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DESIGN CRITERIA FOR AGGREGATE-SURFACED ROADS AND AIRFIELDS

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by

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Failure criteria for aggregate-surfaced pavements subject to vehicular and aircraft loadings are developed. The criteria determine the required thickness making use of the layered elastic design procedure. A literature study was conducted to determine existing design procedures that might be applicable. Types of distresses in aggregate pavements were identified. Those basic parameters that influence performance were defined. Efforts were made to compare the existing design procedures using hypothetical design conditions, and the differences between the procedures were discussed. The technique of reliability analysis was applied on the design procedures. It was found that the reliabilities of some procedures were very low. Field tests on subgrade soils covered with high strength aggregates are recommended. The tests are necessary for the verification of developed failure criteria.

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PREFACE

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The study was conducted by the US Army Engineer Waterways Experiment Station (WES), Geotechnical Laboratory (GL), from 1986 to 1988 by Dr. Y. T. Chou, Pavement Systems Division (PSD). The work was under the general supervision of Dr. W. F. Marcuson III, Chief, GL, and Mr. H. H. Ulery, Jr., Chief, PSD. This report was written by Dr. Chou. Ms. Odell F. Allen, Information Products Division, Information Technology Laboratory, edited this report.

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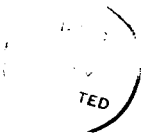

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENTS

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
inches	2.54	centimetres
kips (force)	4.448222	kilonewtons
pounds (force)	4.448222	newtons
pounds (force) per square inch	6.894757	kilopascals
pounds (mass)	0.4535924	kilograms
square inches	6.4516	square centimetres
tons (2,000 pounds, mass)	907.1847	kilograms

DESIGN CRITERIA FOR AGGREGATE-SURFACED ROADS AND AIRFIELDS

PART I: INTRODUCTION

Background

1. Although the bulk of the people in the United States living in cities, towns, and rural areas primarily travel on paved roads and streets, there are many miles of roads in the US that are still aggregate-surfaced. These aggregate roads provide access to farm lands, logging haul roads, and remote area military installations. The aggregate roads generally have relatively low traffic. Some military aircraft such as the C-5 and C-130 are required to land on soft unpaved soil. In the US Army Corps of Engineers (CE), the existing design procedure for aggregate-surfaced roads and airfields is based on modified flexible pavement design criteria.

2. The structural design procedures for both rigid and flexible airfield pavements using the layered elastic method have been developed at the US Army Engineer Waterways Experiment Station (WES) (Headquarters, Departments of the Army and the Air Force, in preparation) in recent years. The advantages of these new procedures are that they permit consideration of many design aspects that are ignored or approximated in other procedures (Headquarters, Department of the Army 1968, 1970, and 1978; Department of the Navy 1973). This report presents design procedures for aggregate-surfaced roadways and airfields using the layered elastic method and other rational procedures which are different from the currently available procedure (Headquarters, Department of the Army 1985).

Purpose and Scope

3. The purpose of this study was to develop a new, practical, and implementable procedure for the structural design of aggregate-surfaced pavements for roads and airfields. This report presents several design procedures currently available and discusses the limitations of these procedures. Three design examples using various procedures are presented, and the

computed results are discussed. The study is limited to the structural design of the pavement section, i.e., the selection of required thickness of gravel layer to carry the design traffic under field conditions. Failure criteria based on layered elastic analysis were developed and are proposed for the design of aggregate-surfaced pavements for both roads and airfields. Miner's rule of linear cumulative damage was used to evaluate pavement performance for various loading groups and, if necessary, in different seasonal conditions. Design examples are given illustrating the use of the proposed design procedure. Requirements for materials, construction, and maintenance are also presented.

Definitions of Terms

4. Several specific terms used in this report are defined below for the reader's convenience:

a. Aggregate-surfaced roads--Unpaved road that has an unbound aggregate material as the surface course.

b. Distress--An undesirable condition of an aggregate-surfaced road, such as rutting and corrugations.

c. Roughness--The riding quality of an aggregate-surfaced pavement. Roughness is used to evaluate the severity levels for corrugations, rutting, and soft spot distress types.

d. Serviceability--The ability of a pavement to serve its intended function at any particular time.

e. Present serviceability index (PSI)--An index number used in the American Association of State Highway and Transportation Officials (AASHTO) road tests that indicates how well a pavement serves the purpose for which it was designed at a given point in time. The PSI ranges from 0.0 (very poor) to 5.0 (excellent) and is related primarily to the roughness of the pavement surface.

f. Terminal serviceability index (TSI)--Value of the present PSI chosen by the designer to be the "failure" level of performance.

g. Rehabilitation--Major restoration after an aggregate pavement reaches the terminal level of PSI. Rehabilitation manually improves the condition of the structure to near the PSI level to which it was originally constructed. The rehabilitation usually consists of placing a layer of aggregate to smooth out the surface and replacing the aggregate loss due to traffic and erosion and is not considered an addition to the structural thickness. Rehabilitation can include realignment, subgrade improvement, and upgrading of the structural capabilities of the pavement structure.

h. Routine maintenance--Actions taken such as grading, blading, patching potholes, cleaning drainage structures, and cutting vegetation to improve the condition of a road or airfield.

i. Periodic maintenance--Extensive operations such as resurfacing with gravel that is required only once every several years to extend the life of a road or airfield.

j. Geotextile--A textile (fabric) used in geotechnical engineering. Other terms used synonymously with geotextile are filter fabric, filter cloth, geotechnical fabric, engineering fabric, civil engineering fabric, and geofabric. In general, membrane is impermeable and fabric is permeable.

PART II: TYPES OF DISTRESSES

5. Successful design and maintenance programs for aggregate-surfaced pavements require a knowledge of types and causes of distress. Studies were conducted at the US Army Engineer Cold Regions Research and Engineering Laboratory (CRREL) to identify and evaluate distresses in unsurfaced roads (Eaton 1985). These studies also developed a pavement condition index (PCI) System in connection with the PAVER Maintenance Management System (Shahin and Kohn 1981). In the study the major types of distress in an unsurfaced road are listed below:

a. Roughness.

b. Dust.

c. Loose aggregate - loss of aggregate.

d. Corrugation.

e. Potholes.

f. Rutting.

g. Loss of crown or surface distortion.

h. Water/drainage damage.

Table 1 presents the distress types for unsurfaced roads according to possible causes as load associated, poor construction, inadequate maintenance practices, and drainage problem. Table 2 presents possible maintenance actions which can be used to correct specific drainage problems, and Table 3 presents suggested possible maintenance actions for each distress type and severity level. Each of the three tables were provided by Eaton (1985).

Table 1

Unsurfaced Road Distress Types by Possible Causes

Distress Type	Probable Causes			
	Volume or Weight Overload	Water/ Drainage	Construction	Maintenance
Corrugations	X			X
Loose aggregate/dust	X	X		X
Potholes	X	X		X
Rutting	X			X
Soft spots, surface heaving, settlement		X	X	

Table 2

Drainage Repair Strategies/Corrective Actions

Drainage Problems	Corrective Action(s)
Ponding	Add material; install culvert and raise grade; provide water turnout/easement; install driveway culverts; Beaver Dam/man-made pond removal
Lack of parallel flow	Clean out vegetation (clean and shape); provide water turnout/easement; install driveway culverts
Erosion	Watershed changes; increase culvert size-elliptical is preferable (squash pipe); stone line ditch (crushed or cobbles)
Trapped road surface water	Reestablish profile of cross section by grading
Mudslides/sloughing of cut slopes	Rock stabilization

Table 3

Unsurfaced Road Repair Strategies/Corrective Action

	Severity Level*		
Distress Type	Low	Medium	High
Corrugations	1	2	2,3
Loose aggregate/dust	1	2	2
Potholes	1	2	3
Rutting	1	2	3,4
Soft spots, surface heaving/settlement	1	3	3,4

* 1 = do nothing, 2 = grade/shape/dust palliative, 3 = add select material/ regrade, and 4 = reconstruct.

6. In a study sponsored by the National Research Council on the structural design of low-volume roads (Transportation Research Board 198?), distress factors leading to structural or functional failure conditions and their effects are tabulated in Table 4. It indicates that many distress factors may act on the gravel surface to increase roughness to a level of functional failure. As a result, an overall design philosophy is to protect against excessive rutting due to shear displacements.

Table 4

Major Distress Types of Low-Volume Roads

Distress Factor	Effect on . . . . .
Dusting	Safety, environment
Surface looseness	Safety, roughness
Gravel loss	Structural deformation, roughness
Surface deformations	Structural deformation, roughness
Shear displacements	
Layer material densification	
Layer material intrusion	
Surface heaving	Roughness*
Frost heave	
Expansive clays	
Corrugations (washboarding)	Roughness
Surface erosion (gulleying)	Roughness
Potholes	Roughness

* Greatly increased if surface profile changes are highly variable in the longitudinal direction.

PART III: PERFORMANCE FACTORS

General

7. Factors affecting design of pavements are traffic, climate, materials, and terrain. For aggregate (unpaved) roads and airfields, the most important factor is climate. The important climatic factors are temperature, rainfall, freezing index, elevation, etc. For aggregate roads and airfields, water can enter the subgrade through the aggregate. Saturated subgrade under heavy traffic loads will rut easily. Strong and durable aggregates can increase and prolong the service life of a gravel pavement. Aggregate requirements are discussed in Part IV of this report.

8. Meyer, Vazquez, and Hicks (1982) reviewed many factors related to climate, and they concluded that temperature and moisture content are the most significant factors affecting the design of low-volume aggregate roads and airfields. They found that dry-hot regions have significant surface wear and deterioration, but cold-wet regions have weak subgrade due to high moisture content. Thus, the main failure mode on these climates is rutting. Excessive moisture in the pavement can cause potholes and surface erosion. Freeze-thaw cycling in cold-wet regions produces detrimental frost heave in the pavement. In general, pavement engineers agree that wet climates are more critical than dry climates, mountainous terrain is more critical than plains, and freeze-thaw climates are more critical than mild climates. Except in cases where other criteria are specifically established, gravel pavements should be designed so that there will be no interruption of traffic at any time due to differential heave or to reduction in load-supporting capacity. Pavements should also be designed so that the rate of deterioration during critical periods of thaw weakening will not be so high that the useful life of the pavements will be less than that assumed as the design objective.

Frost Considerations

9. The detrimental effects of frost action in subsurface materials of gravel pavements are manifested by nonuniform heave of pavements during the winter and by loss of strength of affected soils during the thaw period. Frost-related problems also include possible loss of compaction, development

of permanent roughness, restriction of drainage by the frozen strata, and excessive maintenance requirements. The conditions necessary to cause frost problems are susceptible soil, temperature, and water, and they must be present simultaneously for significant ice segregation to occur in subgrade materials. Therefore, the cold-wet climates are likely to have the most serious problem with frost penetration.

10. The CE (Berg and Johnson 1983) conducted extensive research on the effects of frost penetration on pavement design and performance. For frost design purposes, soils are divided into eight groups, as shown in Table 5, based on percent of 0.02 mm grain size material (Unified Soil Classification System). The first four groups are generally suitable for base materials, and any of the eight groups may be encountered as subgrade soils. Soils are listed in approximate order of decreasing bearing capacity during periods of thaw and increasing order of susceptibility to frost heave, although the low coefficients of permeability of most clays restrict their heaving propensity. More detailed descriptions of the frost-susceptible soils (F1, F2, F3, and F4) are presented in Table 6.

Pavement Design for Frost-Susceptible Soils

11. The CE (Berg and Johnson 1983) developed two methods for determining the thickness design of a pavement that will have adequate resistance to distortion by frost heave and cracking and distortion under traffic loads as affected by seasonal variation in subgrade support. The first method requires a sufficient thickness of pavement, base, and subbase to limit the penetration of frost into the frost-susceptible subgrade to an acceptable amount for the control of pavement distortion caused by frost heave. This is called the limited subgrade frost penetration method or the complete frost penetration prevention method. The second method does not seek to limit the penetration of frost into the subgrade, but to determine the thicknesses of pavement, base, and subbase that will adequately carry traffic loads over the design period of years, each of which includes one or more periods during which the subgrade supporting capacity is sharply reduced by frost melting. This procedure is called the reduced subgrade strength method. In most cases the choice of the design method is made in favor of the one that gives the lower cost. It is found that for paved pavements, complete frost penetration prevention is

Table 5

Frost Design Soil Classification

Frost Group	Kind of Soil	Percentage Finer Than 0.02 mm by Weight	Typical Soil Types Under Unified Soil Classification System
NFS*	Gravels Crushed stone Crushed rock	0-1.5	GW, GP
	Sands	0-3	SW, SP
PFS**	Gravels Crushed stone Crushed rock	1.5-3	GW, GP
	Sands	3-10	SW, SP
S1	Gravelly soils	3-6	GW, GP, GW-GM, GP-GM
S2	Sandy soils	3-6	SW, SP, SW-SM, SP-SM
F1	Gravelly soils	6 to 10	GM, GW-GM, GP-GM
F2	Gravelly soils	10 to 20	GM, GW-GM, GP-GM
	Sands	6 to 15	SM, SW-SM, SP-SM
F3	Gravelly soils	Over 20	GM, GC
	Sands, except very fine silty sands	Over 15	SM, SC
	Clays, PI > 12	--	CL, CH
F4	All silts	--	ML, MH
	Very fine silty sands	Over 15	SM
	Clays, PI < 12	--	CL, CL-ML
	Varved clays and other fine-grained, banded sediments	--	CL and ML CL, ML, and SM CL, CH, and ML CL, CH, ML and SM

* Nonfrost-susceptible.

** Possibly frost-susceptible, but requires laboratory test to determine frost design soil classification.

Table 6

Descriptions of Frost-Susceptible Subgrade Soils

Group	Description
F1	Gravelly soils containing between 3 and 10 percent finer than 0.02 mm by weight
F2	Gravelly soils containing between 10 and 20 percent finer than 0.02 mm by weight; sands containing between 3 and 15 percent finer than 0.02 mm by weight
F3	Gravelly soils containing more than 20 percent finer than 0.02 mm by weight; sands, except very fine silty sands, containing more than 15 percent finer than 0.02 mm by weight clays with plasticity indexes of more than 12
F4	All silts and very fine silty sands containing more than 15 percent finer than 0.02 mm by weight clays with plasticity indexes of less than 12 varved clays and other fine-grained banded sediments

nearly always uneconomical and unnecessary except in regions with a low design freezing index or where the pavement is designed for heavy load aircraft. Freezing index is used as a measure of the combined duration and magnitude of below-freezing temperatures occurring during any given freezing season.

12. Since aggregate-surfaced pavements can be subjected to relatively large distortion without lose of serviceability, the design procedure used to control distortion, i.e., limited subgrade frost penetration method, is not suggested for use in aggregate pavement design. Rather, the reduced subgrade strength method should be used. The procedures to determine the reduced subgrade moduli of the aggregates and the subgrade soils can be obtained from CRREL (Berg and Johnson 1983). When the reduced subgrade strength method is used for F4 subgrade soils, unusually rigorous control of subgrade preparation will be required to ensure that the subgrade is reasonably uniform to prevent or minimize objectionable differential heaving. When a thickness determined by the reduced subgrade strength procedure exceeds that determined for limited subgrade frost penetration, the latter should be used provided that it is at least equal to the thickness required for nonfrost conditions.

13. The CE (Berg and Johnson 1983) developed the frost-area soil support indexes (Table 7) that are used as if they were California Bearing Ratio (CBR) values in the design of pavements in frost areas. The term CBR is not

Table 7

Frost-Area Soil Support Indexes for Subgrade Soils for Flexible Pavement Design

Frost group of subgrade soil	F1 and S1	F2 and S2	F3 and F4
Frost-area soil support index	9.0	6.5	3.5

applied to the soil support index because it is a weighted average value for an annual cycle and cannot be obtained by CBR test.

PART IV: REQUIREMENTS

14. The performance of aggregate-surfaced pavements is affected by climate and is particularly true for frost-susceptible soils in frost areas. The requirements for such a pavement are dependent upon whether or not frost is considered in the design.

Required Layers in Pavement Section

15. When frost is a consideration, the pavement section should consist of a series of layers that will ensure the stability of the system, particularly during thaw periods. The layered system in the granular fill will ideally consist of a wearing surface of fine crushed stone, a coarse-graded base course, and a well-graded subbase of sand or gravelly sand.

16. The wearing surface contains fines to provide stability and a smooth riding surface. The coarse-graded base course is important in providing drainage of the granular fill. It is also important that this material be nonfrost-susceptible so that it retains its strength during spring thaw periods. The sand subbase is used for additional bearing capacity over the frost-susceptible subgrade and as a filter layer between the coarse-graded base course and the subgrade to prevent the migration of the subgrade into the voids in the coarser material during periods of reduced subgrade strength. The sand subbase must be either nonfrost-susceptible or of low frost susceptibility (S1 or S2). The filter layer may not be necessary depending upon the type of subgrade material that underlies the pavement. If the subgrade consists principally of gravel or sand, the filter layer may be replaced by an additional base course if the particle sizes in the base course are such that little migration will occur. If a geotextile is used, the sand subbase/filter layer may be omitted as the fabric will be placed directly on the subgrade and will act as a filter.

17. The subgrade should be compacted to provide uniformity of conditions and a firm working platform for placement and compaction of the subbase. Compaction of the subgrade will not change its frost-area soil support index because frost action will cause the subgrade to revert to a weaker state. Hence, in frost areas, the compacted subgrade will not be considered part

of the layered system of the pavement which should be composed of only the wearing, base, and subbase courses.

18. The relative thicknesses of the base course and the filter layer are variable and should be based on the required cover and economic considerations.

19. An alternate allowable section for economy is to replace the lower 50 percent of the total thickness of granular material with S1 or S2 soils as long as the filter requirement over the subgrade is met. A further alternative is that frost group soils F1 and F2 may be used in the lower part of the base over F3 and F4 subgrade soils. F1 materials may be used in the lower part of the base over F2 subgrades. The thickness of F2 base material should not exceed the difference between the reduced-subgrade-strength thickness requirements over F3 and F2 subgrades. The thickness of F1 base should not exceed the difference between the thickness requirements over F2 and F1 subgrades. Any F1 or F2 material used in the base must meet the applicable requirements of the CE guide specifications for base and subbase materials. The thickness of F1 and F2 materials and the thickness of materials above the F1 and F2 materials must meet the nonfrost criteria in TM 5-822-5.

Material Requirements

20. The use of aggregate in a low volume road is to provide adequate stability to support the repetitive actions of the traffic loads, provide adequate resistance to degradation due to climate and abrasive action of traffic, and provide adequate skid resistance. Two important characteristics of aggregates are gradation of the gravel-sand particles and plasticity properties of the fines (passing No. 40 screen) or silt-clay size particles. The strength of an open (or lean) mix is controlled by the frictional component of shear strength that depends on aggregate-to-aggregate contact. The addition of fines to the mix fills up the void spaces and thus increases the density and shear strength because of added frictional resistance and cohesion provided by the fine particles. When the fine contents become excessive, the fines displace the coarse particles from one another, and the granular particles float in a matrix of fine material. In this situation not only does the density decrease slightly, but significant strength reduction also occurs and the permeability of the mix is drastically reduced. The friction component of shear

strength is greatly reduced through loss of contact between the coarser particles. As a result, the strength of the material is that of the finer soils rather than that of the granular particles. Based on these design guidelines, the requirements for aggregates are discussed below.

Nonfrost areas

21. An aggregate-surfaced pavement should be sufficiently cohesive to resist abrasive action. It should also be graded for maximum density and minimum volume of voids in order to enhance optimum moisture retention while resisting excessive water intrusion. The gradation, therefore, should consist of the optimum combination of coarse and fine aggregates and fines that will ensure minimum void ratios and maximum density. Such a material will then exhibit cohesive strength as well as intergranular shear strength. If the fine fraction of the material does not meet plasticity characteristics, modification by addition of chemicals might be required. Chloride products can, in some cases, enhance moisture retention while lime can be used to reduce excessive plasticity.

22. The gradation requirements shown in Table 8 are suggested for aggregate-surfaced pavements. The coarse aggregate (material retained on the No. 4 sieve) should consist of hard, durable particles of stone, gravel, or slag and have a percent wear according to the Los Angeles Abrasion Test of no more than 50. The fine aggregate (material passing the No. 10 sieve) should be naturally occurring or crushed sand. The maximum liquid limit of the fines should be 35 to limit the clay content in the aggregate, and the plasticity index (PI) should range from 4 to 9. The lower limit is to ensure adequate moisture for good binding quality and a dust free surface; the higher limit is set to ensure adequate stability and skid resistance when wet. The values of the limits may be increased slightly in dry areas. Also, higher limits are allowed for materials when used as subbase than when used as base course.

Frost areas

23. As previously stated, where frost is a consideration in the design of pavements, a layered system should be used. The percentage of fines should be restricted in all the layers to facilitate drainage and reduce the loss of stability and strength during thaw periods.

24. The gradation requirements for the wearing surface, coarse gravel base course, and sand subbase have been developed using standard filter design criteria. This enables water to flow freely through the granular fill and

Table 8

Gradation for Aggregate Surface Courses

Sieve Designation		No. 1	No. 2	No. 3	No. 4
25.0 mm	1 in.	100	100	100	100
9.5 mm	3/8-in.	50-85	60-100	--	--
4.7 mm	No. 4	35-65	50-85	55-100	70-100
2.00 mm	No. 10	25-50	40-70	40-100	55-100
0.425 mm	No. 40	13-30	25-45	20-50	30-70
0.075 mm	No. 200	8-15	8-15	8-5	8-15

prevents the migration of the smaller particles from the wearing surface downward or from the subgrade upward. The gradation limits of the various layers to meet frost design requirements are shown in Table 8. Gradations 3 and 4 may be unstable in frost areas and should be used with caution.

25. Design CBR values and material requirements for select materials and subbases should be selected in accordance with TM 5-825-2/AFM 88-6, Chapter 2.

Maintenance Requirements

26. The primary causes of frequent maintenance on aggregate-surfaced pavements are the environment and aggregate loss due to traffic. Rainfall and water running over the aggregate tend to wash the fines from the surface course reducing cohesiveness and consequently cause loss of surface aggregate under traffic loads.

27. Routine maintenance should be performed at least every 6 months and more frequently if required. It consists of grading, blading, patching potholes, replacing fines, cleaning drainage structures, and cutting vegetation. Periodic maintenance involves more extensive operations such as scarifying the surface layer to bring fines back to the surface, adding additional gravel to restore the thickness, and/or recompacting the wearing surface to the specific density.

28. Proper maintenance is essential in prolonging the service life of an aggregate-surfaced pavement. The most cost-effective design is based on an adopted maintenance strategy which is dependent upon the maintenance cost.

For instance, if the maintenance costs are high, relatively less maintenance may be used and the pavement would be replaced when it failed. However, if the maintenance costs are low, more routine and periodic maintenance may extend the time to failure to such an extent that it is the best maintenance strategy.

Use of Geotextiles

29. Geotextile is one of the geosynthetics used in geotechnical engineering. Geosynthetics include geotextiles, geomembranes, geogrids, mats, nets, and other composite products. In general, geomembrane is impermeable and geotextile is permeable. The latter is used more frequently than the former in pavements. The use of geotextiles (fabric) in pavement structures is relatively recent, and its long-term performance has not been clearly identified or quantified. In aggregate-surfaced pavement geotextiles are primarily used to perform the functions of reinforcement and separation and filtration. These are discussed in the following paragraphs.

a. Reinforcement. The presence of a fabric between two soil layers can strengthen a pavement system by resisting the stresses imposed in the pavement by loads applied to the pavement surface. The effect of reinforcement is proportional to the strength of the fabric used. Based on the very limited traffic test data available, unpaved pavements with geotextile separating the aggregate from cohesive soil performed better than those without geotextile.

b. Separation and filtration. When a geotextile is placed at the subgrade surface for separation, it serves to prevent fines from migrating into the base course and/or prevents base course aggregate from penetrating into the subgrade, thus preventing the mixing of two different materials of different sizes and gradations. The geotextile must have sufficient puncture, burst, grab, and tear strength since the geotextile serves to prevent aggregate from penetrating into the subgrade. The filtration function is primarily one of holding back subgrade soil particles while allowing the passage of water through the fabric to dissipate excess pore water pressure in the subgrade. The water is drained in the gravel layer moving laterally through the plane of the geotextile. This in one of the reasons that geotextile is used more often than geomembrane.

30. The primary function of placing geotextile in a gravel pavement is for the separation of subgrade soil from the gravel layer and the dissipation of excess pore water pressure in the subgrade. The geotextile provides an avenue for lateral drainage in the gravel layer along the plane of the geotextile.

