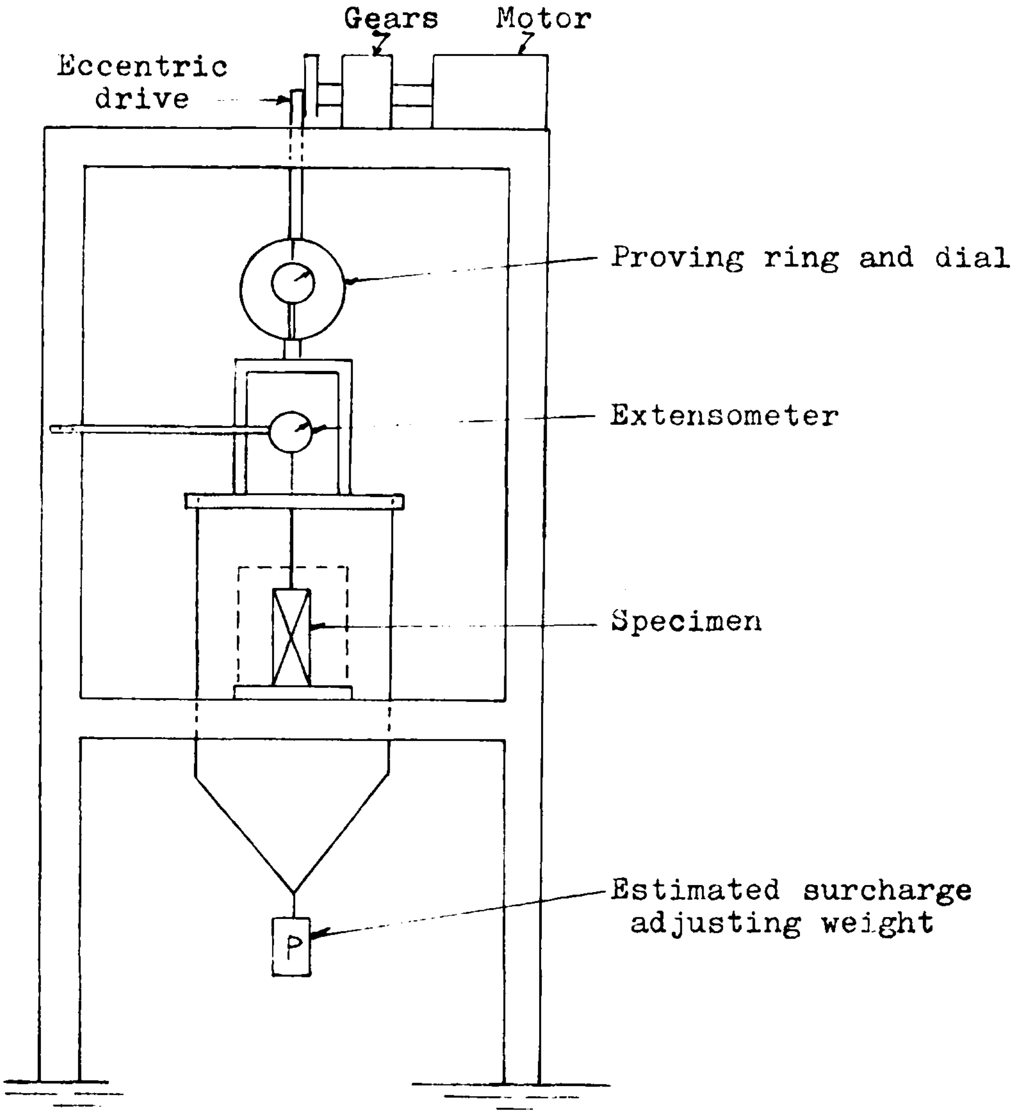

FIGURE 22. SKETCH OF TRIAXIAL TESTING MACHINE EXCEPT PIPING DIAGRAM OF FIGURE 21

The unit is designed to provide an axial load of 500 kg and a confining pressure of 10 kg per sq cm. Constant-strain tests and constant-stress tests can also be performed. A control panel contains all the necessary valves and controls for evacuating, saturating, and measuring the volume change of saturated specimens. The line from the air-pressure cylinder permits saturation of clay specimens by placing the water entering the bottom of the specimen under a known pressure while the top of the specimen is subjected to a vacuum. In this manner, saturation under pressure, consolidation, and shear testing can be conducted without making or breaking a single connection.

Fatigue has been shown to be a function of strain in the surfacing. Since no direct solution for the relation between strain and load has been found, it has to be assumed that deflection of pavement surface is a measure of strain, and it is necessary to use Hveem's maximum permissible deflections as an index of endurance strength of surfaces. Using an endurance limit of $2(10)^6$ repetitions and the thickness-EWL's relation obtained[19] by Drake and Havens converted to a non-dimensional thickness factor, designs for greater or lesser amounts of traffic could be determined, see Figure 23.

The assumption that permissible deflections, suggested[8] by Hveem, produce strains equal to the endurance limit at $2(10)^6$ repetitions on Drake and Havens thickness - log EWL's curve[19] is an empirical approach that seems unavoidable with present knowledge.