PART V: MATERIAL CHARACTERIZATIONS

31. Characterization of the pavement materials requires the quantification of the material stiffness as defined by the resilient modulus of elasticity and Poisson's ratio. For selected pavement components, a fatigue strength is required as defined by a failure criterion. Repeated load laboratory tests designed to simulate aircraft and vehicular loading are used as much as possible to determine the resilient stiffness of the materials. The laboratory procedures for determining the elastic modulus values for aggregates and subgrade soils can be found in TM 5-825-3-1/AFM 88-6, Chapter 3, Section A (Headquarters, Department of the Army, in preparation). For some materials, such as unbound aggregates an empirically based procedure is a better approach for obtaining usable material parameters. Failure criteria have been provided; thus, fatigue testing will not be necessary. If the modulus values are not available, they may be computed from the CBR value from two empirical equations

$$E(psi) = 1{,}500\ CBR \tag{1}$$

$$E(psi) = 1{,}800\ (CBR^{0.7}) \tag{2}$$

where E is the modulus. Equation 1 was derived based on dynamic testing of pavements (Heukelom and Foster 1960), and Equation 2 was developed from other published relationships relating modulus to CBR (Von Til et al. 1972; Luhr, McCullough, and Pelzner 1983). Significant differences exist between the two equations. More recent information (ARE Inc. 1983) suggests that the subgrade modulus should be related to basic soil properties. The information also gives a relationship for granular subgrade soils:

$$\log E_{SG} = 1.94 - 0.0225\ (\%W) \tag{3}$$

and for fine-grained soils:

$$E_{SG} = 36.703 - 0.4566\ (PI) - 0.6279\ (\%W) - 0.1424\ (S200) \tag{4}$$

where

E_{SG} = modulus of the subgrade

%W = percent moisture of the soil

PI = plasticity index of soil

S200 = percent passing the No. 200 sieve

32. The elastic modulus of aggregates can also be determined using a chart in which the modulus of the base or subbase is a function of the modulus of the underlying layer and the base and subbase layer thickness. The chart developed at WES and the procedure for using the chart are given in Appendix A.

33. Figure 1 shows standard correlations between CBR and various soil classifications from the "Soil Primer" (Portland Cement Association 1984). It should be pointed out that any given soil classification can produce a range of CBR's, modulus values, and bearing values. In addition, moisture, compaction, and other placement conditions can alter CBR's for a given soil.

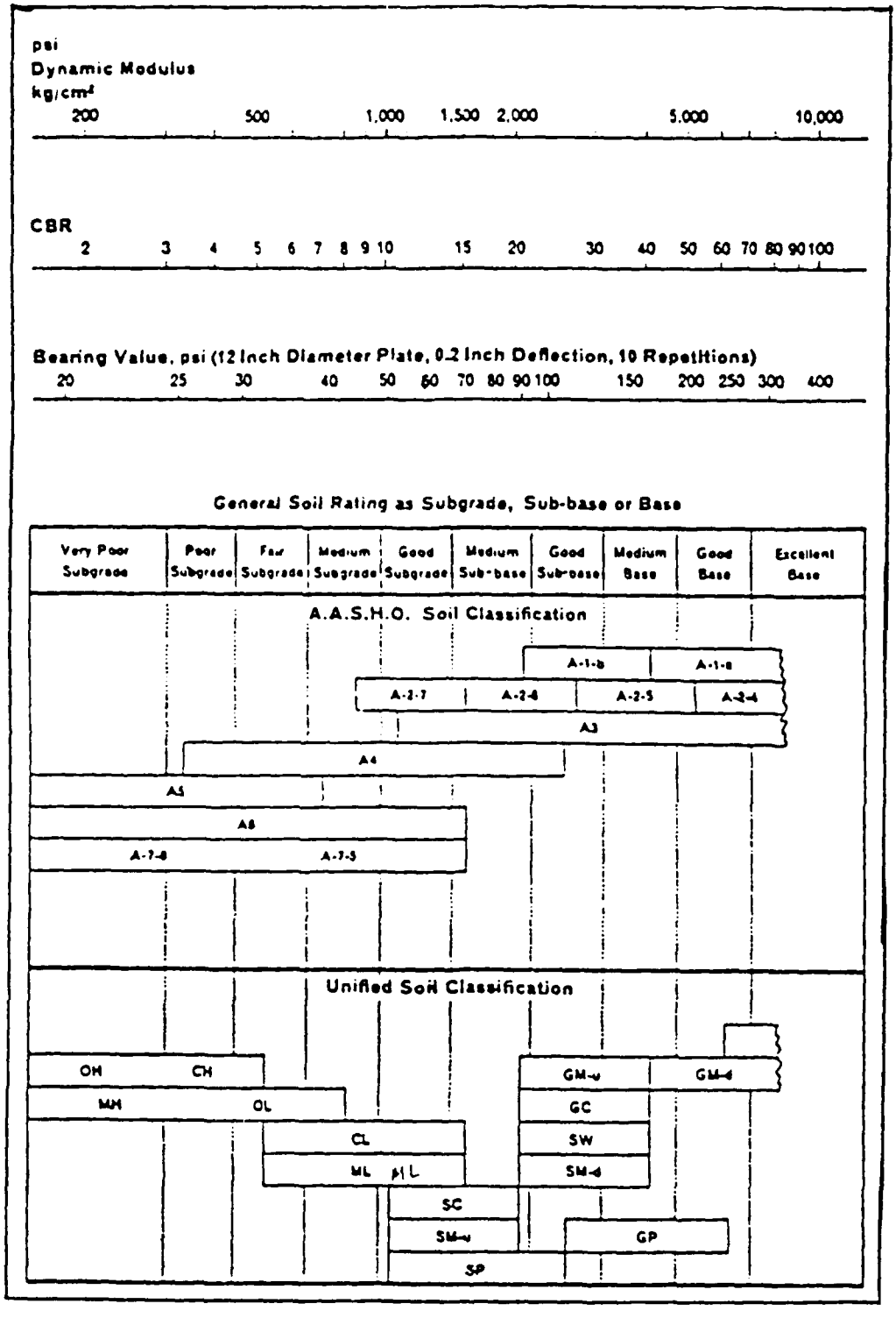


Figure 1. Relationship between soil classifications, CBR, modulus values, and bearing values

PART VI: EXISTING THICKNESS DESIGN PROCEDURES

Roughness and Serviceability

34. The purpose of placing gravel layers on a natural subgrade soil is to help distribute the load and provide a longer lasting surface. The thickness and quality of the gravel layers depend upon the traffic loading conditions and the environmental condition in the locality. For aggregate-surfaced pavements, not only is the load magnitude critical but also the tire inflation pressure. Tires with high inflation pressure tend to rut the surface and increase the pavement roughness.

35. The types of distress in aggregate-surfaced pavements are discussed in Part II. The performance evaluation for an aggregate pavement is complex and might best be defined in terms of roughness (ride quality) and loss of aggregate (loose aggregate) which are the influence of other distresses. Roughness is a quality that can be measured using various types of equipment or evaluated by an experienced observer in a slow-moving motor vehicle.

36. Recent studies on major projects in Bolivia (Carmichael, Hudson, and Sologuren 1979), Kenya (Rolt 1975, Faiz and Staffini 1979), and Brazil (Visser et al. 1979, DeQueirouz 1981) have indicated typical roughness values for low volume road conditions. Table 9 summarizes road roughness readings taken with the Mays meter roughness device on surface-treated and gravel roads in Bolivia. The Mays meter is a portable car road meter, and the measured output is millimetres of roughness per kilometre (or inches per mile). In essence, this value represents the summation of roughness (deviation from a true plane) per unit of road length. As can be seen in the table, roughness values for gravel-surfaced roads are much larger than those for surface-treated roads.

37. Grading is an integral part of routine maintenance for granular-surfaced roads. The effect of grading on roughness is generally quite significant. Studies have indicated that granular roads will return to the same roughness level as before grading within 2 to 3 weeks. Table 10 illustrates the effects of grading on the Mays meter roughness value (Carmichael, Hudson, and Sologuren 1979).

Table 9

Typical Road Roughness Values for Bolivian Roads

R(mm/km)*		R(mm/km)*	
Surface-Treated Roads	Gravel-Surfaced Roads	Surface-Treated Roads	Gravel-Surfaced Roads
927	8,776	2,997	8,179
1,029	12,751	1,245	8,001
813	9,855	1,067	10,820
1,803	12,649		4,394
2,718	14,986		15,596

* Mays meter roughness values.

Table 10

Effect of Grading on Roughness

Section	Mays Meter Roughness (mm,km)		Time
	Before Grading	After Grading	
1	17,272	8,306	Same day
		9,627	24 hr
		8,255	48 hr
		18,288	20 days
2	4,318	2,540	Same day
		3,962	24 hr
		10,262	20 days
3	13,843	8,839	Same day
		12,929	20 days

38. For aggregate-surfaced pavements, ruts contribute greatly to pavement roughness. Critical rut depths are imposed in many design criteria because they cannot be removed through normal maintenance. Although aggregate loss is not a separate failure criterion, the loss of aggregate surfacing reduces the structural integrity and thereby accelerates the deterioration of PSI and development of rutting. The anticipated loss of aggregate should be considered in the design.

39. In the AASHTO road tests, the conditions of the pavements were visually inspected, and the PSI ratings which were primarily associated with pavement surface roughness were determined.

40. The term "serviceability" is used to denote the ability of a pavement to serve its intended function at any particular time. A pavement that has recently been constructed should be relatively smooth and should therefore have a high level of serviceability. With the passage of time and traffic, road roughness will ordinarily increase and serviceability will be lowered.

41. Functional failure occurs when serviceability falls below a predefined value selected by the design engineer. This failure value is called the terminal serviceability.

Thickness Design

42. A number of existing design procedures are presented in this section. It is very important to point out that in the CE's procedures the predicting equations were developed based on test data in which the surface course materials were in many cases cohesive soils (rather than gravels) with relatively low CBR values. The tests were conducted generally in covered areas with controlled moisture conditions. The tests were conducted primarily for establishing criteria for transport vehicles and aircraft on natural soils. For instance, C-5A aircraft are designed to operate on natural soils with a minimum CBR of 9.

43. All design procedures are developed for truck and aircraft wheel loads. For tank trails, i.e., tracked vehicles, the procedures are presented in the example problems in Part IX.

CE design equation

44. In the late 1960's, a research program was conducted at the WES to determine the required strength and thickness of a layer of soil to protect a weak subgrade for roads and airfields (Hammitt 1970). Tests were conducted on

three unsurfaced test sections. Sixteen test items were covered in test Section 1, a 24-in.* clay subgrade of approximately 3-CBR strength and a cover material of approximately 9-CBR strength. Test Section 2 consisted of three lanes of the same thickness arrangement with a 4-CBR subgrade material and an approximately 12-CBR cover material. Fifteen test items were covered in test Section 3, a clay subgrade of approximately 2-CBR strength and a cover material of approximately 17 CBR. The traffic applied to the test items is shown in Table 11.

Table 11

Traffic Test Data

Lane	Wheel Assembly	Type of Tire	Tire Inflation Pressure, psi	Load lb
1	Single-wheel	20.00-20, 20-ply	150	15,000
2	Single-wheel	20.00-20, 20-ply	115	25,000
3	Single-wheel	25.00-28, 30-ply	80	40,000
4	Single-wheel	20.00-20, 20-ply	80	40,000
5	Single-wheel	30.00-11.5, 24-ply	165	15,000
6	Single-wheel	20.00-20, 20-ply	120	40,000
7	Twin-twin*	Three 17.00-20, 24-ply and one 49x17, 22-ply	120	80,000
8	Single-wheel	20.00-20, 20-ply	125	25,000
9	Single-wheel	20.00-20, 20-ply	125	40,000
10	Single-wheel	25.00-28, 30-ply	125	40,000

* Spacing of these tires was 30 in. c-c, 33 in. c-c, and 30 in. c-c. This gear arrangement is similar to the nose gear arrangement proposed for use on the C-5A aircraft.

45. Failure criteria used in the tests were based on permanent deformation or rutting and elastic deflections. When ruts exceeded a 3-in. depth, or when elastic deflection exceeded 1.5 in., an item was judged failed. Failure was also based on overall subsidence in excess of 4 in. measured from a 10-ft straightedge. By following the development of the CBR equation (Turnbull and

* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 4.

Ahlvin 1957) for the design of flexible pavements, the thickness requirements for unsurfaced (unpaved or earth-surfaced) roads and airfields can be computed by using the following equation (Hammitt 1970):

$$t = (0.176 \log_{10} (\text{coverage}) + 0.120) \sqrt{\frac{P}{8.1\ (CBR)} - \frac{A}{\pi}} \qquad (5)$$

where

t = design thickness of gravel layer, in.

P = single or equivalent single-wheel load, lb

A = tire contact area, sq in.

46. Equation 5 was developed following the development of the CBR equation. Therefore when a 15 percent reduction of pavement thickness was imposed on the CBR equation for flexible pavements for roads and streets at a later date, the same reduction factor was applied to Equation 6,* i.e.

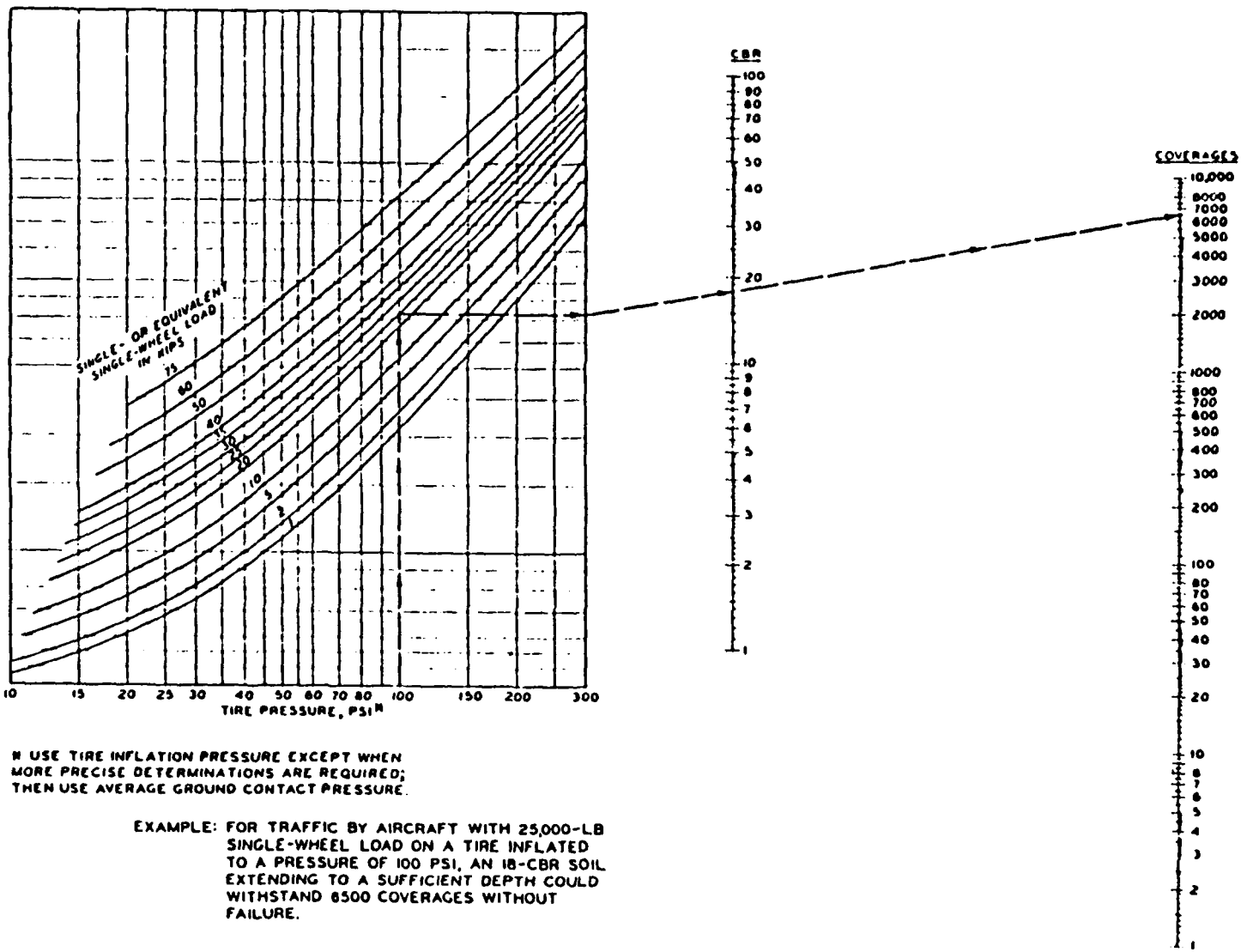
$$t = 0.85\ [0.176 \log_{10} (\text{coverage}) + 0.120] \sqrt{\frac{P}{8.1\ (CBR)} - \frac{A}{\pi}} \qquad (6)$$

47. It should be emphasized that Equation 6 is developed based on test data in which the surface cover materials have low CBR values, i.e., ranging from 7 to 17. Also, maximum coverage level was 700, and only 11 test items had coverage levels above 100. As a result of tests for the MX missiles, the design equation was further modified to the following:

$$t = [0.128 \log_{10} (\text{coverage}) + 0.087] \sqrt{\frac{P}{8.1\ CBR} - \frac{A}{\pi}} \qquad (7)$$

48. Both Equations 5 and 6 determine only the required thickness of the gravel layer; the quality (the CBR value) of the gravel is separately determined in a nomograph shown in Figure 2 as a function of wheel load (or the equivalent single-wheel load), tire inflation pressure, and design coverages. Rut depths up to 2 to 3 in. can be expected when Figure 2 is used. To minimize surface distortion, the nomograph shown in Figure 3 (Ahlvin and Hammitt 1975) can be used. For example, if a 50-kip wheel load with 100-psi tire pressure is designed for a 10,000 coverage level, the required CBR is 30 which

* When Equation 6 is used for roadway design, the pass to coverage ratio for an 18-kip single axle with dual tires is 2.64.



[a] USE TIRE INFLATION PRESSURE EXCEPT WHEN MORE PRECISE DETERMINATIONS ARE REQUIRED; THEN USE AVERAGE GROUND CONTACT PRESSURE.

EXAMPLE: FOR TRAFFIC BY AIRCRAFT WITH 25,000-LB SINGLE-WHEEL LOAD ON A TIRE INFLATED TO A PRESSURE OF 100 PSI, AN 18-CBR SOIL EXTENDING TO A SUFFICIENT DEPTH COULD WITHSTAND 8500 COVERAGES WITHOUT FAILURE.

Figure 2. Required CBR for operations of aircraft on unsurfaced subgrade soil

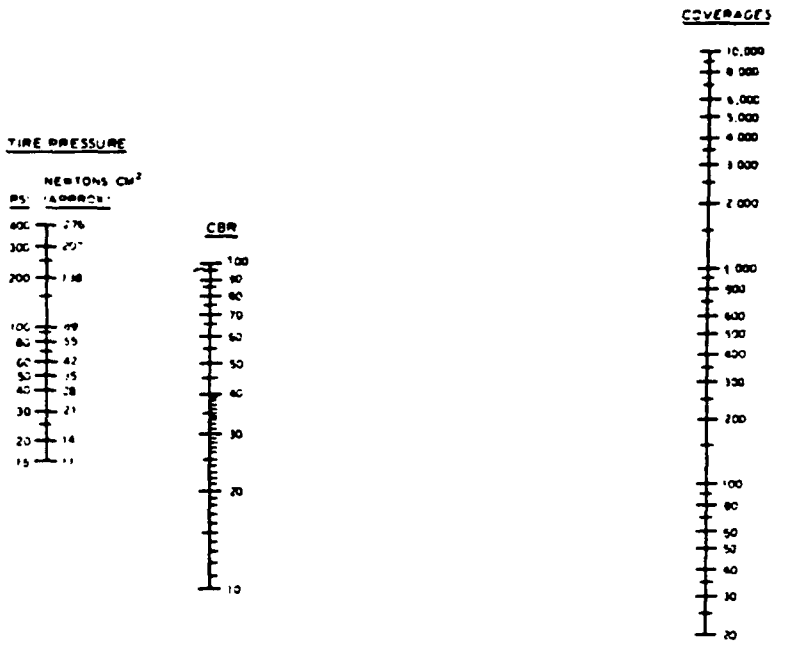


Figure 3. Surface strength requirements for aggregate-surfaced airfields

is expected to have a 2 to 3 in. surface rut. The rut may be minimized if a 100-CBR granular material is used. Figure 3 shows that the load is no longer critical, and only the tire pressure, stress repetitions, and surface strength interact.

CE rutting equation

49. Cosponsored by the US Department of Agriculture Forest Service, Barber, Odom, and Patrick (1978) analyzed the field test data for the prediction of deterioration of pavements. Deterministic equations were developed to predict deterioration in terms of rutting. A total of 254 data points of aggregate-surfaced pavements were analyzed using the regression technique, and the equation considered the best has the form:

$$RD = 0.1741 \frac{(P_k)^{0.4707} t_p^{0.5695} R^{0.2476}}{(\log t)^{2.002} C_1^{0.9335} C_2^{0.2848}} \quad (8)$$

where

RD = rut depth, in.

P_k = equivalent single-wheel load, kips

t_p = tire pressure, psi

R = load repetitions, passes

t = thickness of gravel layer, in.

C_1 = CBR of gravel layer

C_2 = CBR of the natural subgrade

Equation 8 can also be written in the form

$$t = (10^B)^{0.4995} \tag{9}$$

where

$$B = 0.1741 \frac{(P_k)^{0.4707} \; t_p^{0.5695} \; R^{0.2476}}{RD \quad C_1^{0.9335} \; C_2^{0.2848}}$$

50. Appendix B contains the pavement and loading data information for the 254 data points used for the development of Equations 8 and 9. It should be pointed out that the surface course materials for most test sections have relatively low CBR values (between 8 and 17).

CE design index method

51. The design index method (Headquarters, Department of the Army 1985) is the latest version of the CE method for aggregate-surfaced roads and tank trails. In this method the magnitude of wheel load is not directly used in the design. The required gravel thickness is determined based on a design index representing all traffic expected to use the facility during its life. Figure 4 is the design curve used to determine the thickness of aggregate-surfaced roads and streets based on the subgrade CBR and design index. The index is based on typical magnitudes and compositions of traffic reduced to equivalents in terms of repetitions of an 18,000-lb single-axle, dual-wheel load.

52. A class is assigned to the facility depending upon the traffic intensity expected, and a design category is assigned to the traffic depending upon the traffic composition. A design index is then determined for design purposes based on the class and the category.

53. The class of a facility depends upon the traffic intensity and is determined from Table 12. For designs involving rubber-tired vehicles, traffic will be classified into three groups: (a) Group 1 includes passenger cars and panel and pickup trucks, (b) Group 2 includes two-axle trucks, and (c) Group 3 includes three-, four-, and five-axle trucks. Traffic

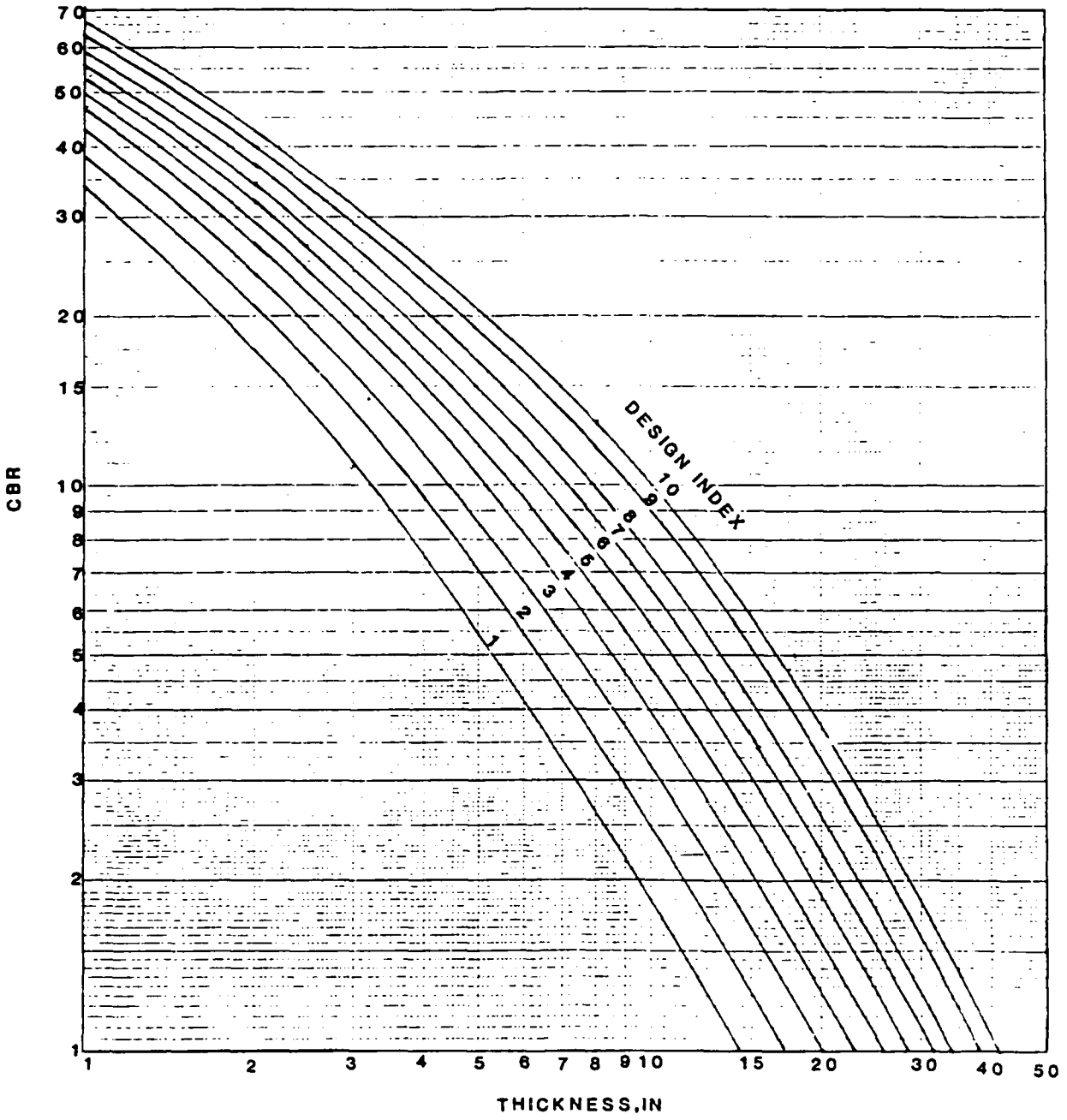


Figure 4. Design curve for gravel-surfaced hardstands

Table 12

Criteria for Selecting Hardstand Class

Hardstand Class	Number of Vehicles per Day
A	10,000
B	8,400 - 10,000
C	6,300 - 8,400
D	2,100 - 6,300
E	210 - 2,100
F	70 - 210
G	under 70

composition will then be grouped under the following categories:

a. Category I. Traffic essentially free of trucks (99 percent Group 1 plus 1 percent Group 2).

b. Category II. Traffic including only small trucks (90 percent Group 1 plus 10 percent Group 2).

c. Category III. Traffic including small trucks and a few heavy trucks (85 percent Group 1 plus 14 percent Group 2 plus 1 percent Group 3).

d. Category IV. Traffic including heavy trucks (75 percent Group 1 plus 15 percent Group 2 plus 10 percent Group 3).

e. Category IVa. Traffic containing more than 25 percent trucks.