It is not recommended that this procedure be put into practice until considerable checking against methods

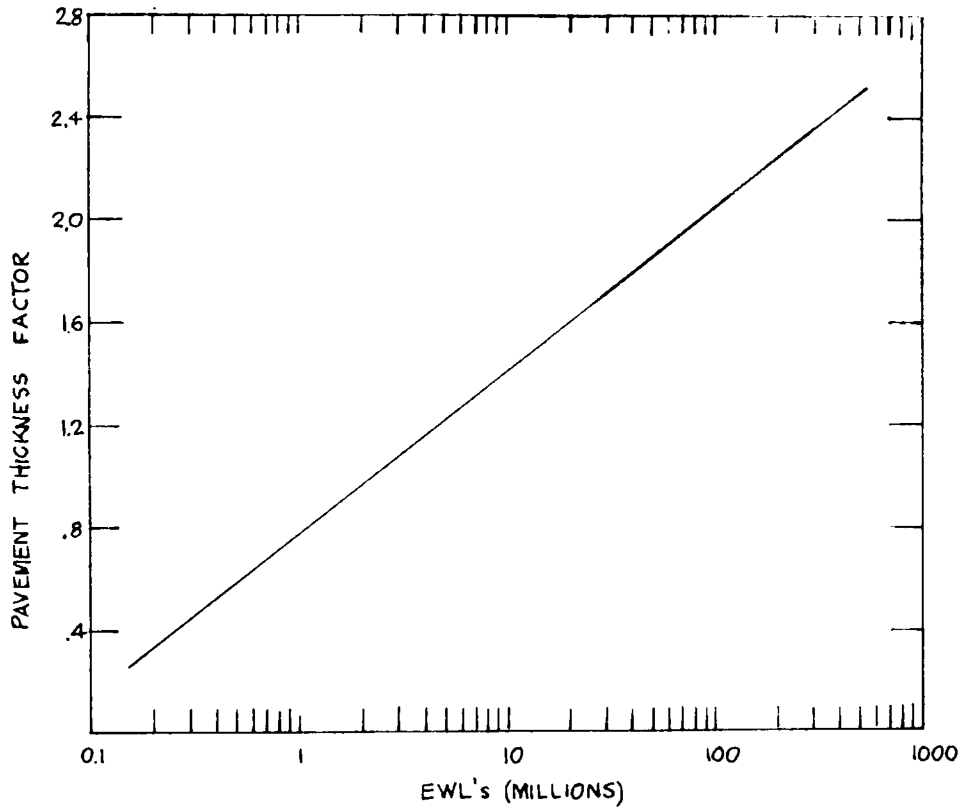

FIGURE 23. ADJUSTMENT OF DESIGN THICKNESS TO GREATER OR LESS TRAFFIC

presently being used has been done; it is suggested as the best approach that can be made at this time toward a completely rational solution.

APPENDIX A: DETERMINATION OF CBR FOR REMOLDED SPECIMENS, AN ABSTRACT FROM SOILS MANUAL FOR DESIGN OF ASPHALT PAVEMENT STRUCTURES, THE ASPHALT INSTITUTE, 1963

The CBR value for a soil will depend upon its density, molding moisture content, and moisture content after soaking. Since the product of laboratory compaction should closely represent the results of field compaction, the first two of these variables must be carefully controlled during the preparation of laboratory samples for testing. Unless it can be ascertained that the soil being tested will not accumulate moisture and be affected by it in the field after construction, the CBR tests should be performed on soaked samples.

Equipment

The equipment and materials required for determining the CBR value of a soil consists of the items described as follows:

a. Cylindrical Mold--6 in inside diameter, 7 in deep provided with a collar extension about 2 in length. A perforated base plate with perforations not greater than 1/16 in diameter is required for specimen preparation. A base plate without perforations is employed for compaction control tests.

The base plate and collar should be made to clamp on either end of the mold. For any group of molds, one extra base plate is desirable since two plates are required when a mold is inverted during specimen preparation.

b. Spacer Disc--metal, 5 15/16 in diameter x 2½ in high.

c. Compaction Hammer--sliding weight or sleeve type, 2 in diameter steel tamping foot, 10 lb weight with an 18-in fall.

d. Sieves--3/4 in and a No. 4.

e. Expansion Apparatus--adjustable stem and perforated plate, tripod, and dial micrometer (reading to 0.001 in) suitable for measuring the expansion of the soil.

f. Weights--one annular and several split 5 lb weights, 5 7/8 in outside diameter and 2 1/8 in inside diameter, suitable to apply as surcharge loads on soil surface during soaking and penetration.

g. Penetration Piston--1.95 in diameter face and sufficiently long to pass through the surcharge weights and penetrate the soil.

h. Loading Device--laboratory testing machine or screw jacks and frame arrangement which can be used to force the penetration piston into the soil specimen at a uniform rate of 0.05 in per minute.

i. Coarse filter paper, wire screen, and cellophane.

j. Equipment for conducting routine soil tests.

k. Miscellaneous Apparatus--mixing bowls, dial micrometers, spatulas, straightedges, trowels, knives, spoons, scales, soaking tank, ovens, moisture content cans or boxes and stop watch.