Where half- or full-track vehicles or forklift trucks are involved in the traffic composition, considerations that should apply are (a) half- or full-track vehicles or forklift trucks having gross weights of less than 10,000 lb may be treated as two-axle trucks in determining design index, (b) half- or full-track vehicles weighing less than 25,000 lb and forklift trucks weighing less than about 15,000 lb may be treated as three-axle trucks in determining design index, and (c) three additional categories are considered to provide for heavy half- or full-track vehicles and forklift trucks as shown in Table 13.

54. The design index to be used in designing a gravel hardstand for the usual pneumatic-tired vehicles will be selected from Table 14. Hardstands sustaining traffic of half- or full-track vehicles having a gross weight less

Table 13

Traffic Categories

Category	Vehicle Weight, lb	
	Tracked Vehicles	Forklift Trucks
V	50,000	30,000
VI	80,000	50,000
VII	120,000	--

Table 14

Design Index for Pneumatic-Tired Vehicles

Class	Design Index			
	Category I	Category II	Category III	Category IV
A	3	4	5	6
B	3	4	5	6
C	3	4	4	6
D	2	3	4	5
E	1	2	3	4
F	1	1	2	3
G	1	1	1	2

than 25,000 lb will be designed in accordance with the pertinent class and category from Table 14. Hardstands sustaining traffic of half- or full-track vehicles heavier than 25,000 lb will be designed in accordance with the traffic intensity and category from Table 15. The design life is assumed to be 25 years. For a lesser life of 2 to 5 years, the design indexes in Tables 14 and 15 may be reduced by one. Design indexes below three should not be reduced.

Transport and Road Research Laboratory (TRRL) design procedure

55. The TRRL of the United Kingdom has developed a design procedure for bituminous-surfaced roads in tropical and subtropical countries (TRRL 1977). The method is applicable to load repetitions up to 2,500,000 equivalent 18,000-lb single-axle loads. The basic TRRL design curves for

Table 15

Design Index for Tracked Vehicles and Forklift Trucks

Traffic Category	Number of Vehicles per Day (or Week as indicated)							
	500	200	100	40	10	4	1	1 Per Week
V	8	7	6	6	5	5	5	--
VI	--	9	8	8	7	6	6	5
VII	--	--	10	10	9	8	7	6

bituminous-surfaced treatment (BST) structures are shown in Figure 5. For granular-surfaced roads, a factor of 0.78* is used for the corresponding thickness given for BST roads. This reduction in thickness is because the design for granular surfaces permits greater deformation at failure than does the BST surfaces.

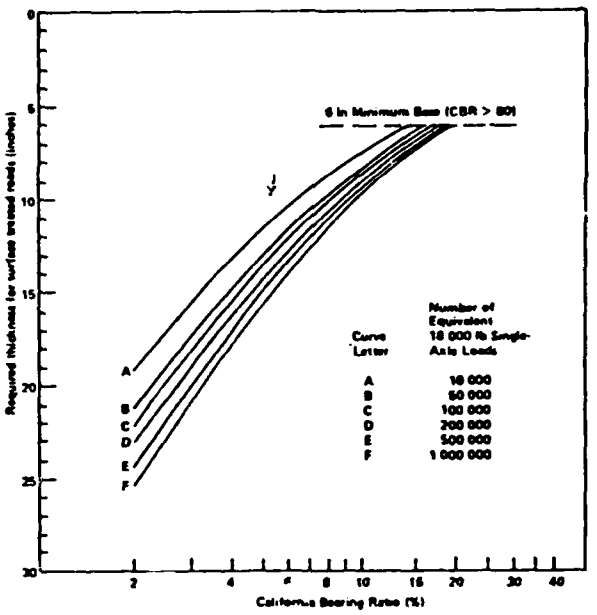


Figure 5. Thickness design curves for surface-treated roads (TRRL)

56. The TRRL procedure recommends a minimum base thickness of 6 in. and a minimum CBR value of 80 percent for the strength of the base material. If a

* In the Corps of Engineers procedure, the reduction factors for the corresponding thickness given for conventional flexible pavements (Hammitt 1970) are 0.85 and 0.75 (Hammit 1970 and Ahlvin and Hammitt 1975, respectively).

subbase is used, minimum values for the subbase material are 4 in. of thickness and CBR of 25 percent at the expected field moisture-density conditions.

US Forest Service procedures

57. The procedures for the structural design of granular-surfaced roads developed by the US Forest Service (US Forest Service 1974) are based on two failure criteria. The first criterion is PSI that begins at an initial index p of 4.0 and reaches a failure index p_t of 1.5 after a period of traffic and time. The second criterion is the rut depth. Failure occurs when rut-depth reaches a specified design value of 2 in.

58. In addition to design values for serviceability index or rut depth, the following three factors are basic to the US Forest Service design procedure.

a. Soil Support (SS) is an empirical soil strength parameter that is not measured directly but that correlates with CBR strength and group index (GI) values (Yoder and Witczak 1975) as shown in Table 16.

Table 16

Correlation Between Subgrade CBR Strength, SS Value, and GI

Subgrade Strength CBR, %	SS	GI
2	2.2	
3	3.0	20.0
4	3.6	17.0
5	4.0	14.0
6	4.3	11.0
8	4.9	5.0
10	5.3	4.0
15	6.1	1.8
20	6.7	1.3
30	7.4	0.6
40	8.0	0.0

b. Structural Number (SN) equals $a_1D_1 + a_2D_2 + \ldots$ where D_1 is the thickness (inches) of the top layer of the pavement structure, a_1 is a coefficient representing the quality of material in the top layer, D_2 is the thickness of the second layer of pavement structure, a_2 represents material quality in the second layer, etc. Relationships between structural number coefficients and CBR strengths of the respective layers are shown in Table 17.

Table 17

Correlation Between CBR Strength of Granular Materials and SN Coefficients (a_i)

Strength of Granular Material CBR, %	Structural Number Coefficients a_i	
	Granular Base or Surfacing	Granular Subbase
20	0.070	0.095
25	0.083	0.103
30	0.093	0.109
35	0.101	0.116
40	0.107	0.120
45	0,112	0.124
50	0.117	0.127
60	0.126	0.130
70	0.132	--
80	0.136	--
90	0.138	--
100	0.140	--

c. Design Life (W_T) equals the number of equivalent 18,000-lb single-axle loads (ESAL) to be experienced in a traffic lane during the design period. Thus, W_T is the accumulation of equivalent axle loads between the times that PSI = 4.0 and PSI = 1.5.

59. The basic design factors are brought together in Table 18, which gives SN values for various combinations of SS values and equivalent single-axle loads (W_T).

60. The rut-depth criterion is based on a maximum rut of 2 in. For a single-layer granular surfacing, Table 19 gives structural numbers for the

Table 18

SN Values for Granular-Surfaced Structures

(USFSPSI Criterion)

$p_i = 4.0$, $p_t = 1.5$

Number (000s) of ESAL's (in thousands) (W_T)	SS Values								
	2	3	4	5	6	7	8	9	10
10	2.08	1.81	1.56	1.34	1.14	0.95	0.78	0.62	0.48
20	2.32	2.03	1.76	1.52	1.30	1.10	0.92	0.75	0.60
50	2.66	2.34	2.05	1.78	1.54	1.32	1.11	0.93	0.76
100	2.94	2.59	2.28	1.99	1.73	1.49	1.28	1.08	0.90
200	3.24	2.87	2.53	2.22	1.94	1.69	1.45	1.24	1.04
500	3.68	3.27	2.90	2.56	2.24	1.96	1.70	1.37	1.25
1,000	4.04	3.60	3.19	2.83	2.49	2.19	1.91	1.66	1.42

Table 19

SN Values for Granular-Surfaced Structures

(USFS Rut-Depth Criterion)

Number (000s) of ESAL's (in thousands) (W_T)	Subgrade CBR, %						
	2	4	6	8	10	15	20
10	3.30	2.25	1.76	1.47	1.26	0.90	0.64
20	3.51	2.39	1.88	1.57	1.33	0.95	0.69
50	3.78	2.58	2.03	1.68	1.44	1.02	0.74
100	3.99	2.73	2.14	1.78	1.53	1.08	0.78
200	4.20	2.87	2.25	1.88	1.60	1.13	0.81
500	4.47	3.05	2.39	1.99	1.71	1.22	0.87
1,000	4.68	3.19	2.51	2.09	1.78	1.26	0.91

equation $SN = a_1 D_1$. Thus, the required thickness is $D_1 = SN/a_1$, where values of a_1 are given in Table 17.

61. For axle loads and axle types other than 18-kip single-axle, the determination of the equivalent damage factors is presented in Appendix C.

62. The US Forest Service design procedures are being revised in terms of a system design approach based on minimization of total life cycle costs (McCullough and Luhr 1979a, 1979b, Roberts et al. 1977). The new procedures, however, are not discussed in this report.

63. AASHTO design procedure. The AASHTO design procedure for aggregate-surfaced roads published in 1986 (AASHTO 1986) is a graphical solution of the US Forest Service mechanistic design procedure (US Forest Service 1974) previously presented in this section. The design steps are as follows:

a. An acceptable serviceability loss (difference between initial and terminal PSI) is set along with the maximum acceptable rut depth during the analysis period.

b. A trial base thickness is selected.

c. The resilient modulus values for the roadbed and base are entered into Columns 2 and 3 of Table 20, respectively, for each season. (See Table 21 for guidance if information is not available.)

d. The total projected 18-kip ESAL's are distributed by season and entered in Column 4 of Table 20.

e. Using Figure 6 and the input parameters from Steps a through d, the allowable 18-kip ESAL's are predicted for each season and entered in Column 5 of Table 20.

f. Column 4 is divided by Column 5 and entered into Column 6. The individual values are summed and entered at the bottom as total damage.

g. Using Figure 7, and the input parameters from Steps a-d, the allowable 18-kip ESAL's are predicted for each season and entered in Column 7 of Table 20.

h. Column 4 is divided by Column 7 and entered into Column 8. The individual values are summed and entered at the bottom as total damage.

i. Steps a through h are repeated for different trial thicknesses to obtain damage summation values less than and greater than one.

j. The damage summations from Columns 6 and 8 are plotted on graphs, the thicknesses are determined for a damage summation of one, and the greatest number is used as the design thickness.

Table 20

Chart for Computing Total Pavement Damage (for Both Serviceability and Rutting Criteria) Based on a Trial Aggregate Base Thickness (AASHTO Design Procedure for Gravelly Surfaced Roads)

Trial Base Thickness D_{BS}, in.				Serviceability Criteria ΔPSI		Rutting Criteria RD, in.	
(1) Season (Roadbed Moisture Condition)	(2) Roadbed Resilient Modulus M_R, psi	(3) Base Elastic Modulus E_{BS}, psi	(4) Projected 18-kip ESAL Traffic w_{18}	(5) Allowable 18-kip ESAL Traffic (W_{18}), psi	(6) Seasonal Damage $\frac{w_{18}}{(W_{18})}$, psi	(7) Allowable 18-kip ESAL Traffic (W_{18}) rut	(8) Seasonal Damage $\frac{w_{18}}{(W_{18})}$ rut
Winter (frozen)							
Spring/Thaw (saturated)							
Spring/Fall (wet)							
Summer (dry)							
		Total Traffic =		Total Damage =		Total Damage =	

Table 21

Suggested Seasonal Roadbed Soil Resilient Moduli M_R, psi, as a Function of the Relative Quality of the Roadbed Material (AASHTO Design Procedure for Gravelly Surfaced Roads)

Relative Quality of Roadbed Soil	Season (Roadbed Soil Moisture Condition)			
	Winter (Roadbed Frozen)	Spring-Thaw (Roadbed Saturated)	Spring/Fall (Roadbed Wet)	Summer (Roadbed Dry)
Very good	20,000*	2,500	8,000	20,000
Good	20,000	2,000	6,000	10,000
Fair	20,000	2,000	4,500	6,500
Poor	20,000	1,500	3,300	4,900
Very poor	20,000	1,500	2,500	4,000

* Values shown are resilient modulus in pounds per square inch.

64. Elastic layered method in cooperation with aggregate loss. Luhr, McCullough, and Pelzner (1983) developed a new design algorithm in that some stress-strain parameter of the pavement was incorporated into a design equation similar to the form of the AASHTO design method. The procedure has a more mechanistic orientation and is adaptable to conditions outside the range of the road-test data. Using the layered elastic method, stresses and strains were computed for the test pavements under various loads. Input data required for the program include the thickness, Poisson's ratio, modulus of elasticity of each layer in the pavement, tire pressure, and magnitude of the applied wheel loads. The material properties used in the computations were normalized values that reflect a range in conditions at the AASHTO road test. Attempts were made to correlate the computed values with results from the AASHTO design equation.* It was found that the most promising parameter was the compressive

* The AASHTO (then called AASHO) pavement design method was developed by using the results from AASHO road test conducted October 1958 through November 1959 near Ottowa, IL. The road test included six loops and 468 test sections of asphalt pavement that were subject to traffic loads ranging from 2-kip single axles to 48-kip tandem axles. Performance was subjectively measured by a panel of raters by using a present serviceability rating that ranges from 0 for very poor to 5 for excellent. The analysis of the road-test data resulted in a design method that for a given pavement structure the number of load repetition before the performance of the pavement reaches a given minimum (or terminal) PSI can be obtained.

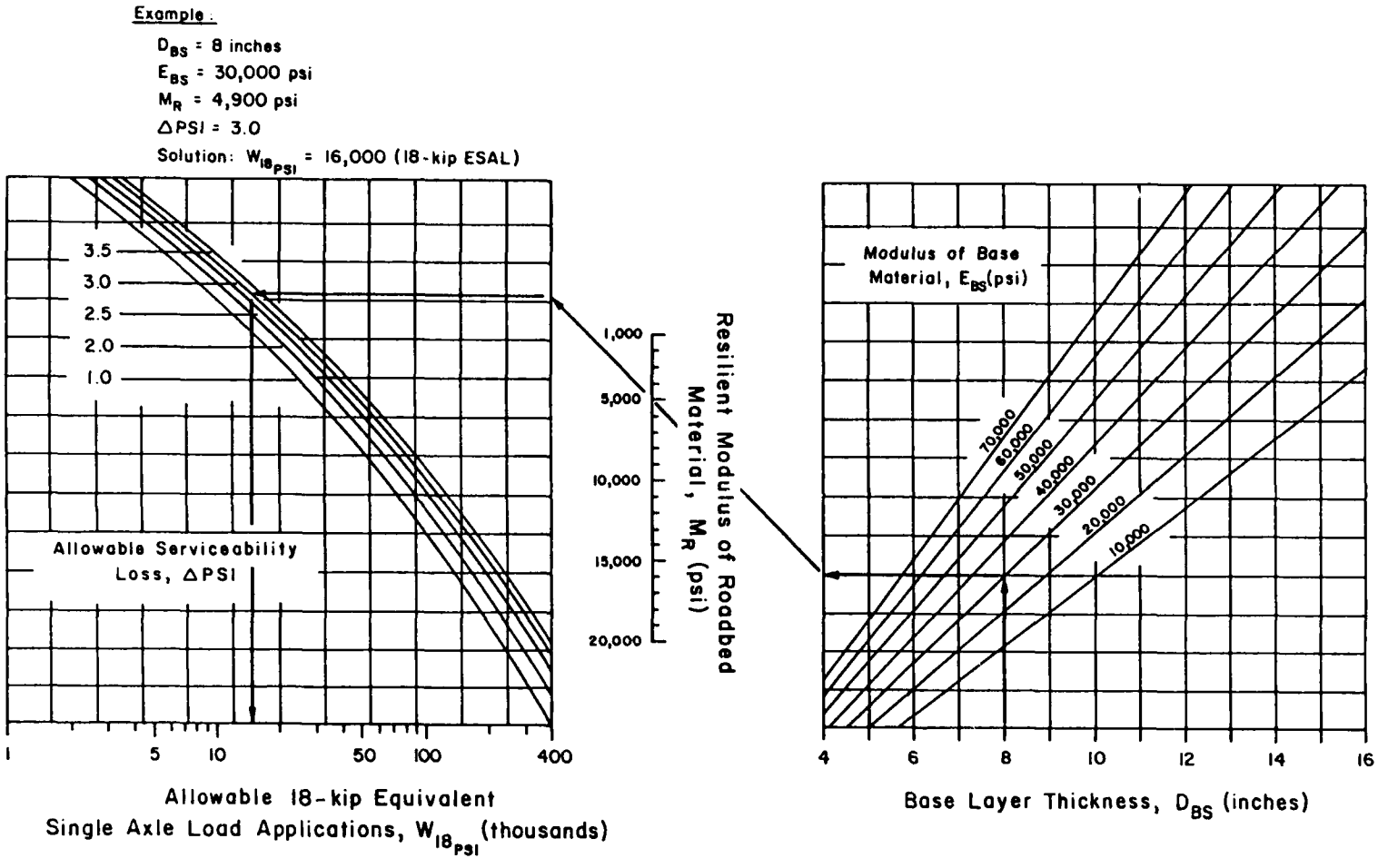


Figure 6. Design chart for aggregate-surfaced roads considering allowable serviceability loss (AASHTO design procedure for gravelly surfaced roads)

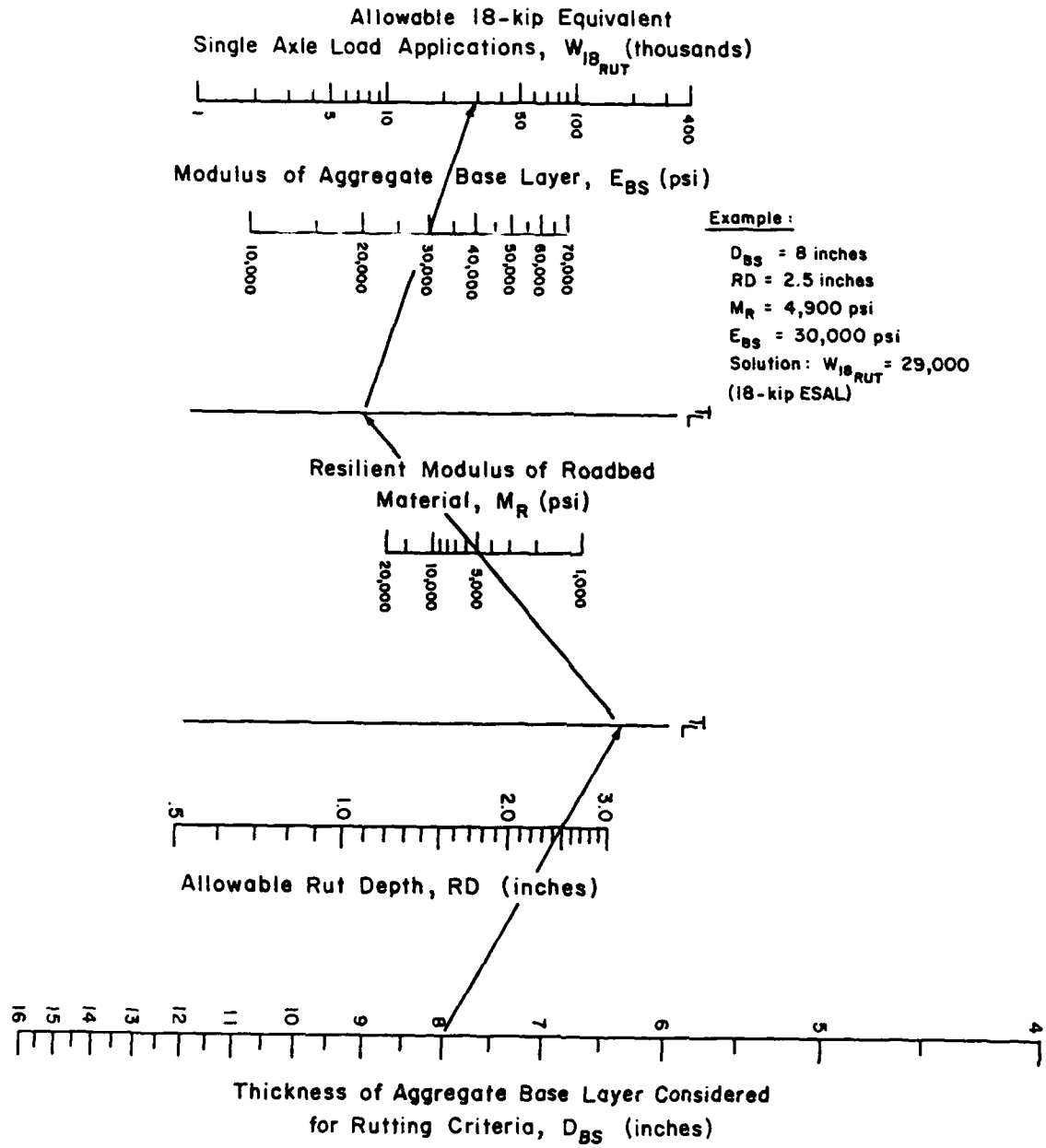


Figure 7. Design chart for aggregate-surfaced roads considering allowable rutting (AASHTO design procedure for gravelly surfaced roads)

strain at the top of the subgrade ε_{SG}. Using regression techniques, the resulted best-fit equation that has the same form as the AASHTO equation is shown in Equation 10:

$$\log_{10} N_{tx} = 2.15122 - 597.662\ (\varepsilon_{SG}) - 1.32967\ (\log_{10} \varepsilon_{SG}) + \log_{10}\ [(p_i - p_t)/(4.2 - 1.5)]^{1/2} \quad (10)$$

where

N_{tx} = number of application of any axle load x

ε_{SG} = compressive strain at the top of the subgrade

p_i = initial PSI of the pavement

P_t = terminal value of PSI (value of PSI used to indicate failure)

Figure 8 shows how well Equation 10 predicts the same 523 road-test observations predicted by the AASHTO equations. The real benefit of the subgrade strain design (Equation 10) is that the equation represents a more rational and mechanistic characterization of the pavement parameters.

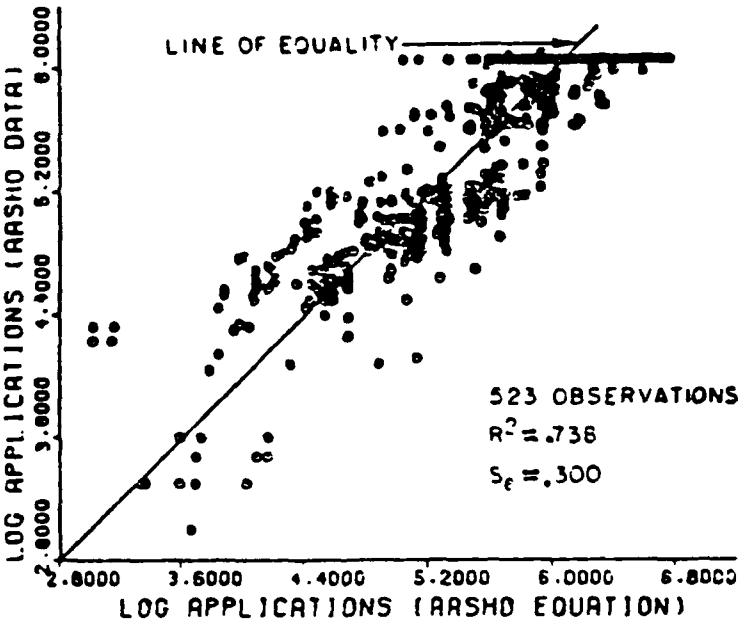


Figure 8. AASHTO equation as predictor of AASHTO data

65. It should be mentioned that the AASHTO road test data relate only to bituminous-surfaced roads; no comparable data are available for aggregate-surfaced roads. In the absence of such information, it was necessary to

design aggregate-surfaced roads using the same concept as for higher type roads by considering the stress-strain parameters in the pavement structure. Since the only parameter considered in Equation 10 is the subgrade strain value, it was assumed that Equation 10 was also valid for aggregate-surfaced pavements as long as the strains were computed using the elastic layered method. It is the writer's opinion, however, that Equation 10 was formulated based on test results of flexible pavements. The assumption that the equation is also applicable for aggregate-surfaced pavements needs to be verified by field tests. It is also important to understand that the failure criteria for aggregate-surfaced pavements may be different from that of bitumen-surfaced pavements. It is again the writer's opinion that some adjustment is needed in using Equation 10 for aggregate-surfaced pavements.

66. In an effort to reduce computations in the simplified procedure, Luhr, McCullough, and Pelzner (1983) established an equation correlating the subgrade strain and pavement input parameters using the regression method. The equation has the form

$$\log_{10} \varepsilon_{SG} = -2.24002 - \left(2.91440 \times 10^{-5} E_{subg}\right) - \left(5.08514 \times 10^{-2} \times D_{AC}\right) - \left(2.02947 \times 10^{-2} \times D_{BS}\right) - \left(5.37288 \times 10^{-8} \times E_{AC} \times D_{AC}\right) - \left(9.37888 \times 10^{-4} \times D_{BS} \times D_{SB}\right) - \left(2.91066 \times 10^{-7} \times E_{BS} \times D_{BS}\right) - \left(8.60253 \times 10^{-7} \times E_{SB} \times D_{SB}\right) \quad (11)$$

where

ε_{SG} = compressive subgrade strain due to 18-kip axle load with 75-psi tire pressure

E_{subg} = elastic modulus of subgrade soil, psi

D_{AC} = thickness of asphalt layer, in.

D_{BS} = thickness of base layer, in.

E_{AC} = elastic modulus of asphalt layer, psi

D_{SB} = thickness of subbase layer, in.

E_{BS} = elastic modulus of base layer, psi

E_{SB} = elastic modulus of subbase layer, psi

For a one-layer aggregate-surfaced pavement, Equation 11 is simplified to

$$\log_{10} \varepsilon_{SG} = -2.24002 - \left(2.91440 \times 10^{-5} \times E_{subg}\right) - \left(2.02947 \times 10^{-2} \times D_{BS}\right) - \left(2.91066 \times 10^{-7} \times E_{BS} \times D_{BS}\right) \quad (12)$$

67. The amount of aggregate loss is estimated and is considered in the layered elastic method. Loss of aggregate surfacing due to traffic is a natural phenomenon that occurs on roads with unbound surfaces. The abrasive action of traffic on aggregate-surfaced pavements loosens the larger aggregate particles from the soil binder. This leads to dusting and loose aggregate particles on the pavement surface and eventually to aggregate loss. The presence of fines in the aggregate will decrease the rate of aggregate loss but will not completely eliminate the loss. The loss of gravel is a significant distress mechanism for granular-surfaced pavements. The need for regraveling pavements may be viewed as equivalent to the need for periodic resurfacing of high-type pavement structures. Gravel loss is significant because it leads to premature or accelerated structural pavement failure.

68. It is desirable to estimate aggregate loss over the design period to predict how much of the pavement structure will be worn or eroded away. Sufficient information is not available to accurately predict the amount of aggregate loss in a road or an airfield. Two major research studies on granular-surfaced roads have been conducted in Kenya (Rolt 1975, Faiz and Staffini 1979) and Brazil (Visser et al. 1979, DeQueirouz 1981). In the Kenya study the annual gravel loss for a particular type of material depended on the annual traffic volume, annual rainfall, and vertical curvature. Selected plots of the relationships are shown in Figure 9 for four types of soils. The annual number of bladings is approximately 6 to 12. The figure also shows that an annual loss of about 95 mm (3.7 in.) of volcanic gravels can be expected when the traffic volume is 400 vehicles per day, the rainfall is 1 m (40 in.) per year, and the vertical curvature is 6 percent.

69. The Brazilian study (DeQueirouz 1981) produced predictive equations for two types of material (lateritic and quartzitic gravels) in which gravel loss is dependent on traffic volume, horizontal curvature, vertical grade, and number of bladings per year. Selected plots of these equations are shown in Figure 9. It is to be noted that Figure 10 excludes the rainfall term that was found to be significant in the Kenya study. Based on the results of Figure 9, Luhr, McCullough, and Pelzner (1983) suggested the following equation

$$GLIN = (B/25.4) \times [0.0045 \times LADT + (3380.6/R) + 0.467 \times G] \quad (13)$$

where

GLIN = aggregate loss during period of time being considered, in.

B = number of bladings during period of time being considered

LADT = average daily traffic (ADT) in design lane (for one-lane road use total traffic in both directions)

R = average radius of curves, ft

G = absolute value of grade, percent

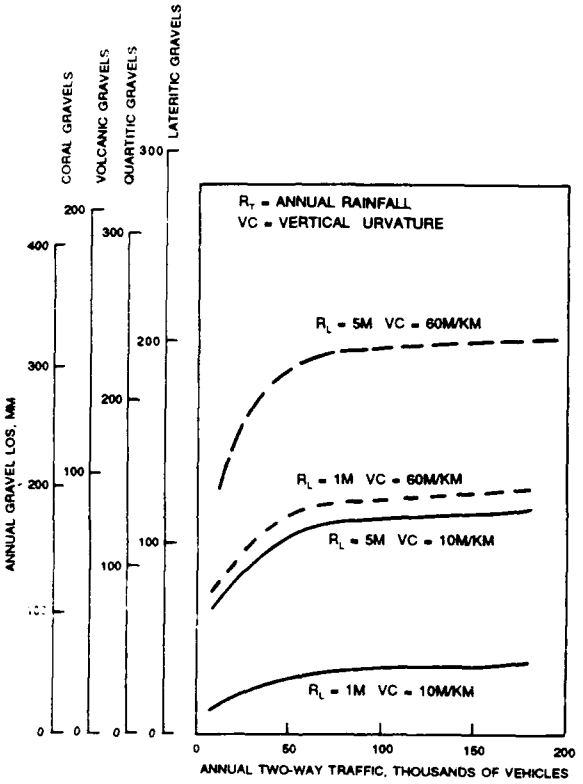


Figure 9. Gravel loss relationships for Kenyan conditions (Rolt 1975)

70. Expected aggregate loss is used in the procedure to reduce the thickness of the surface layer to an average expected thickness over the design life. For example, if 1 in. of aggregate loss is expected every year, a total of 10 in. would be lost over a 10-year design period. If the surface is to be constructed with a thickness of 12 in., the average thickness over the 10-year period would be [12 in. - (10 in./2)] or 7 in. Therefore, 7 in. is used as the surfacing thickness in the design procedure, even though the initial construction is 12 in.

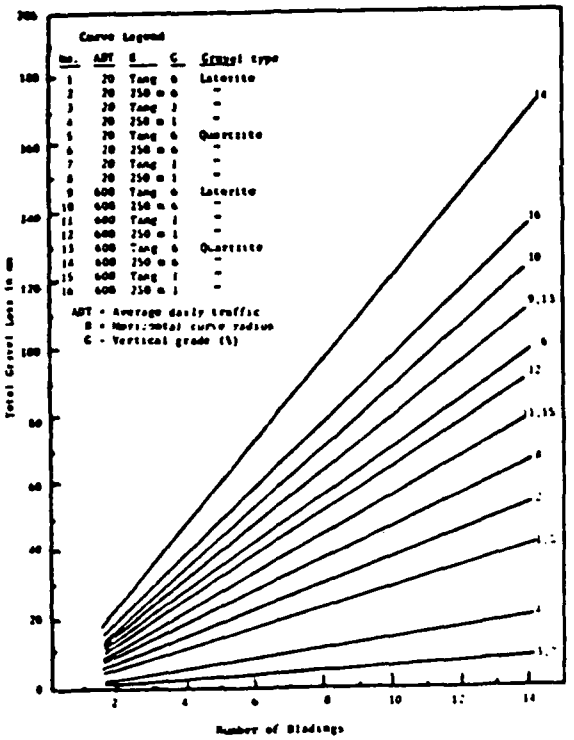


Figure 10. Gravel loss relationships for Brazilian conditions (DeQueirouz 1981)

Computation of Subgrade Strain under Track Loadings

71. To design the gravel-surfaced roadways or hardstands for tank loadings using the layered elastic method, the uniformly distributed track loadings have to be converted into uniformly distributed circular loadings. The subgrade strain induced by the equivalent circular loads can be computed using the BISAR computer program. The procedure is presented in the following paragraphs.

72. For an Army M1 Abrams tank, the track has a width of 25 in. and a contact length (the part contacting the ground surface) of 180 in. Assuming the circular loads have a diameter of 25 in., each circular bed has an area of 490.87 sq in. Each uniformly distributed track load can then be converted to nine equivalent circular loads, and each weighs 6,667 lb. The dimensions of the circular loads are shown in Figure 11. The BISAR computer program can be used to compute the subgrade strain induced by the circular loads. The

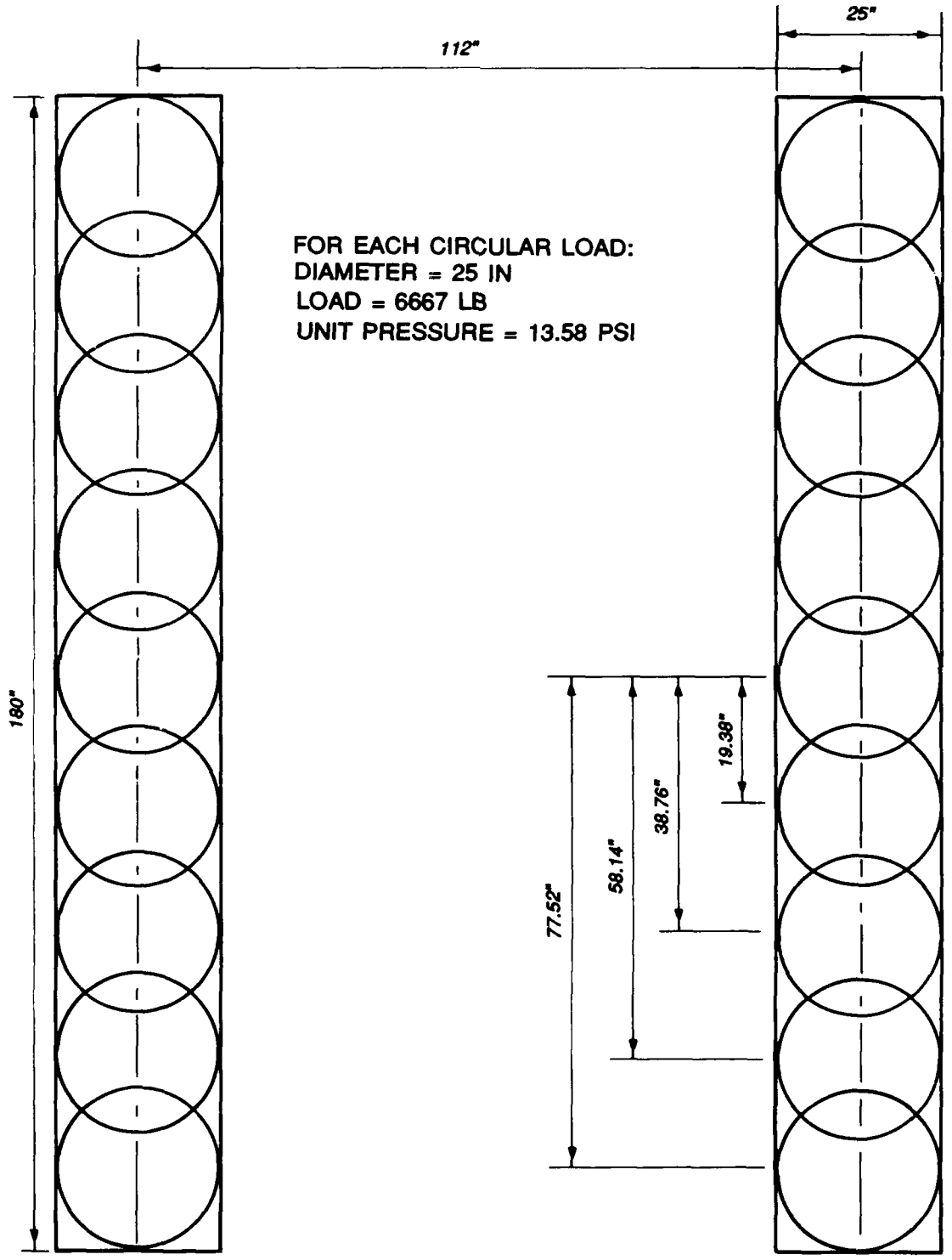


Figure 11. Equivalent circular loads for M1 tank tracks

maximum strain should occur at the track center. For soft soils it is reasonable to assume that the track contact width is the same as the track width. In rigid pavement, however, the actual contact width may be less than the track width. A reduction factor of 0.75 may be used, i.e., the effective contact width is 0.75 = 18.75 in.

73. It is important to note that only vertical pressure is considered in computing the subgrade strain. In reality, a moving track exerts large horizontal forces to the pavement surface and can pulverize the aggregates. The pulverization could accelerate the aggregate loss and shorten the service life of the pavement.

PART VII: PROPOSED LAYERED ELASTIC PROCEDURE

74. The available gravel thickness design procedures developed by various agencies were presented in Part VI. Some procedures were developed strictly for truck loadings, some were for aircraft loadings, and some were suitable for both. In this part design criteria based on elastic layered method are developed for truck, tracked, and aircraft loadings.

Background

75. The original and the new CBR equations for flexible airfield pavements are shown below as Equations 14 and 15, respectively

$$t = \alpha \sqrt{\frac{P}{8.1\ CBR} - \frac{A}{\pi}} \tag{14}$$

$$t = \alpha \left\{ A \left[- 0.0481 - 1.1562 \log \left(\frac{CBR \cdot A}{P} \right) - 0.6414 \log \sqrt{\frac{CBR \cdot A}{P}^{2}} - 0.473 \log \left(\frac{CBR \cdot A}{P} \right)^{3} \right] \right\} \tag{15}$$

where

t = pavement thickness

α = a traffic factor equal to $0.23 \log_{10}$ (coverage) + 0.15 for flexible roads and streets and equal to the value determined from Figure 12 for airfield flexible pavements

P = single-wheel load (or the ESWL in the case of the multiple-wheel loads)

A = tire contact area

CBR = California Bearing Ratio of the subgrade soil

Equation 14 was formulated in the 1950's (Turnbull and Ahlvin 1957, Fergus 1956), and Equation 15 is the form based on more test data formulated in the early 1970's (Hammitt et al. 1971).

76. Once the pavement thickness t is determined, the thicknesses of each component layer may be determined in a number of ways. The conventional way is to have a 3-in. bituminous concrete surface layer, a 6-in. base course

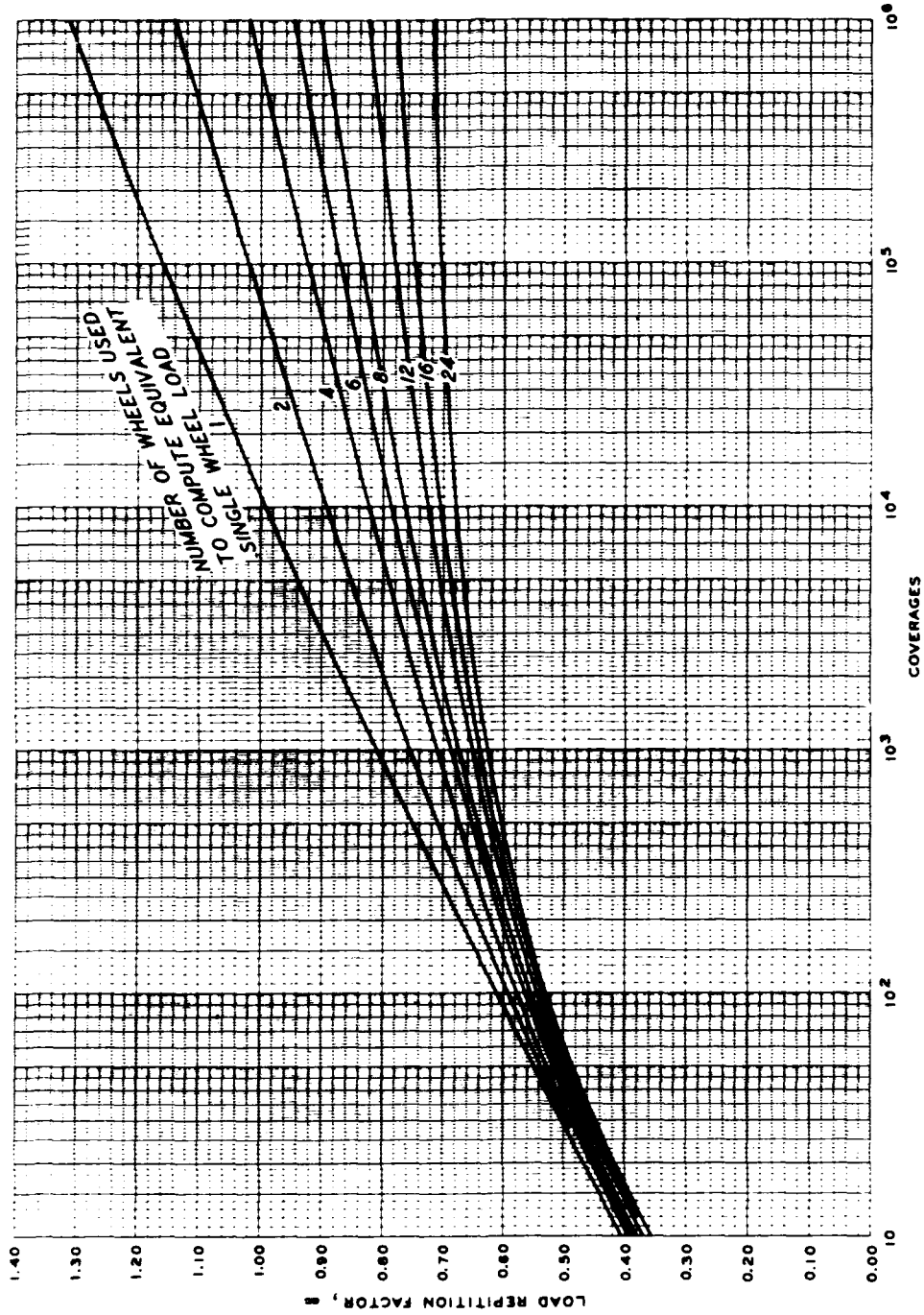


Figure 12. Load repetition factor versus coverages

layer, and a subbase layer with a thickness of (t - 9) in. It is seen that a flexible pavement designed with the CE procedure consists primarily of granular materials covered with a thin layer of bituminous concrete. If the 3-in. bituminous concrete can be converted into an equivalent base course layer, Equation 14 or 15 may be used to design gravel pavements. The proposed layered elastic design procedure is developed based on this principle. Table 22 shows the equivalency ratios for flexible pavement materials

Table 22

Equivalency Factors for Various Materials

Material	Equivalency Factor*
All bituminous concrete (ABC)	1.70
Bituminous-stabilized GW, GP, GM, GC, SW, SP, SM, and SC	1.50
Cement-stabilized GW, GP, SW, and SP	1.60
Cement-stabilized GM and GC	1.45
Cement-stabilized ML, MH, CL, and CH	1.25
Cement-stabilized SM and SC	1.15
Lime-stabilized ML, MH, CL, and CH	1.10
Lime-and-fly-ash-stabilized ML, MH, CL, and CH	1.15
Unbound crushed stone base course	1.40
Unbound granular subbase course	1.00**

* Equivalency factors are based on the use of optimum percent stabilizing agent for durability and strength (from Hammitt, Barker, and Rone 1973).
** Equivalency factor is based on the unbound granular subbase course.

currently used in the CE. The ratio of the bituminous concrete to the granular base material is 1.70/1.40 = 1.22. These ratios are primarily determined based on full-scale accelerated traffic test data conducted at WES. Consequently, these equivalency ratios may be dependent upon the failure criteria adopted in and other conditions in the tests. There is another subject of possible dispute in using Equations 14 and 15 for the design of gravel pavements. The aggregate-surfaced pavement can tolerate more surface deformation or rutting than bituminous concrete-surfaced pavement. The failure criteria used in the tests which form the design equation (Equation 5) for aggregate

pavements were based on permanent deformation (rutting) and elastic deformation. When ruts exceeded a 3-in. depth or when elastic deformations exceeded 1.5 in., a test item was judged failed. Failure was also based on overall subsidence in excess of 4 in. measured from a 10-ft straightedge.

77. In the development of CBR Equations 14 and 15, a pavement item was considered failed when either of the following conditions occurred:

a. Surface upheavel of the pavement adjacent to the traffic lane reached 1 in. or more.

b. Surface cracking occurred to the point that the pavement was no longer waterproof.

78. It is clear that the failure criteria used in the aggregate-surfaced and bitumen-surfaced pavements are quite different. Information presented by Hammitt (1970) indicates that the thickness requirements for unsurfaced pavements (Equation 5) are approximately 75 percent of those determined by the flexible pavement design (Equations 14 and 15). For a given subgrade soil and a given wheel load designed for a given performance level, if the required flexible pavement thickness is t, the required thickness will be $0.75\ t$ when the design is for an aggregate pavement. For convenience of discussion, this fraction is referred to as the "criteria factor." In selecting the criteria factor, the two critical features that should be pointed out are as follows:

a. The criteria factor 0.75 is for earth-surfaced pavements with the CBR values of the cover materials ranging from 7 to 17. It is not known if the factor 0.75 holds true for pavements constructed with high-strength aggregates. Field tests similar to those carried out in Hammitt (1970) should be conducted to determine the criteria factor for pavements constructed with high-strength aggregates. Equation 5 was formulated following the development of the CBR equation for the design of flexible pavements. Hammitt (1969) analyzed the same test data using the regression technique and formulated the following equation for gravel pavements.

$$\begin{aligned}\text{Log thickness} = &- 1{,}02165 + 0.63624 \log \text{pressure} \\ &+ 0.2148 \log \text{load} + 0.2394 \log \text{coverages} \\ &- 0.4028 \log \text{subgrade CBR} - 0.3140 \log \text{cover CBR}\end{aligned} \quad (16)$$

Comparing Equation 16 with Equation 14, the thickness of an aggregate surface for any given loading is about 85 percent of the total thickness of conventional flexible pavements (no equivalencies) above the foundation layer.

b. The criteria factor depends greatly on the failure criteria used in defining failure of the flexible and the earth-surfaced pavements.

79. In conclusion, the proposed layered elastic procedure is developed based on the CBR Equation 14 or 15 in conjunction with predetermined values of equivalency factor and the criteria factor. The effect of the criteria factor is more significant than that of the equivalency factor; the effect of the equivalency factor may be ignored in thick pavements.

Failure Criteria

80. The proposed failure criteria for gravel pavements are established based on two types of loadings. One is the 18,000-lb single-axle dual wheel load, and the other is the C-130 aircraft loading. The 18,000-lb single-axle dual-wheel load has a spacing of 13.5 in. by 58.5 in. by 13.5 in. The tire inflation pressure is 70 psi, and the operation per coverage ratio is 2.64. The C-130 aircraft has such characteristics as gear type (single tandem), tire spacing (60 in. c-c), tire contact area (400 sq in.), gross weight per gear (69,750 lb), and operation per coverage ratio (2.09).