Soil Preparation

a. Air-dry the total sample until it becomes friable under a trowel. Approximately 75 lbs of material passing the 3/4 in screen will be required.

b. Break up soil aggregations, being careful to avoid reducing the natural size of the individual particles.

c. Separate the sample into three fractions over the 3/4 in and No. 4 sieves.

d. Discard all material retained on the 3/4-in sieve, and replace it with an equal portion of original material passing the 3/4-in sieve and retained on the No. 4 sieve.

e. Recombine and thoroughly mix the sample.

f. Place prepared soil sample in a moisture proof container.

g. Oven-dry approximately 500 grams of the material just prepared, to determine its moisture content.

Procedure For Conducting The Compaction Control Test

The compaction control test used essentially corresponds to AASHO Method D, Designation T 180. Modifications made are as follows:

a. The mold shall be as specified above.

b. The compaction hammer shall be of the sliding weight type. It shall consist of a two-inch diameter steel tamping foot, a 5/8-in steel rod, a weight with an 11/16-in hole through the center, and a handle. Construction of the tamping foot and weight shall be such that tamping blows can be applied adjacent to the sides of the mold. The rod shall be attached to the tamping foot with a spring cushion. The maximum allowable weight of the assembled compaction hammer is 17½ pounds.

c. The preparation of the soil shall be as described above.

d. No material shall be re-used, and a separate batch shall be used for each compaction test specimen.

e. The desired amount of mixing water for each test specimen shall be added, mixed well, and the material placed in a container with an airtight cover and allowed to cure for 24 hours. The water content should be redetermined if appreciable condensation occurs on the container wall.

f. Clamp the mold and detachable collar extension to the base plate, insert the spacer disc, and place a coarse filter paper on top of the disc.

g. Compact the specimen in five one-inch layers, each layer receiving the required number of blows of the specified tamper or hammer.

h. For cohesive materials, the water content tested shall range from below to above the estimated optimum; for cohesionless materials, the water content shall range from air-dried to as high as practicable.

i. Modifications to the above procedures may be made when check tests on the specified materials show that the modifications do not affect the results.

j. Place the mold assembly on a concrete floor or pedestal during compaction.

k. Compact a sufficient number of test specimens over a range of water contents to definitely establish the optimum water content and the maximum density. If compaction characteristics of the material are fairly well known, four or five specimens compacted at water contents within the range of ± 2% of optimum water content are usually enough to establish the optimum water content and the maximum density.

l. Plot the test results in the form of a moisture-density diagram, and draw a smooth curve through the points.

Preparation of Test Specimens

a. Assemble the 6-in mold, extension collar, and perforated base plate by clamping the mold with fitted extension collar to the base plate.

b. Insert the spacer disc over the base plate, and place a 6-in diameter coarse filter paper on top of the disc.

c. Compact samples using compacting efforts and molding water content as indicated below. Specimens are usually compacted at several moisture contents and densities to cover the anticipated range that will be experienced in the field. In preparing remolded specimens for the CBR method of design, all subgrades and base courses have been grouped into three classes with respect to behavior during saturation: (a) cohesionless sands and gravels, (b) cohesive soils, and (c) highly swelling soils. The first group usually includes the GW, GP, SW, and SP classifications. Swelling soils usually comprise the MH, CH, and OH classifications. Separate procedures for sample preparations are given for each of the groups.

c-1. Cohesionless sands and gravels. Cohesionless soils usually compact readily under rollers or traffic, and specimens should be prepared at high densities and at a range of water content covering those anticipated in the field and including water contents as high as practicable. Using predetermined and plotted moisture-density relationship, compact samples at optimum moisture content, and on the wet and dry side of optimum. Molding moisture content may be obtained by drying a portion of the sample (100 grams for clays and 500 grams for gravelly soils) at the time the specimen is compacted. Soaking may be omitted in subsequent test on the same material if it does not lower the CBR. Usually the lowest CBR value obtained from this series of specimens is used as the design CBR.

c-2. Cohesive Soils. Soils in this group are compacted and tested in a manner to develop data that will show their behavior over the entire range of anticipated moisture contents for representative samples. Compaction procedures are similar to those outlined above, except that compaction curves are developed for 55, 26, and 12 blows per layer and each specimen is soaked and penetrated to develop a complete family of curves showing the relationship between density, water content, and CBR. To aid in determining the validity of the compaction data, a semilog plot of maximum density versus compaction effort in work per unit volume usually gives a straight-line relationship. The data from a CBR test are plotted as shown in Figure 7, and the resulting family of CBR curves represents the characteristics encompassing a wide range of field conditions. The design CBR should be based on the density and molding-moisture content anticipated in the field. For example, assume that the lean clay soil, for which results are plotted in Figure 7, can be processed to an average moisture content from 13 to 16 percent and that it can be compacted to a density varying from 110.5 (95 percent of AASHO T 180 Method D maximum density) to 115 pounds per cubic foot (see cross-hatched area on left plot of page 8. If construction could be controlled so that the density and moisture were within these ranges, the right-hand plot would indicate that the soil, after moisture conditions had become adjusted, would have a CBR (see crosshatched area on right plot) varying between about 11 (110.5 pounds per cubic foot and 15 percent moisture). The design CBR would be selected near the lower figure, say 12 percent. The right-hand plot shows that close control of moisture content within these limits (13 to 16 percent) is necessary because low CBR values will be obtained if the moisture content during rolling is allowed to increase appreciably above the desired range. For example, if the moisture content is allowed to increase one percent to 17 percent, the right-hand plot indicates the CBR (at 95 percent AASHO T 180 Method D maximum density) would decrease from 26 to 8. Another percent increase in moisture would result in a CBR of about 3.