81. Development of the failure criteria is conducted as follows:

a. Step 1: Calculation of the total thickness of conventional flexible pavement for a given subgrade CBR and for a particular performance level (coverage). The required total thickness of conventional flexible pavement is determined from Equation 13 for the particular loading. In using Equation 13, the traffic factor equals to 0.23 log coverage + 0.15 for flexible roads and streets and equals to the value determined from Figure 12 for airfield flexible pavements. The equivalent single-wheel load is computed from the curves shown in Figures 13 and 14 for vehicular and aircraft loads, respectively. These thicknesses are computed for five subgrade CBR values, i.e., 2, 4, 7, 10, and 20 at a range of coverage levels under the 18,000-lb axle load and the C-130 aircraft load. Once the total thickness t is determined, the thickness is divided into three layers, i.e., a 3-in. bituminous concrete surface layer, a 6-in. base layer, and a (t - 9)-in. subbase layer.

b. Step 2: Determination of elastic moduli of granular layers. Assuming that a 1-in. bituminous concrete is structurally equivalent to a 1.22-in. base course material, the flexible pavement is converted into a 9.66-in. base course and a (t - 9)-in. subbase layer. The subbase layer is then subdivided into layers of 4 to 8 in. Figure A1 is used to determine the elastic modulus of each layer, and Equation 1 is used to

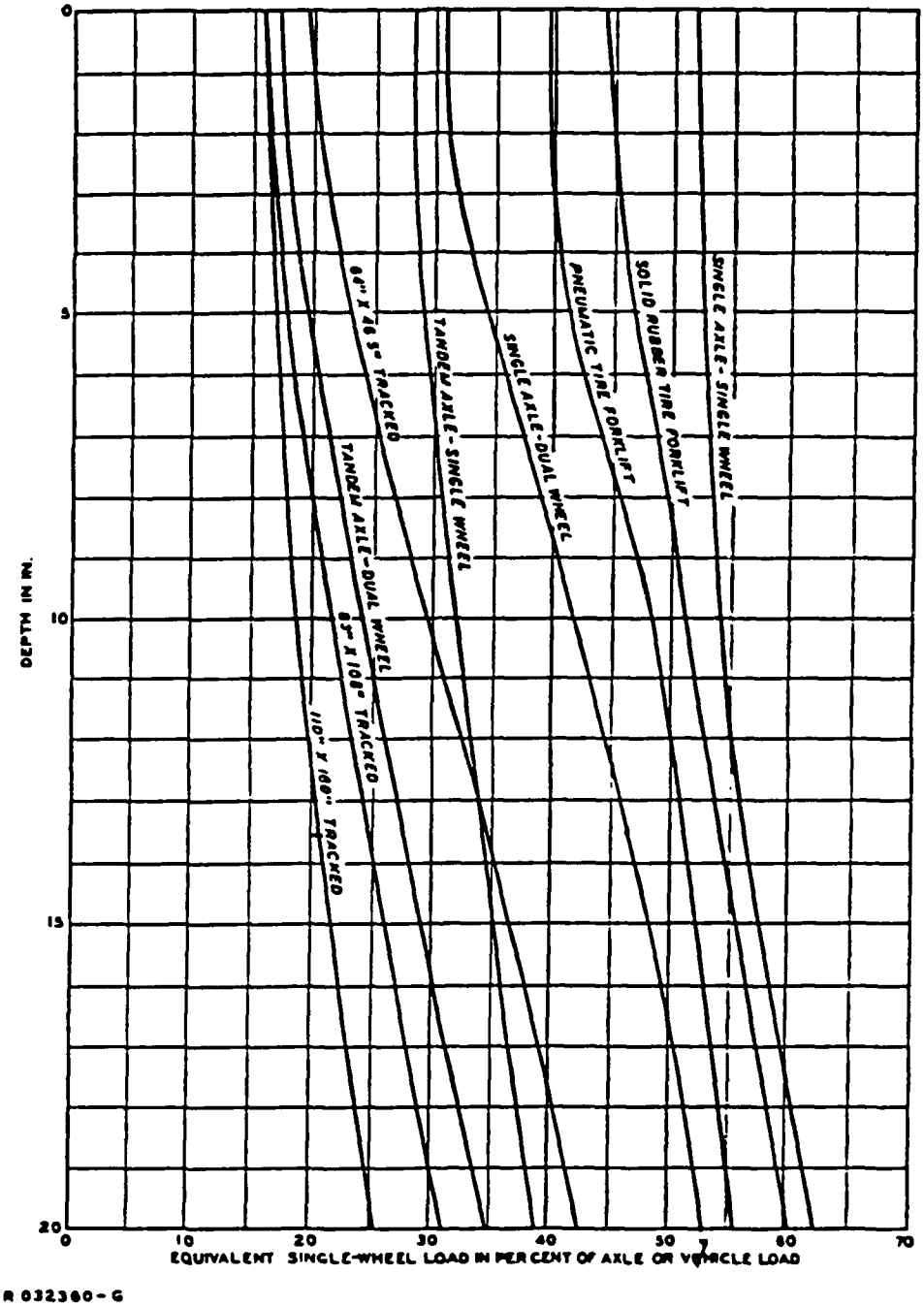


Figure 13. Equivalent single-wheel load in percent of axle or vehicle load versus depth for flexible highway pavement

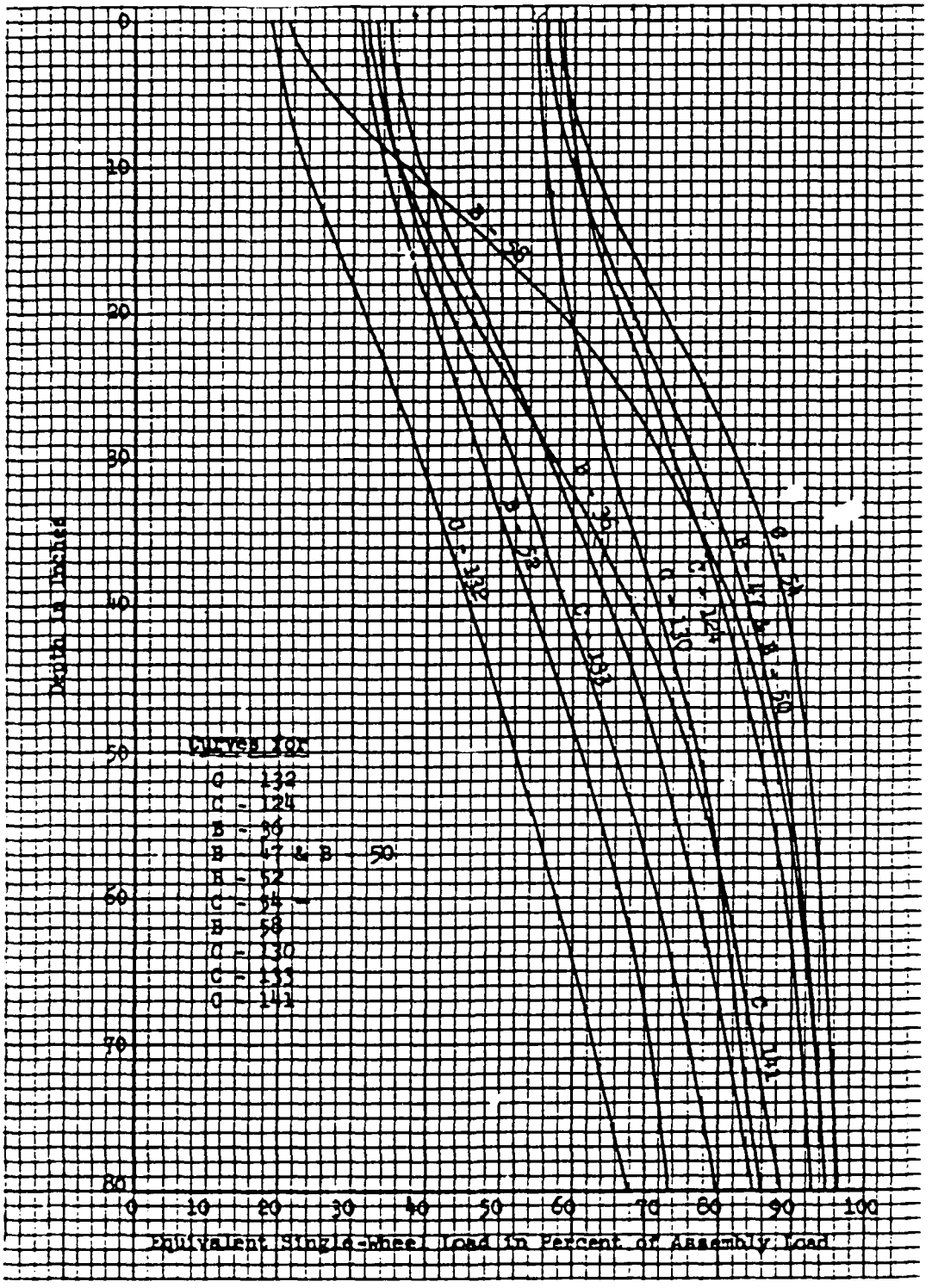


Figure 14. Relationships between multiple-wheel and equivalent single-wheel loads

convert the subgrade CBR to elastic modulus. In establishing the failure criteria two other equivalency ratio's, i.e., 1.5 and 2.0 are also used.

c. Step 3: Failure criteria. The BISAR computer program is used to compute the maximum subgrade vertical strains of the pavements. The relationships between the strain and coverage for various subgrade CBR values are presented in Figure 15. Three sets of failure criteria are presented in the figure, depending upon the equivalency ratio of the bituminous concrete to the base course material; the ratios are 1.22, 1.50, and 2.0. For values between the numbers, an interpolation procedure is necessary.

82. To determine the pavement thickness, Miner's hypothesis is used. The damage factor (DF) is defined as $DF = n/N$, where n is the number of design coverage and N is the number of allowable coverages determined from Figure 15. The cumulative damage factor is the sum of the damage factors for all design vehicles. The required aggregate thickness t_1 is determined for a damage factor of 1. The thickness t_1 is then converted back to the conventional flexible pavement t_2, i.e., a 3-in. bituminous concrete surface layer, a 6-in. base course, and a $(t_2 - 9)$-in. subbase. A criteria factor is then multiplied by t_2 to obtain the required thickness of aggregate pavement. The difference between t_1 and t_2 involves the use of the equivalency factor between the bituminous concrete and the base course material. In fact, the difference is very small.

83. The procedure for the design of gravel pavement using the layered elastic method is illustrated in the design examples presented in Part IX.

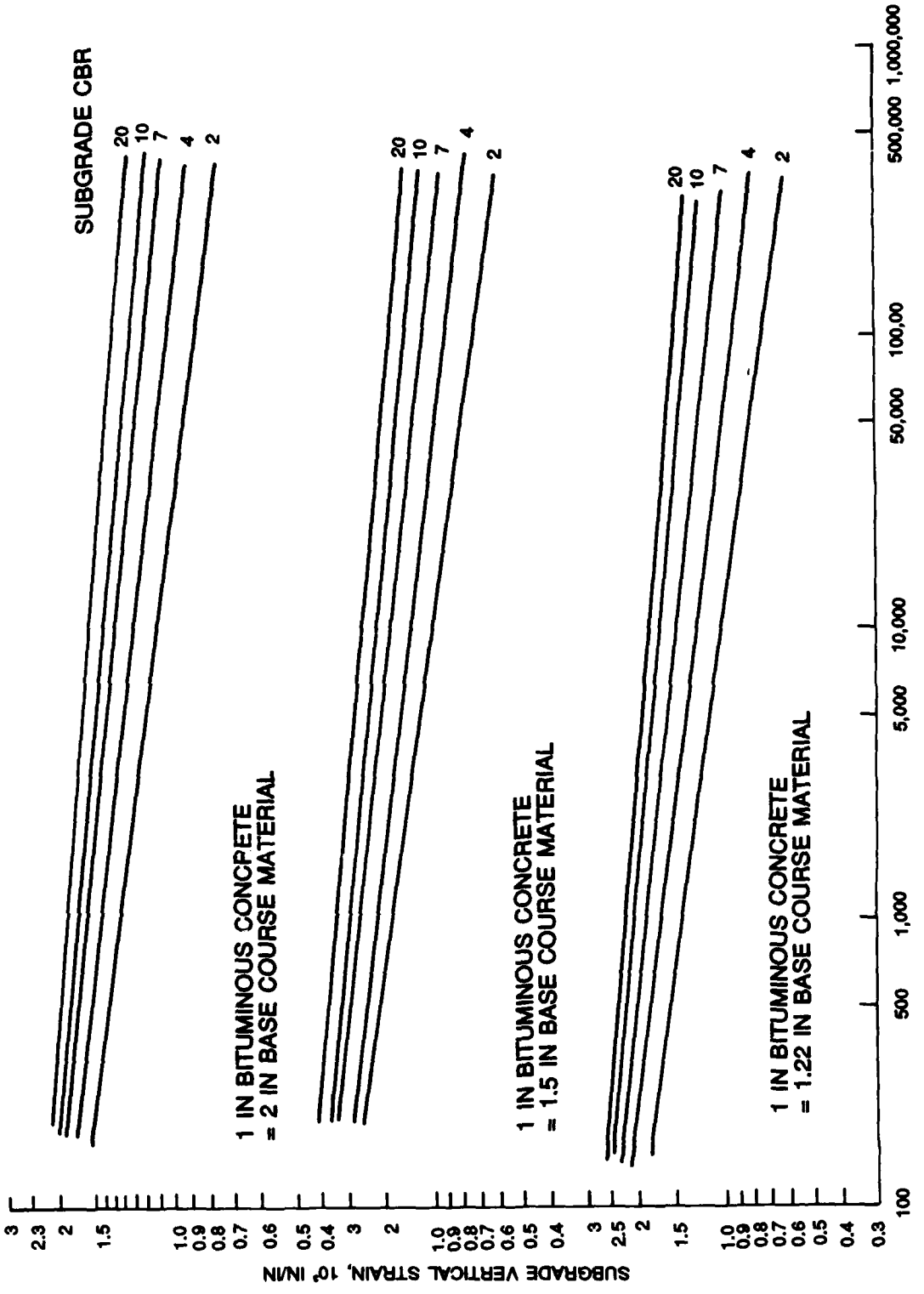


Figure 15. Failure criteria for aggregate-surfaced pavements for roads and airfields

PART VIII: PROBABILISTIC AND RELIABILITY ANALYSIS

84. The three design equations (Equations 9, 6, and 10) presented in the previous sections are all deterministic in nature, i.e., a unique thickness is designed for the unique set of input variables. For instance, in Equation 9 (the rutting equation), a given gravel thickness is determined for given values of subgrade and gravel CBR, wheel load, tire inflation pressure, and design load repetition. However, the effect of input parameter variability on pavement performance is not considered. A quantification of these effects can be accomplished by using the probabilistic approach, and the design procedures can be improved by showing the partial effect of each design parameter on the final design. The final design can be expressed in probability and reliability terms, and the crucial parameters which should be tightly inspected in the construction phases can be stressed.

85. The probabilistic and reliability analysis was conducted for flexible airfield pavements (Chou 1986, 1987). The same method can be used for aggregate-surfaced pavements. The Rosenblueth method (Rosenblueth 1975) is used to evaluate the expected value of the performance function, i.e., $E[f(x)]$, and the variance of the function is computed using the following equation

$$V[f(x)] = E[f^2(x)] - E[f(x)]^2 \qquad (17)$$

i.e., the variance is said to be the mean square minus the square mean. The method is discussed in detail by Chou (1986, 1987) but is not presented in this report.

86. The probabilistic and reliability analysis was conducted on the three design equations and the results are presented below.

Rutting Equation (Equation 9)

87. The dependent variable in Equation 9 is the load repetition, and the independent variables are the CBR's of the aggregate and the subgrade, the thickness of the aggregate layer, the wheel loads, and the tire inflation pressure. The variation in rut depth is not considered in the analysis as the depth of the rut is the criterion for judging failure of the pavement. Using

the Rosenblueth method, the variance and the standard deviation of the load repetition are computed.

88. Assuming the reliability of the rutting equation to be 0.5, i.e., the design curve is drawn through test data points at failure with no consideration of safety factor, and by knowing the standard deviations of the load repetitions, the reliability levels at different load repetitions can be computed. Table 23 shows such results for particular aggregate-surfaced pavements. The results are obtained from the output of a computer program prepared for this study. Reliability is defined as the probability that the pavement system will perform its intended function over its design life and under the conditions encountered during operation (Darter and Hudson 1973).

Table 23

Computed Reliability Values

Reliability of the Design	Load Repetitions
0.5 + 0.499 = 0.999	$14,118 - 3\ \sigma_{rep} = 0$
0.5 + 0.495 = 0.995	$14,118 - 2.5\ \sigma_{rep} = 0$
0.5 + 0.477* = 0.977	$14,118 - 2\ \sigma_{rep} = 0$
0.5 + 0.419 = 0.919	$14,118 - 1.5\ \sigma_{rep} = 0$
0.5 + 0.341* = 0.841	$14,118 - \sigma_{rep} = 0$
0.5 + 0.155 = 0.655	$14,118 - 0.5\ \sigma_{rep} = 5,591$
0.5 + 0 = 0.5	14,118 + 0 = 14,118
0.5 - 0.155 = 0.345	$14,118 + 0.5\ \sigma_{rep} = 22,646$
0.5 - 0.341 = 0.159	$14,118 + \sigma_{rep} = 35,437$
0.5 - 0.419 = 0.081	$14,118 + 1.5\ \sigma_{rep} = 43,964$
0.5 - 0.477 = 0.023	$14,118 + 2\ \sigma_{rep} = 56,755$
0.5 - 0.495 = 0.005	$14,118 + 2.5\ \sigma_{rep} = 69,546$
0.5 - 0.499 = 0.001	$14,118 + 3\ \sigma_{rep} = 78,073$

Note: Repetition computed with mean input values = 14,118; standard deviation of load repetition, σ_{rep} = 21,318 repetitions; reliability of the rutting equation = 0.5; aggregate thickness = 10 in., aggregate CBR = 18, subgrade CBR = 3, load = 9,000 lb, tire pressure = 75 psi, rut depth = 3 in., and the coefficients of variation (CV) of thickness, aggregate CBR, subgrade CBR, load, and tire pressure are 0.1, 0.25, 0.25, 0.1, and 0.1, respectively.

* 0.34 and 0.48 are half of the area within ±1 and 2, respectively, standard deviation under a normal distribution curve.

89. The pavement information for the example pavement shown in Table 23 is gravel thickness = 10 in., CV = 0.1; gravel CBR = 18, CV = 0.25; subgrade CBR = 3, CV = 0.25; wheel load = 9,000 lb, CV = 0.1; tire pressure = 75 psi, CV = 0.1; and rut depth at failure = 3 in., CV = 0. CV is the coefficient of variation which is defined as the ratio of the standard deviation to the mean. For instance, if the mean value of the wheel load is 9,000 lb and the CV is 0.1, the standard deviation of the wheel load will be 900 lb. Since the area within plus and minus one standard deviation under a normal distribution curve is 0.68, the wheel load ranges from 8,100 to 9,900 lb 68 percent of the time.

90. The computed repetition at the mean input values using Equation 8 is 14,118. Assuming the CV's of the aggregate thickness, gravel CBR, subgrade CBR, wheel load, and tire inflation pressure are 0.1, 0.25, 0.25, 0.1 and 0.1, respectively, which are considered representative of field conditions, the standard deviation computed for the 14,118 load repetition is 21,318 repetitions. The extremely large standard deviation indicates that considerable uncertainty is involved in the design of aggregate-surface pavements using Equation 9. Table 23 shows that for the particular pavement, it is not feasible to have a design with a reliability level higher than 0.84. Since the area within ±1 standard deviation is 0.68, one could argue that there is a 68 percent chance that the predicted performance of this aggregate-surface pavement falls within the range of 0 and 35,437 repetitions. This indicates that the design has a rather low confidence level, which is possible due to the nature of aggregate-surfaced pavements. Equation 9 was formulated based on the results of 254 data points using the regression method, but the data were quite scattered.

91. Another possible reason for the low level of confidence in the computed results shown in Table 23 is that the input parameters are not as normally distributed as they were assumed. Figure 16 shows the distribution of aggregate CBR from which Equation 9 is formulated. It is seen that for aggregate CBR less than 20, the distribution can adequately be represented by a normal curve. The presence of a small group of data points with aggregate CBR greater than 20 makes the assumption of normal distribution less desirable. Similar arguments can be formulated for other input parameters. It may be desirable to formulate separate equations for Equation 8 with the conditions that the aggregate CBR is greater and smaller than 20 to increase the confidence level of the design.

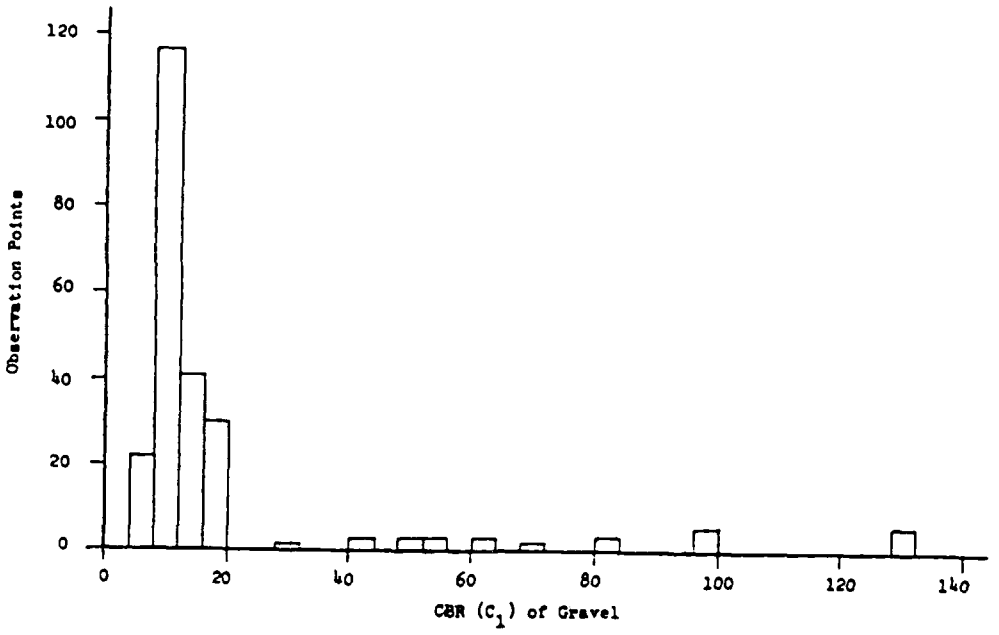


Figure 16. Aggregate-surfaced facility, aggregate CBR distribution

92. Reliability levels presented in Table 23 are for a pavement with a 10-in. aggregate surface. Similar computations were conducted for other thicknesses, and the results are plotted in Figure 17. For a given aggregate thickness, the reliability of the design can be increased (or decreased) when the design performance (load repetitions) is decreased (or increased). At a given design load repetition, the reliability of the design can be increased (or decreased) by increasing (or decreasing) the aggregate thickness. For instance, for a 10-in. pavement, the predicted performance is 14,118 repetitions for a reliability level of 0.5 (which is assumed to be inherent in the rutting equation (Equation 9). The reliability is increased to 0.7 if the same pavement is designed to last only 300 repetitions but is reduced to 0.2 if this pavement is designed to last 31,000 repetitions. Another interpretation is that for a 10-in. aggregate-surfaced pavement designed by the rutting equation, the chance of success that the pavement will last 14,118 repetitions is 50 percent (as the assumed reliability of the rutting equation); the chance that the pavement will last to at least 300 repetitions is increased to 70 percent; and the chance that the pavement will last 31,000 repetitions is decreased to 20 percent. It is noted in Figure 17 that when the thickness of the gravel layer continues to increase, the reliability level of the design cannot be increased beyond a certain limit while the design load repetition

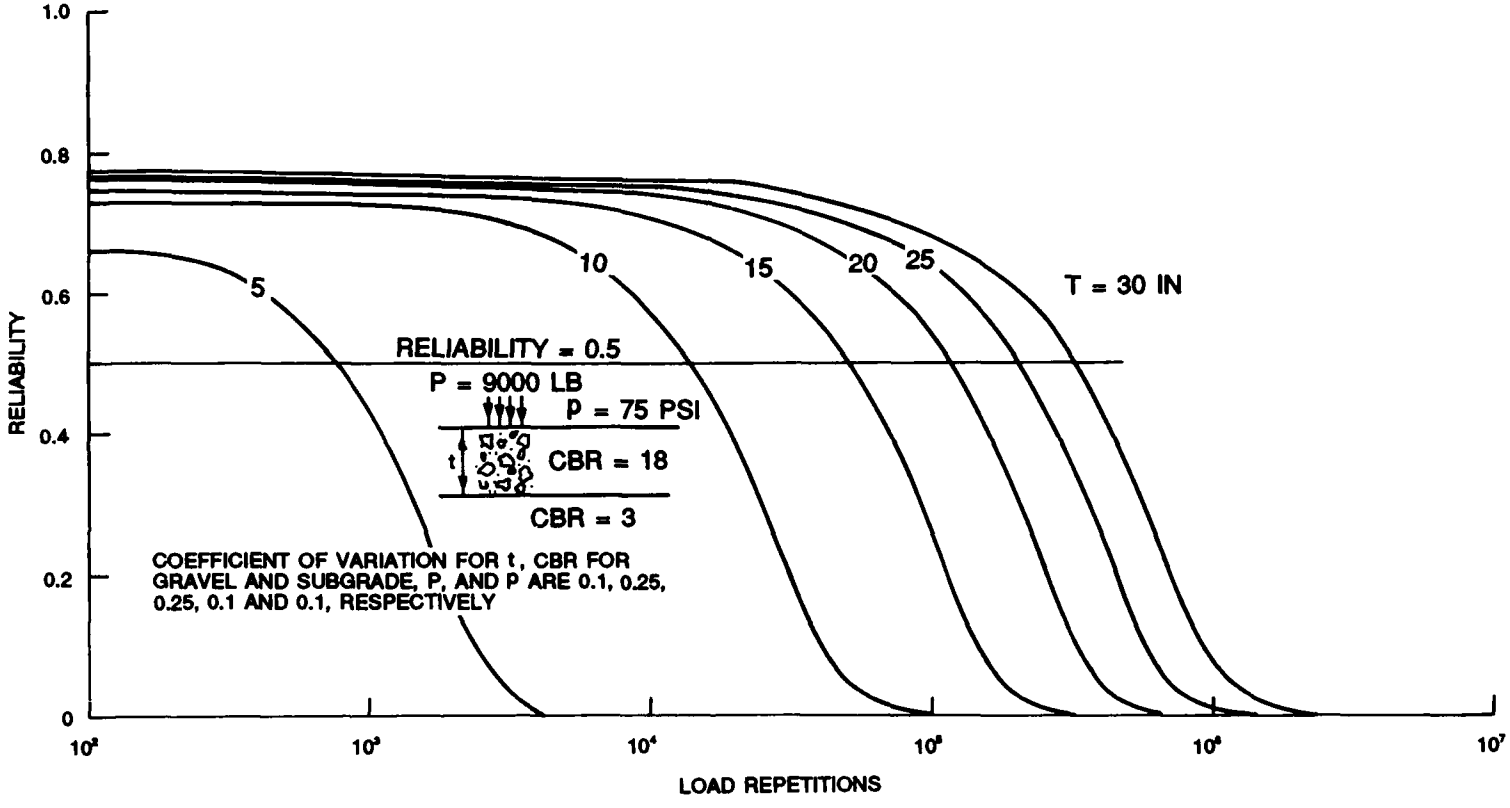


Figure 17. Relationships between reliability and load repetition for varying gravel layer thicknesses, rutting equation with reliability of 0.5

continues to increase. This is caused by the large standard deviation computed for the predicted load repetitions.

93. The results presented in Figure 17 and Table 23 assume that all five design parameters have variations. To study the effect of each individual parameter on pavement performance, computations were made while varying only one parameter at a time (CV = 0.1); the results are tabulated in Tables 24 through 26 for the rutting equation (Equation 9), CE equation (Equation 6), and AASHTO equation (Equation 10), respectively. In the tables, the degree of influence for each input parameter was ranked. The degree of influence indicates the effect of the variability of the particular input parameter on the predicted load repetition, and it is proportional to the standard deviation of the predicted load repetition. A ranking of one indicates that it is most significant, and a ranking of five indicates that it is the least significant. The ranges of load repetitions within ±1 standard deviation are also shown in the tables.

Table 24

Performance Variations as a Function of Input Variables, Rutting Equation*

(Repetition Computed as Mean Inputs, REPE = 14,118)

Coefficient of Variation**					Standard Deviation of Repetition, σ_{rep}	Degree of Influence†	Repetitions	
T	Aggregate	Subgrade	P	psi			REPE- σ_{rep}	REPE+ σ_{rep}
0.1	0.0	0.0	0.0	0.0	4,965	2	9,153	19,083
0.0	0.1	0.0	0.0	0.0	5,366	1	8,752	19,484
0.0	0.0	0.1	0.0	0.0	1,623	5	12,495	15,741
0.0	0.0	0.0	0.1	0.0	2,735	4	11,383	16,853
0.0	0.0	0.0	0.0	0.1	3,325	3	10,793	17,443

Note: Load P = 9,000 lb, gravel thickness t = 10 in., aggregate CBR = 18, subgrade CBR = 3, tire inflation pressure p = 75 psi.

* Headquarters, Department of the Army (1985).

** T, Aggregate, Subgrade, P, and psi stand for the thickness of the gravel layer, aggregate CBR, subgrade CBR, load P, and tire inflation pressure, respectively.

† Degree of influence indicates the effect of the variability of a given input parameter on the predicted performance of the aggregate-surfaced pavement. One is most significant, and five is least significant.

Table 25

Performance Variables as a Function of Input Variables, Corps of Engineers' Equation*

(Repetition Computed as Mean Inputs, REPE = 273)

Coefficient of Variation CV					Standard Deviation of Repetition	Degree of	Repetitions	
T	Aggregate**	Subgrade	P	psi	σ_{rep}	Influence†	REPE- σ_{rep}	REPE+ σ_{rep}
0.1	0.0	0.0	0.0	0.0	213	1	60	486
0.0	0.0	0.1	0.0	0.0	112	2	161	385
0.0	0.0	0.0	0.1	0.0	103	3	170	376
0.0	0.0	0.0	0.0	0.1	11	4	262	284

Note: Load P = 9,000 lb, aggregate thickness t = 10 in., aggregate CBR = 18, subgrade CBR = 9, tire inflation pressure p = 75 psi.

* Headquarters, Department of the Army (1968).

** The aggregate CBR is not involved in the CE equation.

† Degree of influence indicates the effect of the variability of a given parameter on the performance of the aggregate-surfaced pavements. One is most significant, and five is the least significant .

Table 26

Performance Variables as a Function of Input Variables, AASHTO Equation 10

(Repetition Computed as Mean Inputs, REPE = 22,322)

Coefficient of Variation*					Standard Deviation of Repetition	Degree of	Repetitions	
T	Aggregate	Subgrade	P	psi	σ_{rep}	Influence**	REPE- σ_{rep}	REPE+ σ_{rep}
0.1	0.0	0.0	0	0	6,735	1	15,587	29,057
0.0	0.1	0.0	0	0	1,867	3	20,455	24,189
0.0	0.0	0.1	0	0	3,119	2	19,203	25,441

Note: Load P = 9,000 lb, aggregate thickness t = 10 in., aggregate CBR = 18, subgrade CBR = 3, tire inflation pressure p = 75 psi.

* 9,000-lb wheel load with 75-psi tire inflation pressure were held constant in the analysis computations.

** Degree of influence indicates the effect of the variability of a given parameter on the performance variation of the aggregate-surfaced pavement. One is most significant, and five is the least significant.

94. Table 24 shows that the load repetition computed by the rutting equation (Equation 8) is most sensitive to the variation of CBR of the gravel layer and, in descending order, to the variations of the thickness of the gravel layer, the tire inflation pressure, the wheel load, and the subgrade CBR. When only the variation of the gravel CBR is accounted for (CV = 0.1), there is a 68 percent chance (68 percent is the percent of area covered with ±1 standard deviation under a normal distribution curve) that the predicted performance falls within the range between 8,752 and 19,484 repetitions. Where only the variation of the subgrade CBR is accounted for and for the same 68 percent chance, the predicted performance will range between 12,495 and 15,741 repetitions representing a smaller variation. A larger range of predicted pavement performance indicates that the design has a greater amount of uncertainty.

95. It is noted in Table 24 that the performance of a gravel-surface pavement is quite sensitive to the variation of tire inflation pressure. It is understandable that highly inflated tires can rut and deteriorate the gravel surface easily. It is important to point out that in the reliability analysis of flexible airfield pavements (Chou 1986, 1987), it was found that variation of the tire inflation pressure has only a very insignificant effect on the performance of flexible pavements. The relatively much more rigid bituminous concrete surface layer can resist the tire pressure rutting of the pavement better than the gravel layer.

CE Equation (Equation 6)

96. Equation 6 was developed based on traffic tests conducted at WES (Hammitt 1970). The resultant equation was formulated following the same pattern as the CBR equation for flexible pavement (Turnbull and Ahlvin 1957). The thickness equation is shown in Equation 5, and the requirement for gravel CBR is shown in Figure 2. In the reliability analysis of the CBR equation for flexible pavement (Chou 1986), it was found that pavement performance is least (and nearly not) sensitive to the variation of the tire inflation pressure and is most sensitive to the variation of the pavement thickness. The magnitude of the effects of the variation of the wheel load and the subgrade CBR on the pavement performance is practically the same. Since the stiffnesses of the bituminous and granular layers are not considered in the CBR equation for

flexible pavement, their variations were not considered in the reliability analysis (the stiffness requirements are considered in the material, compaction, and construction specifications).

97. Table 25 shows the variations of load repetitions computed for the CE equation (Equation 5) for aggregate-surfaced pavements. The variation of gravel CBR is not involved in the analysis as the gravel CBR is not shown in the equation but is separately determined in Figure 2. The pavement performance is least (and nearly not) sensitive to the variation of the tire inflation pressure and is most sensitive to the variation of the gravel thickness. The magnitude of the effects of the variation of wheel load and subgrade CBR on the pavement are nearly the same. This conclusion is almost identical to these conclusions derived for the CBR equation for flexible pavements but deviates from those of the rutting equation (Equation 9). It seems that Equation 6 behaves similarly to the CBR equation for flexible pavements as it is formulated following the pattern of the CBR equation.

AASHTO Equation (Equation 10)

98. The dependent variables in Equation 10 are parameters which affect the magnitude of compressive strains in the subgrade. When Equation 11 was formulated using the layered elastic method, the magnitudes of wheel load and tire inflation pressure were held constant. Consequently, Equations 10 and 11 are dependent only on the thickness and CBR of the aggregate layer and the subgrade CBR. The results of the reliability analysis of Equation 10 are tabulated in Table 26. The load repetition is most sensitive to the variation of aggregate thickness and then the subgrade CBR; the repetition is least sensitive to the variation of the aggregate CBR.

99. In another study the layered elastic method was used to analyze a three-layer flexible airfield pavement in terms of probability and reliability (Chou 1987). It was found that for subgrade failure criterion (which differs from the asphaltic concrete failure criterion), the pavement performance is sensitive to the variations (in descending order) of the wheel load, the thickness of the aggregate layer, the subgrade modulus, the thickness of the asphaltic layer, the aggregate modulus, and the asphaltic concrete modulus. When the asphaltic surface layer is removed, the performance of the aggregate-surfaced pavement becomes sensitive to the variations (in descending

order) of the wheel load, the thickness of the aggregate layer, the subgrade modulus, and the aggregate modulus.

100. The conclusions presented in Table 26 for the AASHTO equation (Equation 10) are identical to those derived for the three-layer flexible airfield pavement using the layered elastic theory (Chou 1987). This is logical because the AASHTO equation (Equation 10) is also formulated based on the layered elastic method.

101. The AASHTO road test data relate only to asphaltic-surfaced roads; no comparable data are available for aggregate-surfaced roads. The AASHTO equation for flexible pavements is assumed to be applicable for aggregate-surfaced pavement because the load repetition in the equation is formulated in terms of subgrade compressive strain. As long as the strains are computed using the layered elastic method, the equation is assumed applicable even if the pavement does not have a layer of asphaltic concrete on the surface. It was believed that such an assumption was reasonable. In view of the fact that the results of the reliability analysis of the AASHTO equation (Equation 10) are different from those of the rutting equation (Equation 9), e.g. the computed load repetition is more sensitive to the variation of subgrade CBR than to that of the aggregate CBR in the AASHTO equation (Equation 10) while the reverse is true for the rutting equation (Equation 9), it casts some doubt on the applicability of the assumption used in the AASHTO equation (Equation 10).

PART IX: EXAMPLE PROBLEMS

102. Three design examples for granular-surfaced pavements are presented. The first example is for truck loadings of different axle load and type. The second example is for a tank trail, and the third is for aircraft loadings. In each example appropriate design procedures are used to calculate the required thicknesses.

Design Example No. 1, Truck Loadings

103. A two-lane aggregate-surfaced road is to be designed to cross an area of clayey soil where frost design is assumed to be not applicable. According to the Unified Soil Classification System, the soil is classified as CL soil with PI = 15 and is considered as a frost-susceptible F3 soil. The designed CBR of the soil is 5. The anticipated traffic is 1,500 operations per day with 15 percent trucks. The general distribution of truck axle loads is as follows:

Axle Load and Type	Number of Axles per 100 Trucks
12,000 lb, single	102
24,000 lb, single	54
36,000 lb, tandem	44

The design problem is to determine the required thickness of the roadway.

Step 1: Determination of equivalent 18,000-lb single-axle load repetitions

104. In this example only the given truck axle load distribution will be used to determine equivalent 18,000-lb single-axle load repetitions because passenger cars cause insignificant damage to the pavement as compared to the truck loads. The first step is to determine equivalent 18,000-lb axle loads (N_{18}) per 100 trucks. Equivalent factors found in Figure C1 for the CE procedure are 0.16, 5.5, and 6.3, respectively, for the three types of axles for which data are given above. The equivalent factors found in Table C1 for the AASHTO design method are 0.18, 3.62, and 1.38, respectively. Thus, for each 100 trucks

$$N_{18} = 0.16 \times 102 + 5.5 \times 54 + 6.3 \times 44 = 590$$

or 5.9 per truck (CE)

and

$$N_{18} = 0.18 \times 102 + 3.62 \times 54 + 1.38 \times 44 = 275$$

or 2.75 per truck (AASHTO)

The total number of N_{18} in one lane during a 25-year design life is

$$N_{18} = 5.9 \times (0.15 \times 1500 \text{ (vpd)}/2) \times 25 \times 365 = 6{,}056{,}700^{*} \quad \text{(CE)}$$

and

$$N_{18} = 2.75 \times (0.15 \times 1500 \text{ (vpd)}/2) \times 25 \times 365 = 2{,}823{,}000^{*} \quad \text{(AASHTO)}$$

Step 2: Thickness designs

105. CE design equation. In order to use Equation 6 the number of passes have to be converted to coverages. The pass for coverage ratio for an 18-kip single axle with dual tires is 2.64 (Table 7 of US Army Engineer Waterways Experiment Station 1961). For 6,056,700 operations of an 18-kip single-axle load as determined in Step 1, the coverage (C) to be used in Equation 6 is 2,294,200. Assuming the tire inflation pressure is 70 psi, the magnitude of the equivalent single wheel load (ESWL) is computed using the following procedure.

106. Figure 13 shows the relationships between the ESWL and pavement thickness for various gear types and configurations of highway and warehouse loadings. A trial and error procedure has to be used to determine the magnitude of the ESWL as the pavement thickness is determined from Equation 5 based on the magnitude of the ESWL (P).

107. Assuming the aggregate thickness is 12 in., the single-axle dual wheel curve in Figure 13 indicates the ESWL is $18{,}000 \times 44.5$ percent = 8,010 lb and the tire contact area is $4{,}500/70 = 64.3$ sq in. For a subgrade CBR of 5 and $C = 2{,}294{,}200$, the required thickness t is 13.4 in. Assuming $t = 13.4$ in. and following the same procedures, the computed required thickness t is 13.6 in. Thus, $t = 14$ in. is the designed thickness.

108. Note that the CBR value of the aggregate layer is not included in Equation 6. The required CBR value can be determined from Figures 2 and 3.

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* Aggregate-surfaced roads are defined by AASHTO Guide for Pavement Design as having 10,000 to 100,000 passes of 18,000-lb ESWL travel level. The design traffic volume used in the example is obviously too high.

Figure 2 shows that for an equivalent single wheel load of 8,640 lb (= 0.48 × 18,000 lb, where 0.48 is the factor determined from Figure 13), 98 CBR is required with an expected rut of approximately 2 to 3 in. Figure 3 shows that for a tire pressure of 70 psi and an expected 2,294,200 coverage designed life, the required CBR of the aggregate layer is estimated to be about 280 to minimize the surface rutting. In other words, it is not possible to minimize the rutting on an aggregate road designed for such a high coverage level.

109. CE rutting equation. Equation 9 is used to compute the aggregate thickness t . Similar to Equation 6, the thickness depends upon the magnitude of the ESWL which in turn is dependent upon the pavement thickness. Therefore, an interational procedure is needed to determine the required thickness. The required thicknesses t are tabulated in Table 27 for two rut depths and the required CBR of the aggregate layer. The computations are based on the following input information.

110. Table 27 shows that the aggregate layer thickness is extremely sensitive to the CBR of the aggregate used. For instance, for an expected rut depth of 2 in., the required aggregate thickness is 15 in. for an aggregate CBR of 44. If the gravel CBR is increased to 50, the required gravel thickness can be reduced to 10 in.

C_2 (subgrade CBR) = 5

R (load repetition in passes) = 6,056,700

t_p (tire pressure) = 70 psi

Table 27

Relationships Between Aggregate Thickness and Required Aggregate CBR

Aggregate Thickness in.	Expected Rut Depth in.	Required Aggregate CBR
10	2	50
12	2	47
14	2	45
15	2	44
10	3	32
12	3	30
14	3	29
15	3	28

111. CE design index method. For an anticipated traffic of 1,500 operations per day, the facility is classified as Class E. For 15 percent trucks with a distribution of truck axle loads of 22 percent tandem-axle and 78 percent single-axle, the facility will be categorized as Category IV. Table 14 shows that the DI is 4, and the required gravel thickness is 9 in. as determined from Figure 4. The required CBR value of the gravel layer is determined from Figures 2 and 3.

112. TRRL design procedure. Figure 5 shows that for a subgrade CBR of 5 and 2,823,000* repetitions of 18,000 lb single-axle load, the required thickness of surface-treated pavement is 16.5 in. Multiplying by a factor of 0.78, the required gravel layer thickness is 13 in. The TRRL procedure recommends a minimum base thickness of 6 in. with a minimum CBR value of 80. The remaining thickness (subbase) consists of a material having a minimum CBR value of 25.

113. US Forest Service procedures. For design values of serviceability index and rut depth, the following three factors should first be evaluated.

a. Soil support. Table 16 indicates that for a subgrade CBR of 5, the soil support value is 4.0.

b. Structural number. $SN = a_1 D_1$ where D_1 is the thickness (in.) of the gravel layer and a_1 is a coefficient depending upon the CBR value of the gravel layer in Table 17.

c. Design life. The initial index P_I is assumed to be 4.0 and the failure index P_T is designated as 1.5.

114. The basic design factors are brought together in Tables 18 and 19 for determining the values of SN for PSI and rut depth criteria, respectively. The determined SN is 3.65 and 3.1 for PSI and rut depth, respectively. The required gravel thicknesses D_1 are shown in Table 28 for various CBR's of the gravel layer. It is seen that thicknesses designed in this special case using the US Forest Service procedure are much greater than those using the CE procedure. Table 28 also shows that in this particular example the PSI design criterion controls the design.

115. AASHTO design procedure. This procedure is not used because it should give the same design as the US Forest Service design procedure.

116. Elastic layered method (Luhr, McCullough, and Pelzner) procedure. Equation 10 can be used to compute the number of repetitions of the 18-kip

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* It has exceeded slightly the design limitation of 2,500,000 passes.

Table 28

Aggregate Layer Thickness Determined from US Forest Service Design Procedure

CBR of Aggregate Layer	a_1**	D_1*, in. PSI Criterion $D_1 = 3.65/a_1$	D_1*, in. Rut Depth Criterion $D_1 = 3.1/a_1$
20	0.070	52	44
30	0.093	39	34
40	0.107	34	29
50	0.117	31	27
60	0.126	29	25
70	0.132	28	23
80	0.136	27	23
90	0.138	26	22
100	0.140	26	22

* From Table 17.
** For a load repetition N_{18} = 2,823,000.

axle load N_{18} . The last term of the equation is canceled for the condition $P_i = 4.2$. The compressive strain at subgrade surface ε_{SG} is computed from Equation 12. To determine the thickness designed for the 2,823,000 operations (see Step 1) of 18-kip axle load, a series of aggregate thicknesses D_{BS} is selected and for each thickness, a series of aggregate moduli E_{BS} is used to determine the strain ε_{SG} ; the thicknesses and moduli of the aggregate layer corresponding to an N_{18} = 2,823,000 are determined and tabulated in Table 29.

117. Table 29 indicates that the aggregate thicknesses computed using the layered elastic method are extremely sensitive to the CBR value of the aggregate layer. Six inches of aggregate can be eliminated if the CBR is increased merely from 43 to 56. Note that similar conclusions were obtained in the rutting equation (Equation 9).

118. To consider aggregate loss in the design, expressions similar to Equation 13 may be used. Assuming the aggregate loss is 0.5 in. per year, the average loss during a 25-year design life would be $(25 \times 0.5)/2 = 6.25$ in. This amount of aggregate loss should be added to the design computed by Equation 10 shown in Table 29.

Table 29

Aggregate Layer Thicknesses Determined from the Layered Elastic Method

Strength of Aggregate Layer		Aggregate Layer
E, PSI	CBR*	Thickness, in.
30,000	56	10
28,600	52	12
27,450	49	13
26,650	47	14
25,850	45	15
25,040	43	16

* CBR values are computed from the elastic modulus values using Equation 2, $E = 1{,}800(CBR^{0.7})$.

119. Proposed elastic layered method. The configurations of the axle loads are assumed as presented in Table 30.

Table 30

Axle Configurations

Configuration	Axle Load, lb	Dimension, in.	Operations Per Coverage*
Single-axle, single wheels	12,000	72.0	6.29
Single-axle, dual wheels	24,000	13.5-58.5-13.5	2.37
Tandem-axle, dual wheels	36,000	13.5-58.5-13.5-48	1.03

* From US Army Engineer Waterways Experiment Station (1961).

120. Passenger cars are assumed to have no significantly detrimental effect on pavement service life and are thus discarded in computations. The design will be based on the 15 percent trucks which have a total number of $0.15 \times 1{,}500 \times 365 \times 25 = 2{,}053{,}125$ operations. A number of coverages for each axle group are computed as follows:

12,000-lb single-axle, single wheels

$$(2{,}053{,}125/100) \times 102/6.29 = 332{,}939 \text{ coverages}$$

24,000-lb single-axle, dual wheels

$$(2,053,125/100) \times 54/2.37 = 467,800 \text{ coverages}$$

36,000-lb tandem-axle, dual wheels

$$(2,053,125/100) \times 44/1.03 = 877,052 \text{ coverages}$$

121. Assuming the 3-in. bituminous concrete is equivalent to a 3.66-in. base course material (see Table 22) and assuming aggregates satisfying the CE base course materials are used in the design, three base course thicknesses, i.e., 7, 11, and 14 in., are used in the computations. These result in total aggregate thicknesses of 13.66, 16.66, and 17.66 in., respectively. The strains computed using the BISAR program are tabulated in Table 31. In the table the allowable coverages obtained from Figure 15 and the computed damages are included. It is seen that for a damage factor of 1, the required aggregate thickness is 19.16 in. With an equivalency ratio of 1.22, the required flexible pavement

Table 31

Damage Computations

Axle load lb (1)	Design Coverage (2)	Subgrade Strain, in./in. (3)	Allowable Coverage (4)	Damage (2)/(4)
		13.66-in. Pavement		
12,000	332,939	0.00102	60,000	5.55
24,000	467,800	0.00139	7,600	61.55
36,000	877,052	0.00105	50,000	17.54
			Total damage	84.64
		16.66-in. Pavement		
12,000	332,939	0.000617	5,000,000	0.07
24,000	467,800	0.000956	140,000	3.34
36,000	877,052	0.000726	1,500,000	0.58
			Total damage	3.99
		17.66-in. Pavement		
12,000	332,939	0.000407	$> 10^8$	0.00
24,000	467,800	0.000677	3,000,000	0.16
36,000	877,052	0.000513	20,000,000	0.04
			Total damage	0.20

thickness becomes 18.5 in. With criteria factors of 0.75 and 0.85 the required gravel thicknesses are 16 and 15.7 in., respectively.

122. Discussions. Table 32 is a summary of the design values obtained based on various procedures. It is seen that for the same traffic and subgrade strength, the required thickness varies greatly among the procedures. For procedures in which the strength of the gravel layer is considered, the required thickness is found to be very sensitive to the strength of the gravel layer. Unfortunately, accurate measurements of the strength of unbound granular materials are very difficult to obtain; consequently, the reliability of the design procedures is low.

Table 32

Aggregate Thickness, Truck Loadings

Design Method	Required Thickness in.	Required Aggregate CBR	Expected Rutting in.
CE design equation*	15.0	98	2-3
CE rutting equation*	10.0	50	2
	15.0	44	2
	10.0	32	3
	15.0	28	3
CE design index method	9.0	--	--
TRRL	26.0	100	--
	31.0	50	--
	52.0	20	--
Elastic layered method (Luhr, McCullough, and Pelzner)**	15.0	45	--
	13.0	49	--
	10.0	56	--
Proposed elastic layered method	14.0†	Satisfying CE base course requirement	--
	15.7		2-3

* The equation is developed based on results of test pavements with a low CBR value of the aggregate layer.

** The direct application of the regression equation for flexible pavements to aggregate-surfaced pavements may be questionable.

† 14 in. for a criterion factor of 0.75, and 15.7 in. for a criterion factor of 0.85.

Design Example No. 2, Tank Trail

123. Four methods can be used in designing aggregate-surfaced pavements subject to tank loadings. They are (a) design index method, (b) equivalent

18-kip single-axle load method, (c) elastic layered method by Luhr, McCullough, and Pelzner, and (d) the proposed elastic layered method. Assuming that the same subgrade soil condition (CBR = 5) is not used and the consideration of frost action is not necessary, the aggregate thickness is to be determined for an anticipated traffic of 40 passes per day of 60-ton tracked vehicles.

Step 1: Determination of design index

124. From Table 13, select the traffic category for a 60-ton (120,000-lb) tracked vehicle as in Part VI. The DI is then determined from Table 15 to be 10 for 40 passes per day.

Step 2: Determination of equivalent 18-kip single-axle load

125. Table 33 may be used to convert the DI to the equivalent 18-kip single-axle load for gravel-surfaced roads and flexible pavements. For a DI of 10, the equivalent 18-kip single-axle load is 2×10^9 operations or 1×10^9 operations per lane in one direction.

Table 33

Equivalent 18-kip Single-Axle Load

Design Index	Equivalent 18-kip Single Axle Load
1	3.1×10^3
2	1.35×10^4
3	5.9×10^4
4	2.6×10^5
5	1.15×10^6
6	5.0×10^6
7	2.25×10^7
8	1.0×10^8
9	4.4×10^8
10	2.0×10^9

Step 3: Thickness designs

126. CE design equation. For a coverage of 3.788×10^8 (= $1 \times 10^9/2.64$) and a tire pressure of 13.3* psi, the thickness of the hardstand

* Assuming full contact, the contact pressure is calculated as (60 × 2,000)/(2 × 180 × 25) = 13.3 psi.

can be computed from Equation 6. As in design example No. 1, Figure 13 is used to estimate the ESWL. The ESWL p is based on the thickness t . The thickness so determined is 13 in. The required aggregate CBR can be determined from Figures 2 and 3.

127. CE rutting equation. For a load repetition of 1×10^9 passes and a tire pressure of 13.3 psi, the thickness of the hardstand can be computed from Equation 9. Assuming a thickness of 4 in., the ESWL determined from Figure 13 (110 × 160 tracked curve) is 10,200 lb, and the calculated thickness is 2.9 in. for an aggregate CBR (C_1) of 100 and a rut depth of 3 in.

128. DI method. For a subgrade CBR of 5 and a DI of 10, the required aggregate thickness of the hardstand is 18 in. as determined from Figure 4.

129. Elastic layered method (Luhr, McCullough, and Pelnver). To design the aggregate-surfaced pavement for track loadings, the subgrade strain should be computed using the procedure outlined in Part VI. The procedure involves the conversion of uniformly distributed track loading to equivalent circular loads, and the subgrade strains induced by the circular loads are computed using the BISAR computer program.