c-3. Swelling Soils. The sample preparation procedures for highly swelling soils are the same as previously described for cohesive soils; however, the objectives of the testing program are not exactly the same as for cohesive soils. Tests are performed on soils with expansive characteristics to determine a moisture content and a unit weight which will **minimize** expansion. The proper moisture content and unit weight are not necessarily the optimum moisture content and unit weight determined by AASHO T 180 Method D compaction tests. Generally, the minimum swell and highest soaked CBR will occur at a molding moisture content slightly wetter than optimum. It may be necessary, when testing highly swelling soils, to prepare samples for a wide range of moisture contents in order to establish the relationship between moisture content, density, swell, and CBR for a given soil.

Moisture, density, and CBR data should be plotted as shown on Figure 7 just as for cohesive soils. In addition, percent of swell should be plotted versus molding water content for the various compaction efforts in the same way that CBR and density are shown in the plots on the left of Figure 7. A comparison of the plots of swell, CBR, and density versus molding water content will permit selection of specification limits for moisture and density. This also will permit the limitation of swell and, at the same time, give the greatest values of CBR and density which might reasonably be obtained. Then just as for cohesive soils, design CBR and density values would be selected toward the bottom of the ranges in CBR and density, consistent with the specification limits selected.

Where it is desirable to limit swell by the addition of overburden load, tests will have to be conducted to determine the amount of load necessary. These tests can consist of additional specimens, prepared for soaked CBR tests, using various added amounts of surcharge during soaking. The amount of surcharge required to limit swell to a permissible amount then can be used to compute the needed thickness of overburden. The same result may be accomplished with fewer specimens by restricting the swell during soaking and measuring the pressure developed.

d. After each sample has been compacted in the mold, remove the extension ring; strike off excess soil with a straightedge; remove the base plate; and extract the spacer disc.

e. Weigh the mold and compacted soil to determine the density of soil.

f. Place filter paper on the base plate; invert the cylinder so that the bottom during compacting is now on top; re-attach to the base plate; and place filter paper on top of soil in mold.

g. Place the perforated aluminum plate, with adjustable stem attached, on the filter paper.

h. Place surcharge weights on the aluminum plate to produce an intensity of surcharge loading equal to the weight of the base material and pavement within ± 5 pounds, but not less than 10 pounds.

i. Immerse the mold and weights in water to within ½ inch of the top of the mold. Place blocks under the mold to allow free access of water to the bottom of the specimen, and put water inside the mold to the same level as water on the outside of the mold.

j. After immersion, measure the height of the stem or spindle above the edge of the mold with the dial micrometer and tripod assembly. This is the initial measurement for swell.

k. Allow the specimen to soak for four days, maintaining constant water level outside and inside the mold.

l. Repeat step j to obtain the final swell measurement. Compute the swell as a percentage of the initial specimen height.

m. Remove the mold from the water, and pour off free water from inside the mold, being careful not to disturb the soil.

n. Remove the surcharge weights, perforated plate, and filter paper, and allow the specimen to drain for 15 minutes.

o. Weigh the specimen to determine the soil density. The specimen is then ready for the penetration test.

Procedure for Penetration Testing

a. Place one 5-lb annular disc surcharge weight on the soil surface.

b. Place the mold in the loading frame or hydraulic press, and adjust its position until the piston is centered on the specimen.

c. Seat the penetration piston with a 10-lb load, and set both the load dial and strain dials to zero. This initial load is required to insure satisfactory seating of the piston and should be considered as the zero load when determining stress-penetration relations.

d. Add penetration surcharge weights to produce an intensity of loading equal to the weight of the base material and pavement (within ± 5 lb), but not less than 10 lb. If the sample has been previously soaked, the surcharge should be equal to the soaking surcharge.

e. Apply the load to the piston at a uniform rate of 0.05 in of penetration per minute.

f. Record the total load readings at 0.025, 0.050, 0.075, 0.100, 0.125, 0.150, 0.175, 0.200, 0.250, and 0.300 in penetration.

g. Release the load; remove the mold from the loading device; remove the weights; and detach the base plate.

h. For laboratory tests, determine the average moisture content for the entire depth of the sample. For field tests, take a sample of soil from the top inch for moisture content.

i. Remove and discard the remaining soil.

j. From the loads obtained in f, the CBR of the sample is determined, as illustrated below.