130. Figure 11 shows the layout of the equivalent circular loads placed on the two tracks of the M1 tank. Each circular load has a diameter of 25 in. and a load of 6,667 lb. Using the BISAR program, the maximum subgrade strain under the track center induced by the 18 circular loads was computed.

131. Once the maximum subgrade strain is computed for a particular pavement structure (aggregate thickness h and modulus E), the allowable number of applications of the M1 tank may be determined from Equation 10. The drawback of this procedure is that Equation 10 is formulated based on the highway-type truck loadings; the 60-ton M1 tank may be beyond the load range of the AASHTO test.

132. Cumulative damage theory is used to determine the required aggregate thickness under the track loadings of the M1 tank. The procedure is presented in the following steps:

a. For a series of aggregate thicknesses, the maximum subgrade strains induced by the equivalent circular loads of a 60-ton M1 tank (Figure 11) are computed using the BISAR program. The moduli of the gravel layer and the subgrade are assumed to be 49,000 and 5,500 psi, respectively, as computed from Equation 2 for CBR values of 80 and 5, respectively. The Poisson's ratios of aggregate and subgrade are assumed to be 0.3 and 0.4, respectively. The computed strain values at the subgrade surface are tabulated in Table 34.

Table 34

Computation of Aggregate Thickness for M1 Tank

Aggregate Thickness in.	Maximum Subgrade Strain ε, in./in.	Allowable Repetition N	Damage 365,000/N
18	0.000879	489,058	0.75
15	0.000993	355,474	1.03
12	0.001131	247,278	1.48

b. The allowable repetitions corresponding to the strain values are computed using Equation 10.

c. The damage is computed as the ratio of the total number of repetitions of the M1 tank for the 25-year service life to the allowable repetition. The former is computed to be
40 repetitions/day × 365 days × 25 years
= 365,000 repetitions.

d. A plot of the aggregate thickness to damage (Table 34) indicates that the required thickness is 15 in. for a damage of 1.

133. The expected loss of aggregate should be incorporated through the use of Equation 10. However, there are no analytical or empirical expressions available at the present time to estimate aggregate loss under track loadings.

134. Proposed layered elastic method. The procedure used in the layered elastic method by Luhr, McCullough, and Pelzner (1983) presented in the last section is also used in this design procedure, except that the modulus of the aggregate layer determined from the chart presented in Figure A1 is based on the modulus of the underlying subgrade. Table 35 shows the computed damages for four flexible pavement thicknesses. For a damage factor of 1, the required aggregate thickness is 26.66 in. For an equivalency ratio of 1.22, the required flexible pavement thickness becomes 26 in. With criteria factors of 0.75 and 0.85, the required aggregate thicknesses are 19.5 and 22.1 in., respectively.

135. Discussions. Table 36 is a summary of the required thicknesses determined for different procedures. The thicknesses determined using the CE design and rutting equations are extremely thin as compared with those of the DI method and the elastic layered method. It seems that the ESWL does not properly represent the condition of track loadings and can greatly underestimate the required aggregate thickness.

Table 35

Computation of Aggregate Thickness for M1 Tank

Flexible Pavement Thickness* in.	Maximum Subgrade Strain ε in./in.	Allowable Coverage** N	Damage† 1,092,814/N
13†	0.00161	1,000	1.93
18††	0.001158	30,000	36.43
21‡	0.000982	100,000	10.9
25‡‡	0.000796	700,000	1.56

* The 3-in. bituminous concrete is converted into 3.66-in. base material. The subgrade has a modulus of 7,500 psi.

** Obtained from Figure 15.

† The design coverage is determined as 40 × 365 × 25/0.334 = 1,092,814, where 0.334 is the operation per coverage for a track loading presented in US Army Engineer Waterways Experiment Station (1961).

†† The 13.66-in. base material is divided into two layers. The thicknesses are 6.66 and 7 in., and the elastic moduli are 47,000 and 22,000 psi, respectively.

‡ The 18.66-in. base material is divided into three layers. The thicknesses are 6.66, 6, and 6 in., and the elastic moduli are 71,000, 45,000, and 21,500 psi, respectively.

‡‡ The 21.66-in. base material is divided into three layers. The thicknesses are 6.66, 8, and 7 in., and the elastic moduli are 75,000, 51,500, and 22,500 psi, respectively. The 25.66-in. base material is divided into four layers. The thicknesses are 666, 7, 6, and 6 in., and the elastic moduli are 84,000, 70,000, 44,500, and 21,500 psi, respectively.

Table 36

Design Aggregate Thicknesses, Track Loadings

Design Method	Required Thickness, in.	Required Aggregate, CBR	Expected Rutting, in.
CE design equation	4.0	Determined from Equations 5 or 6	2-3
CE rutting equation	2.9	Determined from Equations 5 or 6	3.0
CE design index method	18.0	--	--
Elastic layered method	15.0	--	--
Proposed elastic layered method	19.5* 22.1	Satisfying CE base course requirements	2-3

* 19.5 in. for a criterion factor of 0.75, and 22.1 in. for a criterion factor of 0.85.

Design Example No. 3, Aircraft Loads

136. In the design of aggregate-surfaced pavement for aircraft loads, the gear load can be converted either to the ESWL or the equivalent number of 18-kip single-axle load. In the former case, the CE design and rutting equations (Equations 6 and 9) presented in Part VI are applicable, and the remaining procedures are applicable for the latter case. The CE design and rutting equations and the layered elastic method are believed to be most suitable for aircraft loadings because the other design procedures are primarily developed for truck-type highway loadings. Due to the constraint of the equations, the CE design and rutting equations are not applicable to the design of mixed aircraft. Using the damage ratio concept, the layered elastic method can be used to handle mixed aircraft traffics.

137. The same subgrade soil (CBR = 5) used in design example No. 1 is used in this case. The design aircraft is 5,225 operations of the C-130 aircraft annually for a design life of 25 years. The total design coverage is thus 62,500. The characteristics of the C-130 aircraft are presented in Part VII.

CE design equation

138. Figure 14 shows the relationship between multiple-wheel and ESWL's for a number of aircraft. In using Equation 6 to compute the required aggregate thickness, a trial and error procedure has to be used as the ESWL, and the equation is a function of the aggregate thickness. Table 37 presents the computed values from Equation 6 for three assumed aggregate thicknesses. The ESWL P is determined from Figure 14 based on the assumed aggregate thickness h. The required aggregate thickness for the design condition is 18.3 in. For aggregate CBR requirement, Figure 2 shows that the required aggregate CBR is 27 for a 2- to 3-in. expected rut depth.

CE rutting equation

139. In using Equation 9 to compute the required aggregate layer thickness t, a trial and error procedure also has to be used as the equivalent single-wheel load P_k is a function of the aggregate layer thickness. Table 38 presents the computed values for Equation 8 for a number of aggregate CBR's. The computations are made assuming the aggregate layer has a thickness of 18 in., and the equivalent single-wheel load P_k is determined accordingly from Figure 14. Table 38 shows that the required aggregate thickness is very

Table 37

Aggregate Thicknesses Determined from Equation 6 for C-130 Aircraft*

Assumed Aggregate Layer Thickness t_p to Compute P, in.	Equivalent Single-Wheel Load, P, lb	Required Thickness t Computed from Equation 6, in.
16	40,455	18.2
18	40,800	18.3
19	41,150	18.4

* Coverage = 62,500, subgrade CBR = 5.

Table 38

Aggregate Thicknesses Determined from Equation 9 for C-130 Aircraft*

Assumed Aggregate Layer Thickness t_p to Compute P_k, in.	Equivalent Single-Wheel Load P_k, lb	Anticipated Rutting, in.	Aggregate CBR C_2	Required Thickness t Computed from Equation 9, in.
18	40,800	2	20	76.8
18	40,800	2	30	19.6
18	40,800	2	40	9.7
18	40,800	3	20	251.7
18	40,800	3	30	18.1
18	40,800	3	40	7.3

* Coverage = 62,500, subgrade CBR = 5.

much dependent on the aggregate CBR. For an 18-in. aggregate thickness, the required aggregate CBR is 32 for an anticipated 2-in. rut, and the required aggregate CBR is 30 for a 3-in. rut. It is noted that the thicknesses and aggregate CBR's computed for Equations 6 and 9 are very close.

Elastic layered method, Luhr, McCullough, and Pelzner procedure

140. A particular design aircraft. For a series of aggregate thicknesses, the subgrade strains under the C-130 aircraft loading are computed using the BISAR computer program. The allowable load repetitions are computed from Equation 10. The computed values are tabulated in Table 39. The total design coverage is 62,500 which is equivalent to 130,625 passes based on a pass per coverage ratio of 2.09. Table 39 shows that 25.5 in. of 30-CBR aggregate is needed for the design 62,500 coverages. The required thickness can be reduced to 19 in. if the aggregate CBR is increased to 80.

141. Mixed aircraft. Cumulative damage theory can be used to determine the aggregate thickness when more than one design aircraft is involved. Assuming that 20,000 coverages of C-130 and 400,000 coverages of C-123 are used in the design, the procedure to determine the optimal aggregate thickness is illustrated in Table 40. It is seen that for a 30-CBR aggregate material, a 28-in. aggregate layer is required corresponding to a damage factor of 1. When an 80-CBR aggregate material is used, the thickness of the aggregate layer can be reduced to 19 in.

Proposed layered elastic method

142. For two aggregate thicknesses, the subgrade strains under the C-130 aircraft loading are computed using the BISAR computer program. The modulus values of the aggregate layers are determined from Figure A1. The allowable coverages are determined from Figure 15, based on the computed subgrade strains. Table 41 presents the computed strains and damages. For a damage factor of 1, the required aggregate thickness is 33.66 in. For an equivalency ratio of 1.22, the required flexible pavement thickness becomes 33 in. With criteria factors of 0.75 and 0.85, the required aggregate thicknesses are 24.75 and 28 in., respectively.

Table 39

Aggregate Thicknesses Determined from Equation 10 for C-130 Aircraft*

Aggregate Thickness h_1 in.	Aggregate Layer Modulus E_1, psi	Aggregate Layer CBR**	Subgrade Strain in./in.	Allowable Load Repetition Computed from Equation 9 Passes	Allowable Load Repetition Computed from Equation 9 Coverages
18	45,000	30	0.00263	10,232	4,896
18	120,000	80	0.00153	95,550	45,717
22	45,000	30	0.00198	36,508	17,468
22	120,000	80	0.00114	241,674	115,633
25	45,000	30	0.00163	76,542	36,622
25	120,000	80	0.00095	400,011	191,393
30	45,000	30	0.00124	188,326	90,108
30	70,000	47	0.00097	378,578	181,137
30	120,000	80	0.00072	793,623	379,734

* Subgrade Modulus = 5,000 psi, CBR = 5.
** From Equation 1, E = 1,500 CBR.

Table 40

Aggregate Thicknesses Determined from Equation 10 for C-130 and C-123 Aircraft*

Aggregate Thickness in. (1)	Aggregate Modulus E_2 psi (2)	Subgrade Strain in./in. C-130 (3)	Subgrade Strain in./in. C-123 (4)	Allowable Load Repetition, Coverage C-130** (5)	Allowable Load Repetition, Coverage C-123† (6)	Damages (C-130) 20,000 (5) (7)	Damages (C-123) 40,000 (6) (8)	Total Damages (7+8)
18	45,000††	0.00263	0.00223	4,896	4,225	4.08	9.46	13.54
20	45,000	0.00227	0.00188	9,771	8,581	2.05	4.66	6.71
22	45,000	0.00197	0.00160	17,829	15,633	1.12	2.56	3.68
24	45,000	0.00173	0.00138	29,485	25,760	0.68	1.55	2.23
28	45,000	0.00137	0.00105	65,991	58,345	0.30	0.68	0.98
18	120,000‡	0.00153	0.00122	45,717	37,822	0.20	1.6	1.26
20	120,000	0.00131	0.00102	76,069	63,194	0.26	0.64	0.90
22	120,000	0.00114	0.000865	115,633	97,386	0.17	0.42	0.59

* Subgrade Modulus = 5,000 psi, CBR = 5.
** The passes per coverage ratio for C-130 is 2.09.
† The passes per coverage ratio for C-123 is 5.23.
†† Equivalent to CBR = 30 computed from Equation 1.
‡ Equivalent to CBR = 80 computed from Equation 1.

Table 41

Computation of Aggregate Thickness for C-130 Aircraft

Flexible Pavement Thickness* in.	Maximum Subgrade Strains ε in./in.	Allowable Coverage** N	Damage 62,500/N
30†	0.00120	20,000	3.13
33††	0.00103	80,000	0.78

* The 3-in. bituminous concrete is converted into 3.66-in. base material. The subgrade has a modulus of 7,500 psi.

** The base material has a thickness of 9.66 in. The 28-in. subbase is divided into three 8-in. sublayers. The modulus values starting from the top layer are 63,000, 31,500, 24,000, and 15,000 psi.

† The base material has a thickness of 9.66 in. The 21-in. subbase is divided into three 7-in. sublayers. The modulus values starting from the top layer are 61,000, 30,000, 23,000, and 15,000 psi.

†† Determined from Figure 15.

PART X: CONCLUSIONS AND RECOMMENDATIONS

143. Many design methodologies for aggregate-surfaced pavements are currently available. However, the computed results of design examples indicate that the required aggregate thicknesses determined from the design procedures vary greatly. The divergence of presently available methods is mostly for tracked vehicles. In some design procedures, the required aggregate thickness is extremely sensitive to the stiffness (or CBR) of the aggregate layer. However, the aggregate CBR is difficult to accurately measure. Although the design equation for aggregate-surfaced pavements developed by the CE is widely accepted and used, the fact that the equation was established based on test results involving low-CBR cover materials is often not known or neglected. The layered elastic method developed by Luhr, McCullough, and Pelzner (1983) is believed to be inadequate as the equation was developed for flexible pavements. A reliability analysis was made on design procedures, and it was found that the reliabilities of some procedures were very low.

144. The failure criteria developed in this study are proposed to be used for the design of aggregate-surfaced pavements subject to vehicular, tank, and aircraft loadings. The criteria need to be verified with field test results on pavements with high-strength aggregate surface layers with high CBR's.

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APPENDIX A

PROCEDURES FOR DETERMINING THE MODULUS OF ELASTICITY OF UNBOUND GRANULAR MATERIALS

Procedure

1. The procedure is based on relationships developed for the resilient modulus of unbound granular layers as a function of the thickness of the layer and type of material. The modulus relationships are shown in Figure A1. Modulus values for layer n (the upper layer) are indicated on the ordinate, and those for layer $n + 1$ (the lower layer) are indicated on the abscissa. Essentially linear relationships are indicated for various thicknesses of base and subbase course materials. For subbase courses, relationships are shown for thicknesses of 4, 5, 6, 7, and 8 in. For subbase courses having a design thickness of 8 in. or less, the applicable curve or appropriate interpolation can be used directly. For a design subbase course thickness in excess of 8 in., the layer should be divided into sublayers of approximately equal thickness and the modulus of each sublayer determined individually. For base courses, relationships are shown for thicknesses of 4, 6, and 10 in. These relationships can be used directly or by interpolation for design base course thicknesses up to 10 in. For design thicknesses in excess of 10 in., the layer should also be divided into sublayers of approximately equal thickness and the modulus of each sublayer determined individually.

2. To determine modulus values from this procedure, Figure A1 is entered along the abscissa using modulus values of the subgrade or underlying layer (modulus of layer $n + 1$). At the intersection of the curve applicable to this value with the appropriate thickness relationship, the value of the modulus of the overlying layer is read from the ordinate (modulus of layer n). This procedure is repeated using the modulus value just determined as the modulus of layer $n + 1$ to determine the modulus value of the next overlying layer.

Examples

3. Assume a pavement having a base course thickness of 4 in. and a subbase course thickness of 8 in. over a subgrade having a modulus of 10,000 psi. Initially, the subgrade is assumed to be layer $n + 1$ and the subbase course to be layer n. Entering Figure A1 with a modulus of layer $n + 1$ of 10,000 psi and using the 8-in. subbase course curve, the modulus of the subbase (layer n) is found to be 18,500 psi. In order to determine the modulus value of the base course, the subbase course is now assumed to be layer

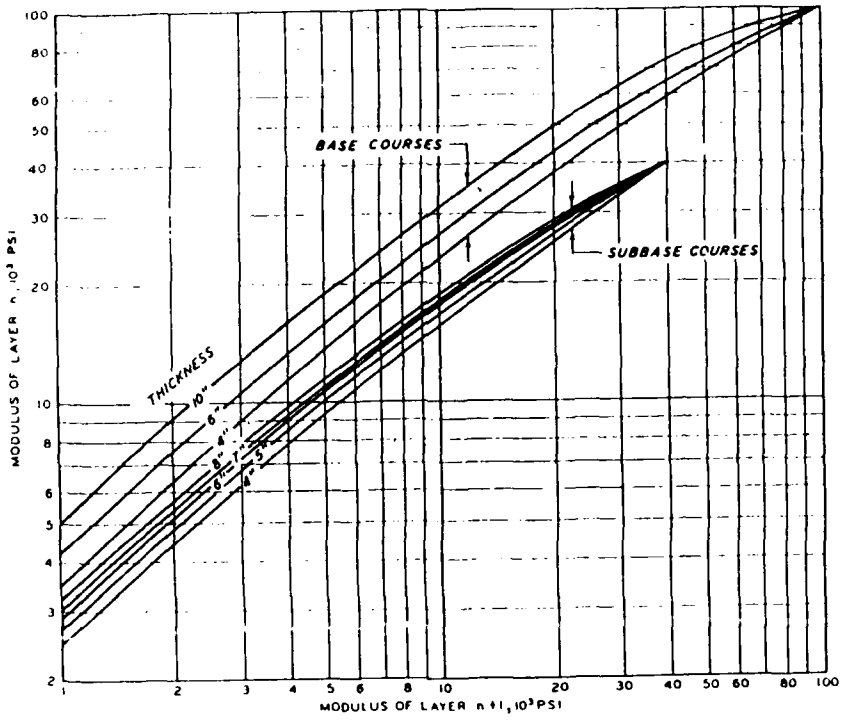

Figure A1. Elastic modulus values for unbound granular materials

n + 1 and the base course to be layer n . Entering Figure A1 with a modulus value of layer n + 1 of 18,500 psi and using the 4-in. base course relationship, the modulus of the base course is found to be 36,000 psi. Modulus values determined for each layer are indicated in Figure A2.

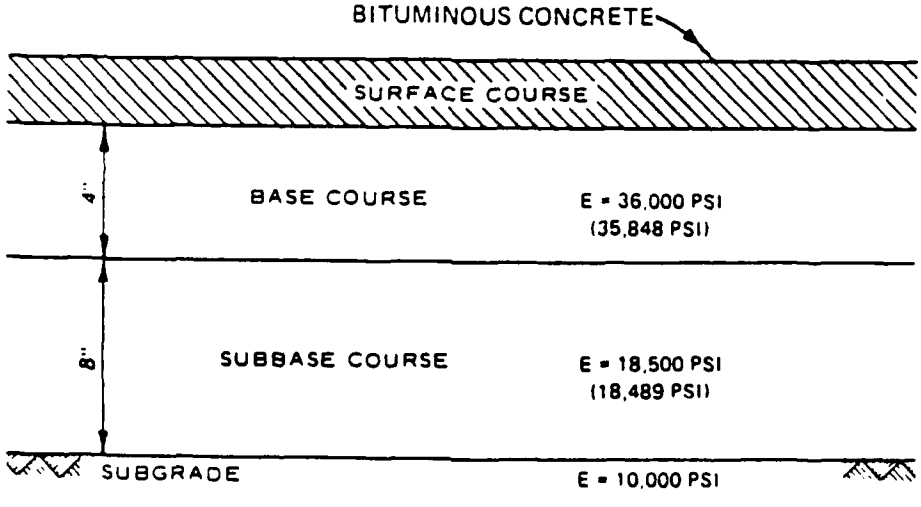

Figure A2. A typical flexible pavement layered system, 8 by 4 ft

4. If the design thickness of the subbase course had been 12 in. in the first example, it would have been necessary to divide this layer into two 6-in.-thick sublayers. Then, using the procedure described above for the second example, the modulus values determined for the lower and upper sublayers of the subbase course and for the base course are 17,500, 25,500, and 44,000 psi, respectively. These values are shown in Figure A3.

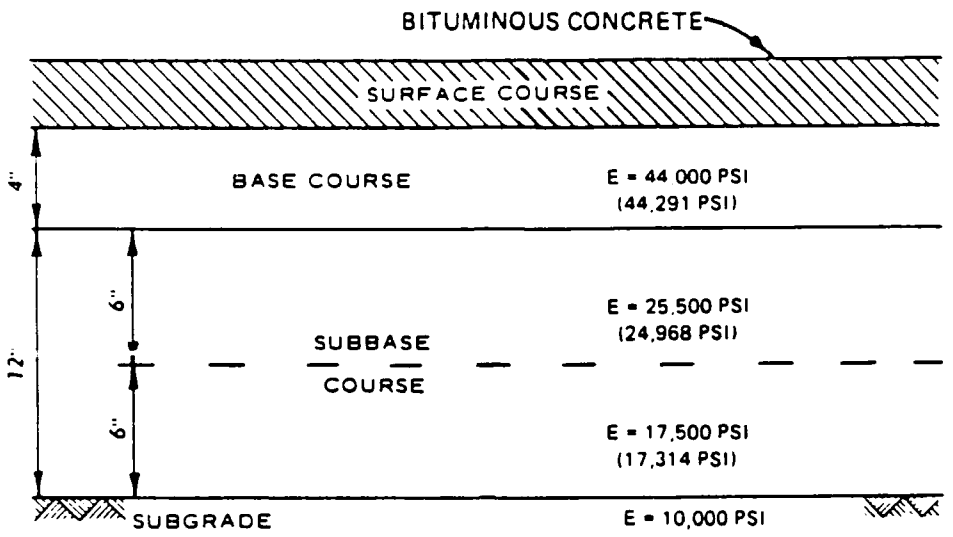

Figure A3. A typical flexible pavement layered system 12 by 4 ft

5. The relationships indicated in Figure A1 can be expressed as

$$E_n = E_{n+1} (1 + 10.52 \log t - 2.10 \log E_{n+1} \log t) \quad \text{(A1)}$$

where

E_n = resilient modulus of layer n , psi

n = a layer in the pavement system

E_{n+1} = the resilient modulus of the layer beneath layer n , psi

t = the thickness of layer n , psi

for base course materials and as

$$E_n = E_{n+1} (1 + 7.18 \log t - 1.56 \log E_{n+1} \log t) \quad \text{(A2)}$$

for subbase course materials. Use of these equations for direct computation of modulus values for the examples given above yields the values indicated in parentheses in Figures A2 and A3. It can be seen that comparable values are obtained with either graphical or computational determination of the modulus value for either material.