Stress-Strain Curve

After the test has been completed, the penetration unit load in psi is calculated and the stress-strain curve plotted on cross-section paper. In order to obtain true penetration loads from the test data, the zero point of the curve is adjusted to correct for surface irregularities and the initial concave upward shape of the curve if it is present. If the curve is uniform as in example No. 1 of Figure 24, the CBR value is calculated from the recorded loads. For surface irregularities as in example No. 3 of Figure 24 extend the straight line portion of the curve to the base to obtain a corrected origin, or zero. If the curve has a reverse bend, or concave upward shape, as in example No. 2, draw a line tangent to the steepest point of the curve (point A), and extend the line to the base to obtain a corrected origin or zero point (point B). Then read the corrected load values for 0.1-in penetration (point C) and 0.2-in penetration (point D).

Calculation of California Bearing Ratio

The CBR value is defined as a ratio comparing the bearing of a material with the bearing of a well-graded crushed stone. The penetration loads for crushed stone are presented in the following table:

Penetration in Inches	Standard Load (lbs.)	Standard Load (PSI)
0.1	3000	1000
0.2	4500	1500
0.3	5700	1900
0.4	6900	2300
0.5	7800	2600

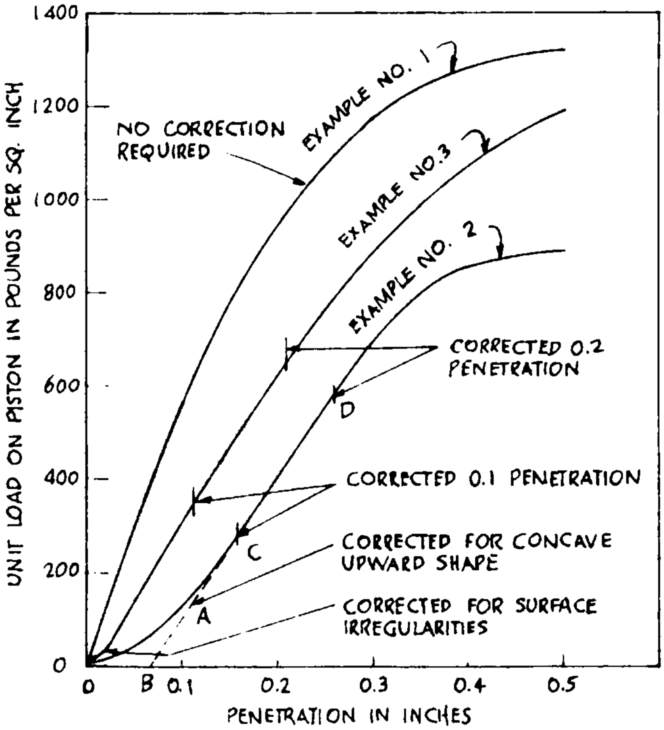

FIGURE 24. CORRECTION OF STRESS - PENETRATION CURVES

The corrected load values, obtained as prescribed above, are determined at 0.1-in and 0.2-in penetration from which the CBR values are determined by use of the following formula:

$$\text{CBR}\ (\%) = 100\ (x/y) \qquad x = aD/3$$

where, x = soil resistance or the unit load on the piston, psi. (for 0.1 in penetration interval)

y = standard unit load, psi.

a = value of one dial division, lbs

D = actual dial reading

The CBR is determined from the corrected load values at 0.1- and 0.2-in penetrations by dividing the loads at 0.1 and 0.2 in by the standard loads of 1000 and 1500 pounds per square inch, respectively. Each ratio is multiplied by 100 to obtain the CBR in percent. The CBR is usually selected at 0.1-in penetration. If the CBR at 0.2-in penetration is greater, the test should be re-run. If check tests give similar results, the CBR at 0.2-in penetration should be used.

APPENDIX B: DETAILED PROCEDURES FOR TRAFFIC ANALYSIS, AN ABSTRACT FROM THICKNESS DESIGN - ASPHALT PAVEMENT STRUCTURES FOR HIGHWAYS AND STREETS, THE ASPHALT INSTITUTE, 1963

The procedure requires the use of detailed traffic survey data, including loadometer studies, to establish the Design Traffic Number.

Definitions

Terms applicable to traffic analysis are defined as follows:

a. Design Lane--The lane on which the greatest number of equivalent 18,000-lb single-axle loads is expected.

b. Design Period--The number of years until the first major resurfacing is anticipated.

c. Design Period Adjustment Factor--A factor used to adjust the 20-year Design Period to lesser or greater lengths of time.

d. Design Traffic Number--The average daily number of equivalent 18,000-lb single-axle load applications expected for the Design Lane during the Design Period.

e. Equivalent 18,000-lb Single Axle Load--The effect on pavement performance of one or more axle loads of any magnitude equated to the number of 18,000-lb single-axle loads required to produce the same effect.

f. Load Equivalency Factor--The ratio of repetitions of any given axle load to an equivalent number of repetitions of an 18,000-lb single-axle load.

g. Loadometer Study--A study in which the following factors are determined:

(1) Weight carried on each axle

(2) Number of axles

(3) Type of truck

h. Traffic.

(1) Light--Traffic conditions resulting in a Design Traffic Number of less than ten (10).

(2) Medium--Traffic conditions resulting in a Design Traffic Number between ten (10) and one hundred (100).