APPENDIX B

GRAVEL-SURFACED FACILITY DATA FOR CE RUTTING EQUATION 9

RUT in.	ESWL lb	TP psi	THI in.	CBR 1	CBR 2	REP
			Gravel-Surfaced Facility Data 1			
0.50	4,680	100	4.5	81.0	19.5	920
1.00	4,680	100	4.5	81.0	19.5	3,240
1.20	4,680	100	4.5	81.0	19.5	6,760
0.60	4,800	70	4.5	69.0	18.5	3,400
0.70	4,800	70	4.5	69.0	18.5	6,480
0.20	5,440	40	4.5	64.0	18.0	1,080
0.50	5,440	40	4.5	64.0	18.0	3,800
0.60	5,440	40	4.5	64.0	18.0	6,800
0.20	5,920	20	4.5	50.0	17.5	1,160
0.45	5,920	20	4.5	50.0	17.5	2,660
0.50	5,920	20	4.5	50.0	17.5	6,120
0.50	5,440	40	10.0	29.0	21.0	1,880
0.60	5,440	40	10.0	29.0	21.0	2,940
0.40	5,920	20	4.5	55.0	9.0	800
0.60	5,920	20	4.5	55.0	9.0	4,000
0.80	5,920	20	4.5	55.0	9.0	6,400
0.20	8,000	20	4.5	44.0	7.0	120
0.80	8,000	20	4.5	44.0	7.0	800
1.60	8,000	20	4.5	44.0	7.0	4,000
2.50	25,000	100	12.0	5.3	4.7	17
3.20	25,000	100	12.0	5.3	4.7	30
2.00	25,000	100	12.0	8.0	5.3	17
2.20	25,000	100	12.0	8.0	5.3	30
2.40	25,000	100	12.0	8.0	5.3	43
2.00	25,000	100	12.0	7.0	4.9	17
2.40	25,000	100	12.0	7.0	4.9	30
2.70	25,000	100	12.0	7.0	4.9	43
2.75	15,000	150	6.0	9.0	3.2	91
3.84	15,000	150	6.0	9.0	3.2	108
4.91	15,000	150	6.0	9.0	3.2	133
1.16	15,000	150	12.0	7.5	3.5	41
1.52	15,000	150	12.0	7.5	3.5	66
2.02	15,000	150	12.0	7.5	3.5	108
2.47	15,000	150	12.0	7.5	3.5	133
3.00	15,000	150	12.0	7.5	3.5	158
3.60	15,000	150	12.0	7.5	3.5	199
1.04	15,000	150	18.0	9.0	3.7	41
1.12	15,000	150	18.0	9.0	3.7	66
1.47	15,000	150	18.0	9.0	3.7	108
1.75	15,000	150	18.0	9.0	3.7	133
1.88	15,000	150	18.0	9.0	3.7	199
2.92	15,000	150	18.0	9.0	3.7	291
3.10	15,000	150	18.0	9.0	3.7	332
3.48	15,000	150	18.0	9.0	3.7	365

(Continued)

RUT in.	ESWL lb	TP psi	THI in.	CBR 1	CBR 2	REP
1.55	15,000	150	24.0	7.6	3.2	41
1.13	15,000	150	24.0	7.6	3.2	66
1.52	15,000	150	24.0	7.6	3.2	108
1.53	15,000	150	24.0	7.6	3.2	133
1.82	15,000	150	24.0	7.6	3.2	199
2.57	15,000	150	24.0	7.6	3.2	291
2.53	15,000	150	24.0	7.6	3.2	332
2.97	15,000	150	24.0	7.6	3.2	365
1.85	25,000	115	12.0	7.5	3.0	29
3.70	25,000	115	12.0	7.5	3.0	109
1.79	25,000	115	18.0	8.2	3.3	29
1.96	25,000	115	18.0	8.2	3.3	57
2.86	25,000	115	18.0	8.2	3.3	109
3.86	25,000	115	18.0	8.2	3.3	144
1.39	25,000	115	24.0	9.0	3.1	29
1.21	25,000	115	24.0	9.0	3.1	57
1.50	25,000	115	24.0	9.0	3.1	109
2.31	25,000	115	24.0	9.0	3.1	144
3.37	25,000	115	24.0	9.0	3.1	333
1.00	40,000	80	12.0	11.0	3.7	11
2.29	40,000	80	12.0	11.0	3.7	56
3.61	40,000	80	12.0	11.0	3.7	90
1.72	40,000	80	18.0	9.3	3.4	187
2.22	40,000	80	18.0	9.3	3.4	262
2.84	40,000	80	18.0	9.3	3.4	337
3.75	40,000	80	18.0	9.3	3.4	449
1.66	40,000	80	6.0	9.0	3.7	8
3.47	40,000	80	6.0	9.0	3.7	17
1.16	40,000	80	12.0	11.0	2.9	17
1.85	40,000	80	12.0	11.0	2.9	55
2.44	40,000	80	12.0	11.0	2.9	76
3.54	40,000	80	12.0	11.0	2.9	98
0.82	40,000	80	18.0	9.7	3.6	17
0.94	40,000	80	18.0	9.7	3.6	55
1.57	40,000	80	18.0	9.7	3.6	76
1.81	40,000	80	18.0	9.7	3.6	98
2.10	40,000	80	18.0	9.7	3.6	157
2.82	40,000	80	18.0	9.7	3.6	212
2.78	40,000	80	18.0	9.7	3.6	233
2.91	40,000	80	18.0	9.7	3.6	254
3.25	40,000	80	18.0	9.7	3.6	297
1.22	40,000	80	24.0	9.7	4.3	212
1.19	40,000	80	24.0	9.7	4.3	233
1.16	40,000	80	24.0	9.7	4.3	254
1.32	40,000	80	24.0	9.7	4.3	297
1.62	40,000	80	24.0	9.7	4.3	424

(Continued)

RUT in.	ESWL lb	TP psi	THI in.	CBR 1	CBR 2	REP
1.72	40,000	80	24.0	9.7	4.3	636
2.25	40,000	80	24.0	9.7	4.3	848
2.57	40,000	80	24.0	9.7	4.3	1,060
2.66	15,000	165	6.0	11.0	4.4	8
3.36	15,000	165	6.0	11.0	4.4	16
1.33	15,000	165	12.0	10.0	3.8	8
1.48	15,000	165	12.0	10.0	3.8	16
0.59	15,000	165	18.0	13.0	4.5	8
0.85	15,000	165	18.0	13.0	4.5	16
1.16	15,000	165	18.0	13.0	4.5	56
1.56	15,000	165	18.0	13.0	4.5	80
2.41	15,000	165	18.0	13.0	4.5	127
2.97	15,000	165	18.0	13.0	4.5	159
3.25	15,000	165	18.0	13.0	4.5	175
0.65	15,000	165	24.0	11.0	4.1	8
0.97	15,000	165	24.0	11.0	4.1	16
1.35	15,000	165	24.0	11.0	4.1	56
1.97	15,000	165	24.0	11.0	4.1	80
2.56	15,000	165	24.0	11.0	4.1	127
2.72	15,000	165	6.0	11.0	4.1	159
3.07	15,000	165	7.0	11.0	4.1	175
2.63	40,000	120	6.0	13.0	3.5	13
3.90	40,000	120	6.0	13.0	3.5	17
1.65	40,000	120	12.0	12.0	4.0	17
3.78	40,000	120	12.0	12.0	4.0	76
1.31	40,000	120	18.0	11.0	4.7	17
2.28	40,000	120	18.0	11.0	4.7	76
2.47	40,000	120	18.0	11.0	4.7	127
2.81	40,000	120	18.0	11.0	4.7	170
3.20	40,000	120	18.0	11.0	4.7	212
0.88	40,000	120	24.0	11.0	5.1	17
1.53	40,000	120	24.0	11.0	5.1	76
1.65	40,000	120	24.0	11.0	5.1	127
2.04	40,000	120	24.0	11.0	5.1	170
2.57	40,000	120	24.0	11.0	5.1	212
2.66	40,000	120	24.0	11.0	5.1	254
2.75	40,000	120	24.0	11.0	5.1	297
3.25	40,000	120	24.0	11.0	5.1	339
0.78	26,600	120	12.0	10.0	4.3	5
1.88	26,600	120	12.0	10.0	4.3	49
1.97	26,600	120	12.0	10.0	4.3	82
2.50	26,600	120	12.0	10.0	4.3	114
3.38	26,600	120	12.0	10.0	4.3	147
1.31	26,600	120	18.0	9.9	4.1	49
1.57	26,600	120	18.0	9.9	4.1	114
1.97	26,600	120	18.0	9.9	4.1	147

(Continued)

RUT in.	ESWL lb	TP psi	THI in.	CBR 1	CBR 2	REP
2.28	26,600	120	18.0	9.9	4.1	196
2.29	26,600	120	18.0	9.9	4.1	245
2.47	26,600	120	18.0	9.9	4.1	293
2.78	26,600	120	18.0	9.9	4.1	342
3.16	26,600	120	18.0	9.9	4.1	391
1.57	26,600	120	24.0	11.0	4.4	49
1.66	26,600	120	24.0	11.0	4.4	114
1.94	26,600	120	24.0	11.0	4.4	147
2.07	26,600	120	24.0	11.0	4.4	196
1.94	26,600	120	24.0	11.0	4.4	245
2.00	26,600	120	24.0	11.0	4.4	293
2.16	26,600	120	24.0	11.0	4.4	342
2.72	26,600	120	24.0	11.0	4.4	391
2.50	26,600	120	24.0	11.0	4.4	440
3.52	26,600	120	24.0	11.0	4.4	473
2.38	25,000	125	15.0	18.0	2.7	431
2.63	25,000	125	15.0	18.0	2.7	545
2.94	25,000	125	15.0	18.0	2.7	689
3.56	25,000	125	15.0	18.0	2.7	861
4.06	25,000	125	15.0	18.0	2.7	941
2.19	25,000	125	18.0	17.0	2.9	712
2.69	25,000	125	18.0	17.0	2.9	861
2.81	25,000	125	18.0	17.0	2.9	941
2.65	25,000	125	18.0	17.0	2.9	1,091
2.85	25,000	125	18.0	17.0	2.9	1,538
3.00	25,000	125	18.0	17.0	2.9	1,722
3.25	25,000	125	18.0	17.0	2.9	1,866
4.00	25,000	125	18.0	17.0	2.9	2,003
1.69	25,000	125	21.0	17.0	2.6	1,866
1.63	25,000	125	21.0	17.0	2.6	2,003
1.56	25,000	125	21.0	17.0	2.6	2,153
1.66	25,000	125	21.0	17.0	2.6	2,296
1.69	25,000	125	21.0	17.0	2.6	2,440
1.75	25,000	125	21.0	17.0	2.6	2,583
1.81	25,000	125	21.0	17.0	2.6	2,727
1.88	25,000	125	21.0	17.0	2.6	2,870
2.06	40,000	125	15.0	15.0	2.4	42
2.48	40,000	125	15.0	15.0	2.4	85
2.83	40,000	125	15.0	15.0	2.4	127
3.93	40,000	125	15.0	15.0	2.4	170
2.12	40,000	125	18.0	15.0	2.9	42
2.43	40,000	125	18.0	15.0	2.9	85
3.00	40,000	125	18.0	15.0	2.9	127
3.31	40,000	125	18.0	15.0	2.9	170
3.62	40,000	125	18.0	15.0	2.9	233
1.87	40,000	125	21.0	14.0	2.6	233

(Continued)

RUT in.	ESWL lb	TP psi	THI in.	CBR 1	CBR 2	REP
2.13	40,000	125	21.0	14.0	2.6	276
2.13	40,000	125	21.0	14.0	2.6	318
2.38	40,000	125	21.0	14.0	2.6	424
2.44	40,000	125	21.0	14.0	2.6	530
2.69	40,000	125	21.0	14.0	2.6	636
2.81	40,000	125	21.0	14.0	2.6	742
2.81	40,000	125	21.0	14.0	2.6	848
2.87	40,000	125	21.0	14.0	2.6	954
2.87	40,000	125	21.0	14.0	2.6	1,060
2.94	40,000	125	21.0	14.0	2.6	1,166
3.00	40,000	125	21.0	14.0	2.6	1,272
3.25	40,000	125	21.0	14.0	2.6	1,484
3.13	40,000	125	9.0	12.0	2.4	11
5.62	40,000	125	9.0	12.0	2.4	19
2.13	40,000	125	12.0	13.0	2.3	11
2.62	40,000	125	12.0	13.0	2.3	19
3.25	40,000	125	12.0	13.0	2.3	37
1.75	40,000	125	15.0	16.0	2.2	37
2.75	40,000	125	15.0	16.0	2.2	75
3.06	40,000	125	15.0	16.0	2.2	105
3.31	40,000	125	15.0	16.0	2.2	116
2.06	40,000	125	18.0	14.0	2.9	116
2.13	40,000	125	18.0	14.0	2.9	150
2.25	40,000	125	18.0	14.0	2.9	187
2.25	40,000	125	18.0	14.0	2.9	224
2.50	40,000	125	18.0	14.0	2.9	262
2.62	40,000	125	18.0	14.0	2.9	299
2.75	40,000	125	18.0	14.0	2.9	337
2.81	40,000	125	18.0	14.0	2.9	374
2.87	40,000	125	18.0	14.0	2.9	411
2.94	40,000	125	18.0	14.0	2.9	486
3.08	40,000	125	18.0	14.0	2.9	524
3.20	40,000	125	18.0	14.0	2.9	561
3.08	40,000	125	18.0	14.0	2.9	598
3.31	40,000	125	18.0	14.0	2.9	636
3.50	40,000	125	18.0	14.0	2.9	673
1.75	40,000	125	21.0	17.0	2.4	673
1.78	40,000	125	21.0	17.0	2.4	748
1.88	40,000	125	21.0	17.0	2.4	860
1.98	40,000	125	21.0	17.0	2.4	935
2.08	40,000	125	21.0	17.0	2.4	1,047
2.09	40,000	125	21.0	17.0	2.4	1,103
2.13	40,000	125	21.0	17.0	2.4	1,167
2.22	40,000	125	21.0	17.0	2.4	1,290
2.31	40,000	125	21.0	17.0	2.4	1,403
1.3	25,000	123	12.0	10.0	4.3	57
2.2	25,000	123	12.0	10.0	4.3	115

(Continued)

RUT in.	ESWL lb	TP psi	THI in.	CBR 1	CBR 2	REP
2.6	25,000	123	12.0	10.0	4.3	172
3.3	25,000	123	12.0	10.0	4.3	230
3.8	25,000	123	12.0	10.0	4.3	287
1.5	25,000	123	12.0	10.0	3.9	57
2.1	25,000	123	12.0	10.0	3.9	115
2.4	25,000	123	12.0	10.0	3.9	172
3.2	25,000	123	12.0	10.0	3.9	230
4.5	25,000	123	12.0	10.0	3.9	287
2.3	25,000	123	12.0	10.0	3.8	115
2.7	25,000	123	12.0	10.0	3.8	172
3.4	25,000	123	12.0	10.0	3.8	230
4.1	25,000	123	12.0	10.0	3.8	287
0.11	10,000	100	8.0	100.0	6.2	35
0.19	10,000	100	8.0	100.0	6.2	141
0.21	10,000	100	8.0	100.0	6.2	353
0.23	10,000	100	8.0	100.0	6.2	706
0.29	10,000	100	8.0	100.0	6.2	3,530
0.70	10,000	100	8.0	100.0	6.2	6,001
0.12	10,000	100	11.0	132.0	6.2	35
0.15	10,000	100	11.0	132.0	6.2	141
0.20	10,000	100	11.0	132.0	6.2	353
0.20	10,000	100	11.0	132.0	6.2	706
0.19	10,000	100	11.0	132.0	6.2	1,765
0.20	10,000	100	11.0	132.0	6.2	3,530
0.30	10,000	100	11.0	132.0	6.2	6,001

APPENDIX C: TRAFFIC EQUIVALENT DAMAGE FACTORS

Axle Loads and Axle Types (American Association of State Highway and Transporation Officials (AASHTO) and Corps of Engineers)

1. Equivalent damage factors (F-values) for axle loads and axle types other than 18-kip single axle can be computed directly from the fundamental definition of the F-value. In the AASHTO road test F-values were found to be dependent on the pavement type, structural number of the pavement, and the failure present serviceability index (PSI) (P_T). Table C1 shows F-values for a flexible pavement with a structural number (SN) of 2.0 and a failure PSI (P_T) of 2.0. F-values for other conditions may be found in McCullough and Luhr (1979) and Roberts et al. (1977).*

2. Table C1 shows that for a 30-kip single-axle load the F-value is 10.03. Thus, this axle load is about 10 times as damaging as the 18-kip single-axle load. Therefore, it requires about 10 repetitions of an 18-kip single-axle load to cause as much damage as one repetition of the 30-kip single-axle load.

3. The US Army Corps of Engineers (CE) also developed equivalent damage factors for tandem- and single-axle loads. Figure C1 (Ahlvin and Hammitt 1975) shows such a relation by which the factor is the number of repetitions of an axle load to be multiplied to yield equivalent 18-kip single-axle loads ESAL. It should be noted that the damage factors developed by the CE (Figure C1) are greater than those of AASHTO road tests (Table C1). This is particularly true in the case of tandem-axle loads. For instance, Figure C1 shows that the damage factor for a 30-kip single-axle load is 21 as compared with 10.03 shown in Table C1, and it is 40 for a 48-kip tandem-axle load as compared with 4.98 determined in Table C1. It should also be pointed out that in the CE's procedures (Figure C1), one pass of the tandem-axle load is considered to be two passes of the single-axle load. It is possible that the use of deflection in estimating the equivalent single-wheel load in the CE procedure contributes to the overestimate of damage factors for multiple-axle loads.

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* References cited in this appendix are included in the References at the end of the main text.

Table C1

Equivalent Damage Factors, AASHTO

Single-Axle Load		Tandem-Axle Load	
pounds (in thousands)	F-Value	pounds (in thousands)	F-Value
2	0.0002	10	0.01
4	0.003	12	0.02
6	0.01	14	0.03
8	0.04	16	0.05
10	0.08	18	0.08
12	0.18	20	0.12
14	0.34	22	0.17
16	0.60	24	0.24
18	1.00	26	0.34
20	1.59	28	0.46
22	2.44	30	0.62
24	3.62	32	0.82
26	5.21	34	1.07
28	7.31	36	1.38
30	10.03	38	1.75
32	13.51	40	2.19
34	17.87	42	2.73
36	23.30	44	3.36
38	29.95	46	4.11
40	38.02	48	4.98

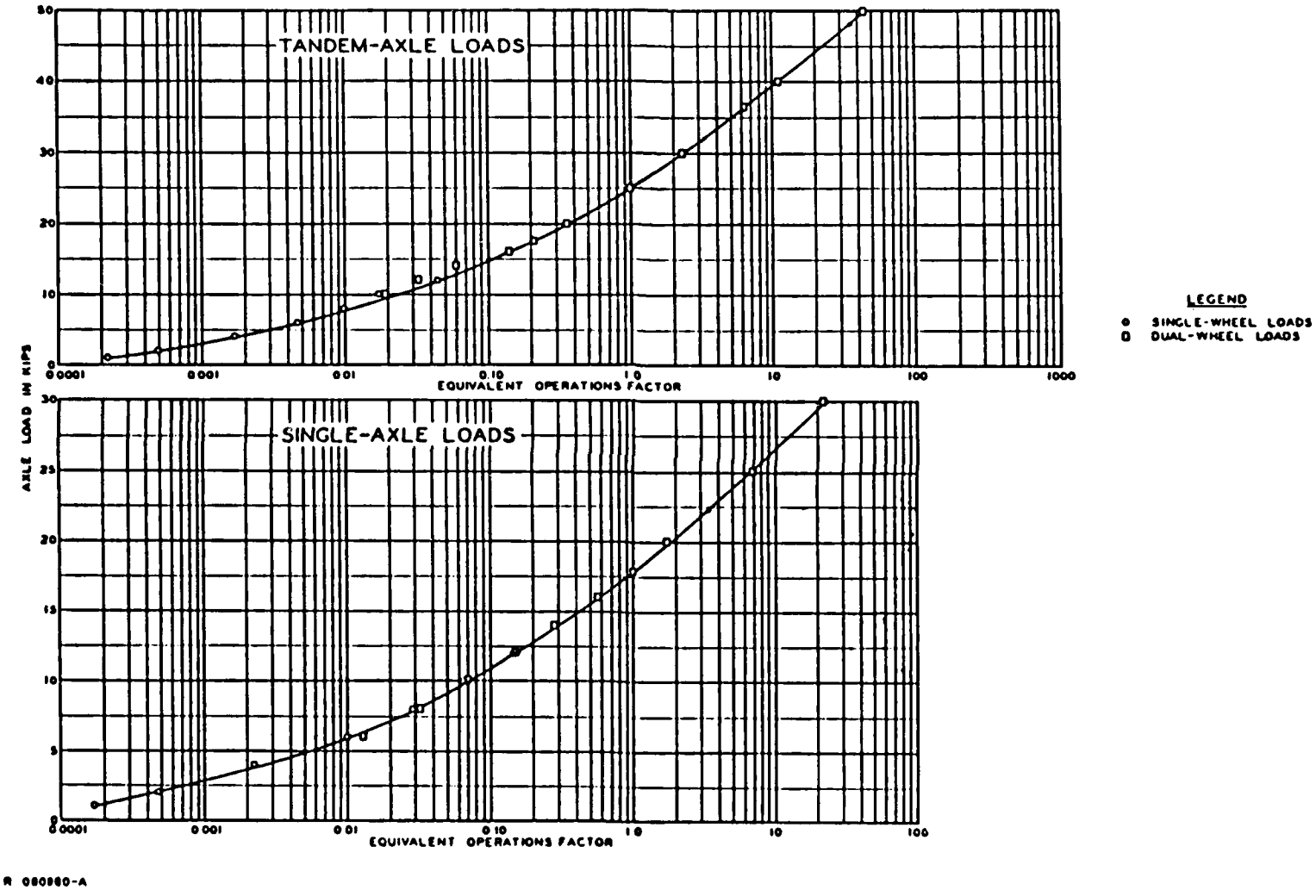

Figure C-1. Equivalent operation factors for tandem-axle loads and single-axle loads, CE

Vehicle Classes

4. The Arizona Department of Transporation collected data for different classes of trucks on highways, and Table C2 was adapted from the data showing the average vehicle equivalency factors for five vehicle classes shown in Figure C2. These factors are multiplied by the total number of trucks in each class over the design period or life. The products are summed over the five vehicle classes to obtain the total number of 18-kip ESAL's. This is divided by two to obtain total 18-kip ESAL's in one direction. This number may then be used for design of the pavement structure.

Table C2

Average Vehicle Equivalency Factors for Five Vehicle Classes

Vehicle Class	Vehicle Equivalency Factor 18-kip ESAL/Vehicle Type
LT	0.004
MT	0.25
TS	1.0
TT	1.3
TST	1.4

Equivalent Damage Ratios Between the 18-kip Axle Load and Other Types of Loadings

5. Equation 10 is used to predict the allowable 18-kip axle load based on subgrade strain. The equation is not directly applicable for other types of loadings, such as aircraft loadings or tracked vehicle loadings. A procedure was developed to find the equivalent damage ratio between the 18-kip axle loads and types of loading other than axle load for which the aggregate-surfaced pavement is designed. This procedure is presented in steps as follows:

a. Select an aggregate layer thickness t and a modulus value of the gravel E and compute the subgrade strain ε_1 for the 18-kip axle load using the BISAR computer program.[1]

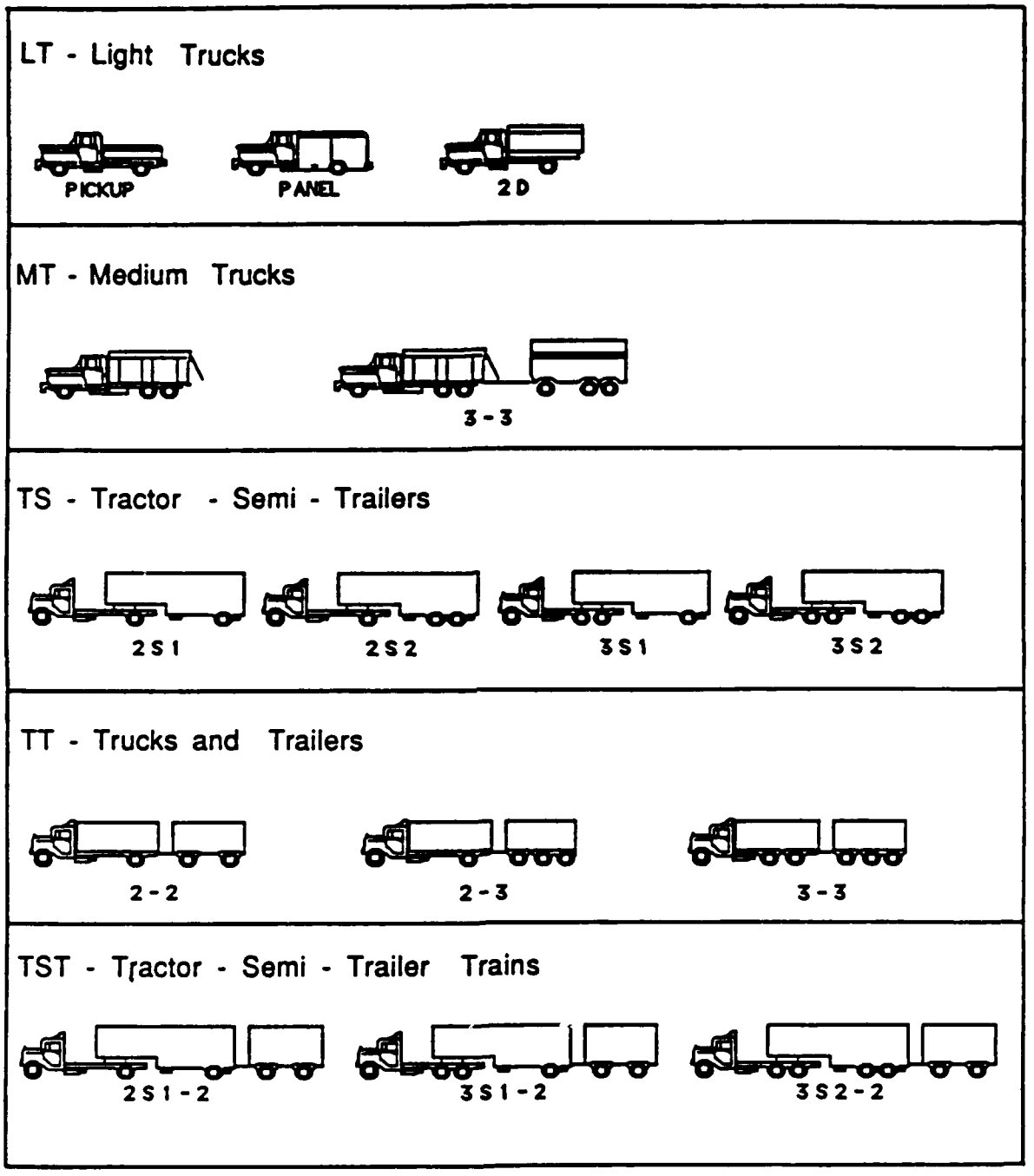

Figure C2. Traffic classification categories

b. Determine the allowable 18-kip axle load $N_{18}(1)$ from Equation 10.

c. For the same thickness t and modulus E , compute the subgrade strain ε_2 for the specific type of loading (such as the 60-ton M1 tank) and determine the allowable 18-kip axle load $N_{18}(2)$ from Equation 10.

d. The equivalent damage ratio between the specific type of loading (the M1 tank) and the 18-kip axle load is $(N_{18}(2)/N_{18}(1))$ number of operations of 18-kip axle load for this particular pavement structure.

6. The apparent drawback of this procedure is that when the magnitude of the designed vehicle or aircraft is much greater than the 18-kip axle load, such as the 60-ton M1 tank, Boeing 747, or B-52 bomber, the resulted $N_{18}(2)$ will be extremely large and will fall in the region well beyond the range of the AASHTO road tests upon which Equation 10 is based.