(3) Heavy--Traffic conditions resulting in a Design Traffic Number of one hundred (100) or more.

i. Traffic Count--A traffic count is a determination of the average number of all types of vehicles using a facility within a given period of time. Traffic count data are usually expressed in terms of Average Daily Traffic (ADT), in units of vehicles per day, but are not distinguished as to direction or number of lanes.

j. Traffic Classification Survey--A survey in which vehicles are counted by general types, such as:

(1) Cars

(2) Light trucks (panel, pickup and light delivery)

(3) Heavy trucks

k. Truck Factor--The average number of equivalent 18,000-lb single-axle loads per truck when all types of trucks are considered.

Traffic Data

Obviously, no traffic data are available which are directly applicable to a new pavement facility. The designer must therefore depend upon traffic studies of similar facilities, and community or regional planning studies, to provide the information needed for a detailed traffic analysis.

Traffic Growth

Pavements must be designed to serve adequately traffic needs over a period of years. Traffic growth must therefore be anticipated in determining structural requirements of the pavement. Traffic growth for comparable facilities, as well as community and regional planning programs, provides a basis for the estimate. Experience has indicated an increase of 50 to 100 percent, uniformly over about a 20-year period. For these conditions, the Traffic Growth Factor for the average increase is 1.25 and 1.50 respectively.

Design Period

A pavement may be designed to support the cumulative effects of traffic for any period of time. The selected period, in years, for which the pavement is to be designed is termed the Design Period. Thickness design curves are based on a Design Period of 20 years. For a different Design Period the Design Traffic Number is adjusted. A Design Period Adjustment Factor for Design Periods other than 20 years and an Adjusted Design Traffic Number may be computed as follows:

a. Design Period Adjustment Factor = 0.05 X desired period in years

b. Adjusted Design Traffic Number = Design Period Adjustment Factor X Design Traffic Number (20 years)

Cumulative Effects of Traffic

Both the number of vehicles and the weight on each wheel of each vehicle affect the structural requirements of the pavement facility. These occur in an infinite number of combinations so for pavement design purposes, the cumulative effects of these factors must be reduced to some "common denominator."

Load Equivalency Factor and Truck Factor

A Load Equivalency Factor is used to convert a single or tandem axle load of given magnitude to an Equivalent 18,000-lb Single-Axle Load. The Truck Factor, the average number of equivalent 18,000-lb single-axle loads per truck, is used to compute the Design Traffic Number.

Figure 25 provides Load Equivalency Factors for single-axle loads of 10,000-lb and above and for tandem-axle loads of 17,000-lb and above. Where significant volumes of heavily loaded trucks are anticipated, the effects of automobile and light truck loadings on thickness design requirements are of little or no significance and often may be disregarded for design purposes.

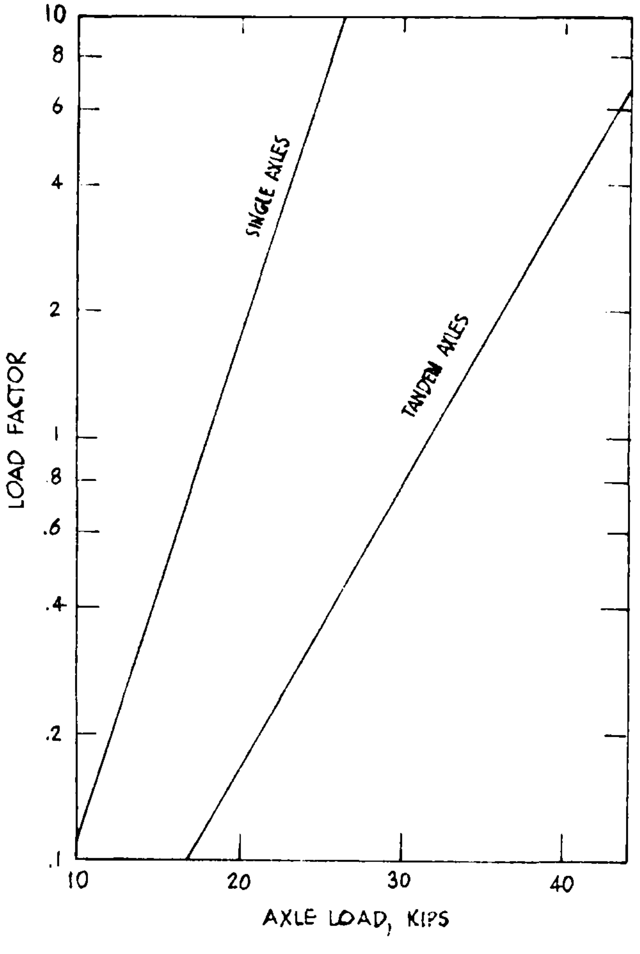

FIGURE 25. LOAD EQUIVALENCY FACTOR FOR LOADS EQUAL TO OR GREATER THAN 10,000-lb

For convenience, axle loads are grouped. The Load Equivalency Factor for the average axle load of each group is determined from Figure 25. The Axles Per Day, Per 1000 Trucks and Combinations, are obtained from traffic survey. The number of Equivalent 18,000-lb Single-Axle Loads is the Truck Factor.

If the Truck Factor is less than 0.05, Truck Factor of 0.05 should be used as a practical minimum value.

Design Lane

For two-lane streets and highways, the Design Lane may be either lane of the pavement facility. Under some conditions, more heavily loaded trucks may be anticipated in one direction than in the other. This then becomes the Design Lane for two-lane pavements.

Determining the Design Traffic Number

A Design Traffic Number for the anticipated volume of trucks is determined as follows:

a. Establish the daily number of all trucks, in both directions, initially expected to use the pavement.

b. Establish the Traffic Growth Factor.

c. Establish the Design Period Adjustment Factor.

d. Calculate the Load Equivalency Factor per truck, or Truck Factor.

e. Determine the decimal equivalent of the percent of trucks using the Design Lane.

f. The Design Traffic Number, based on truck traffic only, is then the product of all factors in Steps a through f above.

If the Design Traffic Number established in Step f above is 10 or more (Medium or Heavy Traffic) the relative effects of light axle loads on pavement thickness design requirements will be negligible and may be disregarded. If this Design Traffic Number is less than 10 (Light Traffic), it must be adjusted to compensate for the effects of the light axle loads on thickness design requirements.

Adjusting the Design Traffic Number for Light Traffic Conditions

Figure 26 provides a means for adjusting Design Traffic Numbers of less than 10, light traffic conditions.

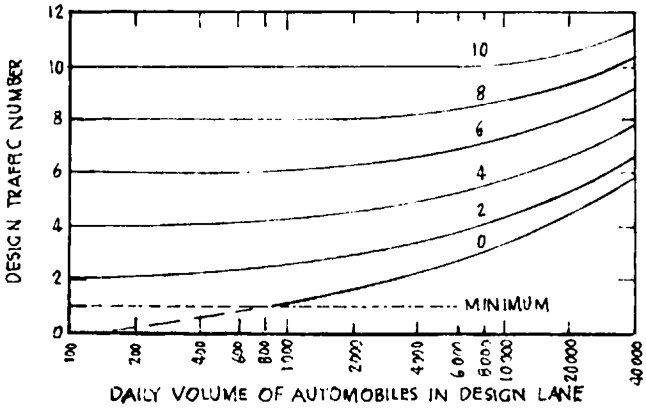

FIGURE 26. CHART FOR ADJUSTING TRAFFIC NUMBER FOR DAILY VOLUME OF AUTOMOBILES.

APPENDIX C: THICKNESS DESIGN OF PAVEMENT STRUCTURES, AN ABSTRACT FROM <u>THICKNESS DESIGN - ASPHALT PAVEMENT STRUCTURES FOR HIGHWAYS AND STREETS</u>, THE ASPHALT INSTITUTE, 1963

The Design Traffic Number and CBR value of the subgrade from Appendices A and B are the basic data needed for thickness determinations. Figure 27 is the basic design chart to determine total thickness of pavement structure and the thickness of all components. T_A is the thickness required of asphalt concrete; alternate thickness designs using base and subbase may be derived by using substitution ratios as follows: 2 in base for 1 in asphalt concrete; 2.7 in subbase for 1 in asphalt concrete; 1.35 in subbase for 1 in base. Alternate designs, considering different combinations, should be prepared to select the most economical pavement structure.

The A-line on the chart is used to establish minimum thickness of asphalt concrete surface and asphalt concrete base. The B-line determines if subbase can be used, and is used to determine thickness of each. Thickness of surface course must not be less than 1 in for light traffic to 2 in for heavy traffic.

For full depth asphalt concrete pavement read the chart directly unless the CBR value is to the right of the A-line; in this case use the A-line in place of a vertical line from the CBR value.

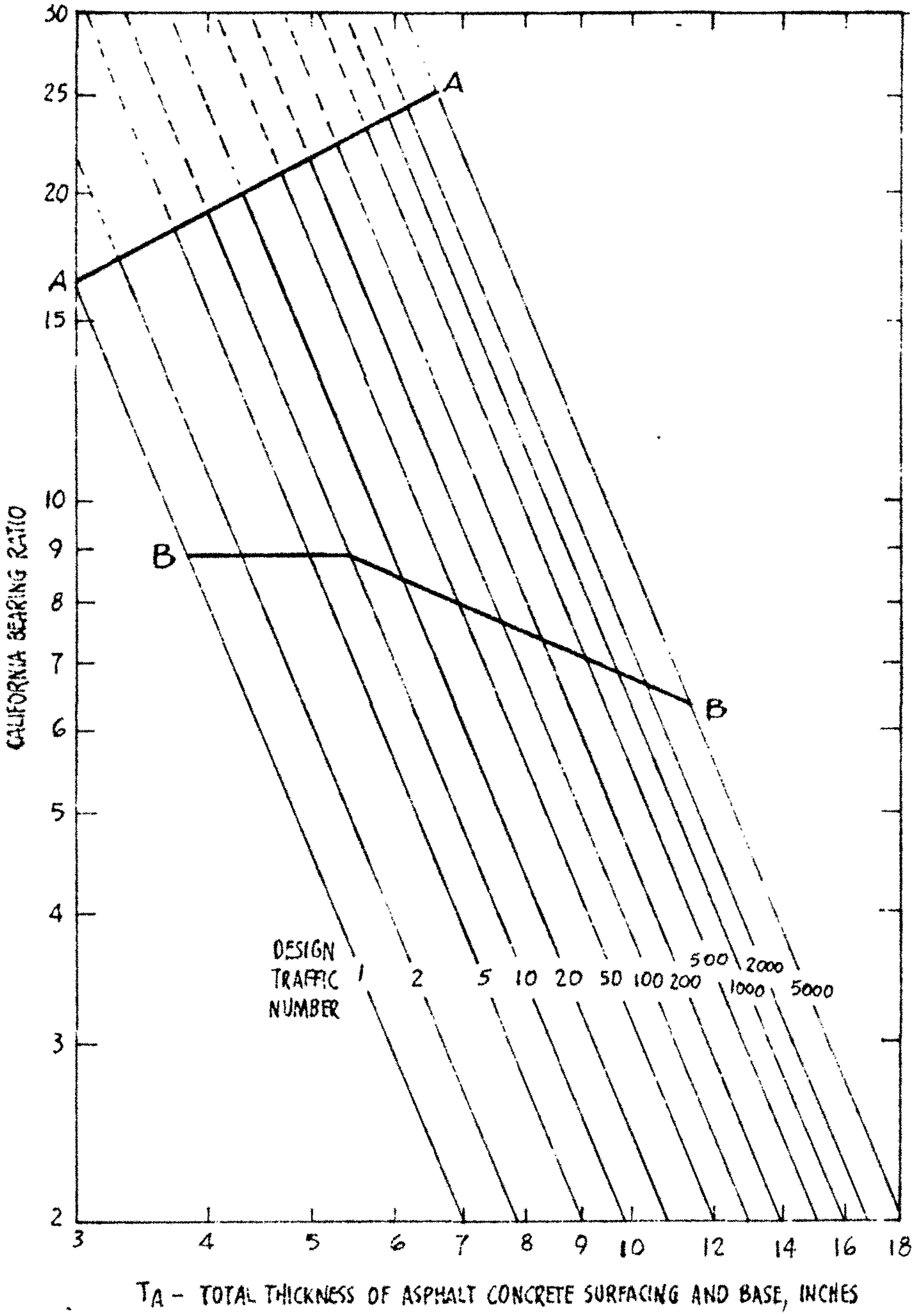

FIGURE 27. THICKNESS REQUIREMENTS FOR ASPHALT PAVEMENT STRUCTURES USING CBR VALUES

To substitute granular base for part of the asphalt concrete determine minimum asphalt surface and base from the intersection of Design Traffic Number and A-line; subtract this value from the thickness required for full depth asphalt concrete and multiply the remainder by 2 to determine the thickness of the granular base.

Asphalt concrete and subbase may be used if the CBR value and the Design Traffic Number intersect left of the B-line. In this case minimum asphalt concrete is determined from the intersection of the Design Traffic Number with the B-line; this value is subtracted from thickness required for full depth asphalt concrete and the remainder multiplied by 2.7 to determine the thickness of subbase.

Asphalt concrete surface, granular base and subbase may be used if the intersection of the CBR value with the Design Traffic Number line is left of the B-line. The minimum asphalt concrete with granular base is determined from the intersection of the Design Traffic Number line with the A-line; the minimum thickness of asphalt concrete if surface and base are of asphalt concrete is determined from the intersection of the Design Traffic Number line with the B-line; and the difference is the thickness of asphalt concrete for which granular base may be substituted at a 2:1 ratio. The difference between the required thickness of pavement if all asphalt concrete is used and that determined from the intersection of the Design Traffic Number line with the B-line is the thickness for which granular subbase may be substituted at a 2.7:1 ratio.

APPENDIX D: NOTATION

The following symbols have been adopted for use in this paper:

A	area of contact (between tire and pavement or between plate and ground)
a	equivalent radius of contact
C	per cent of area cracked
CBR	California Bearing Ratio
c	unit cohesion
D_1, D_2, D_3	thickness, in inches, of surfacing, base and subbase
E	modulus of elasticity
EWL	equivalent wheel loads
F	bearing capacity factor (subscript denotes type)
h	thickness of pavement
K	base course constant (McLeod)
k	modulus of subgrade reaction (McLeod)
L_1	single axle load, kips
L_2	0 for single axle or 1 for tandem
m	traffic coefficient (Kansas)
P	load (total), patching in per cent of area
P_h	transmitted horizontal pressure (stabilometer)
P_v	applied vertical pressure (stabilometer)

P/A	perimeter to area ratio (plate bearing test).
p	unit pressure, serviceability index
R	resistance value (stabilometer)
RD	average depth of ruts
S	stability value (stabilometer), subgrade support (McLeod)
s	unit shear strength
SV	slope variance
T	thickness (total)
t	thickness of a pavement component
TI	traffic index
W	applications of a given load
z	depth
γ	unit weight
δ	unit strain
Δ	total deflection
μ	Poisson's ratio
σ	direct stress (subscript denotes direction)
τ	shear stress (subscript denotes direction)
ϕ	angle of internal friction

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